CERTIFICATE OF SEISMIC PERFORMANCE LEVEL

☒ **UC-Designed & Constructed Facility**

☒ **Campus-Acquired or Leased Facility**

Campus: UC Berkeley
Building Name: International House
CAAN ID: 1390
Auxiliary Building ID: N/A
Address: 2299 Piedmont Avenue
Site location coordinates: Latitude 37.869665 Longitudinal -122.251462
UCOP SEISMIC PERFORMANCE LEVEL (OR "RATING"): V

I, Michael J. Korolyk a structural engineer, duly licensed by the State of California, have verified that the UCOP Seismic Performance Level is presumptively permitted by the following UC Seismic Program Guidebook provision:

☐Contract documents indicate that the original design and construction of the aforementioned building is in accordance with the benchmark design code year (or later) building code seismic design provisions for UBC or IBC listed in Table 1 below.

☑The existing SPL rating is based on an acceptable basis of seismic evaluation completed in 2006 or later.

☐ Contract documents indicate that a comprehensive building retrofit was done in accordance with the

- ☐ the 1997 Uniform Building Code or later;
- ☐ the 1998 California Building Code or later; or
- ☐ a design completion year of 2000 or later,

Retrofit designs were based on ground motion paramters corresponding to:

- ☐ BSE-1N and BSE-2N
- ☐ BSE-1E (or BSE-R) and BSE-2E (or BSE-C)

Michael J. Korolyk	Principal	AFFIX SEAL HERE
Print Name	Title	
S5274	2019-06-30	
CA Professional Registration No.	License Expiration Date	
Mike Korolyk	2018-12-20	
Signature	Date	

REGISTERED PROFESSIONAL ENGINEER
MICHAEL J. KOROLYK
No. S5274
Exp. 6-30-19
STRUCTURAL
STATE OF CALIFORNIA

Tipping Structural Engineers, 510-549-1906
1906 Shattuck Ave., Berkeley CA 94704
Firm Name, Phone Number, and Address

This form is to be used in connection with Guidebook, Version 1.2, Section III.A.3.d or Section III.A.3.e.

BUILDING DATA

ASCE 41-17 Model Building Type:

a. Longitudinal Direction: C2: Concrete Shearwalls with Regular Openings
b. Transverse Direction: C2: Concrete Shearwalls with Regular Openings

Square Footage: 185,223
Number of stories *above* grade: 5 to 7
Number of basement stories *below* grade: 1 to 2

Year Original Building was Constructed: 1930
Original Building Design Code & Year: NA
Retrofit Building Design Code & Code (if applicable):
NA, NA

SITE INFORMATION

Site Class (A-F): C Basis: Langan (Hearst at La Loma), 2017-02-13
Geologic Hazards:
Fault Rupture: Unknown Basis:
Liquefaction: Low Basis:
Landslide: Low Basis:

COST RANGE TO RETROFIT (if applicable): Low: less than $50 per square foot

ATTACHMENT

Seismic Evaluation: UCB International House Seismic Evaluation, Structural Tipping, 2018-12-12, ASCE 41-17

Table 1: Benchmark Building Codes and Standards

Building Type[a,b]	Building Seismic Design Provisions UBC	IBC
Wood frame, wood shear panels (Types W1 and W2)	1976	2000
Wood frame, wood shear panels (Type W1a)	1976	2000
Steel moment-resisting frame (Types S1 and S1a)	1997	2000
Steel concentrically braced frame (Types S2 and S2a)	1997	2000
Steel eccentrically braced frame (Types S2 and S2a)	1988[g]	2000
Buckling-restrained braced frame (Types S2 and S2a)	[f]	2006
Metal building frames (Type S3)	[f]	2000
Steel frame with concrete shear walls (Type S4)	1994	2000
Steel frame with URM infill (Types S5 and S5a)	[f]	2000
Steel plate shear wall (Type S6)	[f]	2006
Cold-formed steel light-frame construction—shear wall system (Type CFS1)	1997[h]	2000
Cold-formed steel light-frame construction—strap-braced wall system (Type CFS2)	[f]	2003
Reinforced concrete moment-resisting frame (Type C1)[i]	1994	2000
Reinforced concrete shear walls (Types C2 and C2a)	1994	2000
Concrete frame with URM infill (Types C3 and C3a)	[f]	[f]
Tilt-up concrete (Types PC1 and PC1a)	1997	2000
Precast concrete frame (Types PC2 and PC2a)	[f]	2000
Reinforced masonry (Type RM1)	1997	2000
Reinforced masonry (Type RM2)	1994	2000
Unreinforced masonry (Type URM)	[f]	[f]
Unreinforced masonry (Type URMa)	[f]	[f]
Seismic isolation or passive dissipation	1991	2000

Note: This table has been adapted from ASCE 41-17 Table 3-2. Benchmark Building Codes and Standards for Life Safety Structural Performed at BSE-1E.

Note: UBC = *Uniform Building Code*. IBC = *International Building Code*.

[a] Building type refers to one of the common building types defined in Table 3-1 of ASCE 41-17.

[b] Buildings on hillside sites shall not be considered Benchmark Buildings.

[c] not used

[d] not used

[e] not used

[f] No benchmark year; buildings shall be evaluated in accordance with Section III.J.

[g] Steel eccentrically braced frames with links adjacent to columns shall comply with the 1994 UBC Emergency Provisions, published September/October 1994, or subsequent requirements.

[h] Cold-formed steel shear walls with wood structural panels only.

[i] Flat slab concrete moment frames shall not be considered Benchmark Buildings.

Image from Google Maps

Seismic Evaluation of UC Berkeley International House

December 12, 2018

For questions or comments, contact:
Michael J. Korolyk, Principal
Alec Zavala, Project Engineer

Introduction

UC Berkeley has contracted Tipping Structural Engineers (TSE) to perform a seismic evaluation of the International House (I-House), a concrete residential structure built into the sloping grade near the southwest corner of UC Berkeley campus, adjacent to Memorial Stadium and within 100 feet of the Hayward Fault. The structure was built in the late 1920's mainly of reinforced concrete. Following the grade, the structure is terraced, and various wings of the building are different heights above adjacent grade. The tallest point of the structure is the rounded cupola at the intersection of two main wings at about 140' above the ground floor. The two east residential wings top out at about 90' above the ground floor. The largest floor plate (not including the auditorium) has an area of about 30,000ft^2, and the total building area is about 180,000 ft^2.

The gravity framing system comprises 6" slabs supported by beams, columns, and a concrete facade with regular window openings. There are many locations in the building where columns are transferred by beams to form open spaces below. This is the case in particular for the cupola, the corners of which land on transfer beams. The foundation comprises shallow strip and spread footings.

Lateral stability derives from typically 8" thick concrete walls. The typical residential wings have window openings forming a regular pattern of column piers that alternate in width between 3'-2" and 8'-10". Along these piers, pilasters are built with additional vertical reinforcement.

Outline of Sections

Executive Summary

Tipping Structural Engineers has contracted with UC Berkeley to provide a seismic evaluation of the International House at UC Berkeley (I-House). Our evaluation comprises extensive review of the original design drawings and previous evaluation reports written by others and made available to us. The goal of the seismic evaluation is to provide a rating in accordance with Table A.1 of the UCOP Seismic Safety Policy.

In accordance with our contractual scope of work, and with consideration for the information we have gained from a large amount of nonlinear response history analysis of three models of the structure with varying elemental assumptions, we recommend a rating of V for the structure in its existing condition. This rating implies "significant structural and nonstructural damage and/or falling hazards that would represent appreciable life hazards" (UCOP Seismic Safety Policy, pg. 16, description of *Poor* rating). Since the recommended rating is greater than IV, our scope of services includes retrofit design at a conceptual level deemed necessary to achieve a rating of IV. We provide retrofit recommendations in the following section.

The pivotal issue in our rating recommendation is the condition of the cupola with its heavy weight high atop the structure and supported by beams (not columns or walls). Secondarily, certain beams that carry significant but not nearly a majority of the tributary area of the structure appear susceptible to shear failure during an earthquake.

We deem the vulnerabilities of the existing structure to be local in nature, and with or without retrofit, the analysis results do not indicate a global collapse mechanism. Therefore, we recommend addressing the vulnerable beams under the cupola and elsewhere and believe an evaluation of the retrofitted structure would warrant a revised rating of IV without extensive augmentation of the lateral force resisting system.

Retrofit Recommendations

We recommend the following retrofit measures with the intent to improve the rating of the structure to IV: strengthen or provide additional supports for beams below cupola and beams identified that support discontinuous concrete walls or columns.

Rating IV implies "structural and nonstructural damage and/or falling hazards that would represent low life hazards" (UCOP Seismic Safety Policy, pg. 16, description of *Fair* rating). The recommended retrofit measures are the minimum we deem are required to limit life hazard and areas of potential collapse.

The following content identifies areas where we recommend strengthening measures and conceptual diagrams of how the strengthening might be achieved. It is beyond the scope of this study to provide details and plans ready for construction; the intent is to convey an idea of the disruption that might be involved and inform a rough order of magnitude (ROM) cost estimate.

The areas of the building affected are as follows, prioritized in order of urgency (see following illustrations):

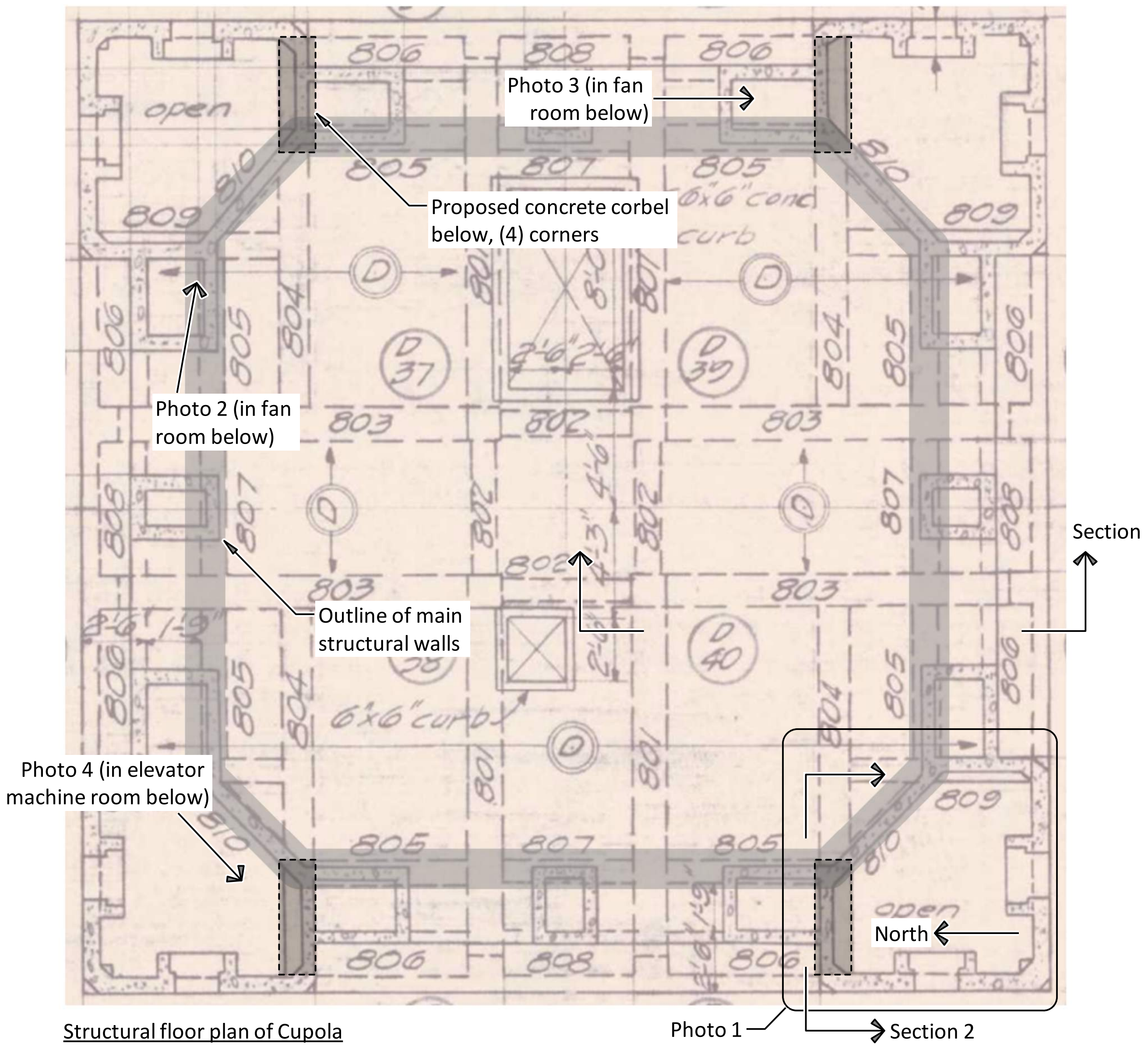
1. The cupola and the room(s) underneath it. The original architectural plans call this a fan room on the seventh floor;
2. Transfer condition above certain units on the west side of the building on the first floor;
3. The roof above the Foyer on the ground floor;
4. The Entrance Hall on the ground floor;

Cupola

Narrative:

The cupola is constructed with double walls, 6" thick, to present a large mass, but the main bearing walls are inset 2'-6" from solid 8" concrete walls below. The vulnerability of the structure is potential shear mechanisms in all of the beams that support the main bearing walls. This mechanism is likely to occur locally between where the walls of the cupola land and the solid walls below as the weight of the cupola tips to one side or corner during an earthquake.

The retrofit strategy is to effectively increase the shear strength of the vulnerable corners. We believe this could be achieved with new deep concrete corbels that would be doweled into the concrete walls and beams in the corners of the fan room below the cupola. The eventual design challenge will be to find the corbel or other strengthening scheme that minimizes the impact to equipment currently located in the fan room: the elevator machinery, large air handler, ductwork, and access stair.



Structural floor plan of Cupola

Cupola

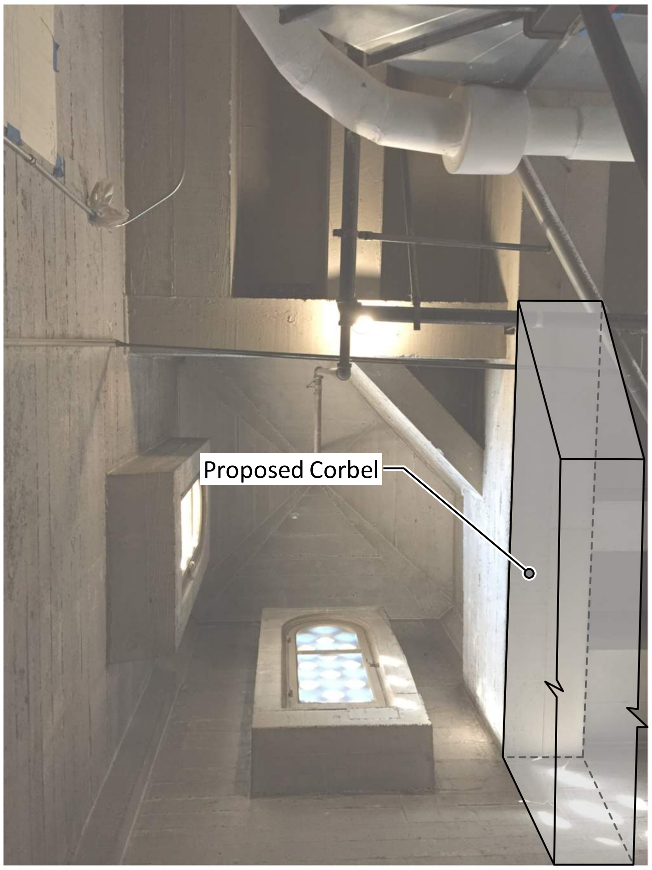
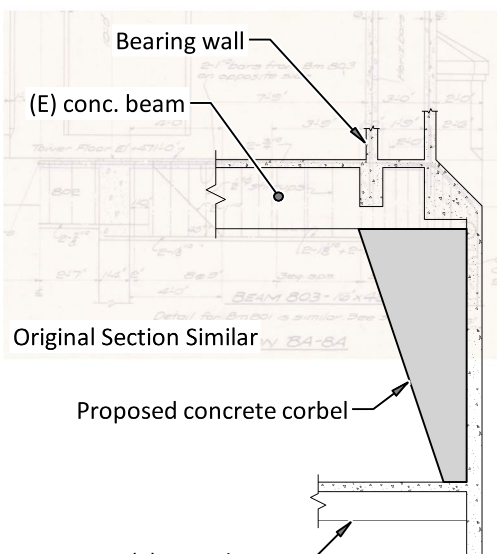
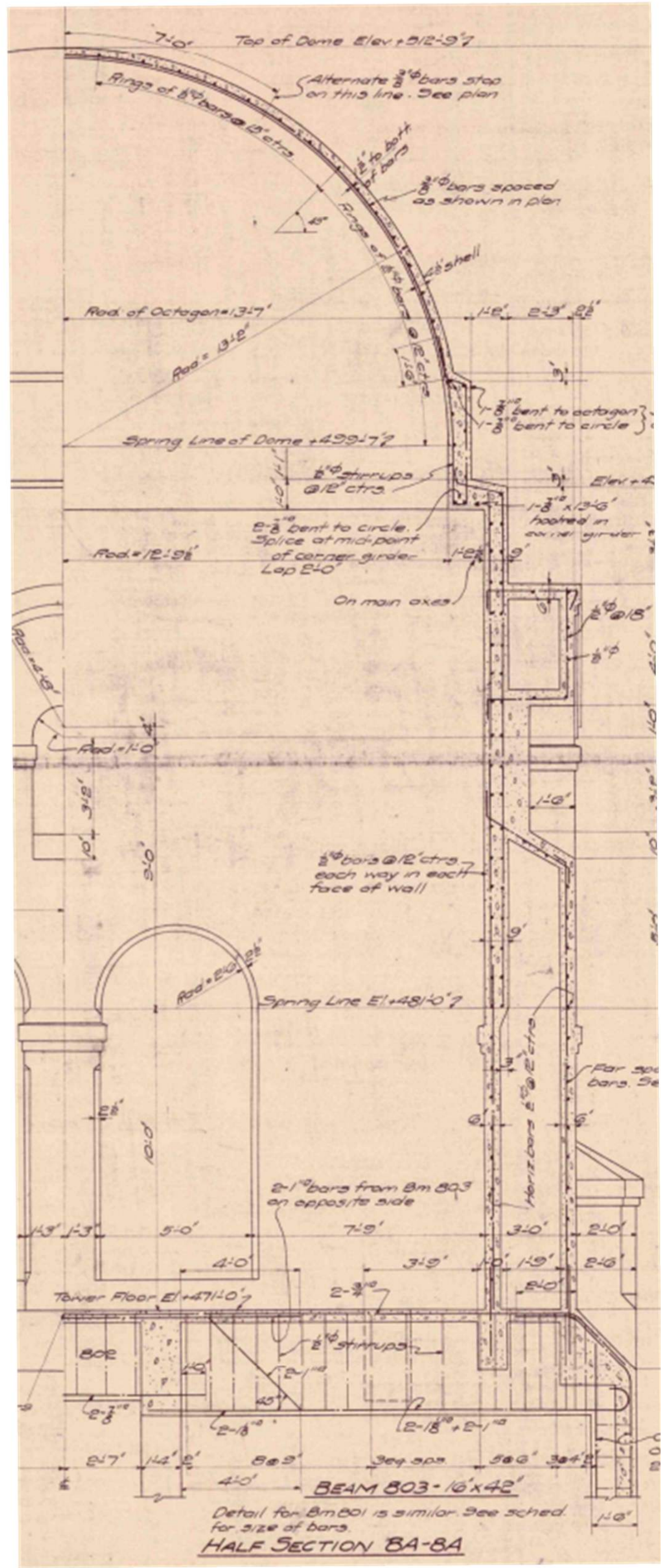


Photo 1



Section 2: Proposed Corbel



Section 1: Section from Original Drawings

Cupola

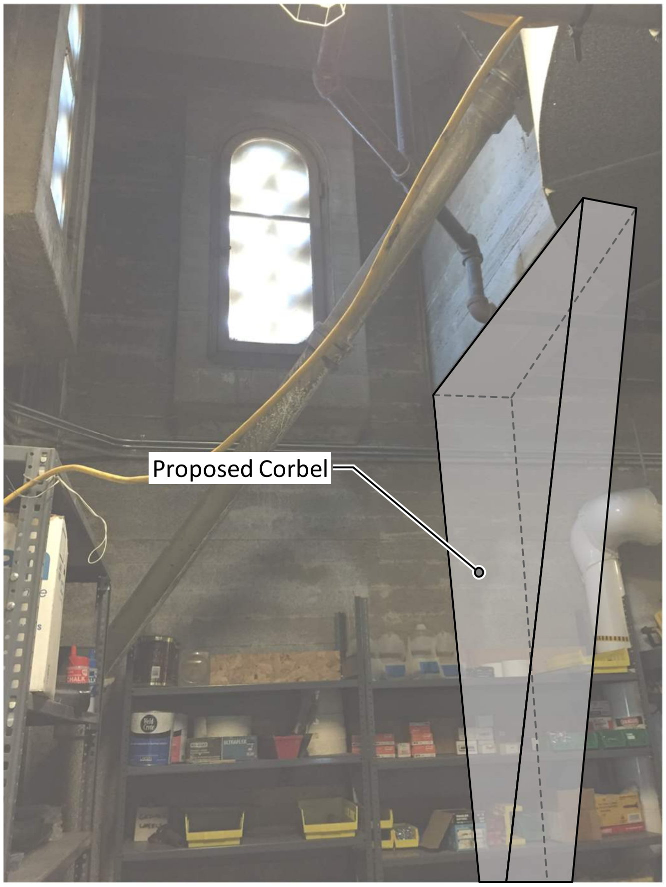


Photo 2

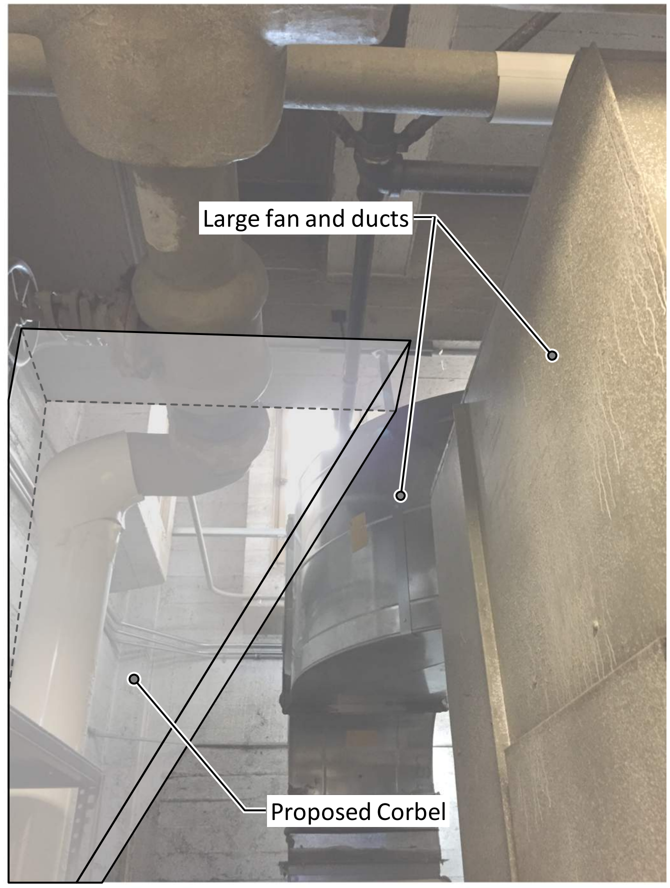


Photo 3

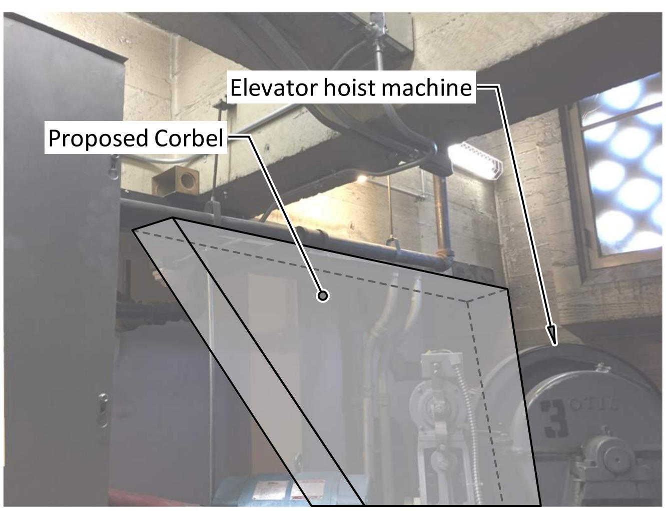


Photo 4

West Facade Over Units

Narrative:

Analysis results indicate potential shear failures in beams supporting discontinuous walls that form part of the west facade due to four existing columns concealed in the corridor walls that are offset.

We recommend augmenting the existing columns concealed in the corridors to address the offset.

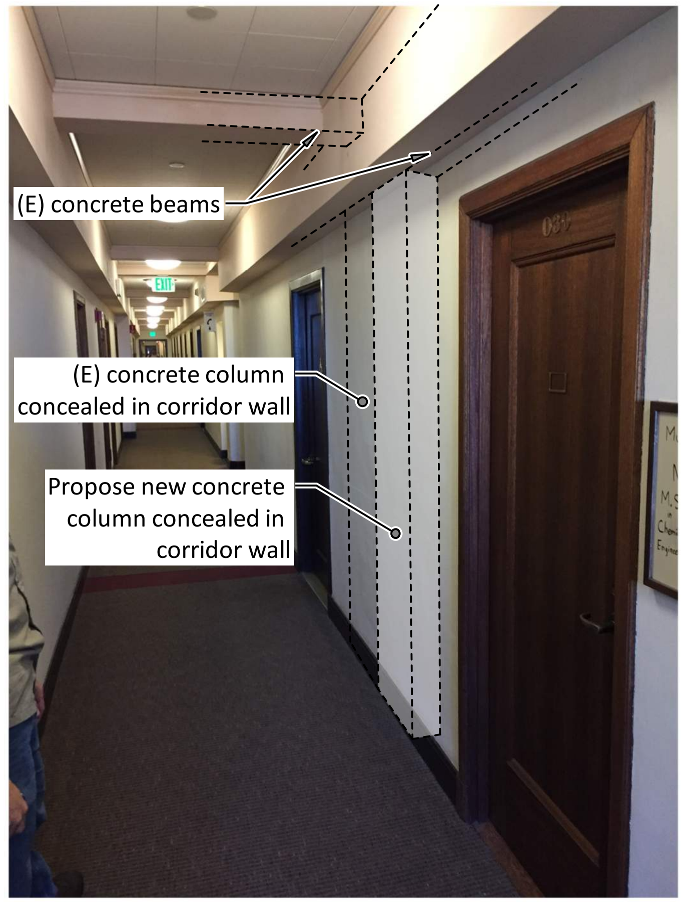
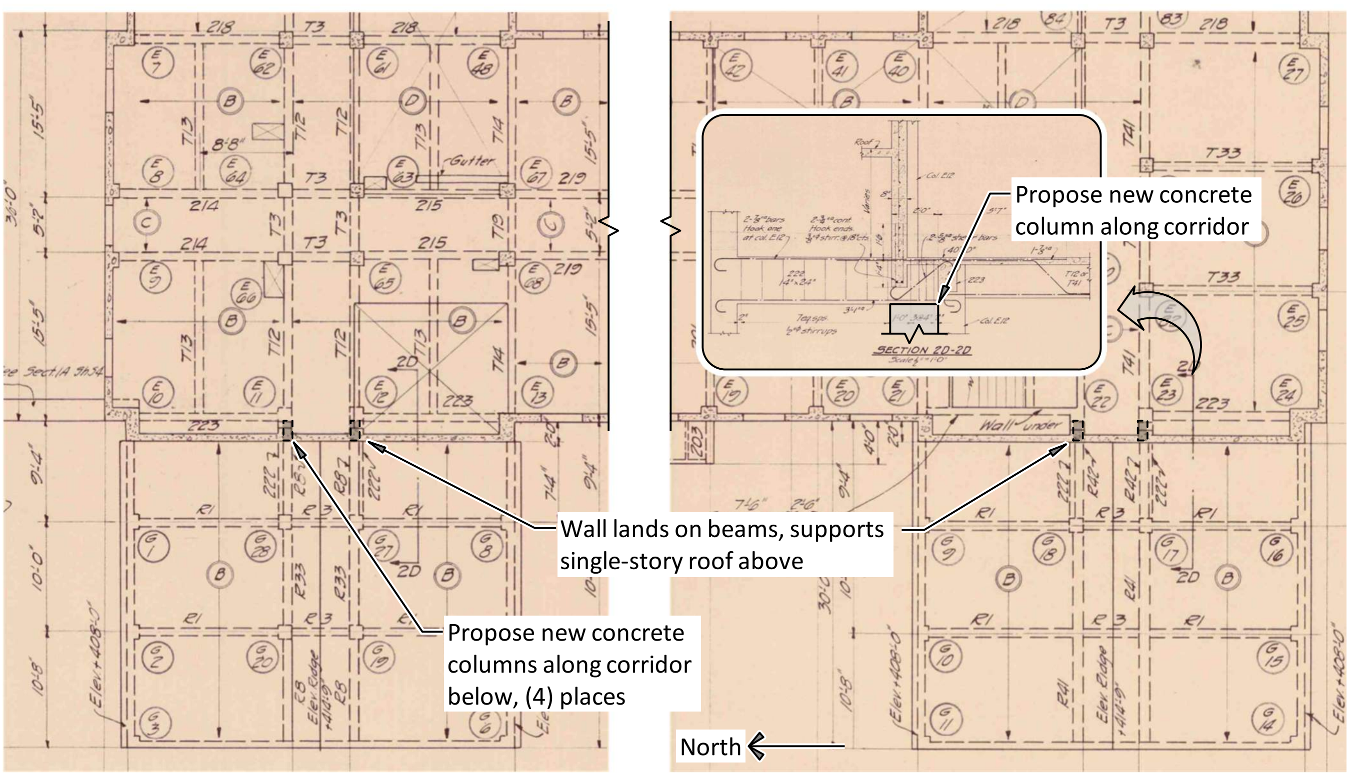


Photo 2



Partial Second Floor structural plan of roof at west facade

Foyer Roof

Narrative:

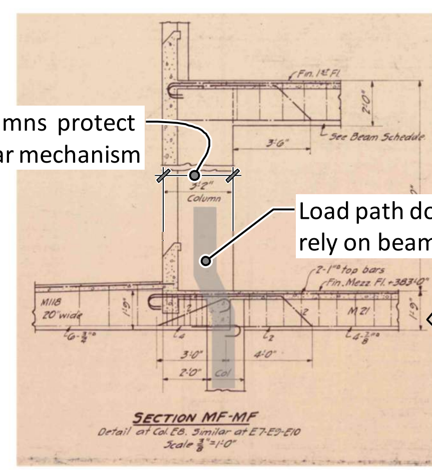
Analysis results indicate potential shear failures in beams supporting discontinuous walls above the Foyer. There are are two such conditions:

1. An offset in the north facade of a residential wing with several stories of gravity load above, and
2. A wall supporting a nominal amount of roof load.

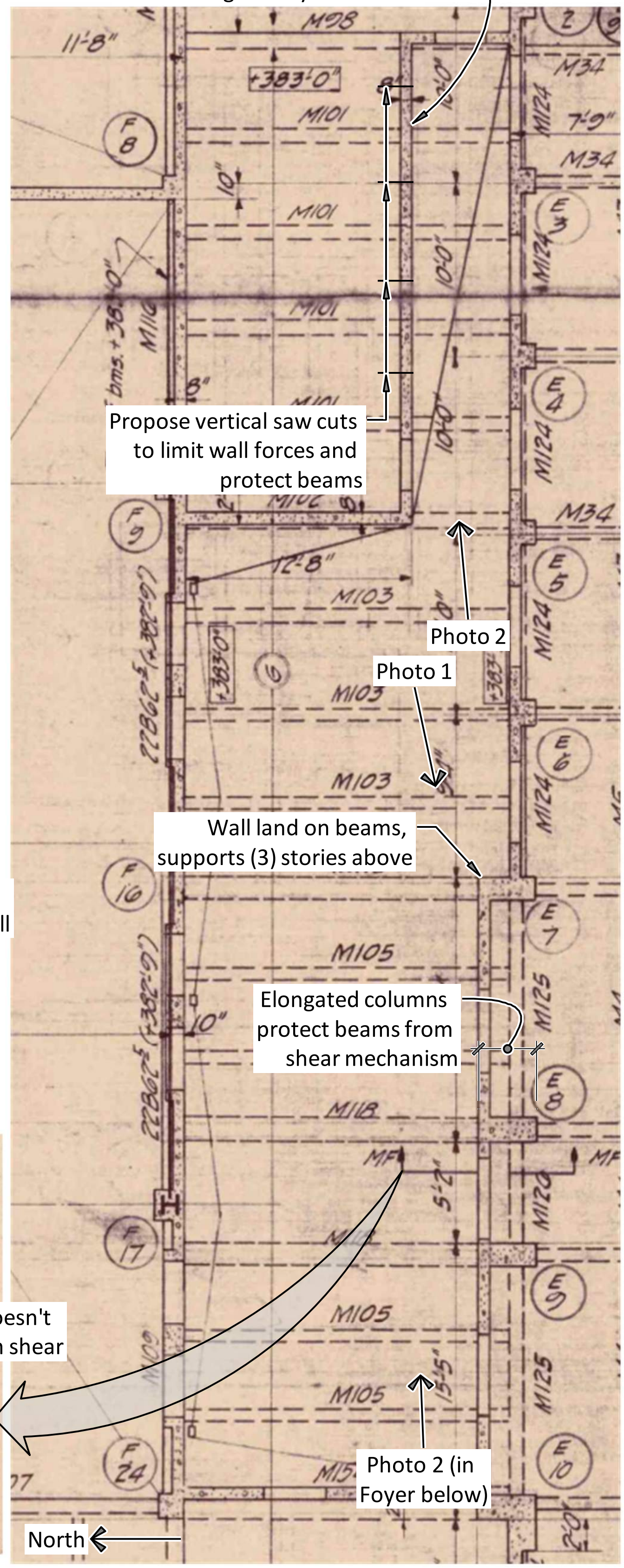
In the first case, upon further review, we do not recommend retrofitting these beams because we believe the model does not properly account for secondary mechanisms that would alleviate the concerns. In the case of the beams carrying several stories of gravity load, the columns appear to have been elongated in such a way as to protect the beams and avoid a shear mechanism.

In the second case, we recommend vertical saw-cuts on the inside of the wall but not all the way through to the exterior. The cuts are intended to limit the amount of overturning forces the walls can transfer to the beams below, effectively protecting the beams from excessive shear.

Photo 1: Offset in Facade Wall



Section from original drawings



Structural floor plan of roof above Foyer

Foyer Roof

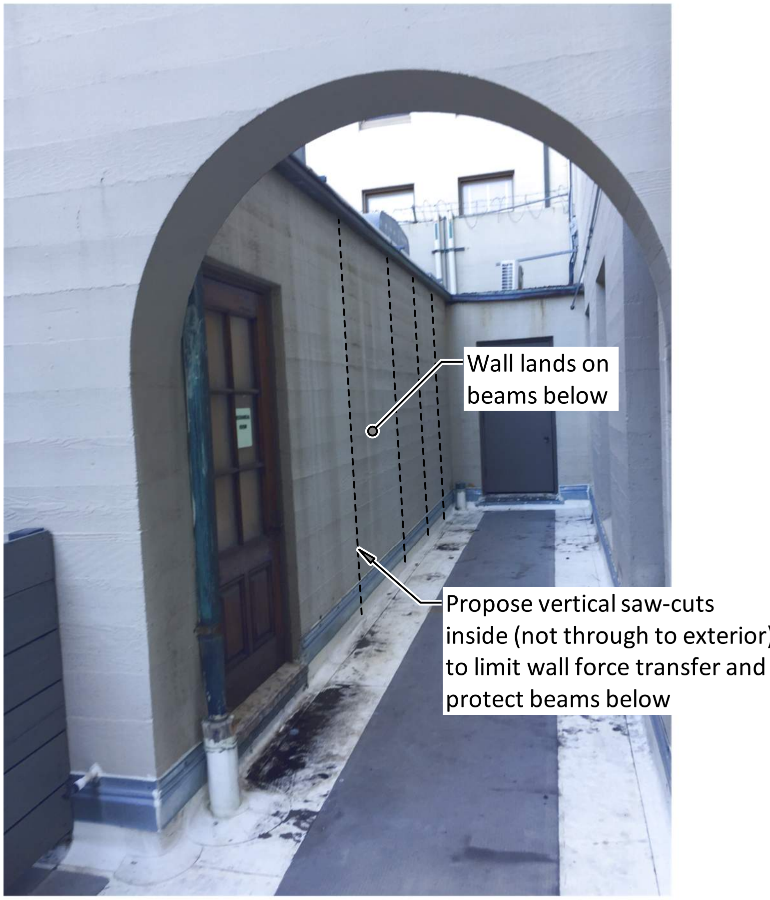


Photo 2

Photo 3: Foyer

Entrance Hall

Narrative:

Above the Entrance Hall is a condition where several columns land on steel transfer beams in order to provide the open space below. Above the four corners of the Hall, the columns above are offset from those below and land on concrete beams that are vulnerable to a shear mechanism. Given that the offset of the column center-lines is 30" and the concrete beams are 36" deep, it is possible that the concern over beam shear is false; unfortunately, it is difficult be certain, and the consequences of beam shear failure in this case would be somewhat severe.

A conservative approach would be to add new concrete columns to overlap the existing columns either on the floor above or below the offset. We have shown new concrete columns to be concealed in walls on the mezzanine level above the Great Hall.

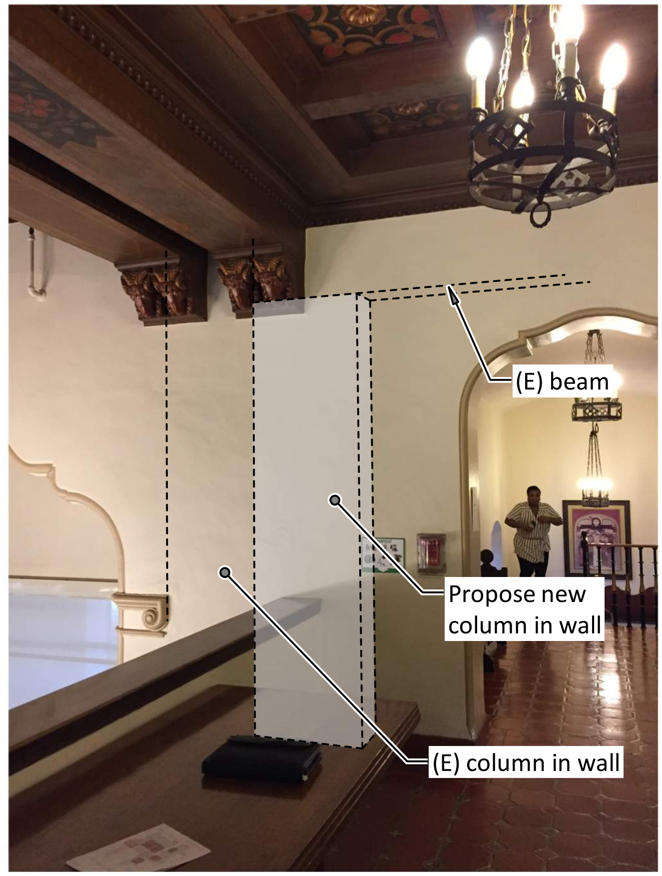
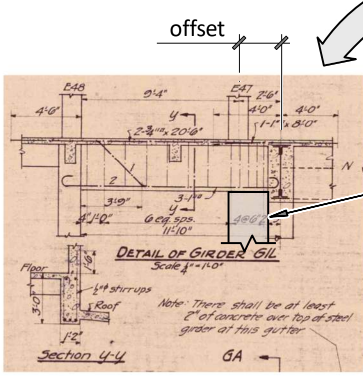
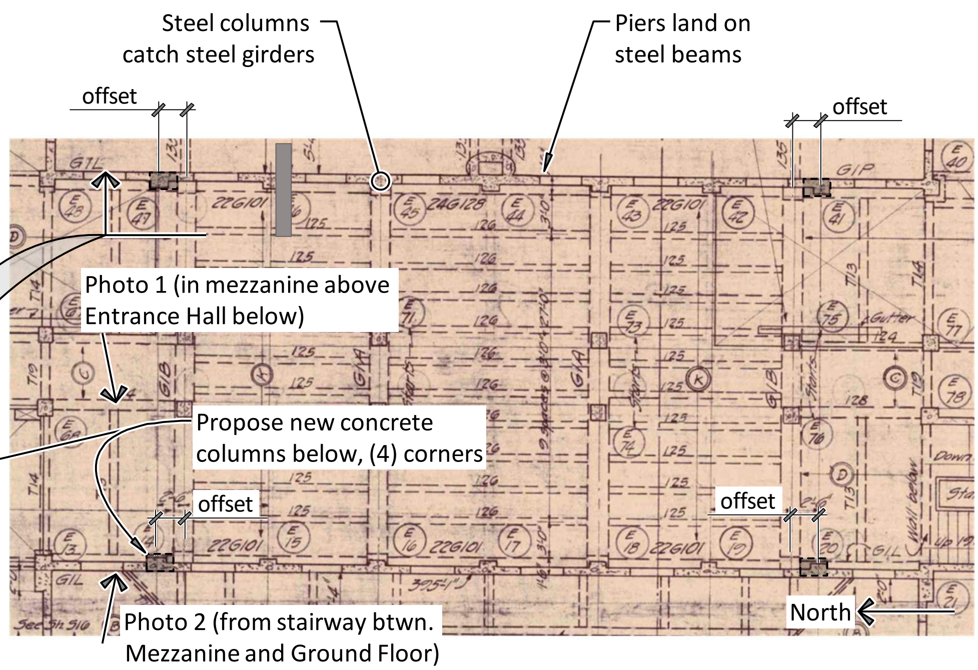


Photo 1: Mezzanine above Entrance Hall





Section from original drawings

Partial First Floor structural plan of floor above Entrance Hall

Entrance Hall

Photo 2: Between Mezzanine and Ground Floor

Evaluation Summary

With respect to the expected seismic performance of the International House, we note the following key observations evident in our review of the existing drawings and in the results of the analyses:

1. The I-House shows a large amount of lateral strength relative to its weight, attributable primarily to the 8" concrete walls with window openings that form its exterior facade and secondarily to miscellaneous interior concrete walls.
2. The walls are typically lightly reinforced relative to modern standards with a single curtain of reinforcement, which is consistent with other buildings of this vintage.
3. The response of the structure is such that interstory drift is expected to be low even for a large earthquakes; the average drift results are summarized as:
   1. BSE-2N analyses: between 1.0% and 1.5%, depending on modeling assumptions.
   2. BSE-1N analyses: between 0.5% and 1%. For context, a new building would be allowed 2% to 2.5% drift for a BSE-1N analysis.
4. Despite the low drift, the structure dissipates energy through non-ductile shearing primarily of the spandrel beams and secondarily through a mixture of flexural yielding in walls and shearing in certain wall elements. These mechanisms are not as reliable as ductile mechanisms encouraged by modern building code. In modeling the structure, we attempt to take into account the lack of ductility and increased uncertainty of these mechanisms in order to substantiate our conclusions.
5. The configuration of the exterior facade is such that there are alternating longer and shorter column piers that support gravity bearing beams. The shorter column piers are subject to shearing mechanisms. However, it appears that the geometry (depth relative to length) of the spandrels is such that arching action may transfer gravity loads to the longer piers and circumvent the shearing piers.
6. The cupola represents a large seismic mass (roughly 300 tons) perched at the top of the building where resonance of the structure during an earthquake may cause very large spectral accelerations. Set back from the exterior facade, the edges of the cupola are supported by beams rather than concrete walls. These beams appear to have insufficient shear strength to resist loads imparted by the cupola under both the BSE-1N and BSE-2N hazards.
7. Some beams in the building carry gravity loads transmitted through concrete walls above. The analyses show that certain of these beams may have insufficient shear strength to resist loads imparted by the walls under the BSE-2N hazard.

The UCOP Seismic Safety Policy Table A.1 provides ratings that ultimately refer to the ASCE 41 standard; the key question in this evaluation is whether this structure might be rated IV, which is to say it meets the Life-Safety and Collapse Prevention performance levels under the BSE-R and BSE-C hazard levels.

For a structure as complicated as the I-House is, it would be inappropriate to evaluate the performance levels in the strictest and most microscopic way possible, meaning that if any element does not meet a performance level, than the building is deemed as a whole to not meet the performance level. However, this is how ASCE 41 is structured. If the strictest interpretive method is not followed, subjective judgment must be used to ultimately delineate the boundary where a certain performance level is met.

In this case, we draw the distinction clearly between IV and V in the following way. On one hand, we note that the lateral drifts are low and that the gravity frame is expected to remain intact enough that a "global" collapse is not expected. On the other hand, we are compelled to rate this structure V because of the likelihood of severe damage to the support structure of the cupola and the extremity of the potential consequences of such damage. The shear-critical beams lend similar influence to our conclusion, although not as dramatically.

We note that we did not actually analyze the structure with ground motions scaled to the BSE-R and C hazard levels; instead, we used ground motions scaled to BSE-1N and 2N, respectively, which are somewhat larger than BSE-R and C (see Spectra Section, page 6). We do not believe analyzing at the BSE-R and C levels would change our conclusions and rating given the prominence of the vulnerability of the cupola, particularly at the BSE-1N level, which is less than the BSE-C. In other words, since the cupola appears vulnerable at the BSE-1N level, it would also appear vulnerable at the BSE-C, so the rating IV would not be applicable.

Previous Evaluations

The I-House has been the subject of more than one evaluation in the past; the most recent in 1994 (Paul Fratessa) and in 1999 (ISEC).

The 1994 report is well-written and comprehensive and includes peer review comments, though the peer reviewers are not named. The report refers to and makes use of a previous report in 1974 by Frank McClure which is not available. The evaluation follows a capacity vs. demand procedure. Seismic forces are estimated as 10% of building weight (i.e. base shear ratio = 0.1), and distributed to punched opening facade walls based on tributary area. Treating the facades as moment frames, the report indicates that the capacities of spandrel and column elements are slightly larger than estimated demands. The rates the building as "Fair" but with improved Life-Safety expectations related to addressing the hollow clay tile partitions, which, we understand, has largely been done.

The 1999 report is quite brief and appears to focus mainly on certain girders in the building. The report is apparently an earlier rendition of time-history analysis and indicates that the building might remain elastic under a design-basis earthquake an assertion contradicted by the peer review report by Rutheford and Chekene. Due to its brevity, lack of detail and explicit recommendations, we find the 1999 report of little use.

Key Features of Construction

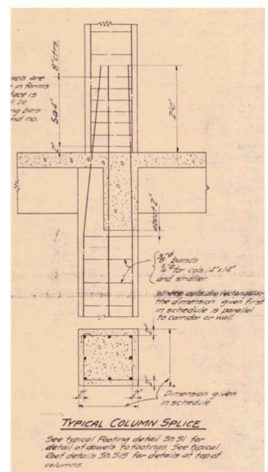
Neither the concrete nor reinforcement strength are evident in the drawings or reports available. We assume for now values based on ASCE 41-13 of 3ksi and 40 ksi, respectively.

Generally speaking, the concrete walls are lightly reinforced by today's standards. For example, the 8" wall reinforcement is typically single curtain $\frac{5}{8}$" sq. @ 18" o.c. vertical and $\frac{1}{2}$" sq. @ 12" o.c. horizontal.

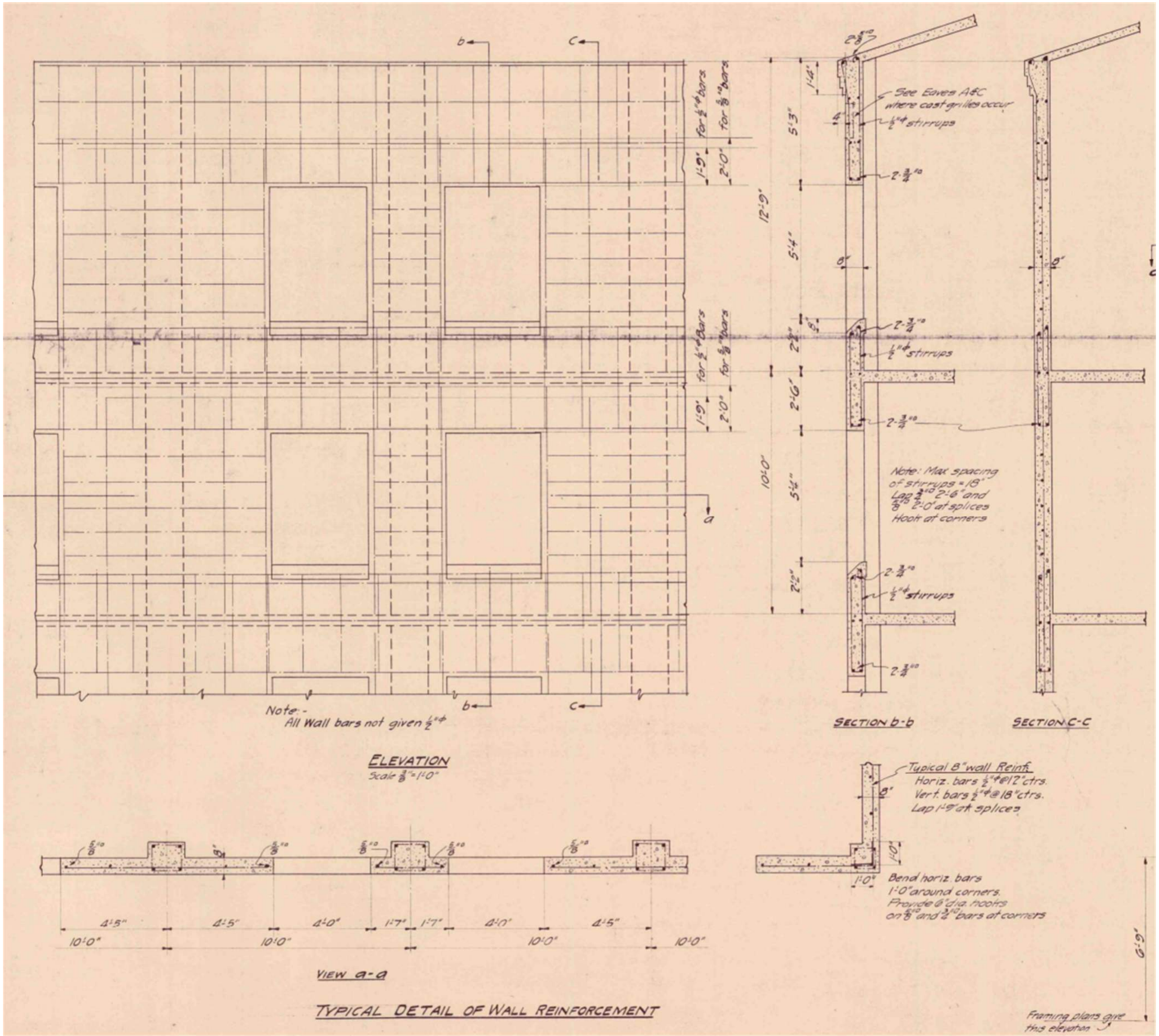
The typical exterior facade is 8" thick concrete wall with window openings (see illustration next page). The exterior columns are built integrally with the facade. The column/wall piers have a pattern alternating length: 3'-2" and 9'-10".

The spandrel beams are typically 4'-8" deep and have double-legged (vertical) stirrups $\frac{1}{2}$" sq. @ 18" o.c. and (2) $\frac{3}{4}$" sq. bars top and bottom. The shear force corresponding to yielding end-moments is quite close to the nominal shear capacity. Based on the geometry (i.e. depth to length ratio $\approx$ 1:1) and, they are most likely controlled by a shear mechanism (diagonal crushing or yielding of shear reinforcement).

Column dimensions vary: they are as small as 14x14 and as large as 26x26. Vertical reinforcement ratios are typically small (less than 2%), and ties are $\frac{3}{8}"\phi$ ($\frac{5}{16}"\phi$ for 14x14 columns) @ 8" o.c. (@ 4" o.c. over the splice). With this spacing, tie reinforcement is effective for shear strength only for columns with the smallest dimension greater than about 20". Many of the interior columns at the lowest three levels of the building fall into this category.



Typical Column Details



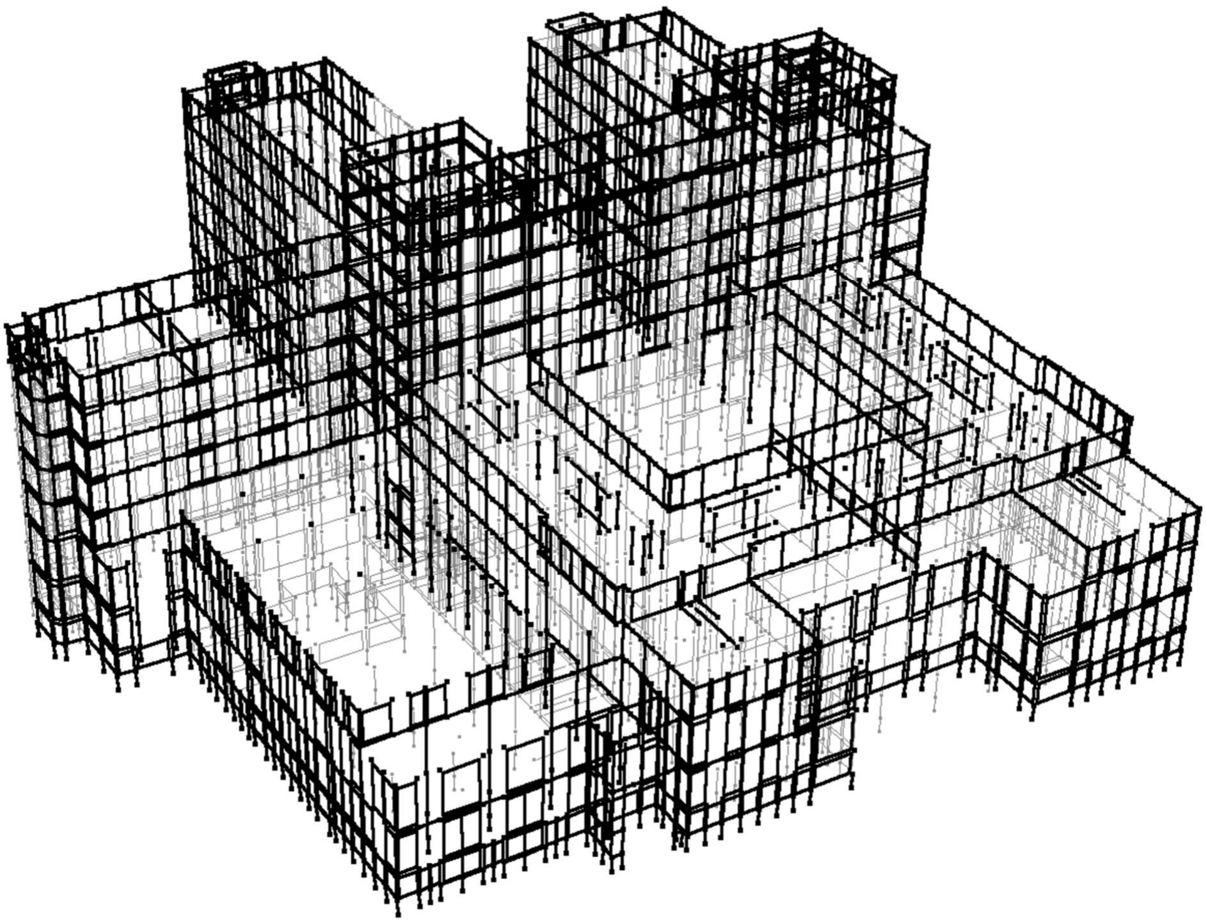
Typical Pier and Spandrel Elevation and Section

Current Study

TSE built a computer model of the structure using CSI Perform 3D. The model represents to the degree practically possible the complexity in the structure, including:

1. Nonlinear shear material and fiber cross sections in wall elements with expected material assignments;
2. Column elements with nonlinear flexural hinges top and bottom and a nonlinear shear hinge in the center;
3. Spandrel elements with a single shear hinge at the middle;
4. Diaphragms broken into various wings of the building and joined by elements with fiber cross section hinges and shear hinges to afford some diaphragm flexibility and, if needed nonlinear behavior;
5. Compression-only soil spring elements to allow uplift.

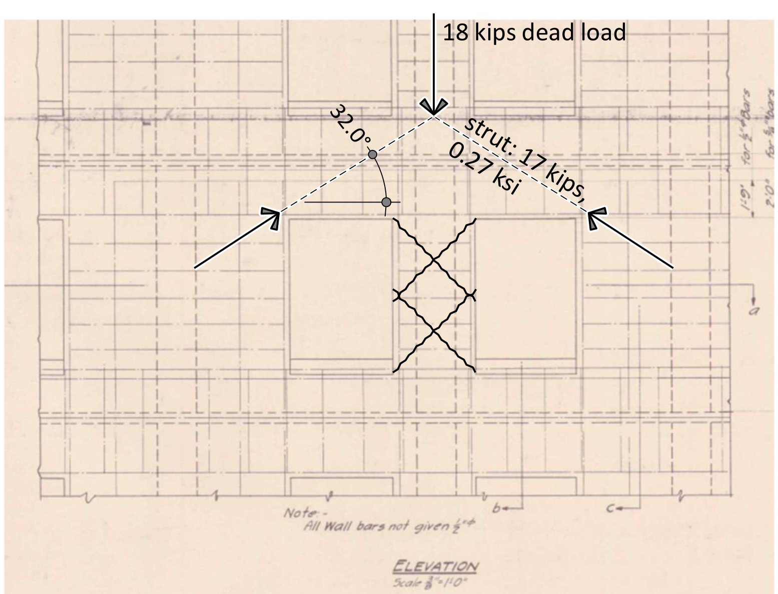
Ground motions were selected for the Upper Hearst Residential project with peer review by LCI. These same ground motions are used for the current study as the proximity to the Hayward Fault is the same and soil conditions are assumed to be the same. There are (11) ground motions scaled to the BSE-1N. To run BSE-2N analyses, a scale factor of 1.5 is applied in the model.



Summary of Analysis Results

The model has been run for the BSE-2N and BSE-1N hazards and results extracted for review and synthesis. The following list enumerates key findings based on the results:

1. The primary lateral force resisting system is the concrete facade walls with window openings. The spandrel beams across the windows form moment frames with the column/wall piers.
2. Average interstory drift is quite low, less than 1% for the BSE-2N.
3. Recall that along the exterior facade, there is a pattern of alternating 3'-2" wide and 9'-10" wall piers integral with columns:
    1. A significant number of the shorter column piers show:
        1. Average inelastic shear strain greater than 1%, which may indicate loss of gravity bearing capacity.
        2. Average inelastic flexural hinge rotation greater than 1%, which may indicate loss of gravity bearing capacity.
    2. The longer piers show only a few instances of shear strain greater than 1%, and rebar tensile and concrete compressive strains are not excessive.
4. We have run side studies on interior gravity beams and columns which show that the beams crossing the corridor are susceptible to shear mechanism at very low interstory drift. The other interior longer beams appear to yield in flexure in areas where tie spacing is less than d/3 and do not appear susceptible to a shear mechanism.
5. The cupola at the top of the structure is expected to experience accelerations amplified by approximately a factor of 4.0 relative to ground due to resonance of the base structure.The edges of the cupola are set back from the facade walls and supported by 16"x42" deep beams with ½" square stirrups with varying spacing. We have run a sub-structure analysis on the cupola using acceleration time histories from the main model and find likelihood of shear failure of the supporting beams under the cupola.
6. There are many locations in the building where discontinuous walls are supported by concrete beams which may cause localized shear mechanisms. These beams typically do not support large areas of the building, but their failure would raise concerns over falling hazards and blocking egress.
7. Based on a simple analysis of the exterior facade, it appears feasible that, if a short column pier is compromised by shear deformation, the spandrel is deep enough to bridge across to the larger piers through strut action.



Peer Review Meeting Notes - October 26, 2018

During our meeting on October 26, 2018, we heard feedback from the peer review team and subsequently took into account their comments in our analysis and reporting.

- Comment: Plot response spectra of selected scaled ground motions in comparison with BSE-R and BSE-C spectra in accordance with CBC 2016

  Response: A graph is now included in the of page 6 of the Spectra Section of the report showing the comparison requested.

- Comment: Run analyses with variations of the base model: variation 1 includes horizontal soil springs and shear strain at strength loss of 0.003, and variation 2 further includes softening of the elastic response of column and spandrel members.

  Response: Variations in the modeling are summarized below. Analyses were performed at the BSE-2N hazard level and results are reported for drift, column, wall and spandrel deformations. Snapshots of the deformed shape of the model are taken from an individual record with approximately the highest lateral drift; elements are color-coded based on shear strain. The following list summarizes the Base Model and variations:

  - Base Model:
    - No horizontal soil springs (fixed horizontally at terraced levels)
    - Shear strain at strength loss, $DL = 0.007$ for columns, walls, and spandrels
    - Elastic Modulus
      - Columns: $0.5 E_c$
      - Spandrels: $0.35 E_c$
      - Walls: $1 E_c$ (cracking in fiber sections)
  - Variation 1 = Base Model plus
    - Horizontal Soil Springs
    - Shear strain at strength loss, $DL = 0.003$ for columns and walls
  - Variation 2 = Variation 1 plus
    - Columns, $E_{eff} = 0.35 E_c$
    - Spandrels:
      - $E_{eff} = 0.2 E_c$
      - Shear hinge changed to Trilinear (was EPP) with $FY = 2 \cdot \sqrt{f'_c} \cdot b_w \cdot d$ and $DL = 0.003$

- Comment: Substantiate further that interior columns are not shear-controlled, particularly at lower parts of the structure where columns cantilever out of the stiff lower box up into the more flexible upper part of the structure.

  Response: In our previous reporting, we showed a graph of column shear capacity (V_n) relative to shear demand corresponding to fixed-end yield moments $\frac{2 M_{ye}}{h}$. For simplicity, we took $h = 64\,in$ because it is the height of column piers along the facade. In the revised Column, Wall, and Spandrel Results section, on page 2, we plot the ratio $\alpha_V = \frac{M_{ye}}{V_n h}$, taking more detailed account of column height, which is particularly important with regard to the interior columns. Where $\alpha_v < 0.5$, columns are flexure-controlled considering no beam flexibility (i.e. fixed/fixed); $\alpha_V = 1$ corresponds to the flexure control limit assuming a pinned/fixed condition. The reality is somewhere in between, i.e. $0.5 \le \alpha_V \le 1$. As the graph shows, only two interior columns show α_V>0.75, and these by a small margin. This reinforces our perception that only exterior facade columns appear vulnerable to shear failure, with two exceptions.

Comments on Results of Models with Variations

We make the following observations with regard to the results of the model variations described above:

1. The effect of the variations is to increase lateral displacement (ref. Drift section pgs. 2-5).
2. With Variation 1, the pattern of the interstory with respect to floor level parallels that of the Base Model, but the pattern of Variation 2 shows larger interstory drift at the upper levels of the structure (Drift section pg. 2, top graph).
3. Despite the increase, mean drift ratio results of the model variations are not excessive, approximately 1.5% for the BSE-2N
4. For the Base Model and variations, drift results in the lower stories are larger in the fault parallel direction whereas in the upper stories, drift is larger in the fault normal direction. Noting that the drift plots show the envelope of all extreme locations of the building, we perceive that embedment into the hillside reduces drift perpendicular to the hillside but not parallel, or at least not for certain locations.
5. The drift results do not all show uniform dispersion about the mean. In particular, the mean drift of Variation 2 in the fault normal direction is dominated primarily by one outlier record and secondarily by two other records (Drift section, pg. 5).
6. In the Column, Wall, and Spandrel Results section, we plot element results for the Base Model and variations.
7. Column shear strain results (pgs. 4-6) of the Base Model and variations are not dramatically different from one another. Variation 2 typically shows the smallest values probably due to elastic softening and shifting the yielding mechanism into spandrels.
8. Wall shear strain results (pgs. 12-14) show an increasing number of wall elements with shear strain above 0.01 comparing the Base Model to the variations. Variation 2 typically shows the largest values.
9. Spandrel shear strain results (pgs. 19-21) show an increasing number of elements with shear strain above 0.01 comparing the Base Model to the variations. Variation 2 shows a dramatically larger number of spandrel elements with high shear strain. This is consistent with the large interstory drift ratios that appear in the upper levels of the Variation 2 results. Generally, the coupling beam (moment frame) effect is lost where the spandrel beams lose shear strength, which occurs at a smaller drift ratio for Variation 2 where the strength loss strain of the spandrels, $DL = 0.003$.
10. In the Snapshots section, we show the deformed shape of the Base Model and variations for Record 5 (RSN 0767, 1989 Loma Prieta, scaled by 1.84) at a time stamp of 5.4 sec., which is approximately the condition of largest interstory drift response from any record at any time. We focus on the response in the fault normal direction along two primary elevations of the structure. We show the deformed shape (deformations are scaled by a factor of 10), with element color corresponding to shear strain.
11. The snapshot illustrations in conjunction with the drift ratio and element results plots clarify how the structure tends to deform. Although the wall and column shear strain plots indicate elements with high shear strain (especially Variation 2), the drift ratio profile does not resemble a story mechanism; why not? In the snapshots (pgs. 3 and 6), we note that walls with high shear strain are typically elements representing below grade retaining walls that are trapped between two longer wall piers above. Above grade, the longer piers do not show high shear strain, but the spandrel elements show very high shear strain. Thus, the longer concrete piers above grade appear strong enough to constrain the deformation mode as a flexural cantilever rather than a weak story.

Peer Review Meeting Notes - December 5, 2018

During our meeting on December 5, 2018, we discussed with the peer reviewers the results of the model variations with the hope of finding consensus around a rating and potential retrofit scope. The following are notes from this meeting

1. Typographical errors were found in the interstory drift plots: the drifts Fault Parallel drifts in Variation 1 and 2 were mislabelled Base Model and Variation 1, respectively.
2. The behavior of the building is vexing:
    1. With certain wall elements experiencing shear strains beyond shear strength loss, still no story mechanism is evident
    2. Variation 2, with shear strain at strength loss (DL_V) set at 0.003 for all elements, shows larger interstory drift but less shear strain in critical bearing wall elements, whereas in Variation 1 (in which $DL_V = 0.007$) the drift is less, but more walls show strain exceeding DL_V.
3. The period of the building is unexpectedly short.
4. The mass of the building has not been verified by the peer reviewers in detail.
5. There is a question whether the building tends to move more toward rather than away from the hillside.
6. The target spectrum for the BSE-2N records is not plotted.

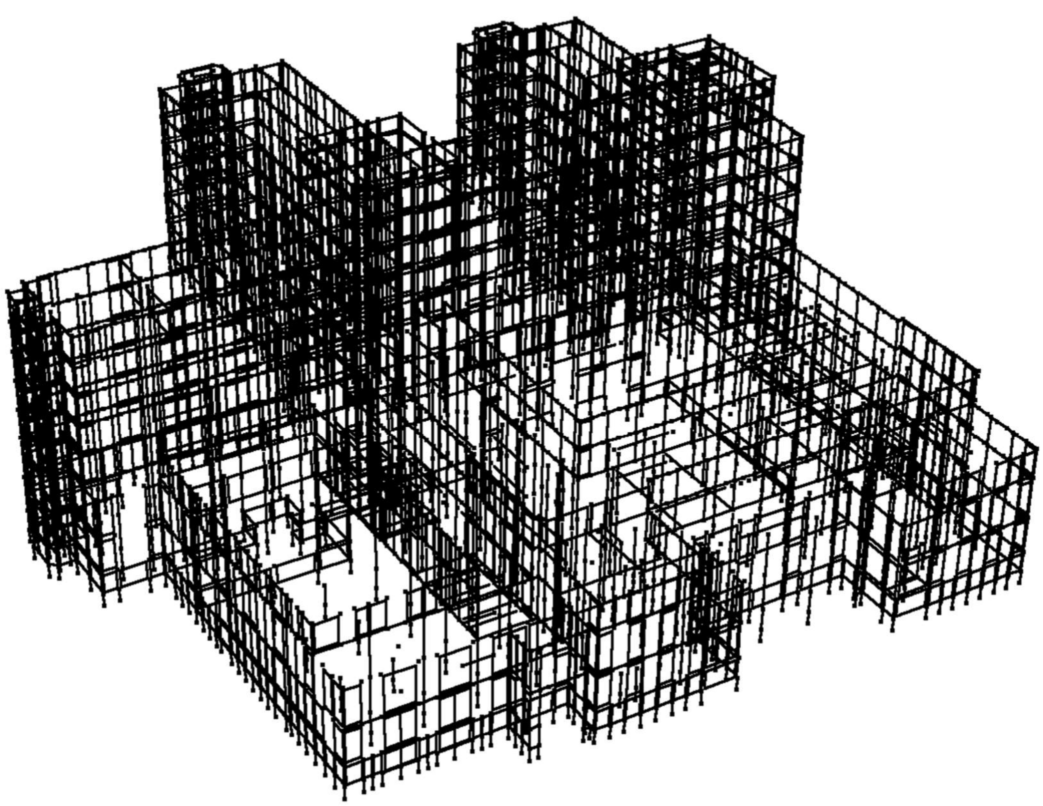
Further Study in Response to Comments

We make the following response to the comments above:

1. The typographical errors are corrected in the revised Draft report.
2. We have further investigated the shear strain results of wall elements and find that elements showing high shear strain are typically either part of below-grade retaining walls or are wall elements acting as coupling beams. Annotations to this effect have been added to the wall shear strain plots. This explains why certain elements may show high shear strain without the story mechanism drift profile.
3. With regard to the period of the structure, we have created a copy of the model and applied softening factors to the wall and soil elements. In the nonlinear analysis, the concrete and soil materials are very stiff in the elastic range in compression but have essentially zero stiffness in tension; this effect is taken into account in an elastic modal analysis by applying softening (cracking) factors. Factoring the concrete and soil stiffness by about ⅓, the period of the structure is 0.5 sec.
4. We have done a blind, double-check of the weight of one of the typical residential wings and confirmed that the mass of the model appears correct.
5. We have added interstory drift plots for the model Variation 2, BSE-2N separating drift by direction (Drift Results, pg. 6). Both the average response and peak response of the dominant records show larger drifts away from the hillside 1.4% vs. 1.0%.
6. We have plotted the target spectrum to the Spectra section of the report, page 6. We selected/scaled to the BSE-1N spectrum and run BSE-2N analyses by scaling the BSE-1N records uniformly by 1.5.

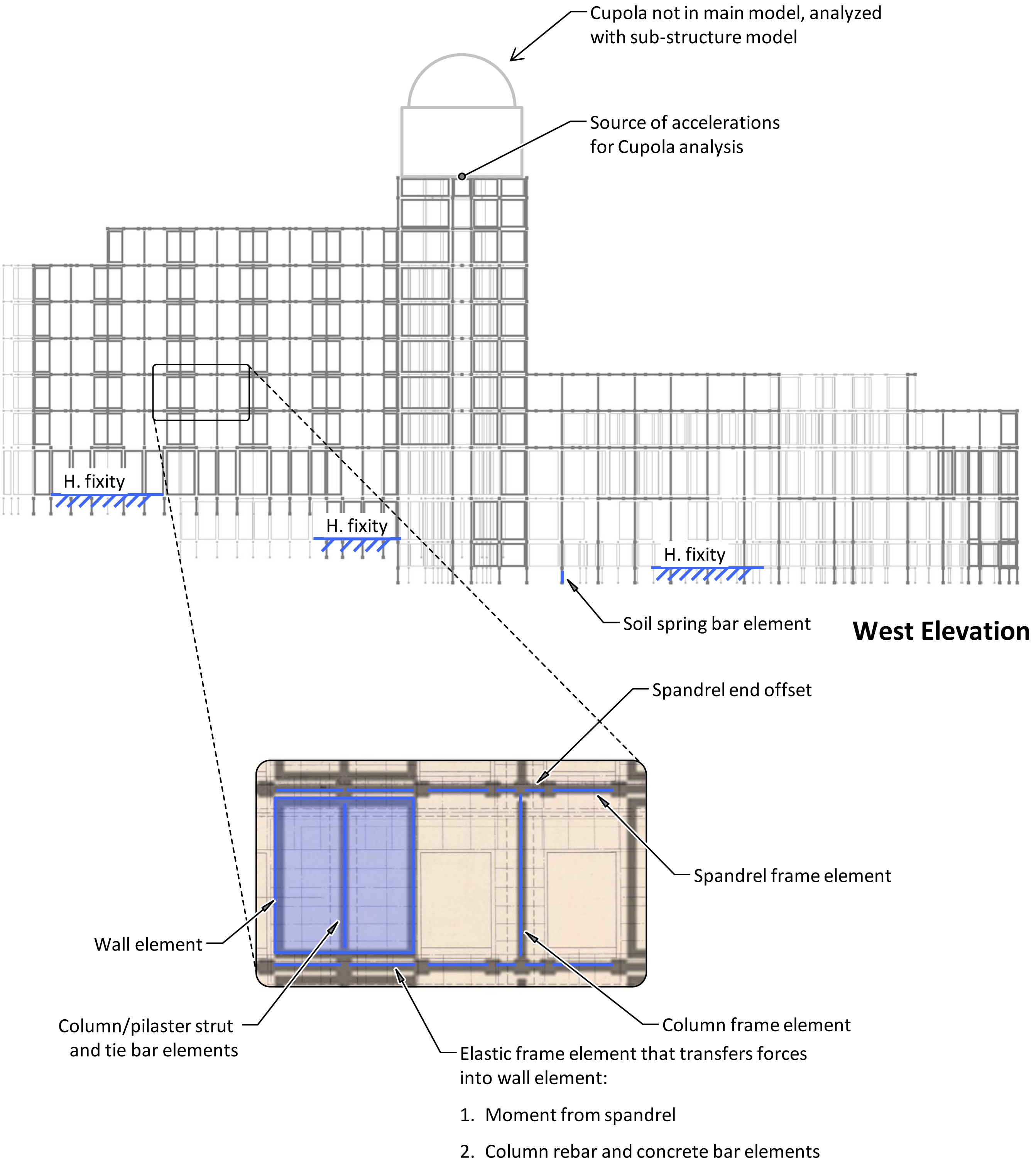
Building Model

1. Perform-3D (product of Computers and Structures, Inc.) analysis software was used to perform the UCB International house NRHA study.
2. For rebar material, expected rebar yield strength is estimated as $f_{ye} = 50\,ksi$, and for concrete material $f'_c = 3\,ksi$ is assumed.
3. Elements groups include:
    1. Four-node "shearwall" elements;
    2. Two-node "frame elements" for columns part of the exterior facade frame;
    3. Two-node "bar elements" for column reinforcement and concrete area integral with long wall piers;
    4. Two-node "frame elements" for spandrel beams part of the exterior facade frame;
    5. Two-node "frame elements" for elastic transfer of moments from spandrel and column elements to walls elements;
    6. Two-node "bar elements" for compression only soil springs;
    7. Two-node "frame elements" for critical intersections of the diaphragm;

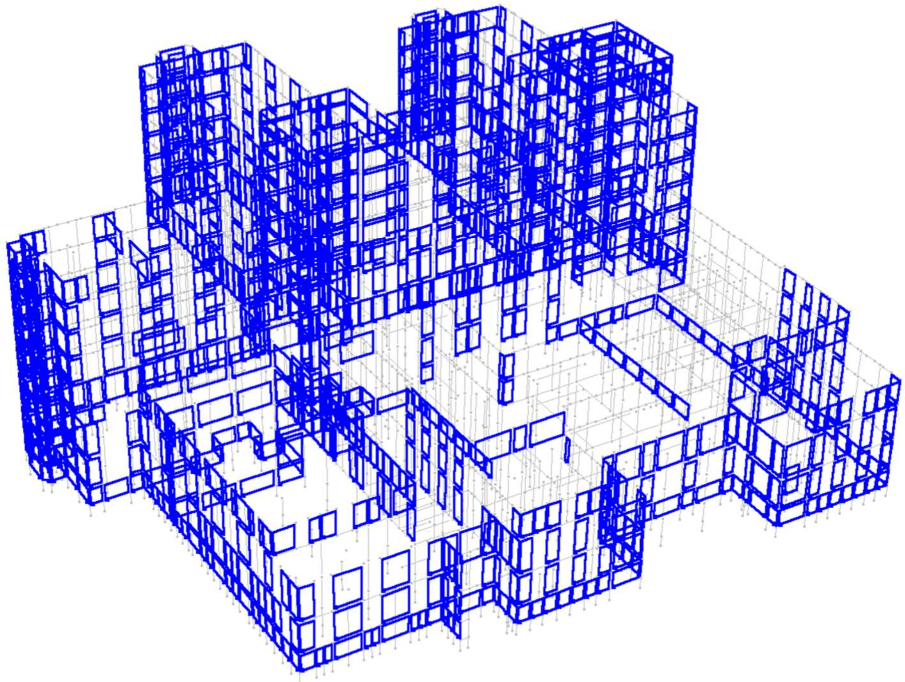


Screenshot of Perform-3D Model from North-West Perspective

Typical Meshing

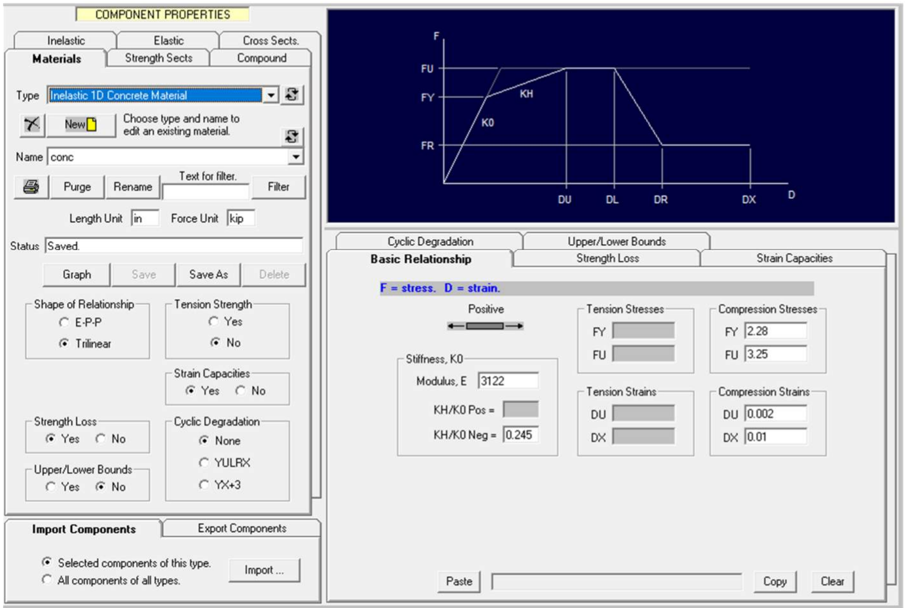


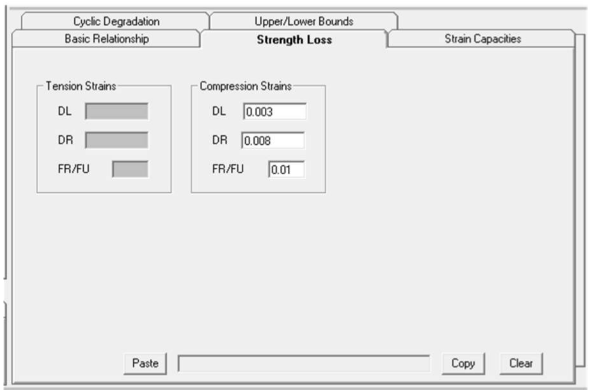
Wall Elements



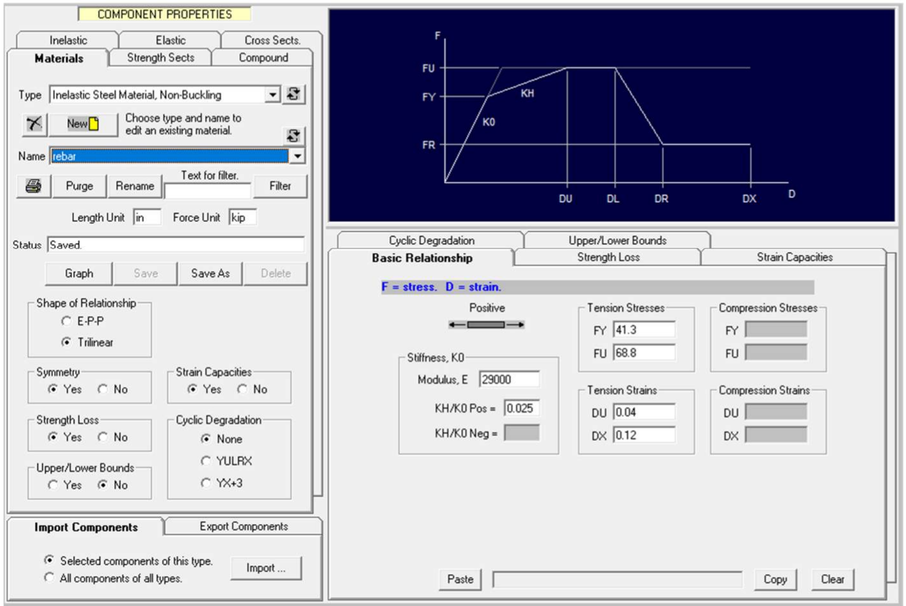
Four-node "shearwall" elements include inelastic shear material and fiber sections for bending and axial actions. Typically, all walls in the building are 8" with reinforcement as indicated on the first sheet of the original drawings: single curtain ½"ϕ @ 12"H and ½"ϕ @ 18"V. Retaining walls are 14" thick with the same reinforcement, except double curtain.

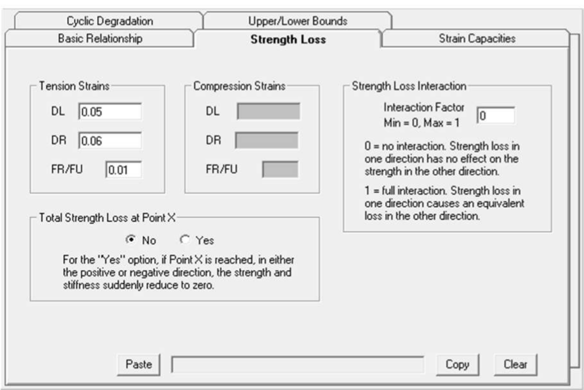
Concrete Material for Wall Elements



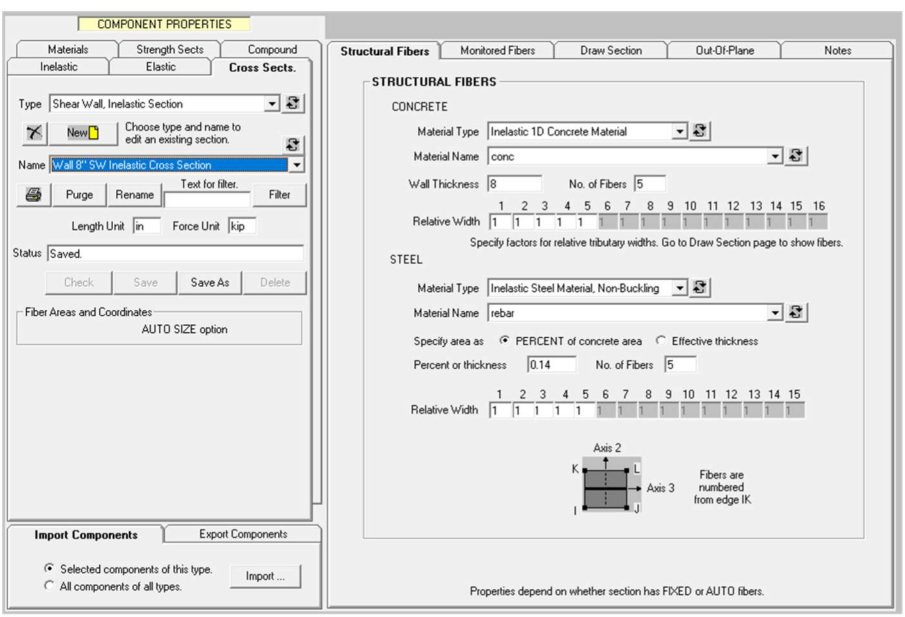


Rebar Material for Wall Elements

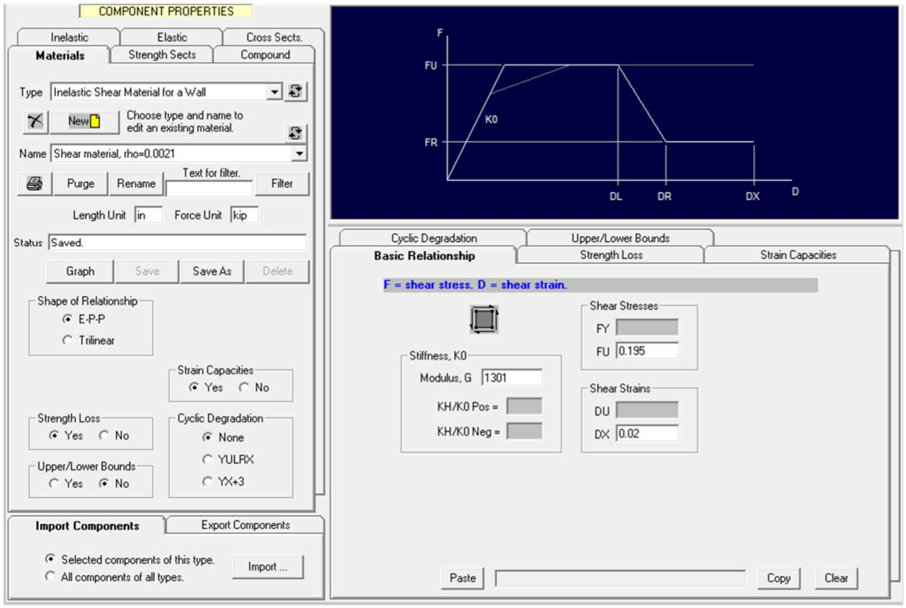


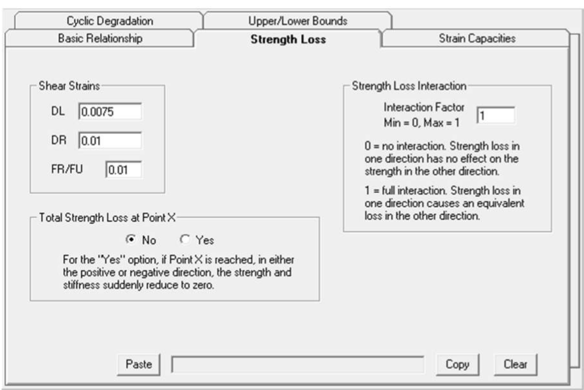


Inelastic Shearwall Cross Section

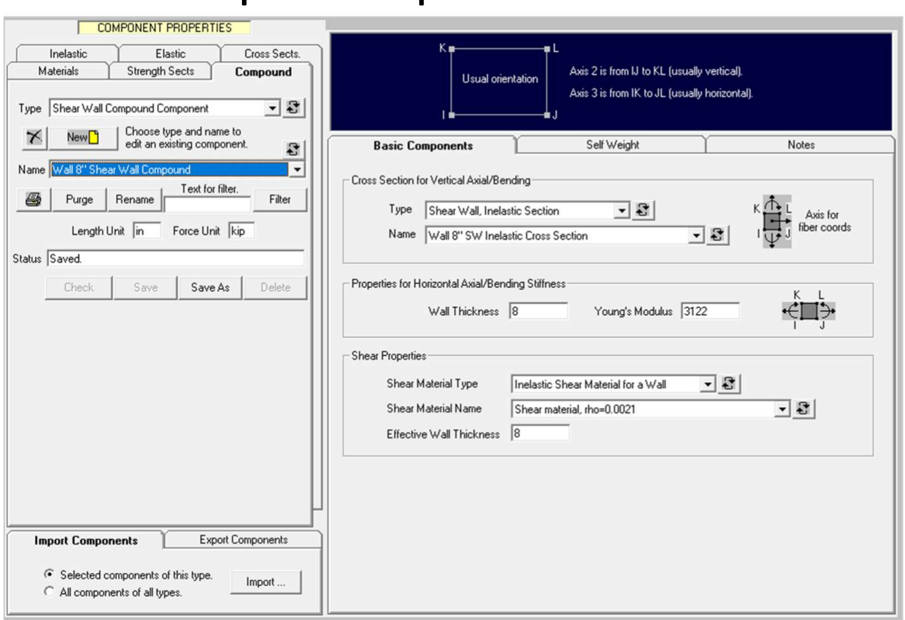


Shear Material for Wall Elements

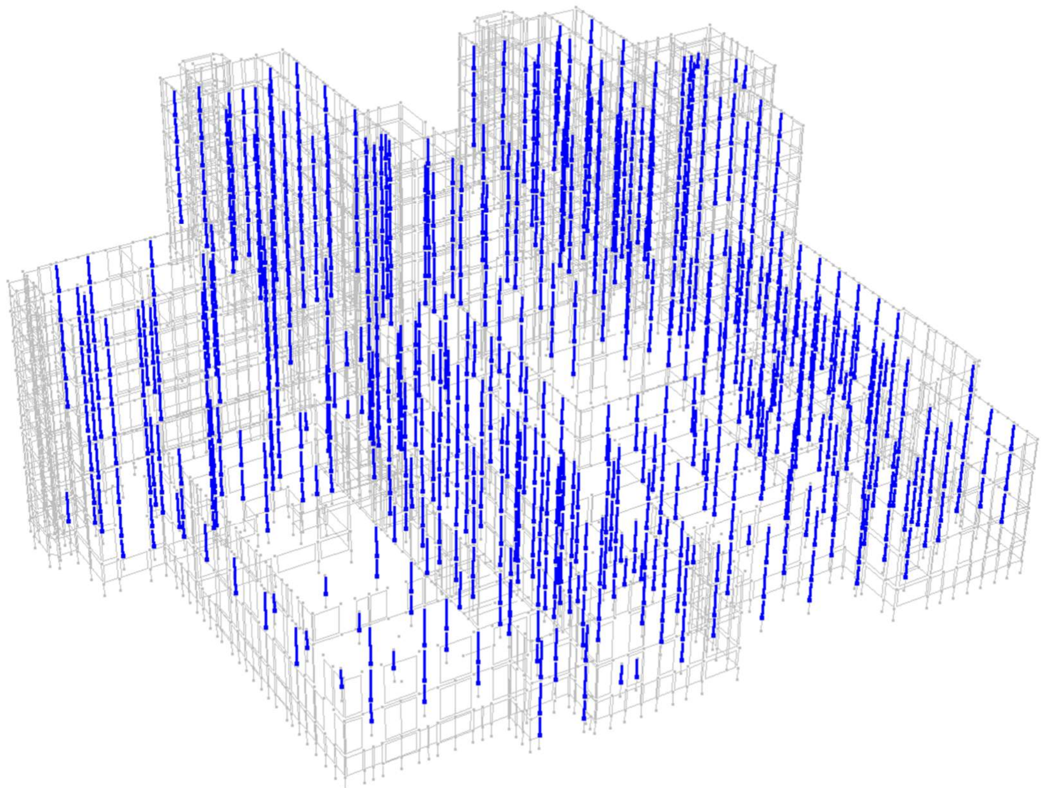




Shearwall Compound Component



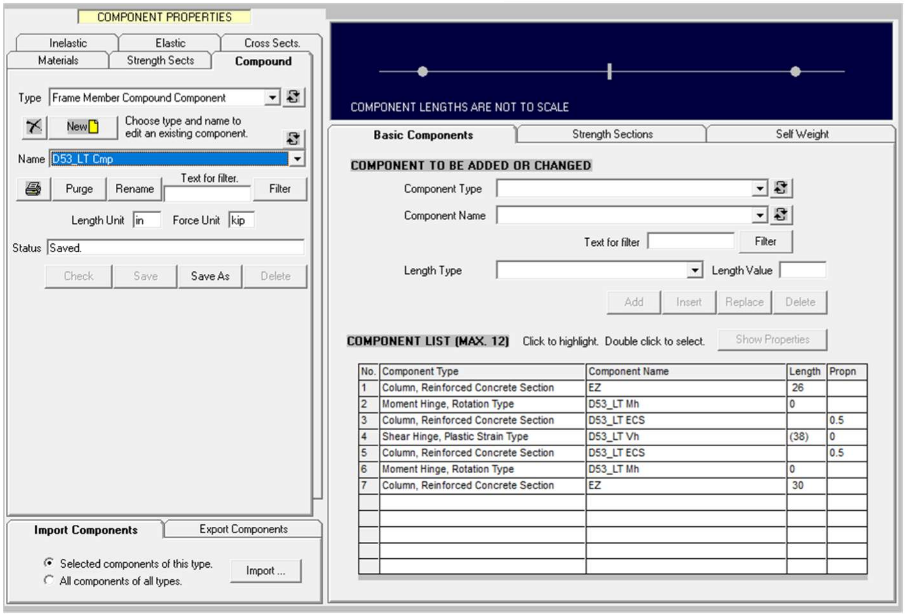
Column Frame Elements



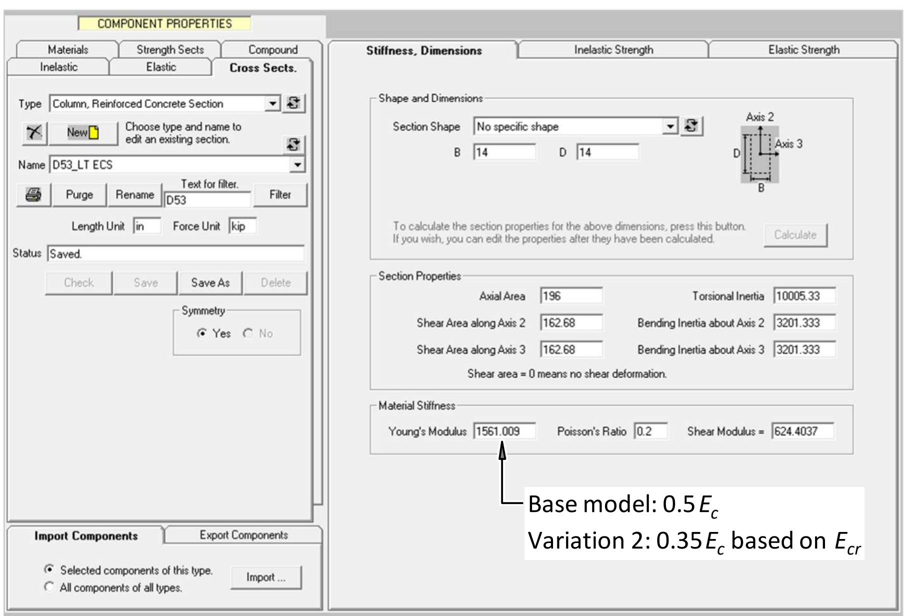
Two-node "frame elements" represent exterior columns, including short integral wall piers.

1. The column sub-components and compound components were created by importing definitions into Perform. We tabulated all the columns in the column schedule on the original drawings, recording dimensions and reinforcement. We divided the columns into groups as to whether they would be bar elements (i.e. they are integral with long wall piers) or frame elements (short wall piers). We imported components and assigned elements based on a process we've developed in-house to write data to the Perform input files.
2. Both the shear and moment hinges assume (for simplicity) no axial load. Higher moment and shear capacity due to axial load may cause the shearing mechanism to prefer the spandrels, so we reason that the chosen assumption is conservative with regard to column shear failure.
3. The sequence of sub-components is:
    1. Elastic end zone,
    2. Moment-rotation hinge: yield moment (M_y) assuming (for simplicity) full area of steel at center of column and zero axial load; ultimate moment of $1.25M_y$; rotation at strength loss, DL = 0.01 rad.
    3. Elastic zone, 0.5E
    4. Shear hinge, plastic strain type: elastic, perfectly plastic with strength, $FU = 2 \cdot \sqrt{f'_c} + \frac{A_{sv} \cdot d}{s}$; shear strain at strength loss, DL = 0.007.
    5. Elastic zone,
    6. Moment-rotation hinge, and
    7. Elastic end zone.

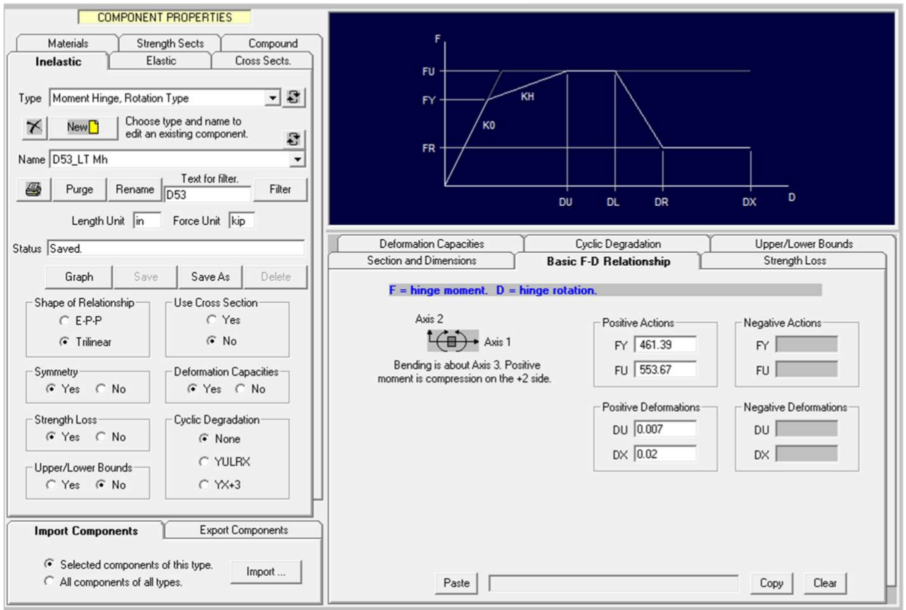
Column Compound Component

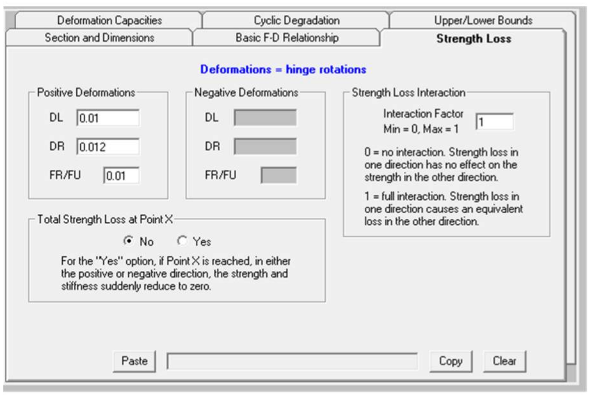


Column Elastic Section

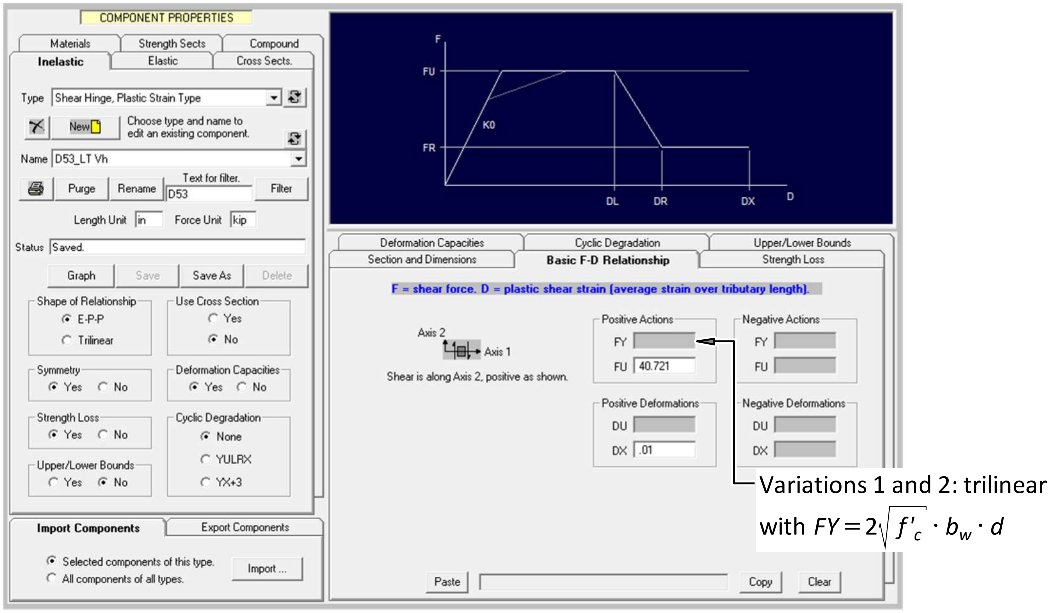


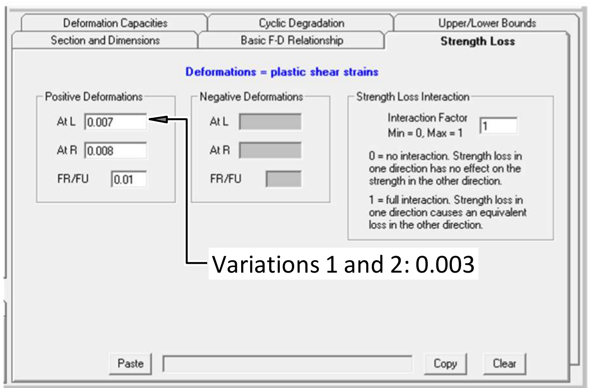
Column Moment-Rotation Hinge



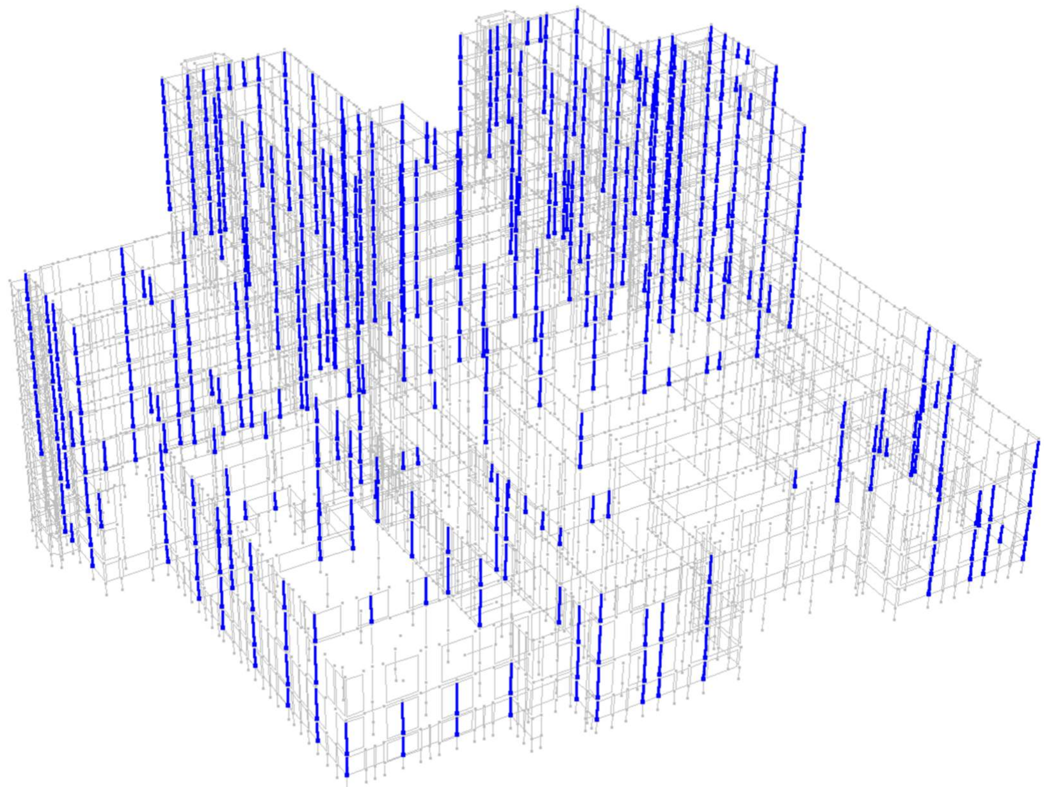


Column Shear Hinge



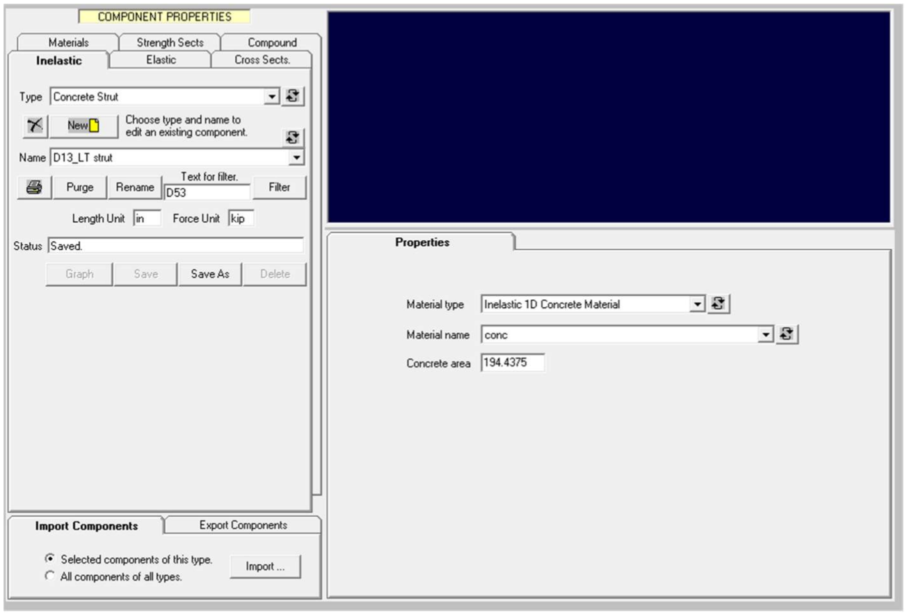


Column Strut and Tie Bar Elements

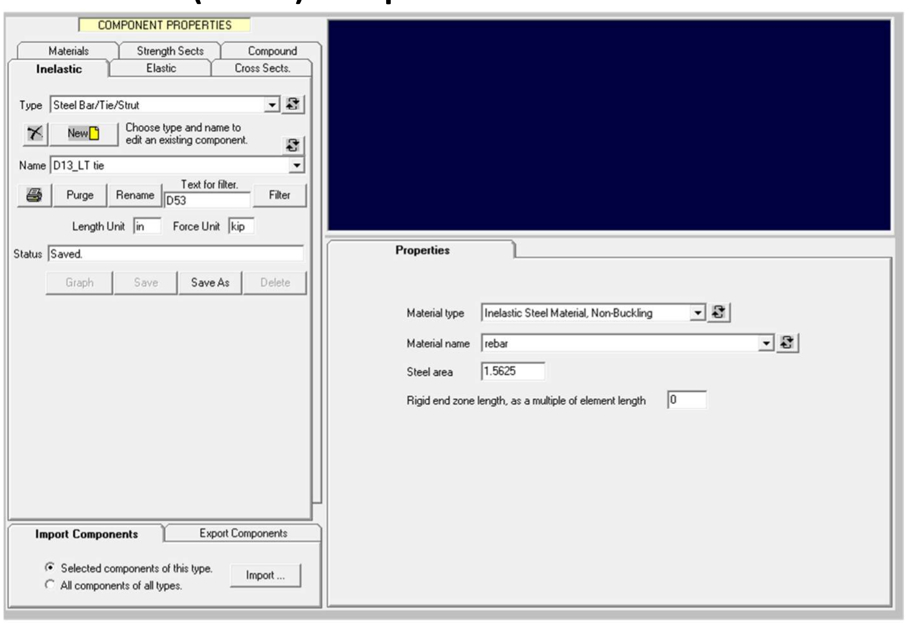


Two-node "bar elements" for column reinforcement and concrete area integral with long wall piers: elements are assigned appropriate area of concrete and rebar material, respectively.

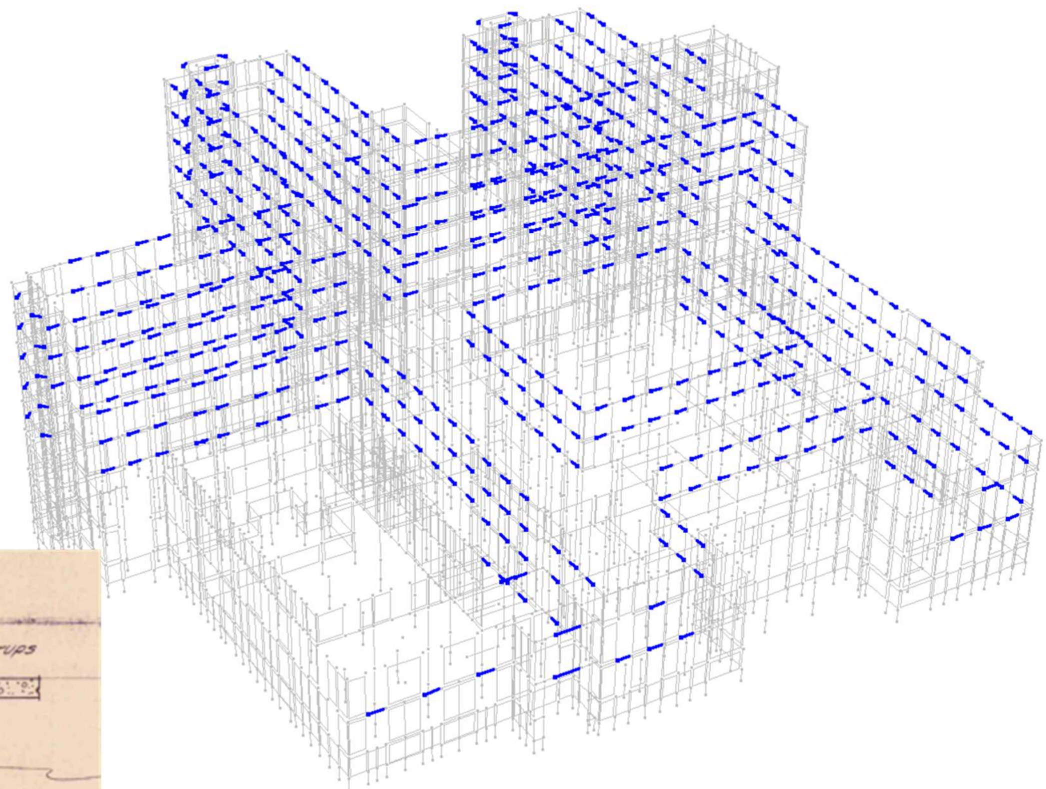
Column Strut (Concrete) Component

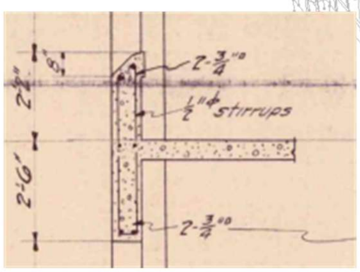


Column Tie (Rebar) Component



Spandrel Frame Elements

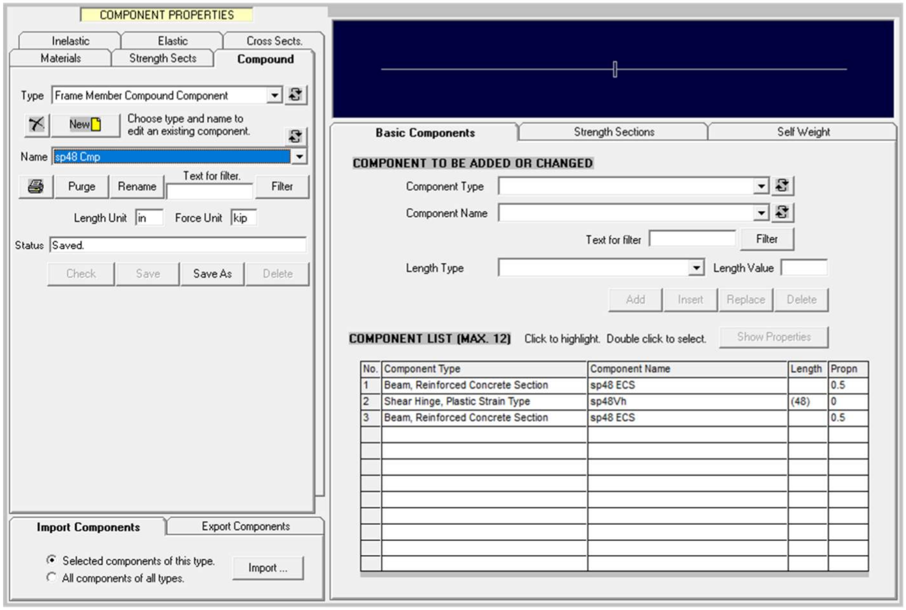




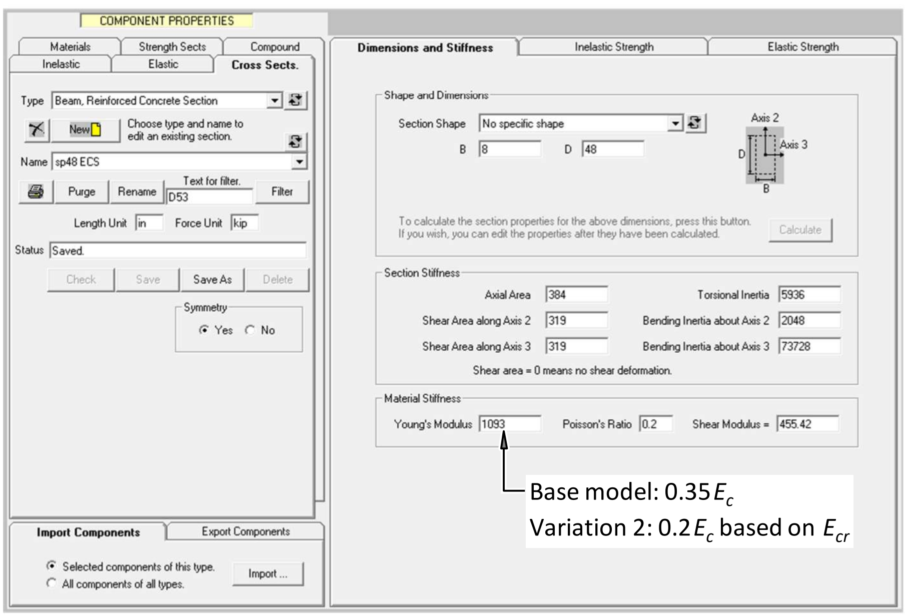
Two-node "frame elements" for spandrel beams part of the exterior facade frame

1. The typical spandrel beam is 8" thick and 4'-8" deep with (2) ¾" sq. bars top and bottom and ½" sq. stirrups spaced 18" apart. Though there are some variations throughout the building, this typical configuration is used for all spandrel beam elements.
2. End-zones were modeled explicitly as separate elements (rather than as sub-components) because they vary in length.
3. The sequence of sub-components is:
    1. Elastic zone, 0.5EI
    2. Shear hinge, plastic strain type: elastic, perfectly plastic with strength, $FU = 2 \cdot \sqrt{f'_c} + \frac{A_{sv} \cdot d}{s}$; shear strain at strength loss, DL = 0.007, and
    3. Elastic zone.

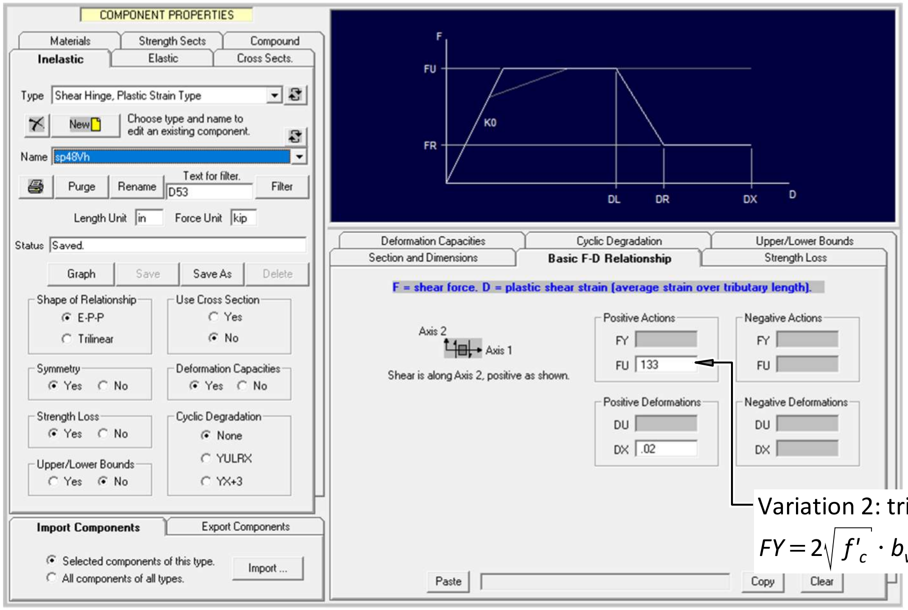
Spandrel Compound Component



Spandrel Elastic Section

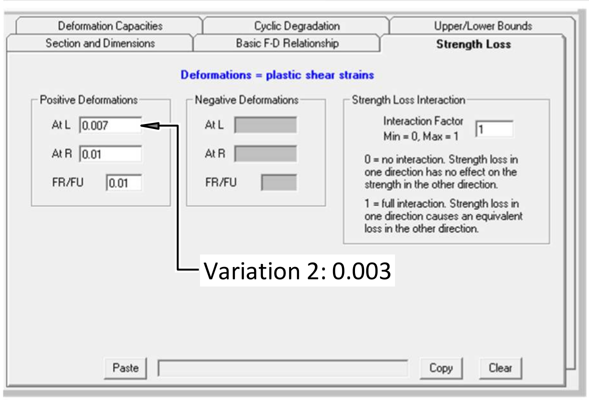


Spandrel Shear Hinge



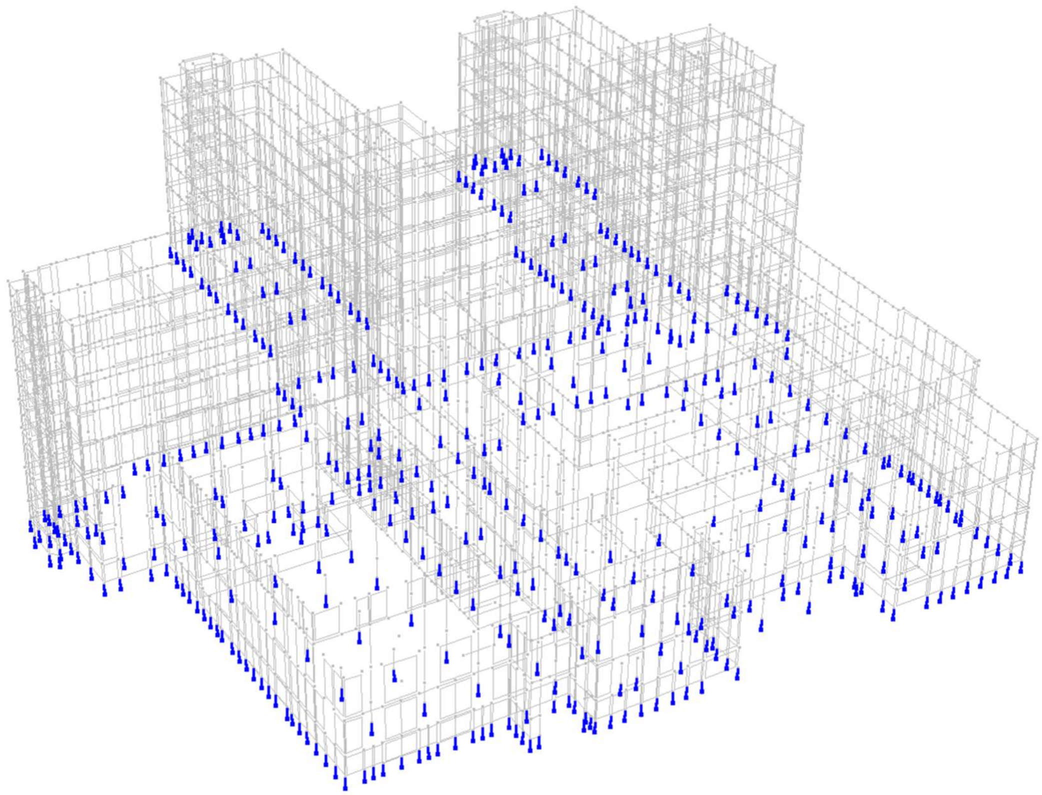
Variation 2: trilinear with

$FY = 2\sqrt{f'_c} \cdot b_w \cdot d$



Variation 2: 0.003

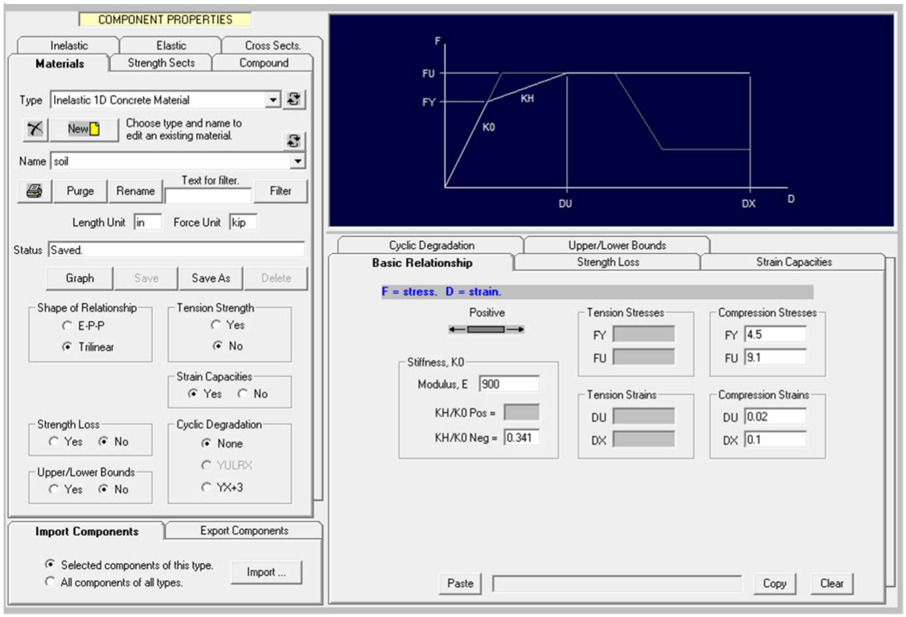
Compression Only Soil Elements



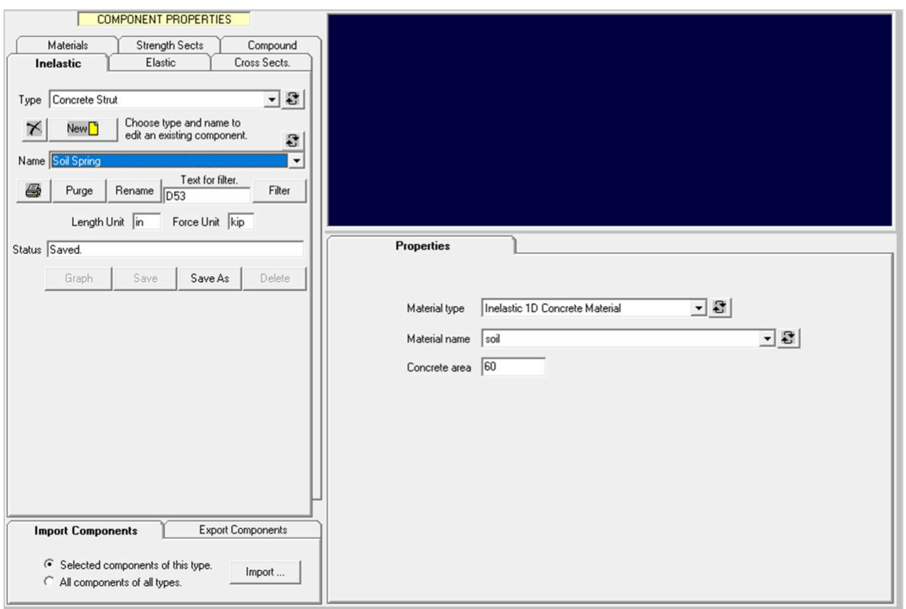
Two-node "bar elements" represent compression only soil springs.

1. A "concrete" material is defined based on an estimate of soil strength derived from various footings loads.
2. The spring is defined to soften at ¼" of displacement.
3. An area of 60 ft^2 is assigned to the typical element, and they are dispersed throughout the model.
4. The goal is to have reasonable foundation flexibility and model compression-only response.

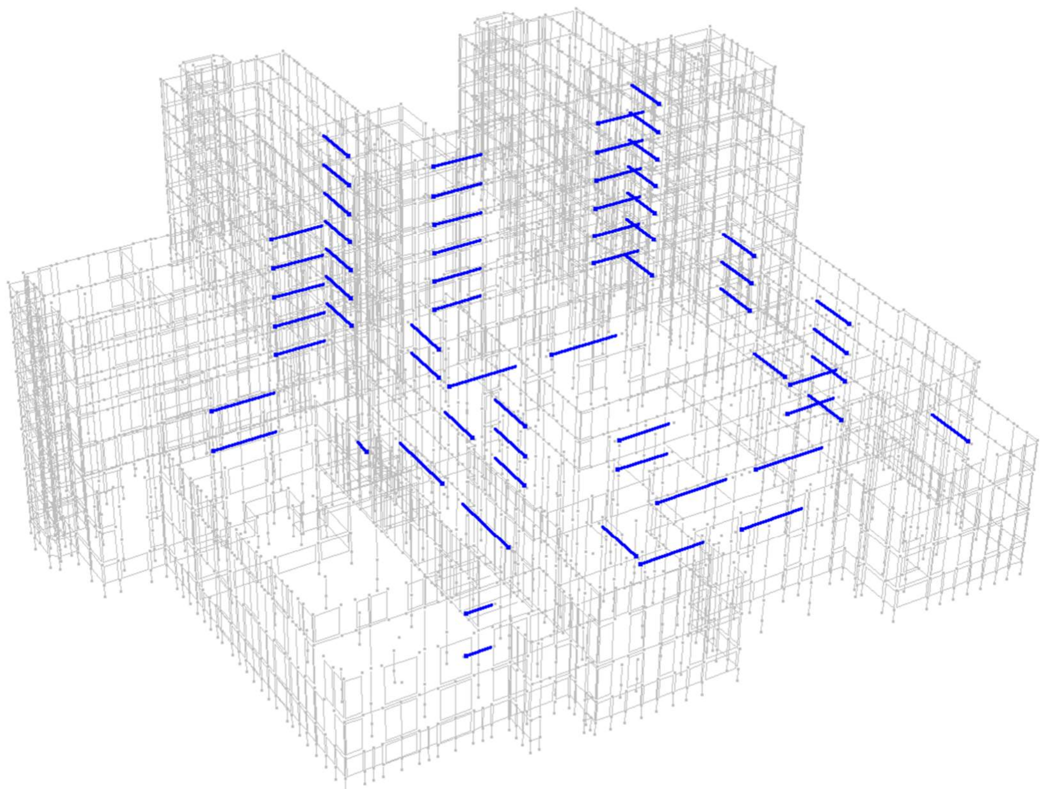
"Concrete" (Compression Only)Material for Soil Elements



Soil Strut Component



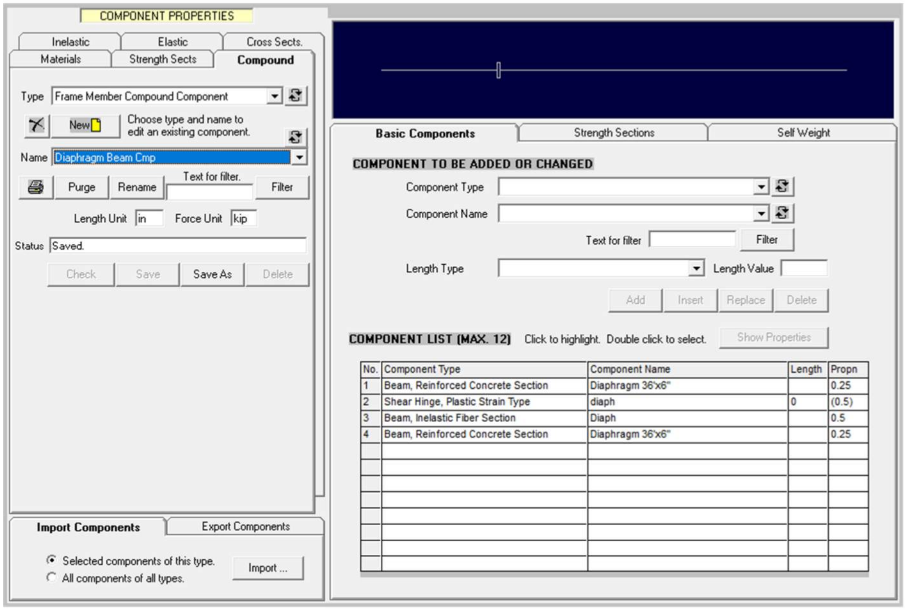
Diaphragm Frame Elements



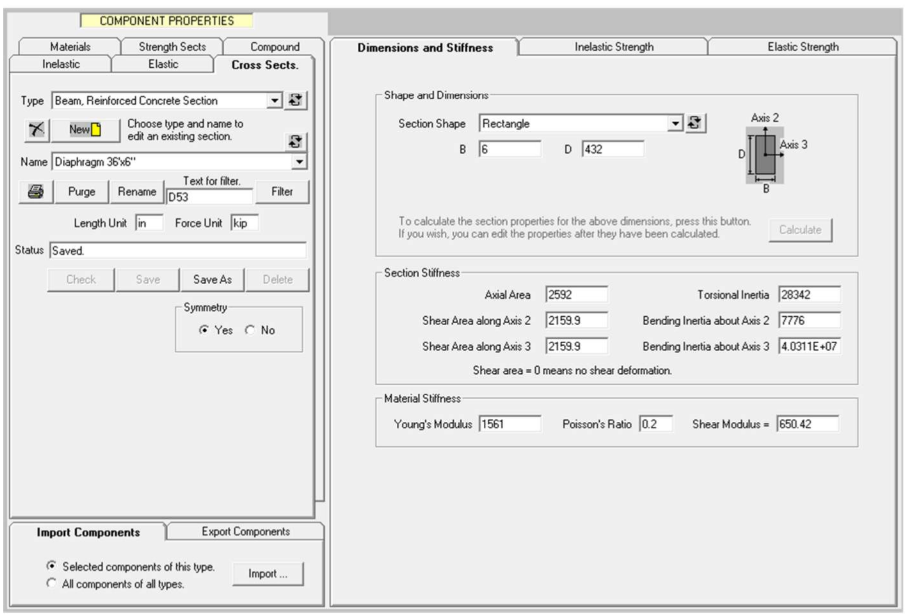
Two-node "bar elements" represent critical intersections of the diaphragm.

1. The building has many re-entrant corners as the diaphragms emanate in various wings.
2. The base of the building is terraced such that the east side is founded two stories above the west. Therefore, for westward movement the building may tend to "hang up" on the east part of the building fixed at an elevation two stories higher than the west. The diaphragm link elements allow for flexibility between these parts of the structure.
3. The typical diaphragm link represents a 36' wide x 6" thick of typical concrete floor slab with beams along the corridor and walls at the edges.
4. The sequence of sub-components is:
    1. Elastic zone, 0.5EI
    2. Shear hinge, plastic strain type: elastic, perfectly plastic with strength, $FU = 2 \cdot \sqrt{f'_c} + \frac{A_{sv} \cdot d}{s}$; shear strain at strength loss, DL = 0.007.
    3. "Beam" Fiber Section hinge with fibers representing concrete and rebar areas of beams and slab, and
    4. Elastic zone.

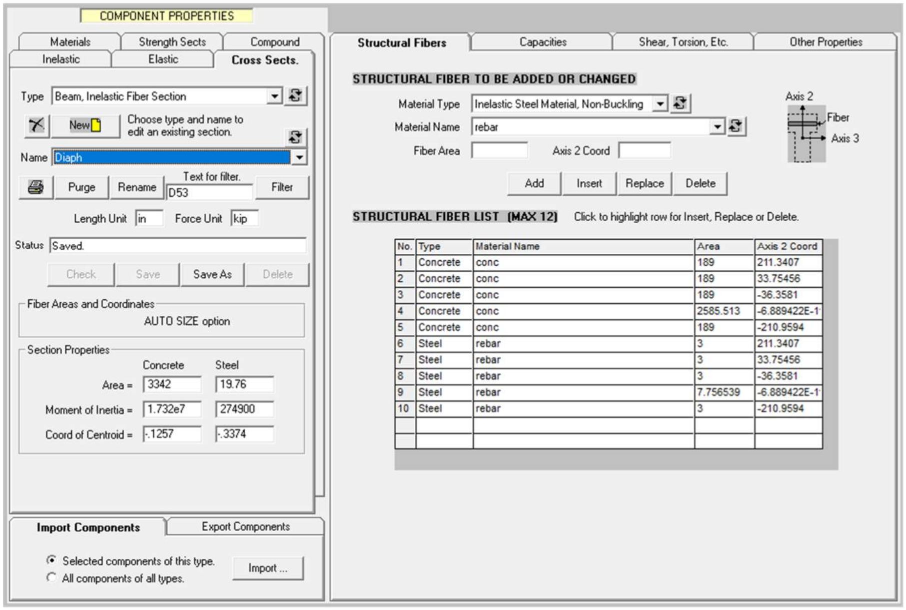
Diaphragm Link Compound Component



Diaphragm Link Elastic Section

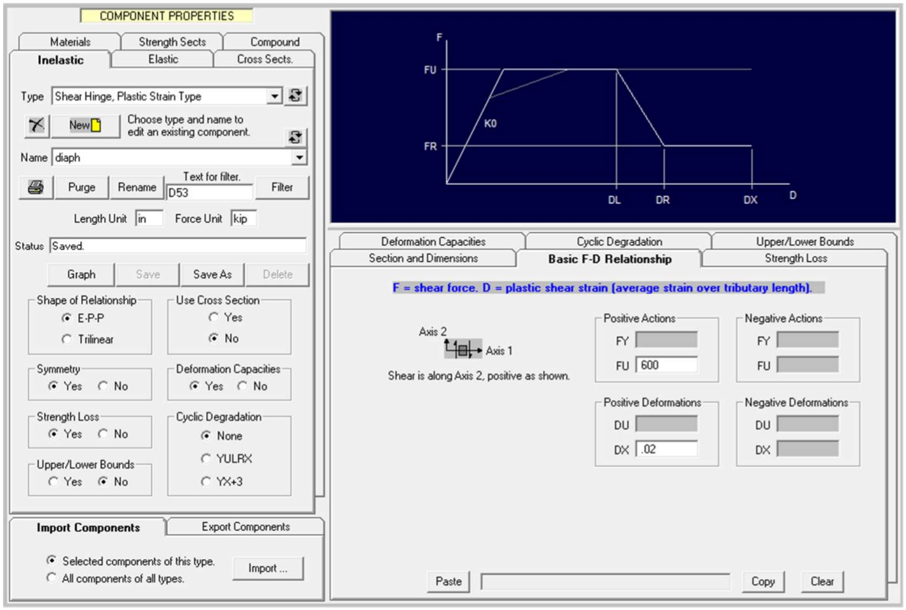


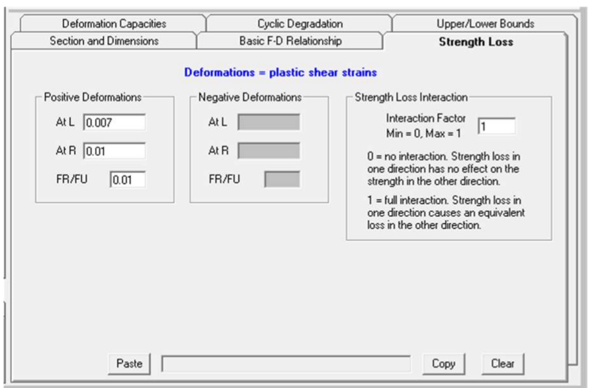
Diaphragm Link Fiber Section Hinge



No.	Type	Material Name	Area	Axis 2 Coord
1	Concrete	conc	189	211.3407
2	Concrete	conc	189	33.75456
3	Concrete	conc	189	-36.3581
4	Concrete	conc	2585.513	-6.889422E-1
5	Concrete	conc	189	-210.9594
6	Steel	rebar	3	211.3407
7	Steel	rebar	3	33.75456
8	Steel	rebar	3	-36.3581
9	Steel	rebar	7.756539	-6.889422E-1
10	Steel	rebar	3	-210.9594

Diaphragm Link Shear Hinge





Ground Motion Selection Process

1. The 2015 URS report provides site-specific spectra for the various site conditions found on campus and for various hazard levels.
2. UCB contracted with Lettis Consultants, Inc. (LCI) to perform peer review services and provide recommendations with regard to the ground motion selection and scaling.
3. The Hearst @ La Loma site is peculiar in that it is adjacent to the Hayward Fault and that the rock strata is sloping underneath the site. Thus, it is unclear how to categorize the site; it has attributes of *Site Class C* but also the *Rock, Shear Zone* condition listed in the URS report.
4. LCI drafted a memo dated May 25, 2018 indicating their recommendations, summarized here:
    1. Target spectrum should envelope 80% of ASCE 7-10 Site Class C, site-specific spectra from URS report for 36 to 75 ft. of soil and for Rock, Shear Zone
    2. (11) records are to be selected
    3. (7) should have velocity pulse content
    4. Appropriate 5-95% duration range is 7.7 to 18.5 sec (this requirement was later relaxed in preference for records out of range but with higher magnitudes).
    5. Rotated fault-normal/fault-parallel
    6. Scaled such that the maximum-direction spectrum of each record set generally matches the target spectrum and the average of the maximum-direction spectra meets or exceeds the target spectrum for the period range of interest. It was allowed to have the mean spectrum fall below the target in a period range small enough to be considered "insignificant"
    7. Scaled records consistent with Design response spectrum may be scaled by a factor of 1.5 for the MCE_R level.
5. We applied a reduction to the target spectrum for base-slab averaging based on the recommendations of ASCE 41-13 and using an area of 14,494 ft^2, which is about half of the residential/garage footprint.
6. We selected a large suite of ground motions from the PEER Strong Motion database (over 200), and sifted them based on the criteria listed above and with regard to best fit to the shape of the target spectrum.
7. Through interactions with LCI, additional selection criteria were added, including magnitude (should be greater than 6.3) and Arias Intensity (of scaled motion, around 8.5 m/sec.). LCI also suggested certain specific ground motions for consideration.
8. The final selection set is presented on the following pages and meets with LCI's approval.

$S_S = 2.472$

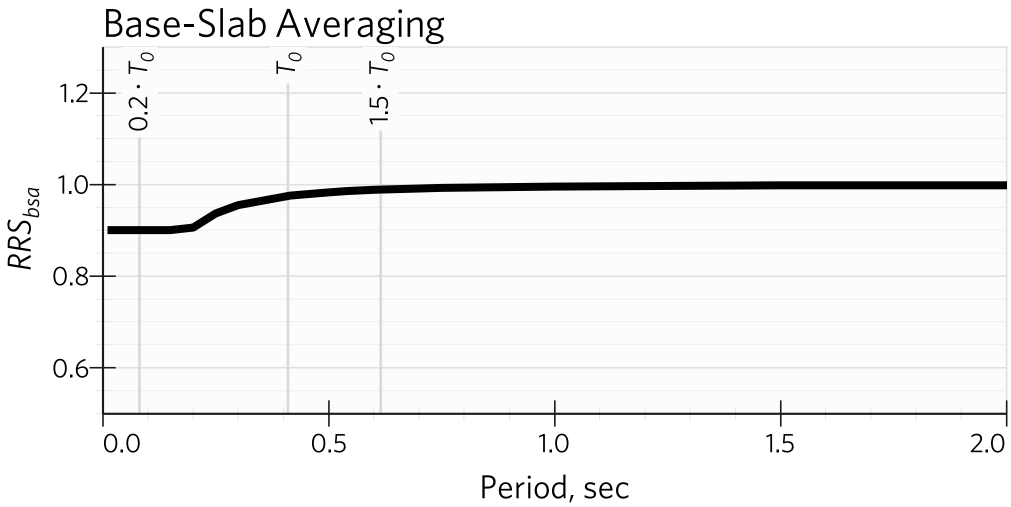
$S_1 = 1.027$

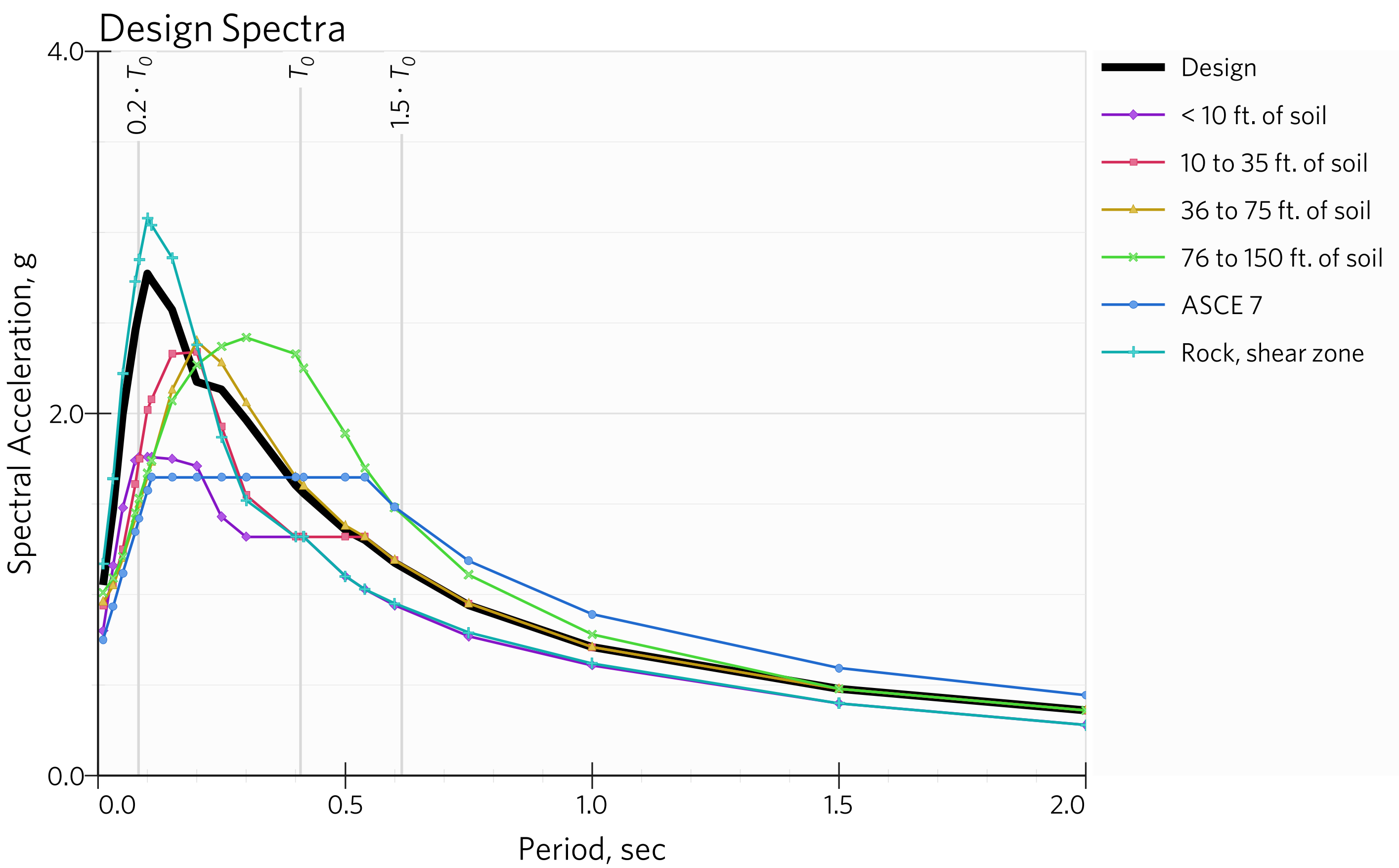
$SiteClass = C$

$F_a = getFa\left(SiteClass\ ,\ S_S\right) \rightarrow 1$

$F_v = getFv\left(SiteClass\ ,\ S_1\right) \rightarrow 1.3$

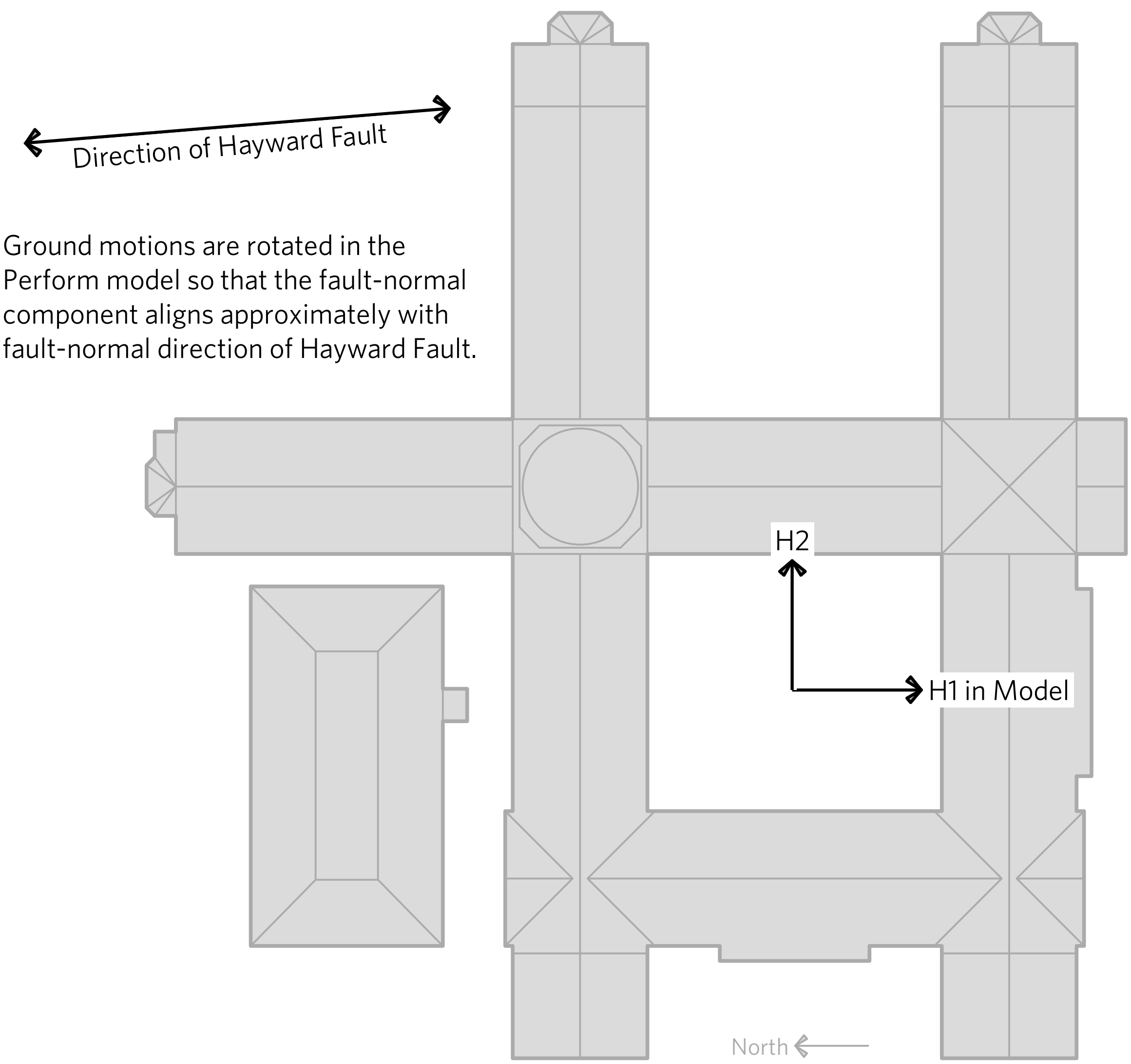
$T_0 = 0.41\ sec$

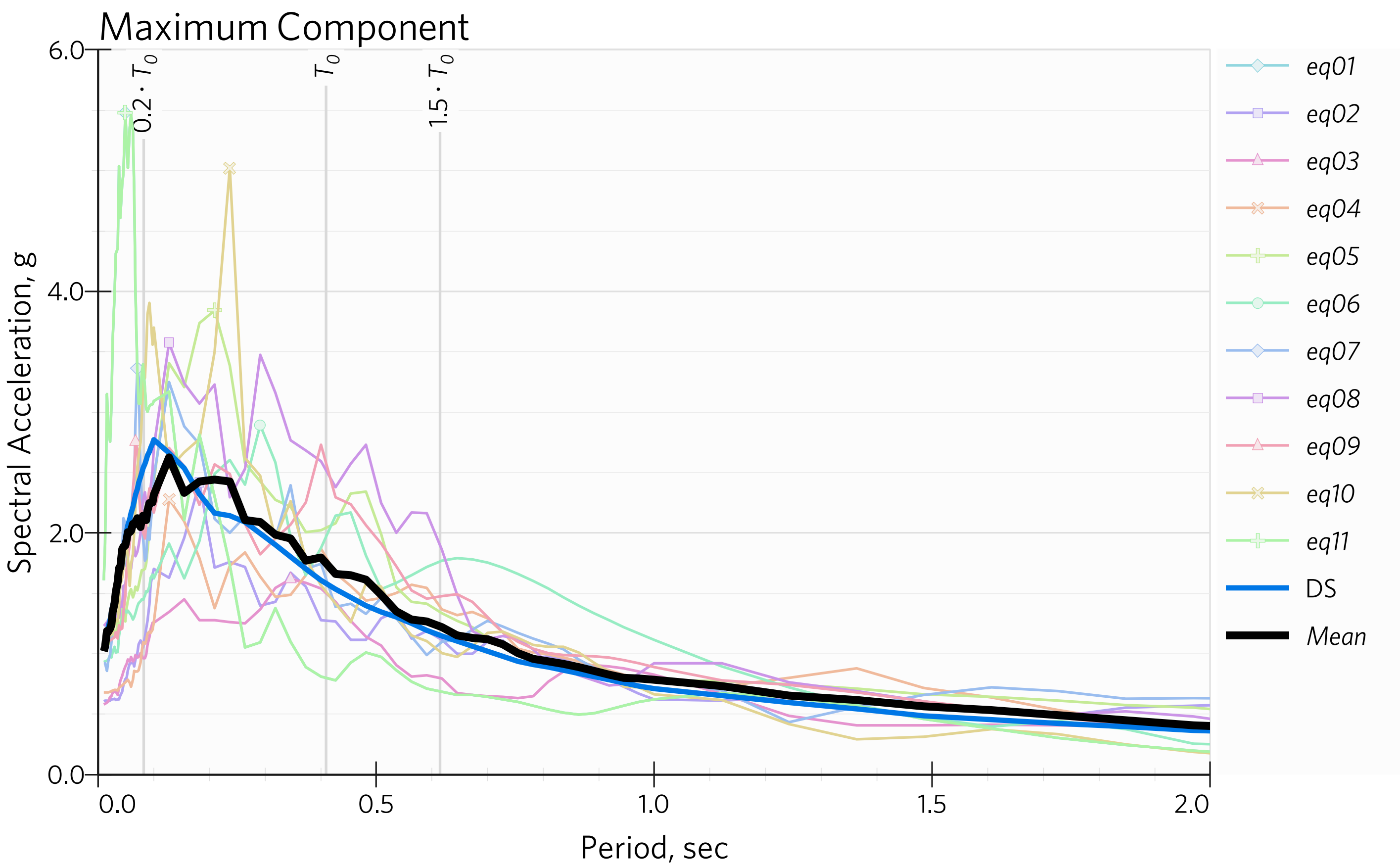


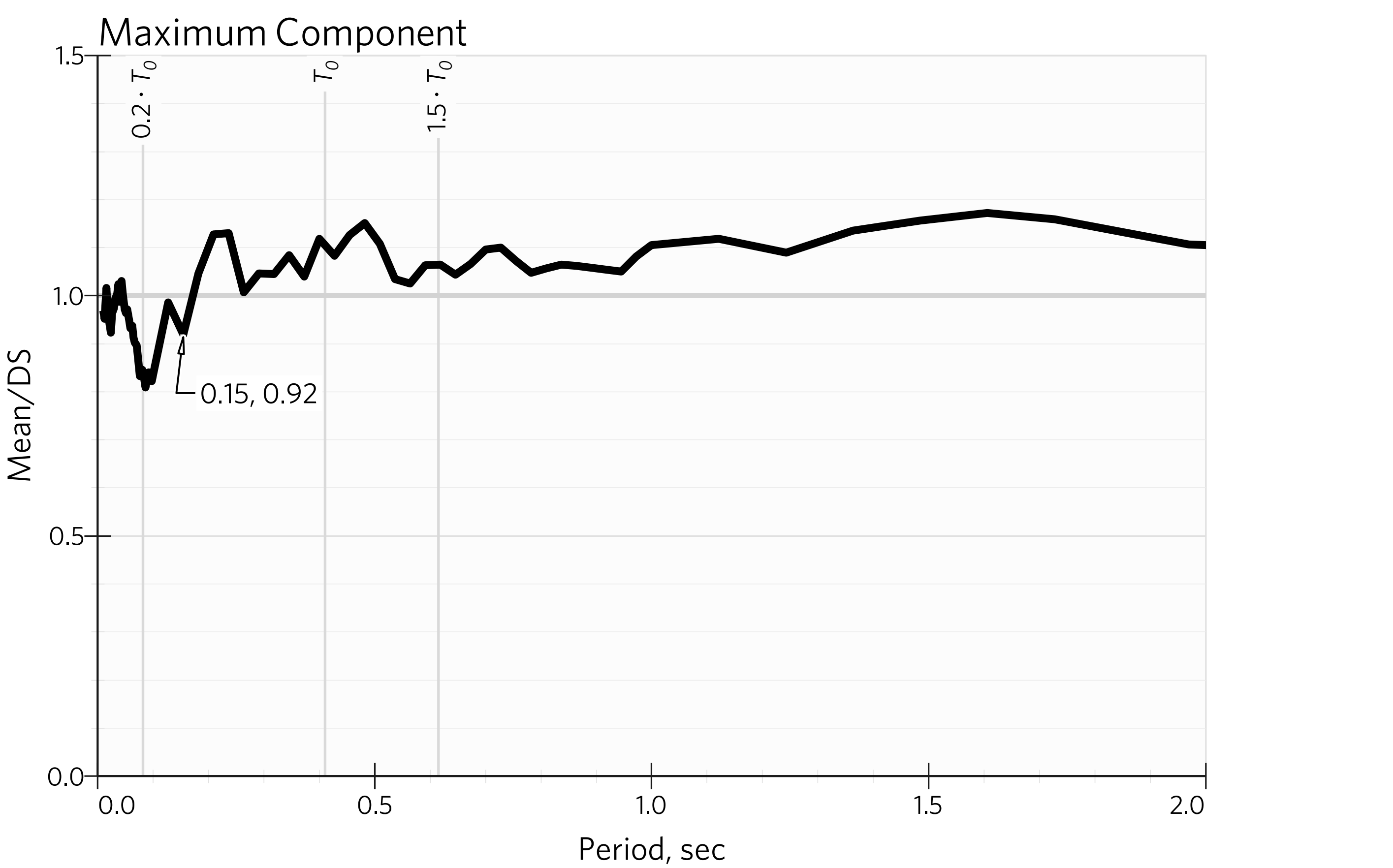


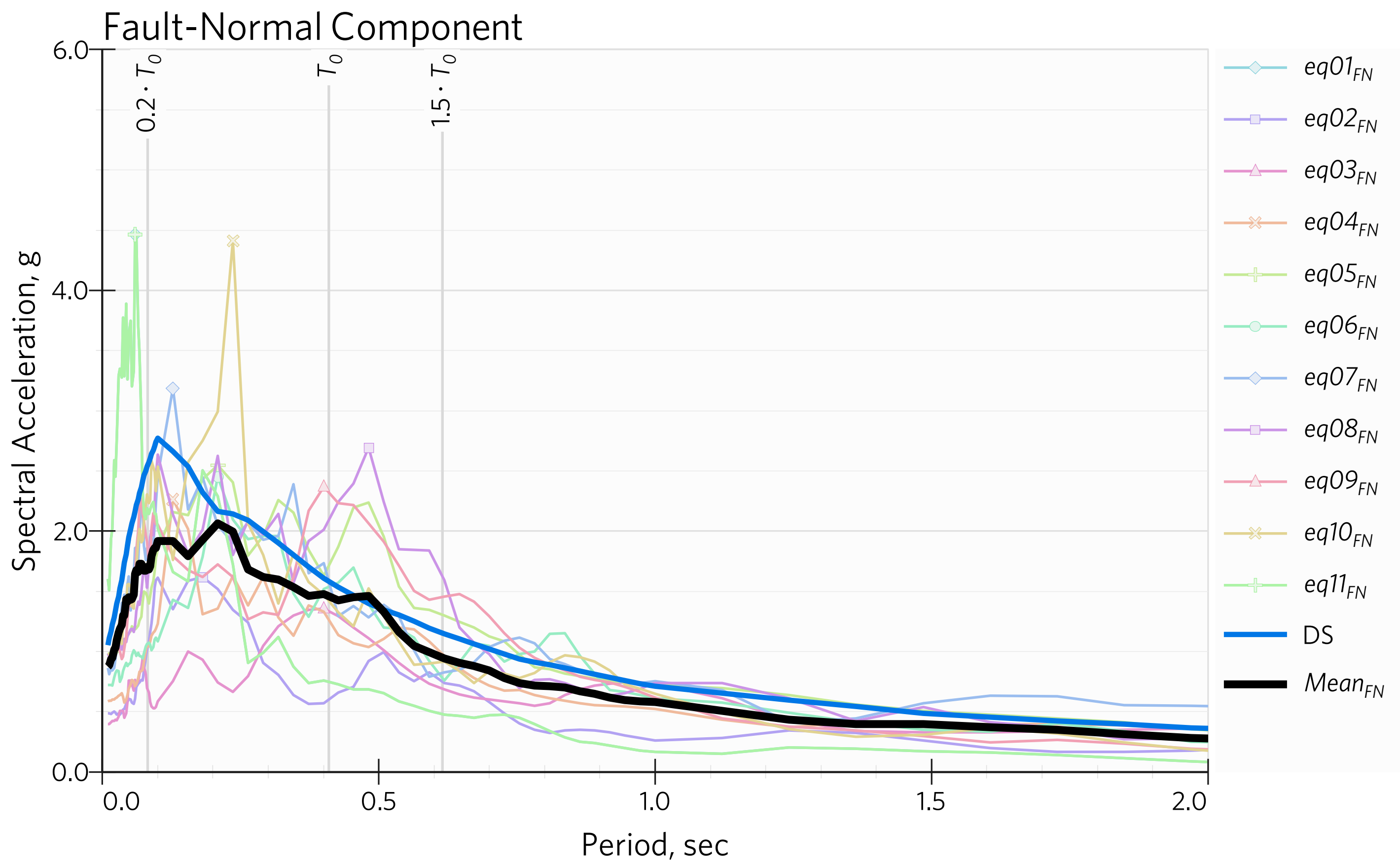
tblSelection

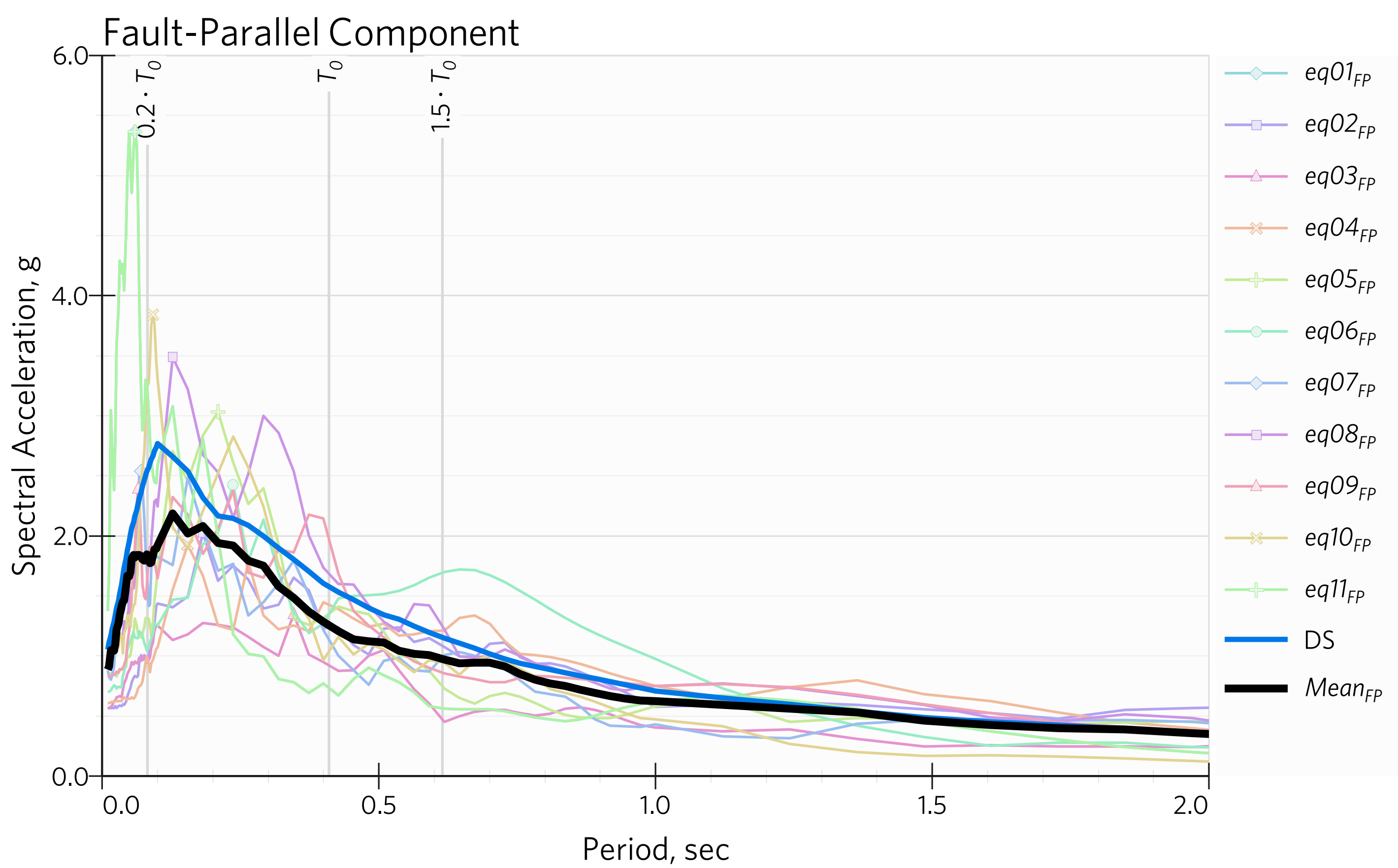
eq	*rsn*	*SF*	AI_{scaled}	URS_{ref}	T_{pulse}	$D_{5,95}$	*Magn*	*EarthquakeName*	*Year*	*StationName*	*Rrup*	*Vs30*
1	143	1.5	12.1	x	6.19	16.5	7.35	Tabas_ Iran	1,978	Tabas	2.05	767
2	178	2.07	7.92	x	4.5	14.1	6.53	Imperial Valley-06	1,979	El Centro Array #3	12.9	163
3	180	1.04	3.66	x	4.13	9.6	6.53	Imperial Valley-06	1,979	El Centro Array #5	3.95	206
4	184	1.38	7.24		6.27	7	6.53	Imperial Valley-06	1,979	El Centro Differential Array	5.09	202
5	767	1.84	11.5		2.64	11.4	6.93	Loma Prieta	1,989	Gilroy Array #3	12.8	350
6	1,004	0.98	11.2		0.93	8.5	6.69	Northridge-01	1,994	LA - Sepulveda VA Hospital	8.44	380
7	1,633	1.5	21.7			29.1	7.37	Manjil_ Iran	1,990	Abbar	12.6	724
8	4,031	2.25	17.6			10.3	6.52	San Simeon_ CA	2,003	Templeton - 1-story Hospital	6.22	411
9	4,228	1.77	11.5	x	1.8	12.2	6.63	Niigata_ Japan	2,004	NIGH11	8.93	375
10	4,229	2.5	21.4			10.1	6.63	Niigata_ Japan	2,004	NIGH12	10.7	564
11	5,656	2.5	18.8	x		26.8	6.9	Iwate_ Japan	2,008	IWTH24	5.18	486

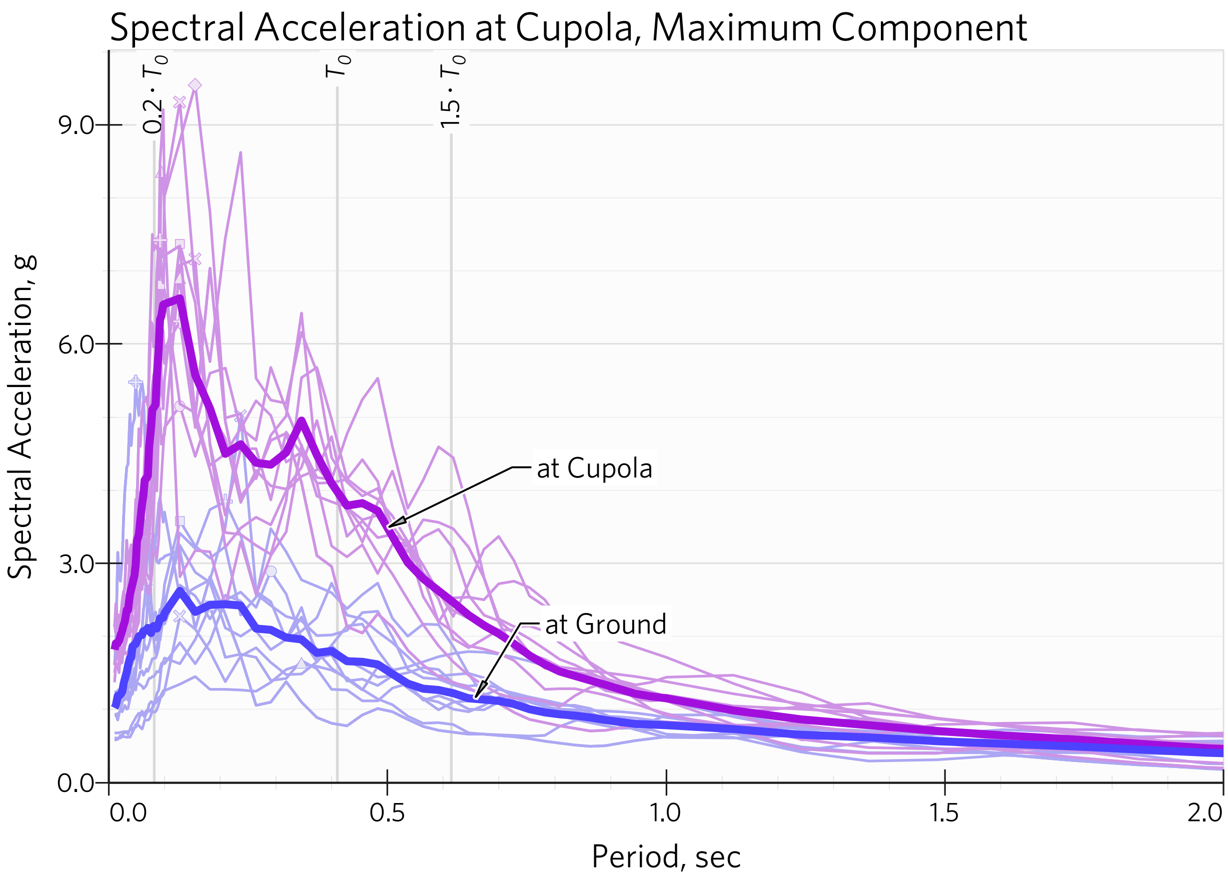


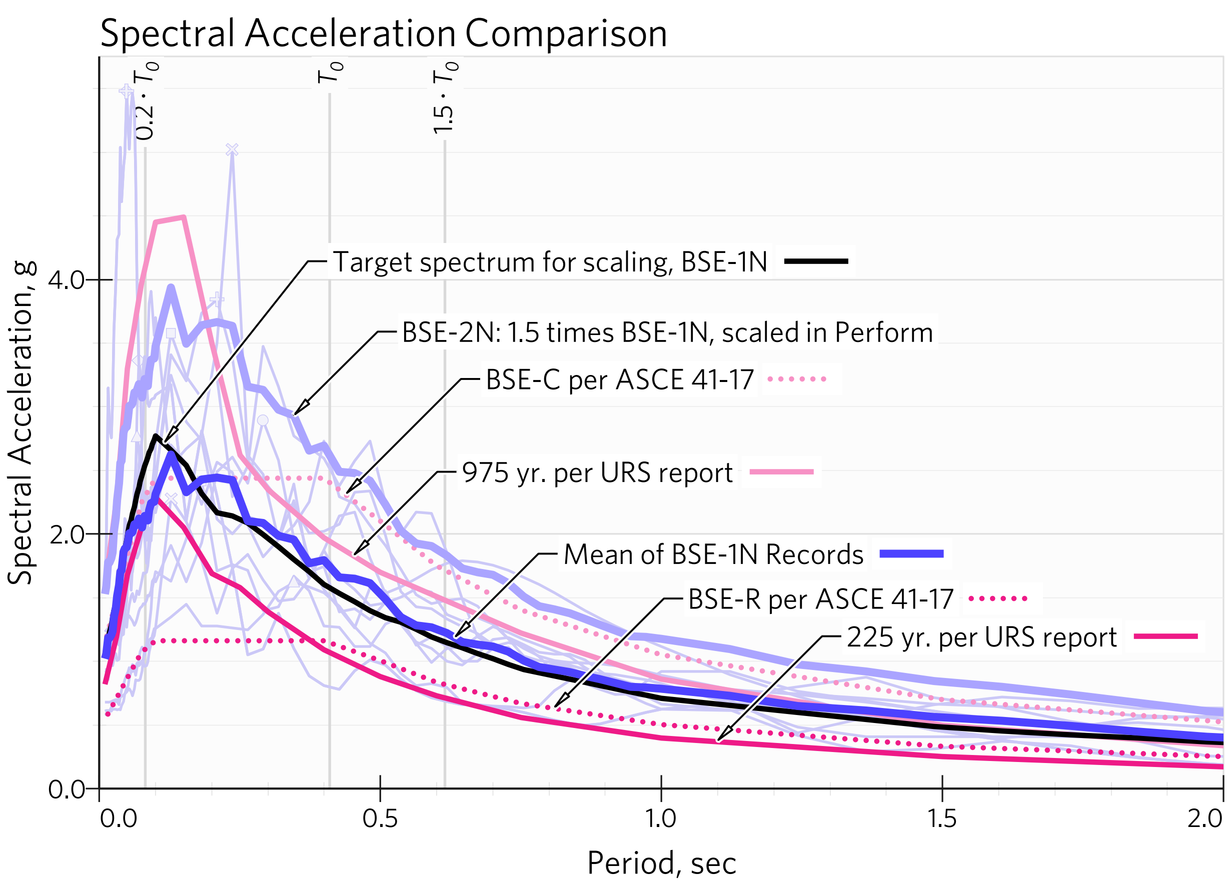

Maximum Component
Spectral Acceleration, g
6.0
4.0
2.0
0.0
0.2 · T_0
T_0
1.5 · T_0
0.0
0.5
1.0
1.5
2.0
Period, sec
eq01
eq02
eq03
eq04
eq05
eq06
eq07
eq08
eq09
eq10
eq11
DS
Mean


Maximum Component
Mean/DS
1.5
1.0
0.5
0.0
0.2 · T_0
T_0
1.5 · T_0
0.15, 0.92
0.0
0.5
1.0
1.5
2.0
Period, sec

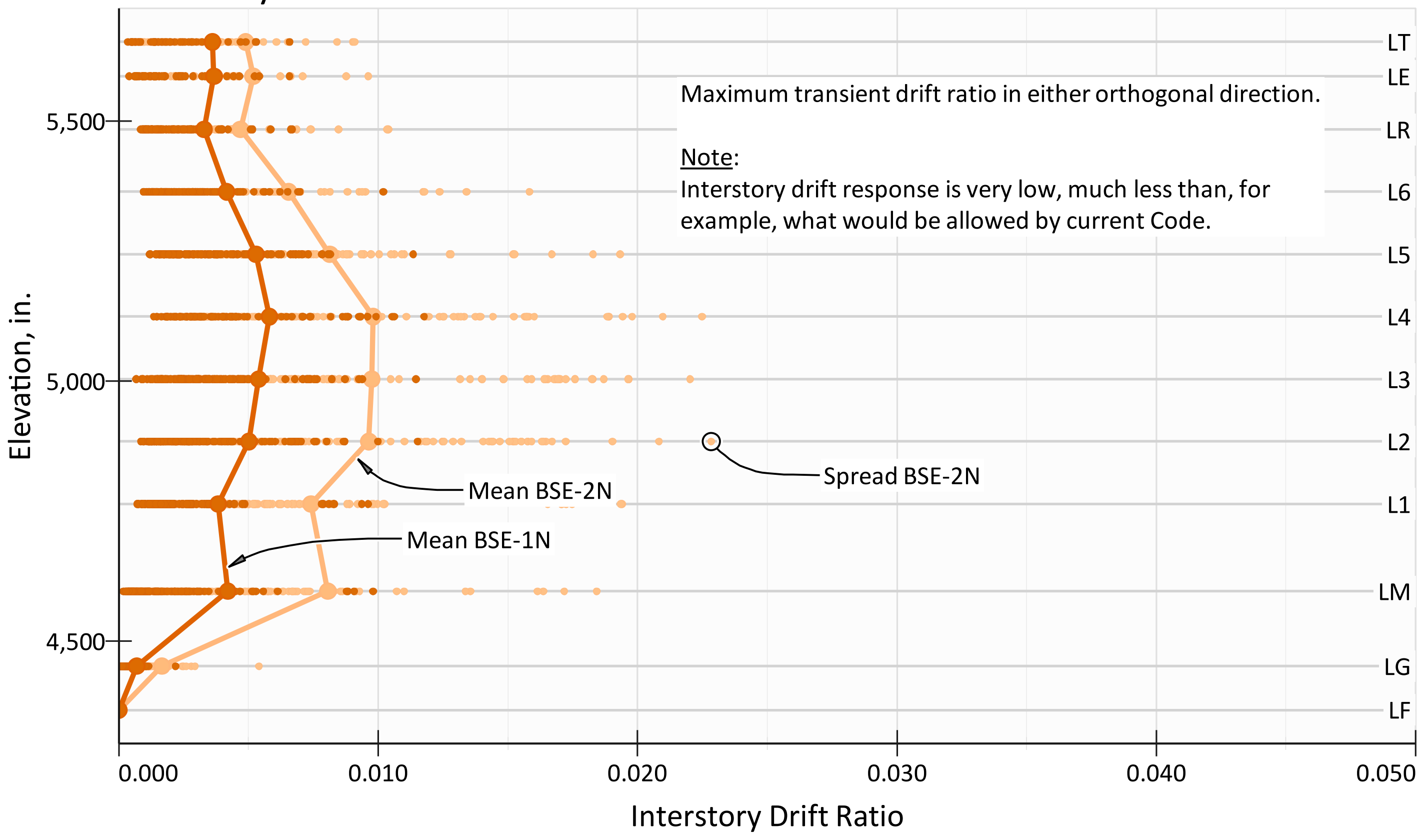

Fault-Normal Component
Spectral Acceleration, g
Period, sec
0.0
2.0
4.0
6.0
0.0
0.5
1.0
1.5
2.0
0.2 · T_0
T_0
1.5 · T_0
$eq01_{FN}$
$eq02_{FN}$
$eq03_{FN}$
$eq04_{FN}$
$eq05_{FN}$
$eq06_{FN}$
$eq07_{FN}$
$eq08_{FN}$
$eq09_{FN}$
$eq10_{FN}$
$eq11_{FN}$
DS
$Mean_{FN}$


Fault-Parallel Component
Spectral Acceleration, g
Period, sec
0.0
2.0
4.0
6.0
0.0
0.5
1.0
1.5
2.0
0.2 · T_0
T_0
1.5 · T_0
$eq01_{FP}$
$eq02_{FP}$
$eq03_{FP}$
$eq04_{FP}$
$eq05_{FP}$
$eq06_{FP}$
$eq07_{FP}$
$eq08_{FP}$
$eq09_{FP}$
$eq10_{FP}$
$eq11_{FP}$
DS
$Mean_{FP}$

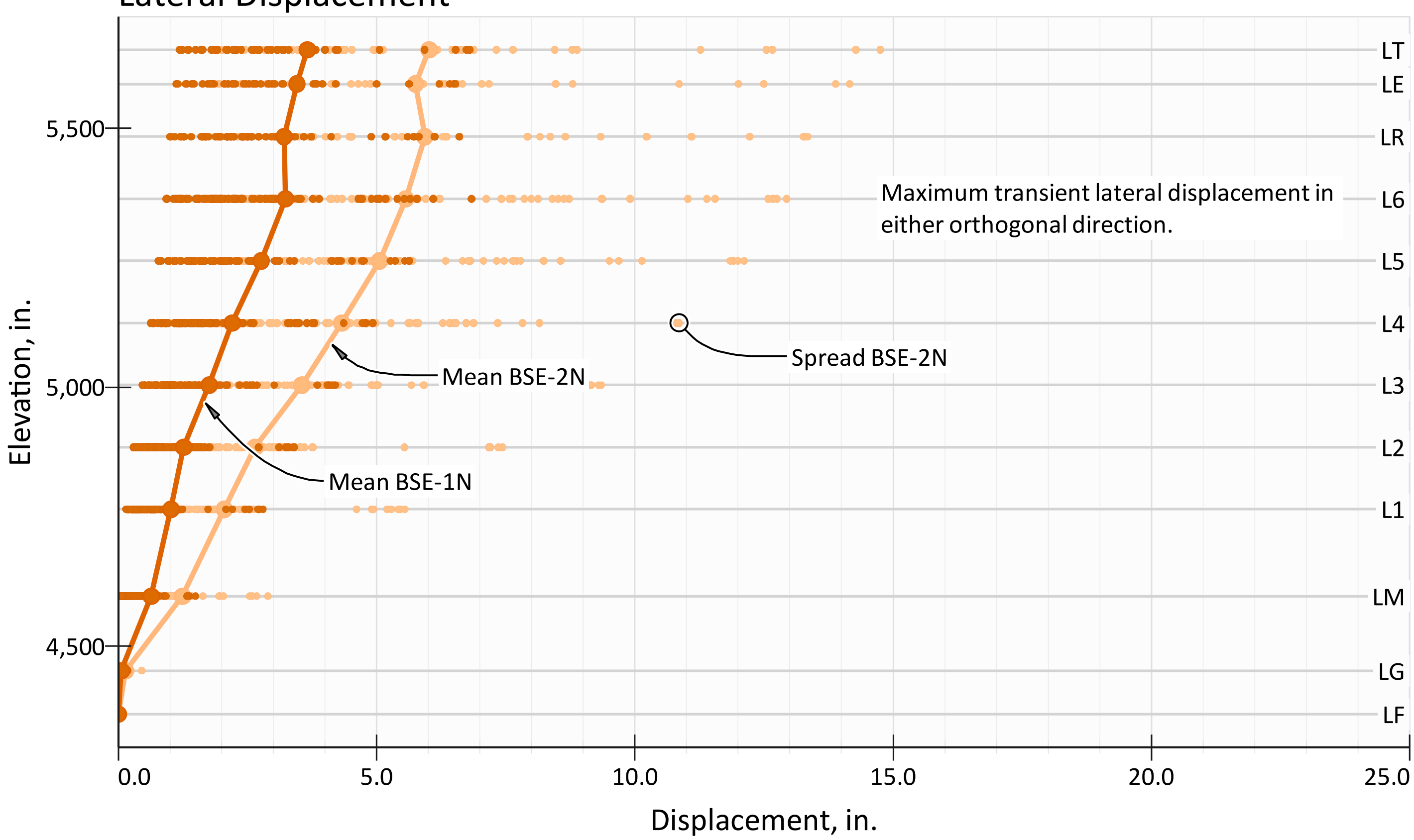

Spectral Acceleration at Cupola, Maximum Component
0.2 · T_0
T_0
1.5 · T_0
9.0
6.0
3.0
0.0
Spectral Acceleration, g
at Cupola
at Ground
0.0
0.5
1.0
1.5
2.0
Period, sec


Spectral Acceleration Comparison
0.2 · T_0
T_0
1.5 · T_0
4.0
2.0
0.0
Spectral Acceleration, g
Target spectrum for scaling, BSE-1N
BSE-2N: 1.5 times BSE-1N, scaled in Perform
BSE-C per ASCE 41-17
975 yr. per URS report
Mean of BSE-1N Records
BSE-R per ASCE 41-17
225 yr. per URS report
0.0
0.5
1.0
1.5
2.0
Period, sec

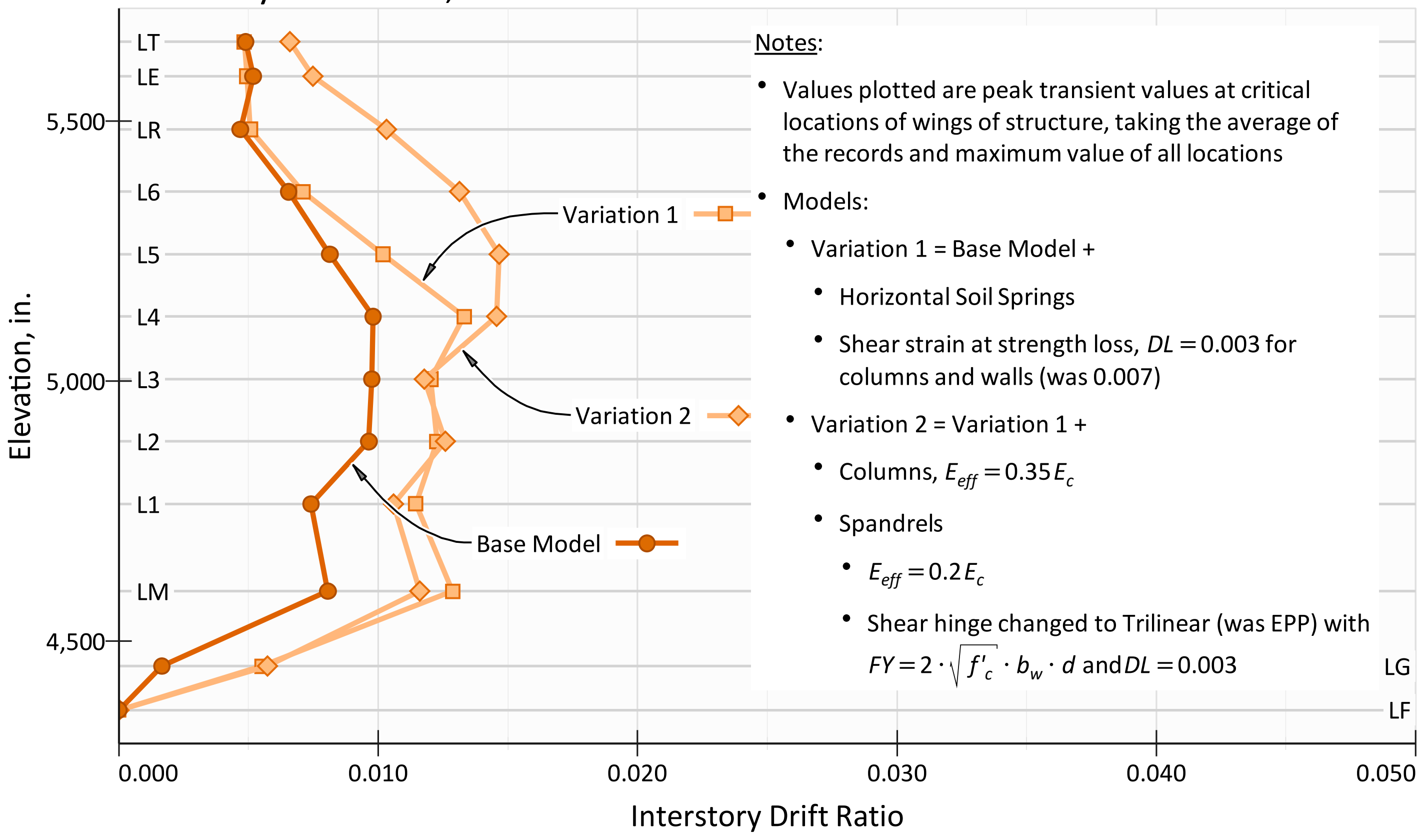
Interstory Drift Ratio



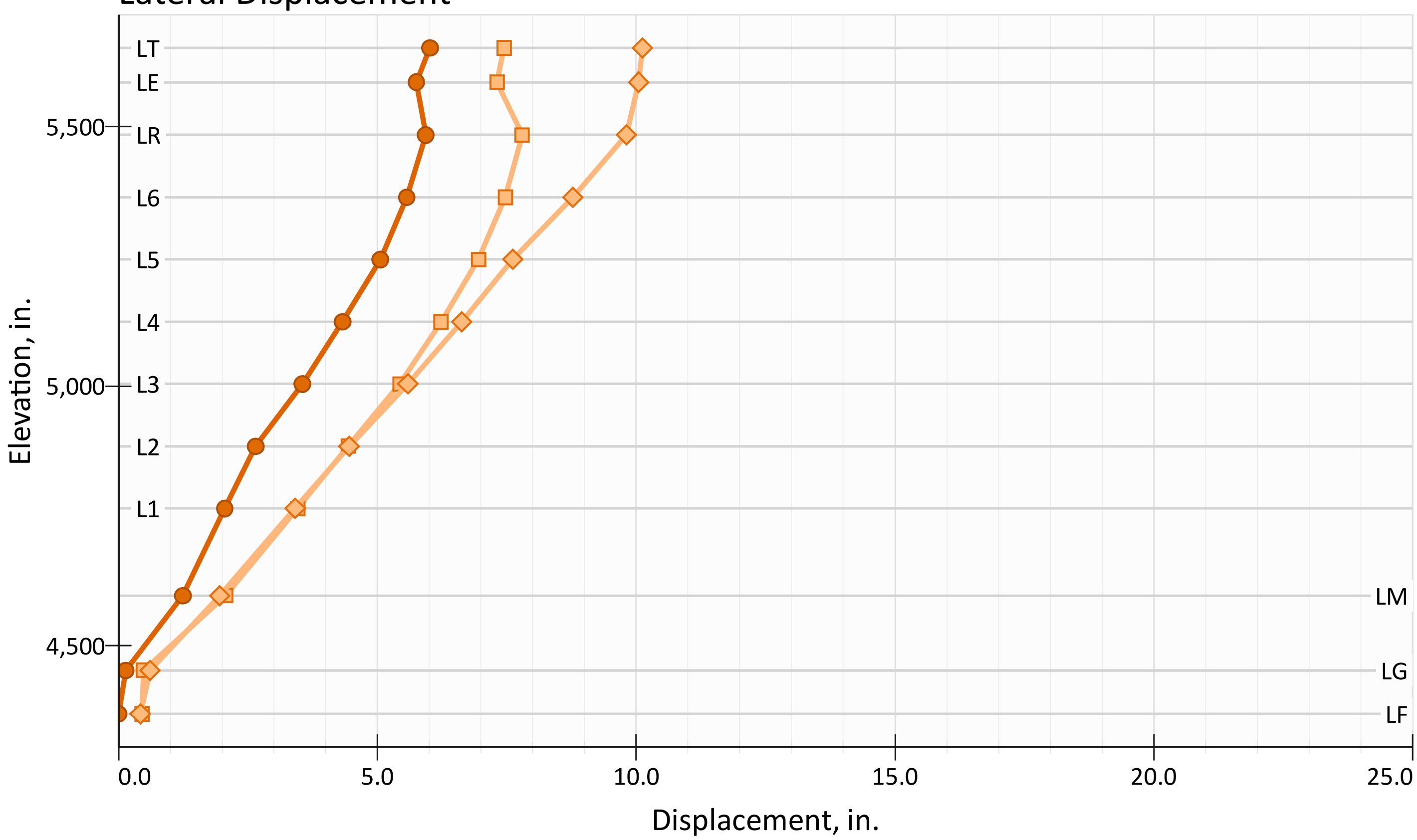
Lateral Displacement



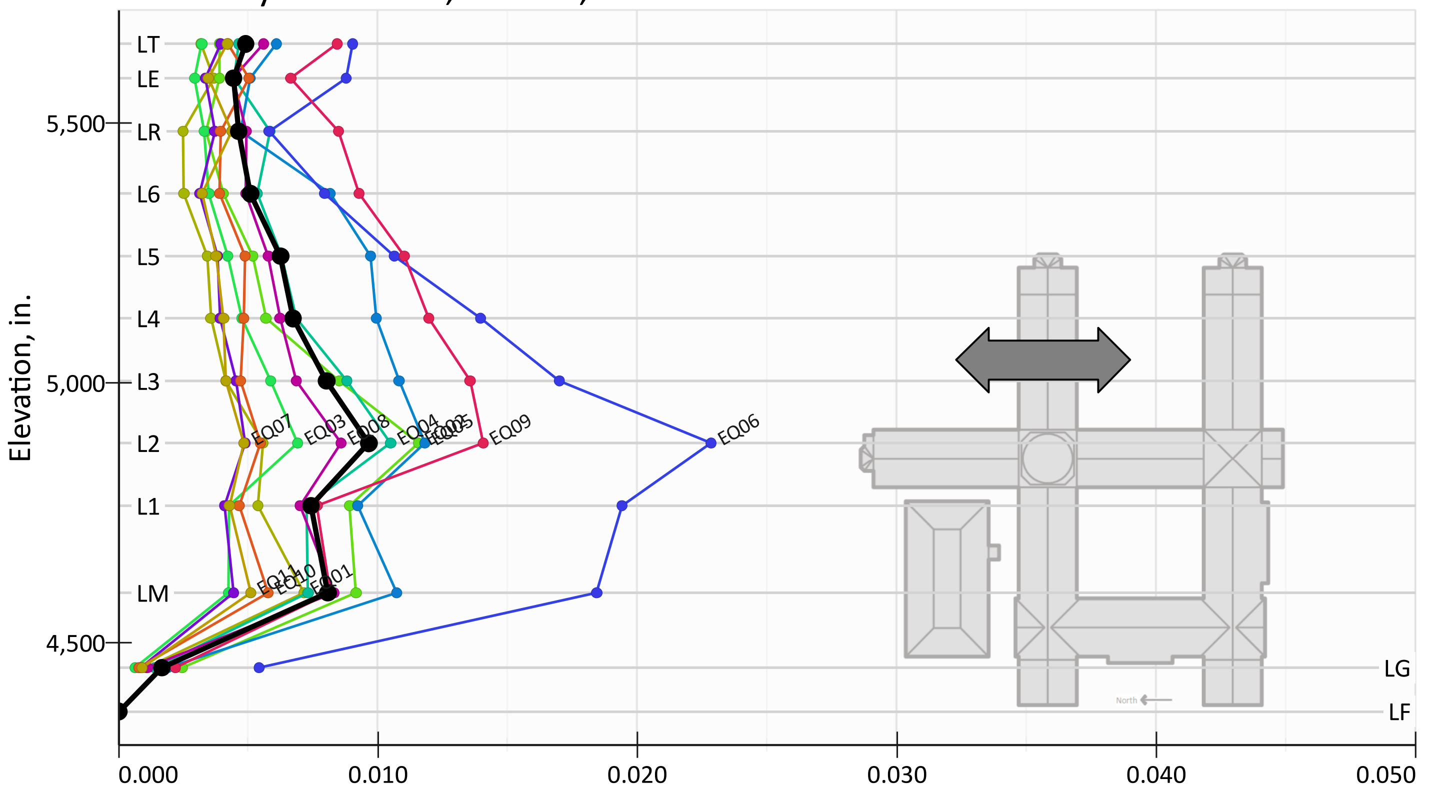
Interstory Drift Ratio, BSE-2N



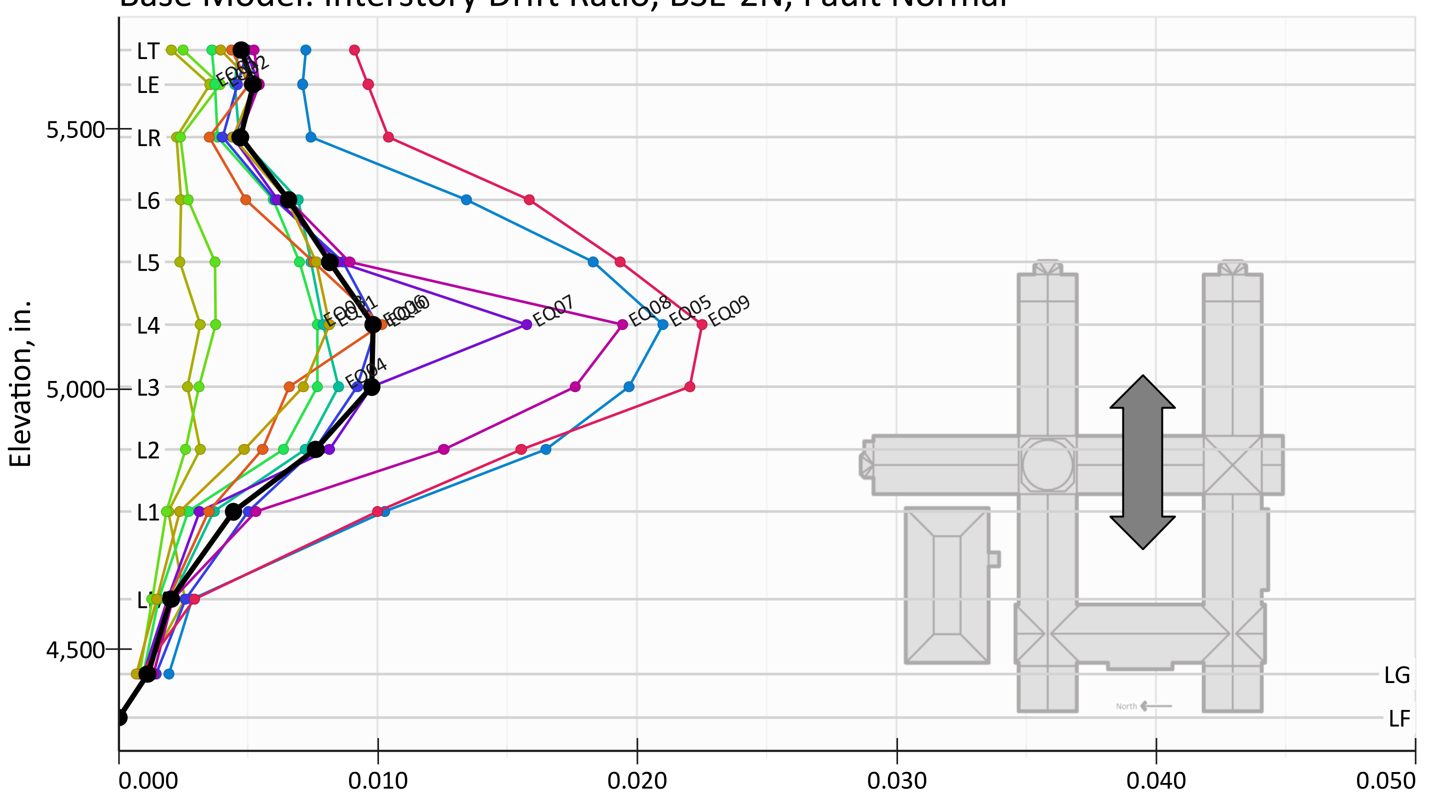
Lateral Displacement



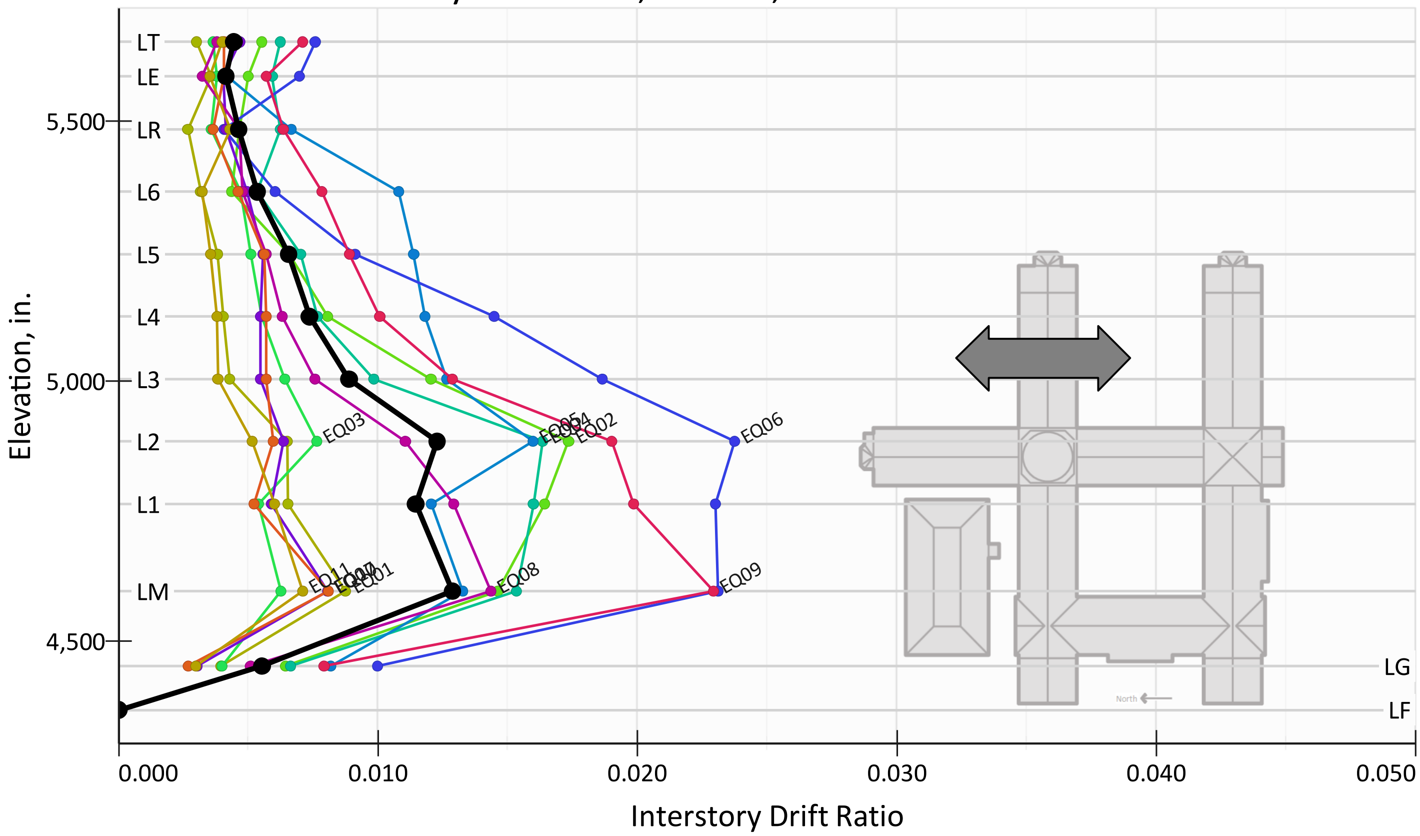
Interstory Drift Ratio, BSE-2N, Fault Parallel



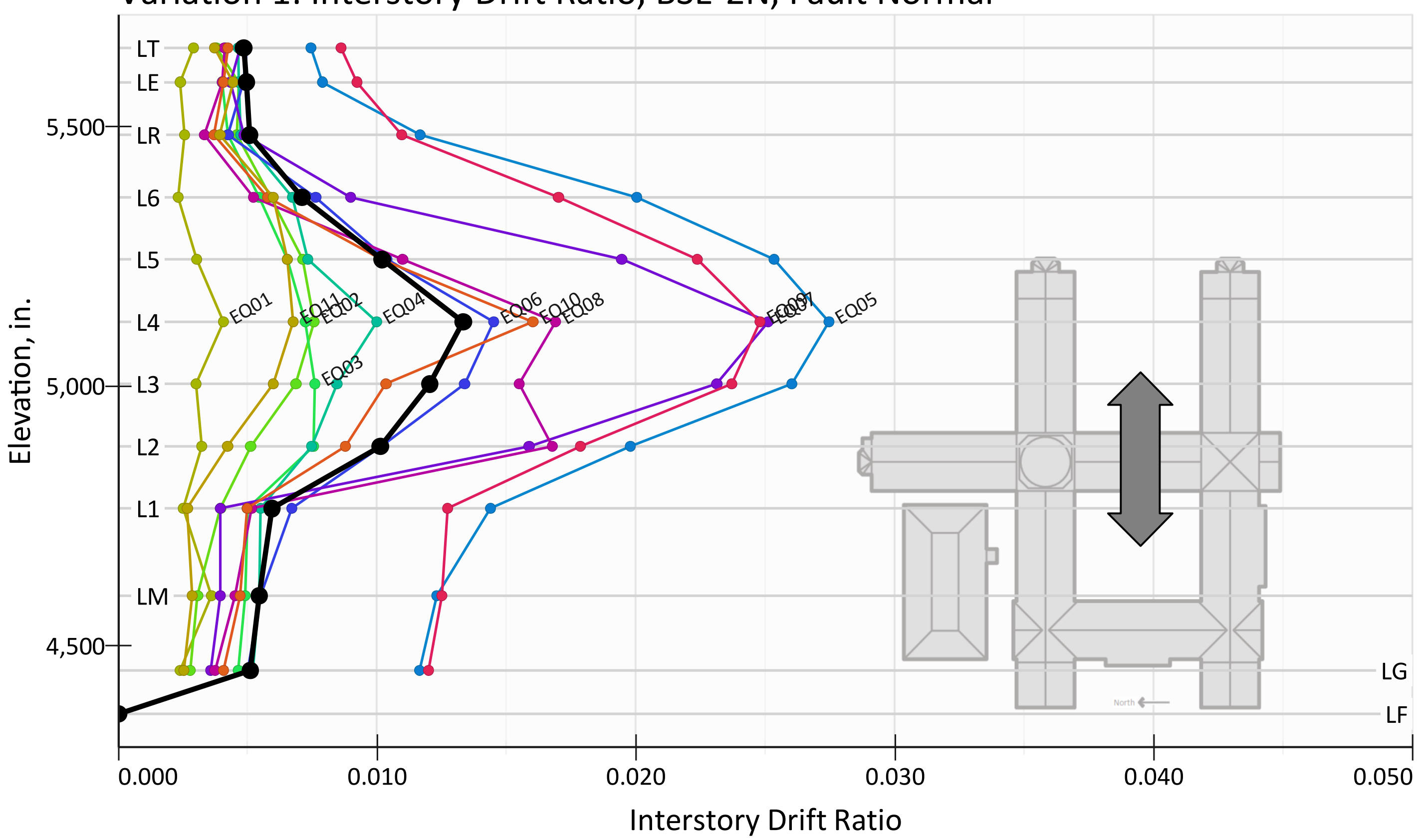
Base Model: Interstory Drift Ratio, BSE-2N, Fault Normal



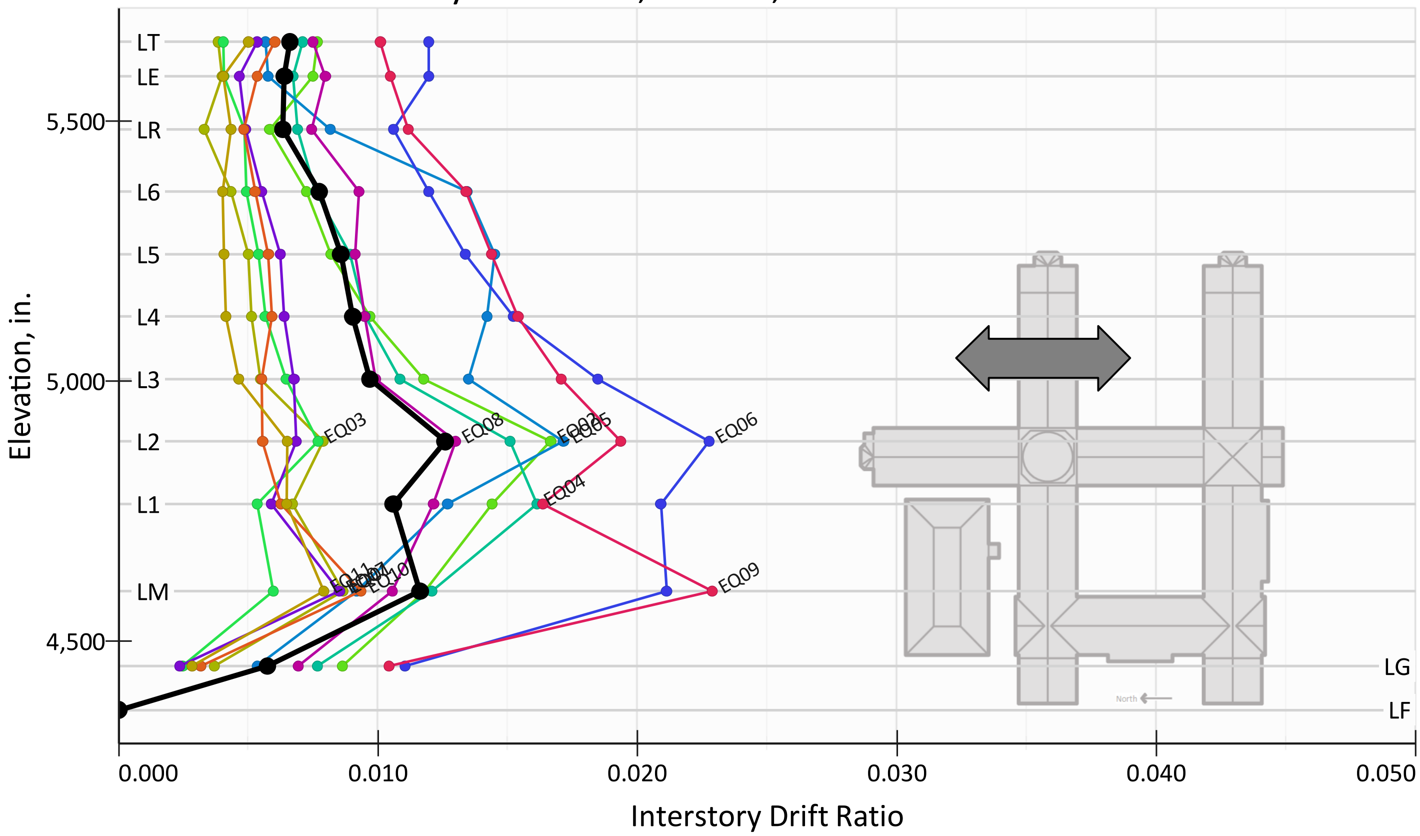
Variation 1: Interstory Drift Ratio, BSE-2N, Fault Parallel



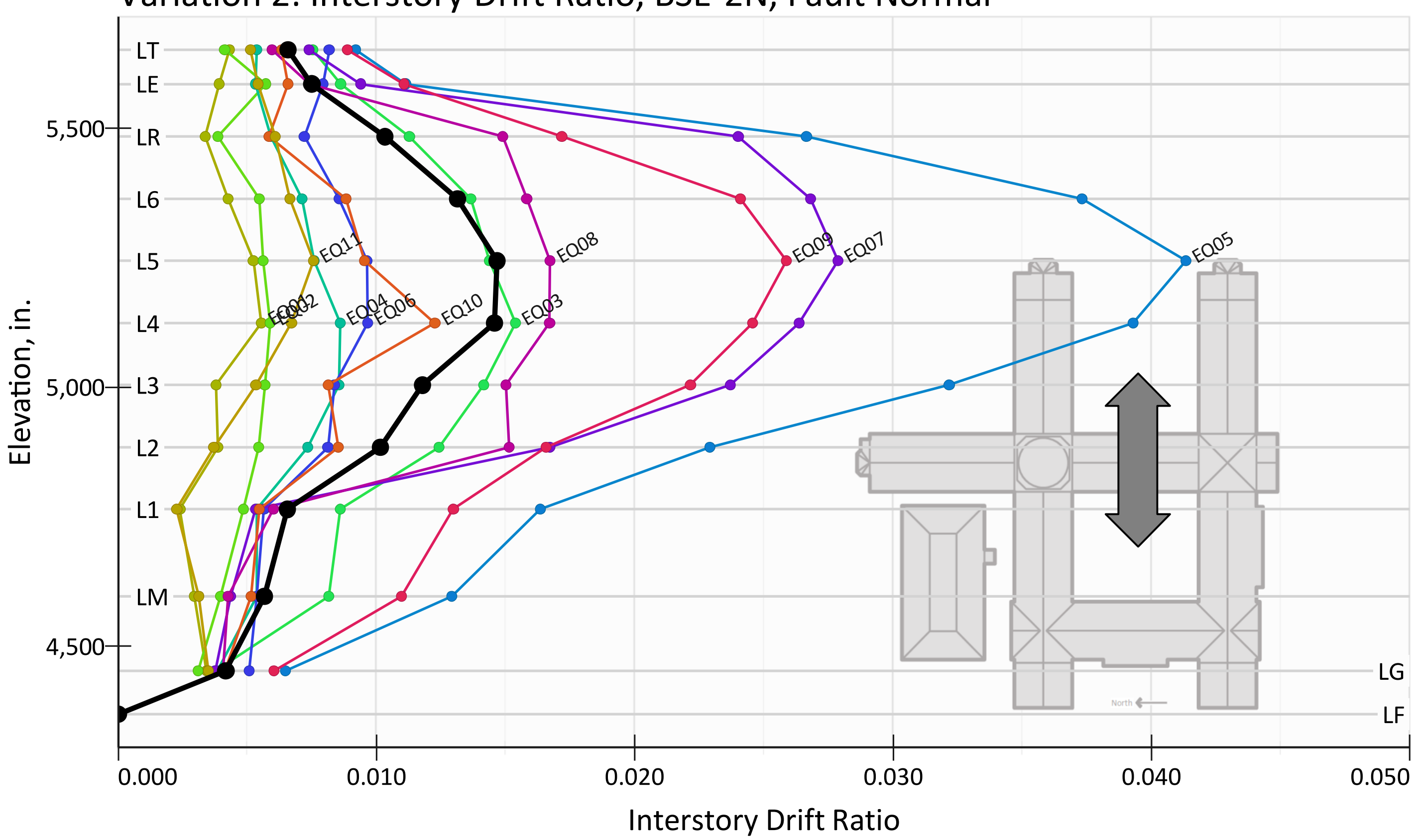
Variation 1: Interstory Drift Ratio, BSE-2N, Fault Normal



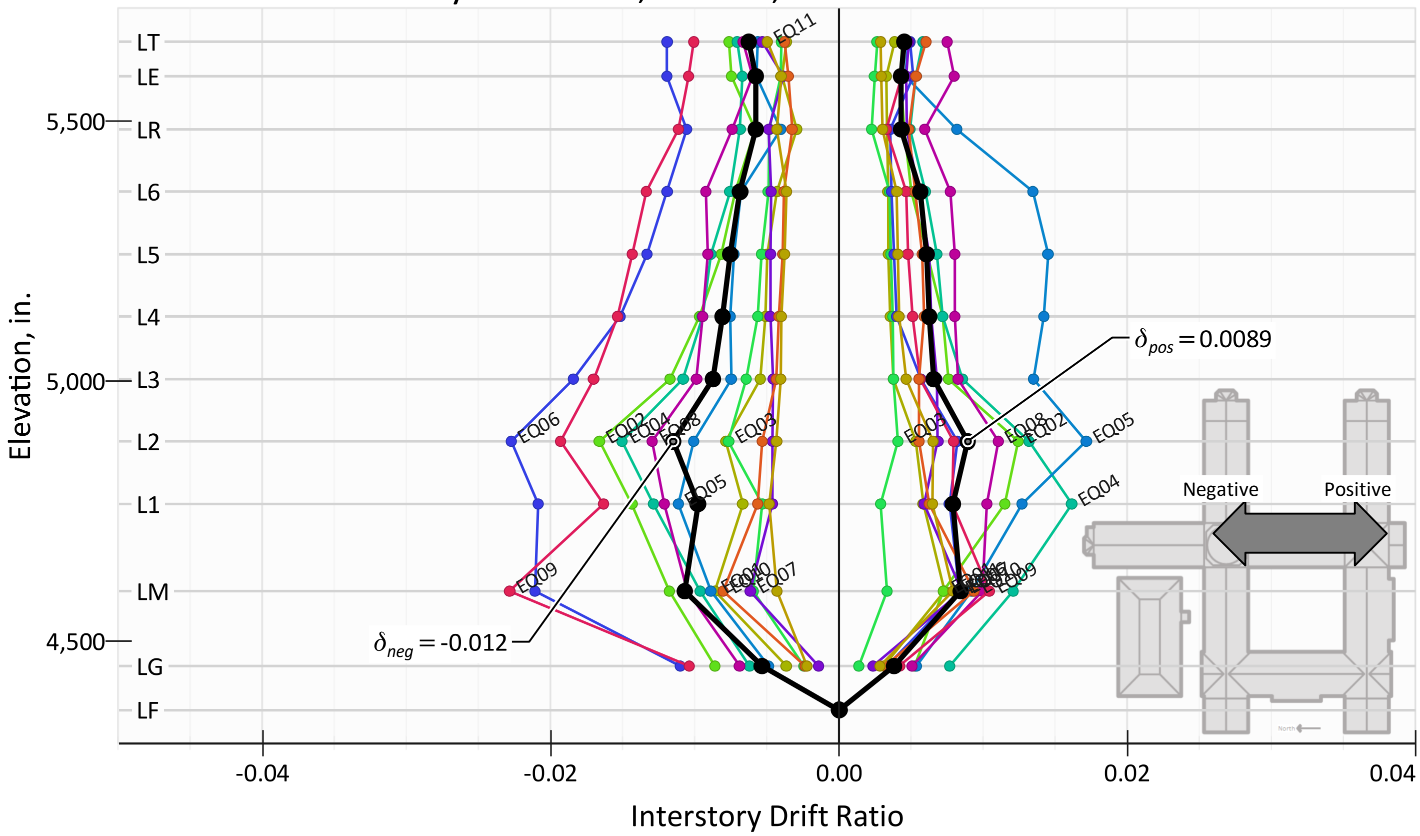
Variation 2: Interstory Drift Ratio, BSE-2N, Fault Parallel



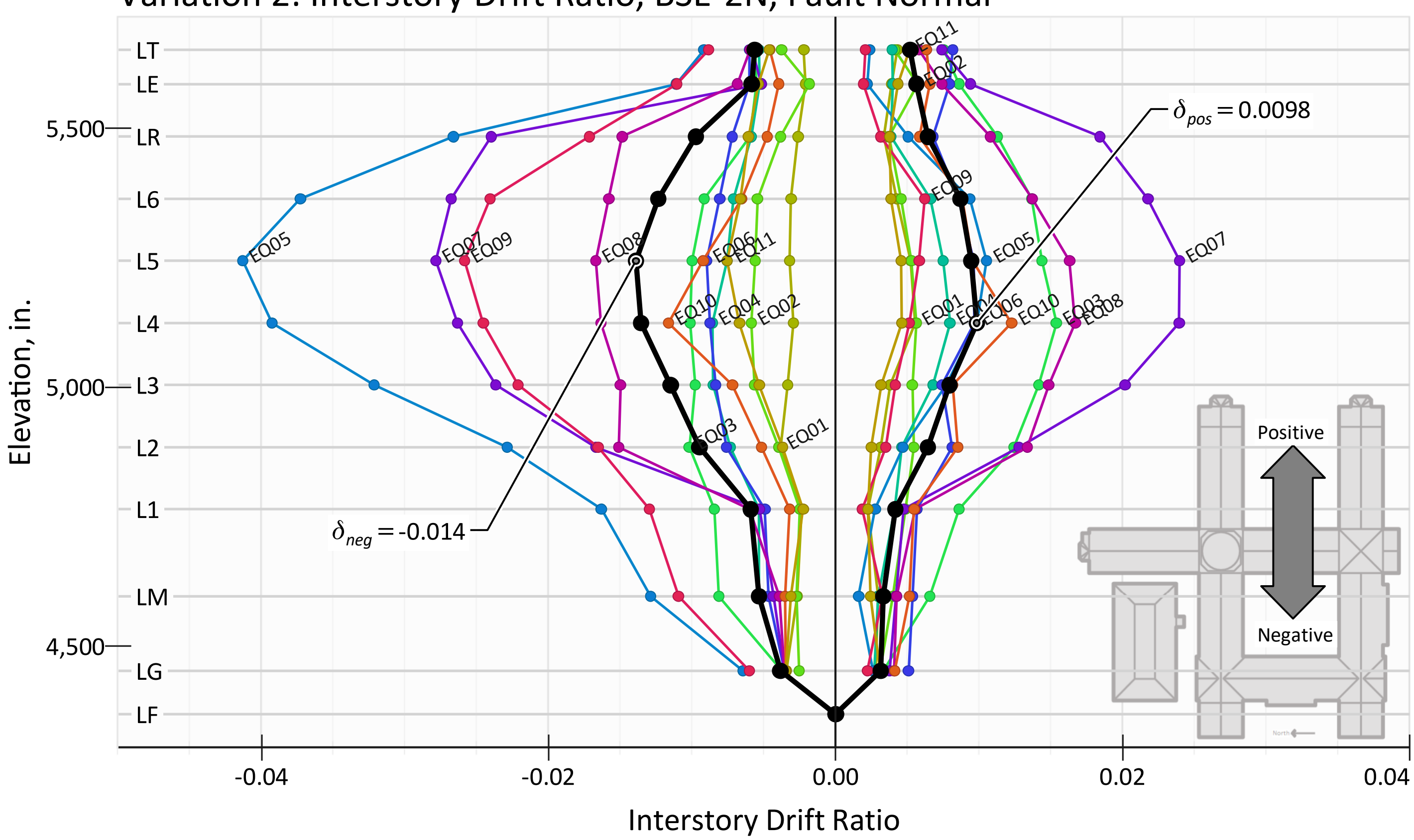
Variation 2: Interstory Drift Ratio, BSE-2N, Fault Normal



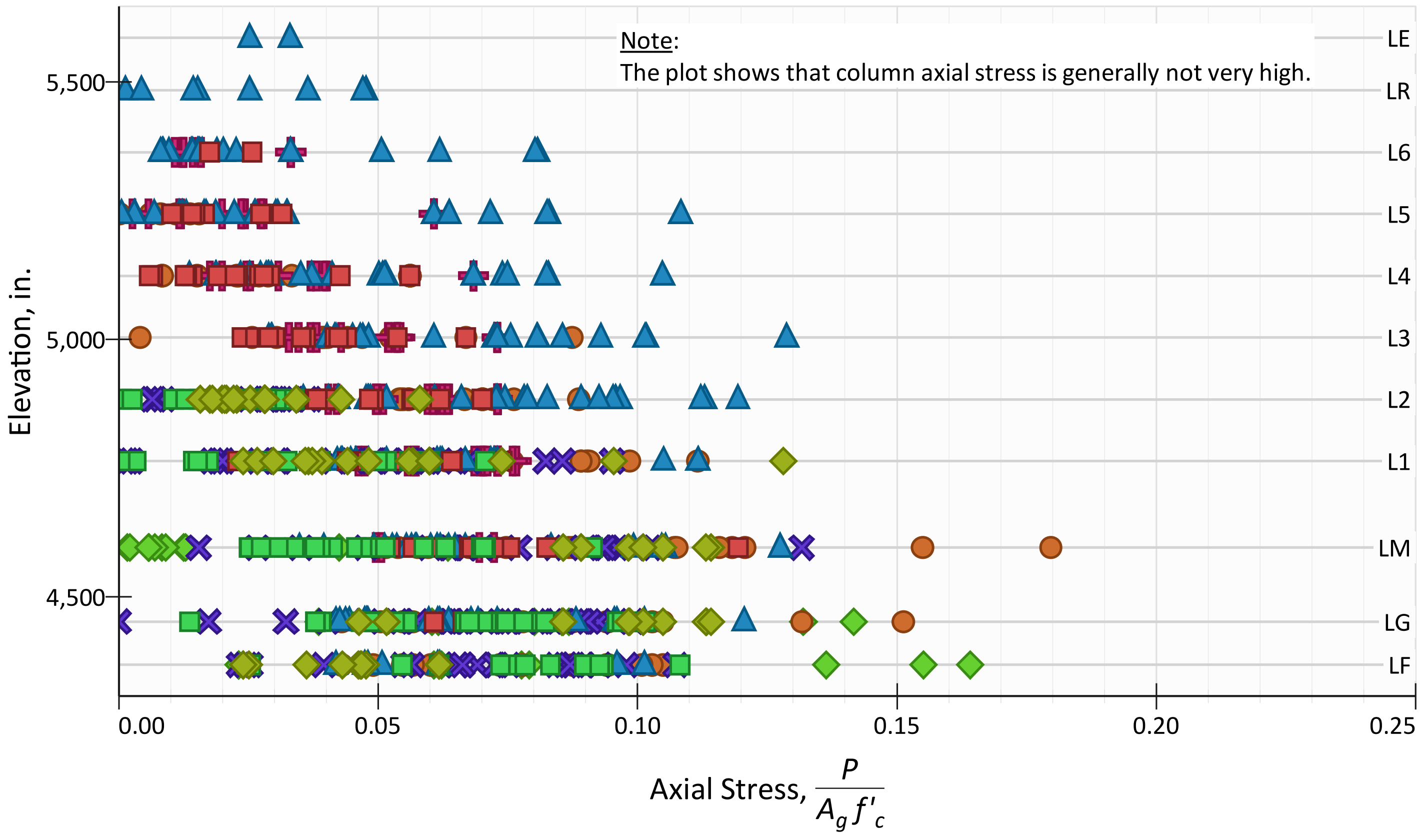
Variation 2: Interstory Drift Ratio, BSE-2N, Fault Parallel

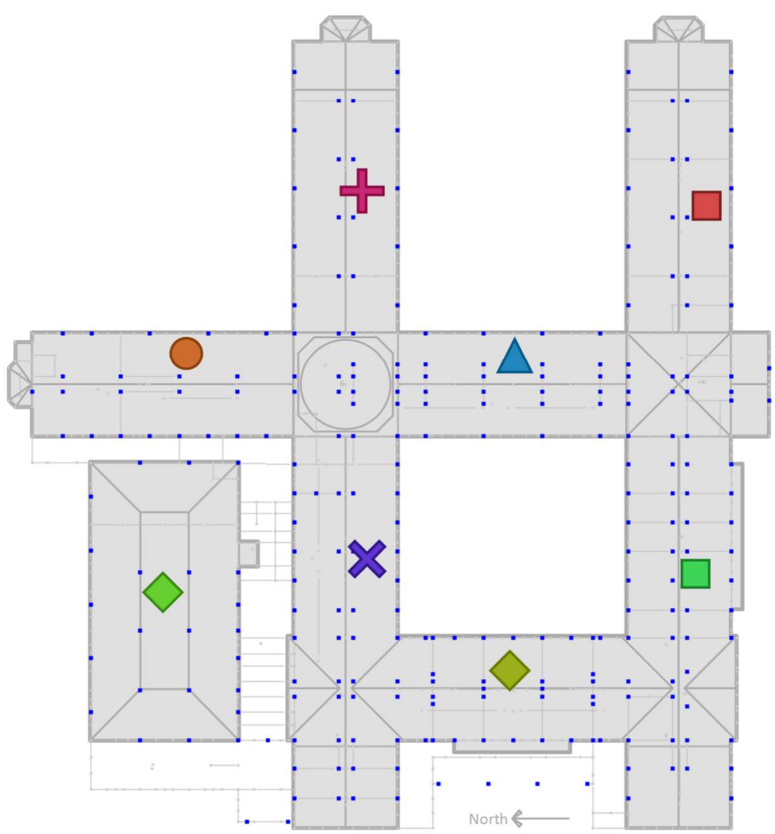


Variation 2: Interstory Drift Ratio, BSE-2N, Fault Normal

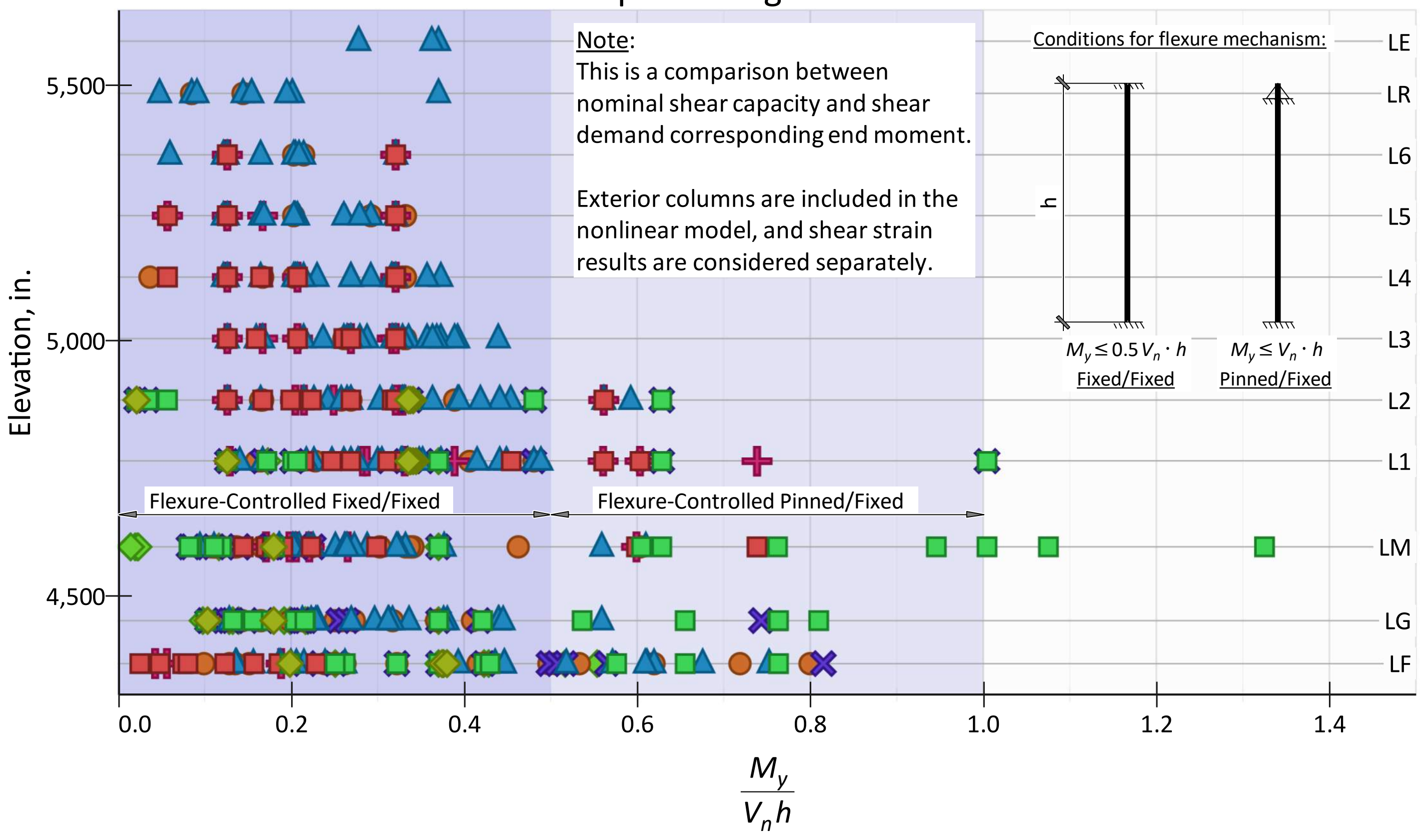


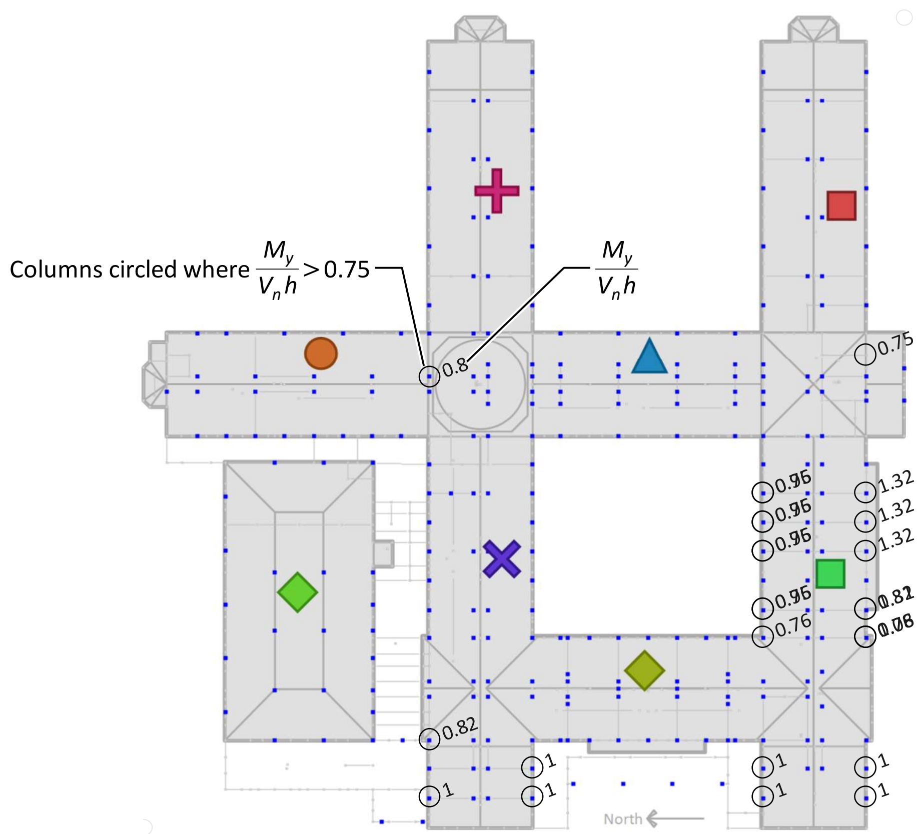
Column Dead Load Axial Stress



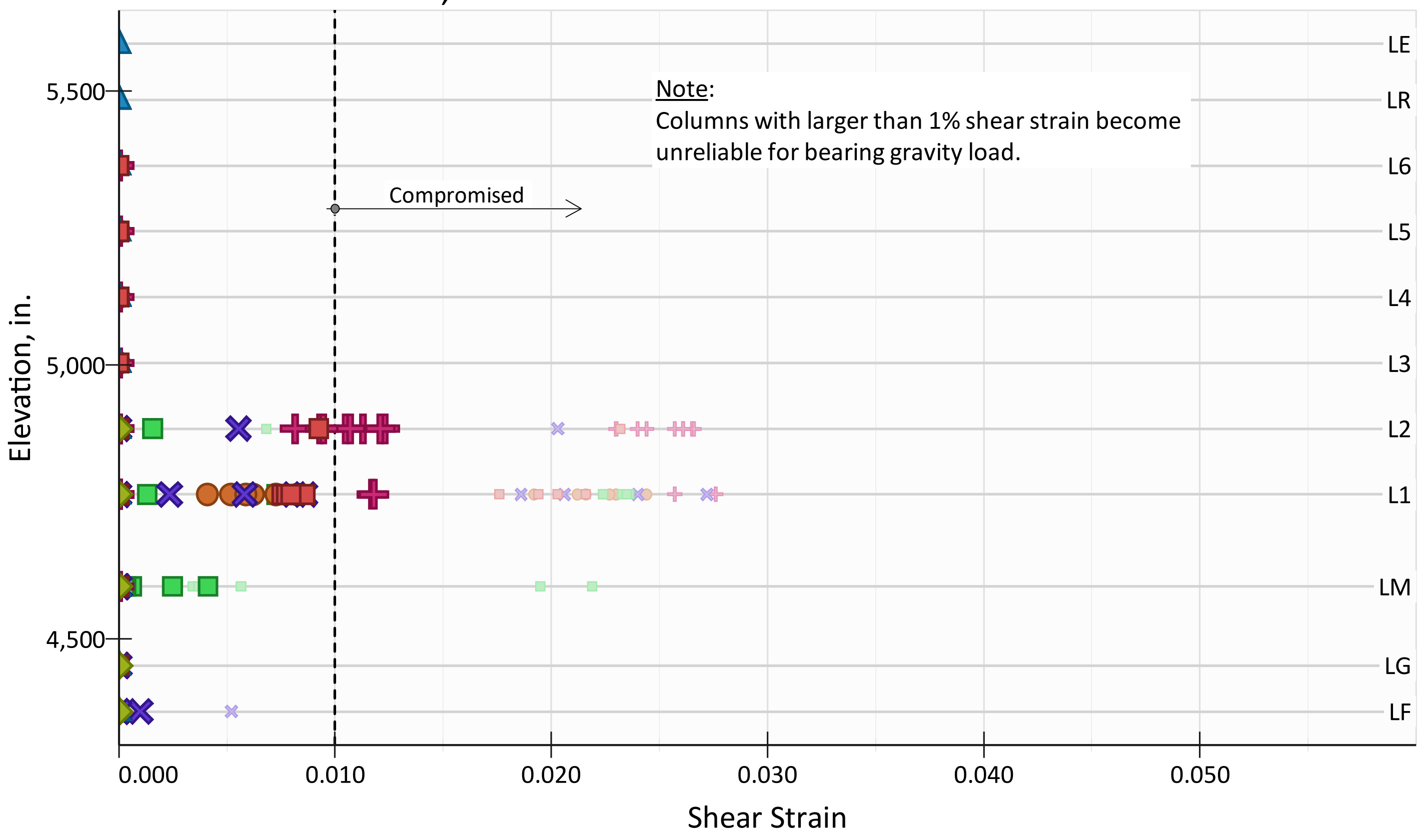


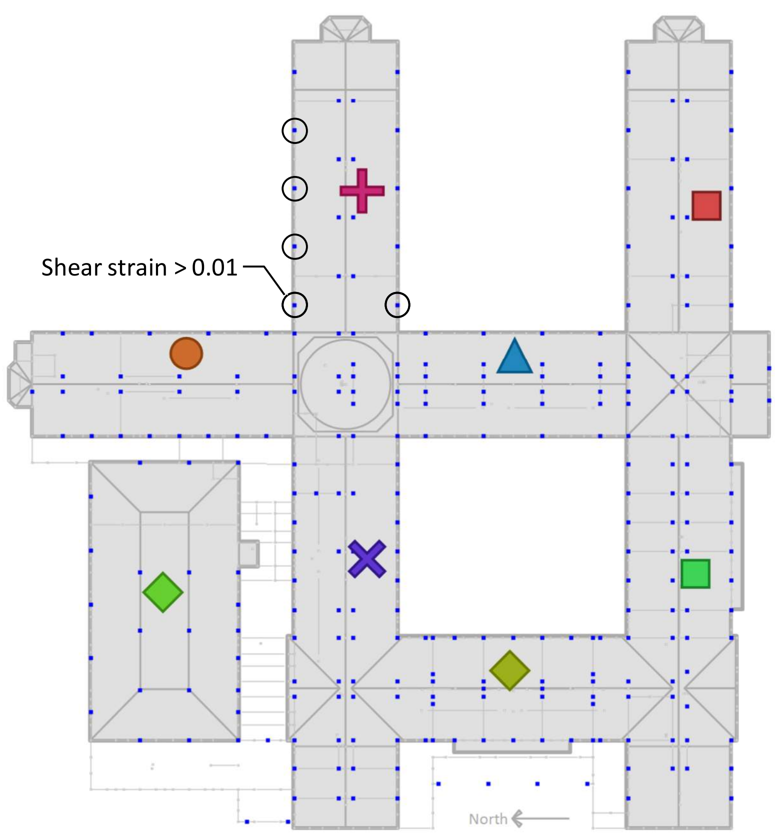
Columns Moment vs. Shear Proportioning



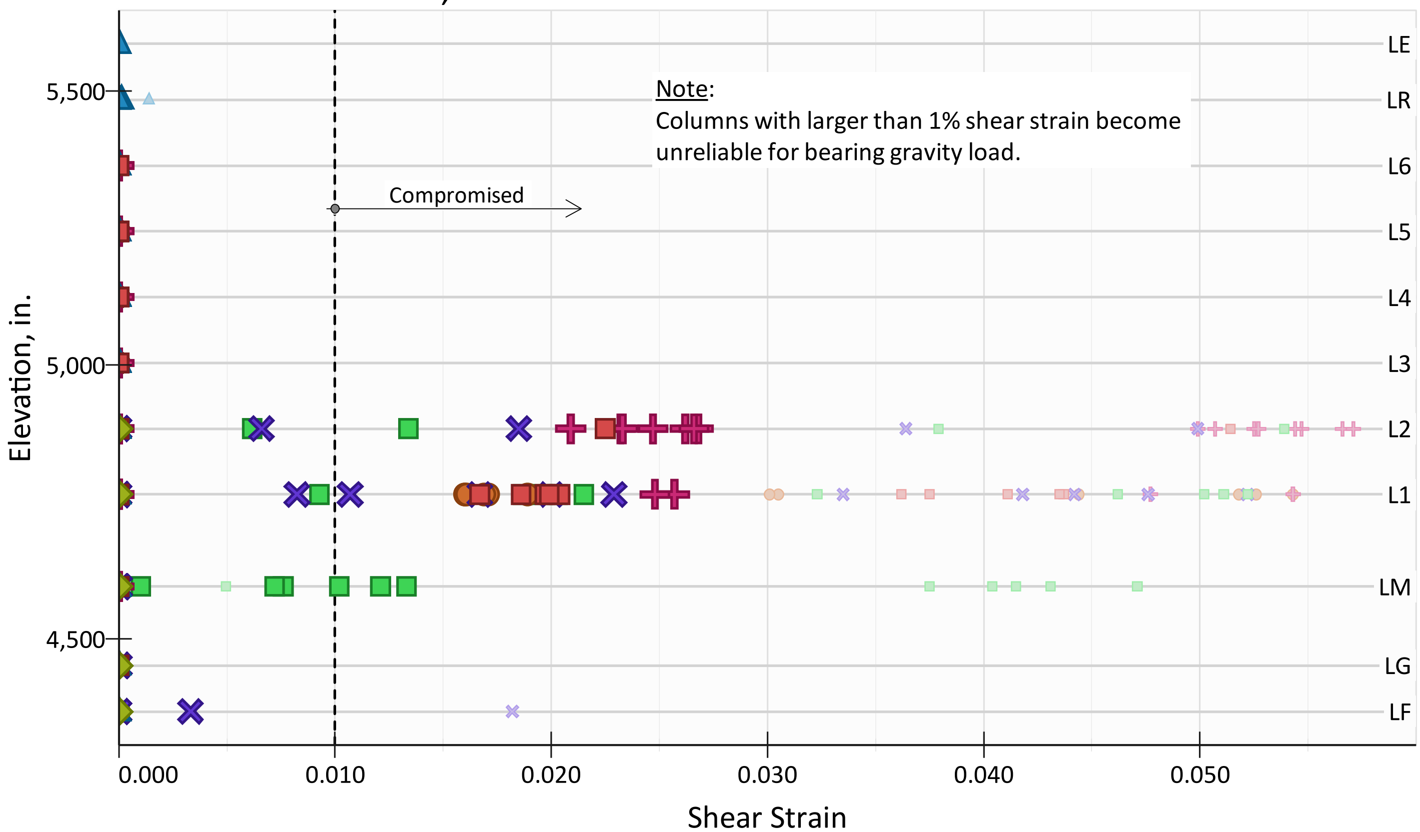


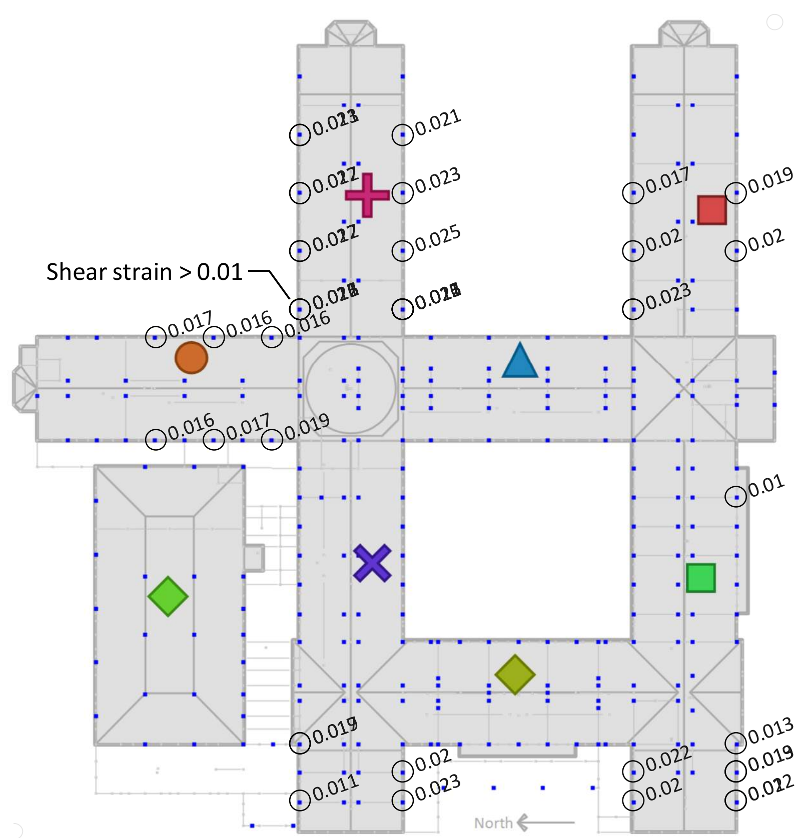
Column Shear Strain, BSE-1N



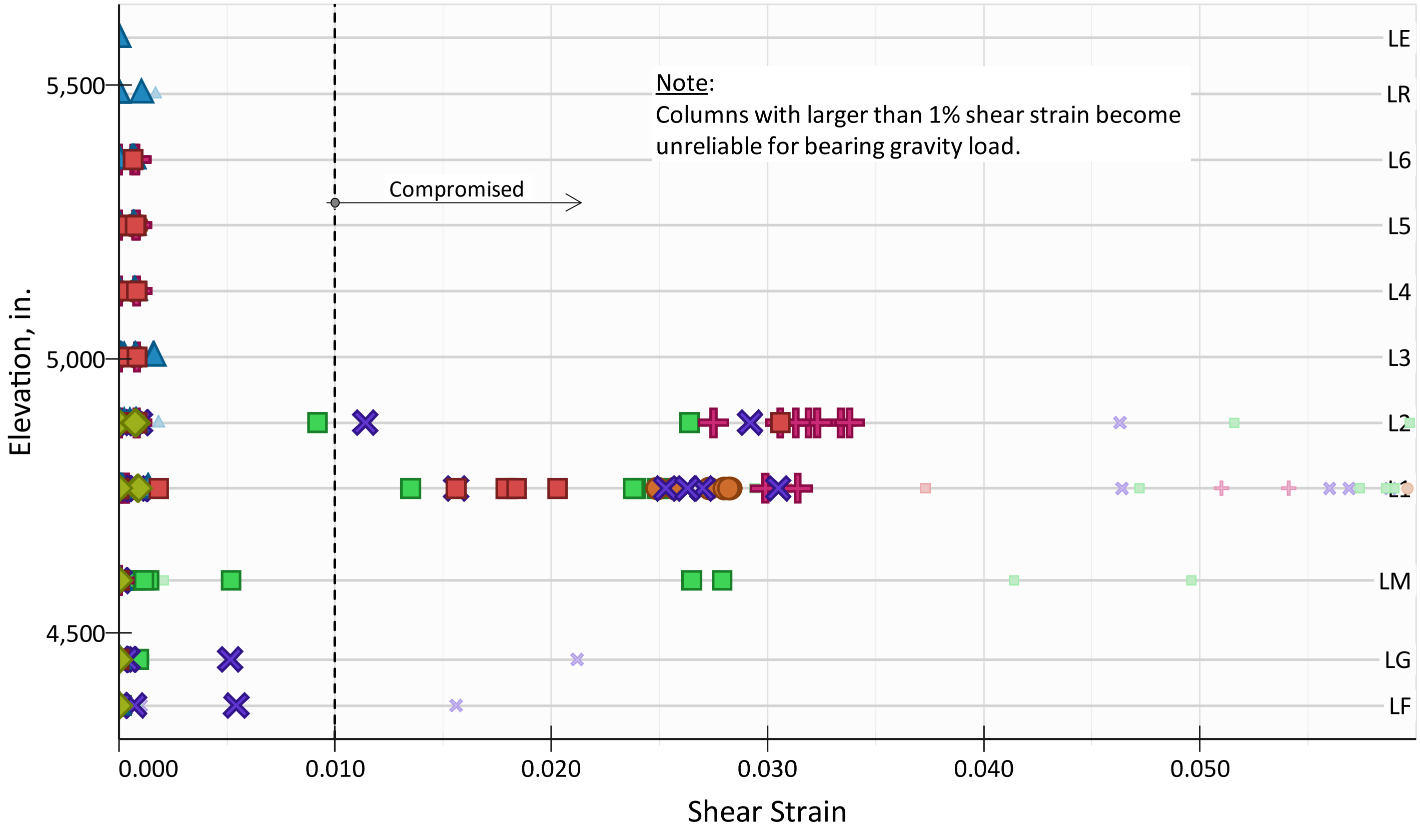


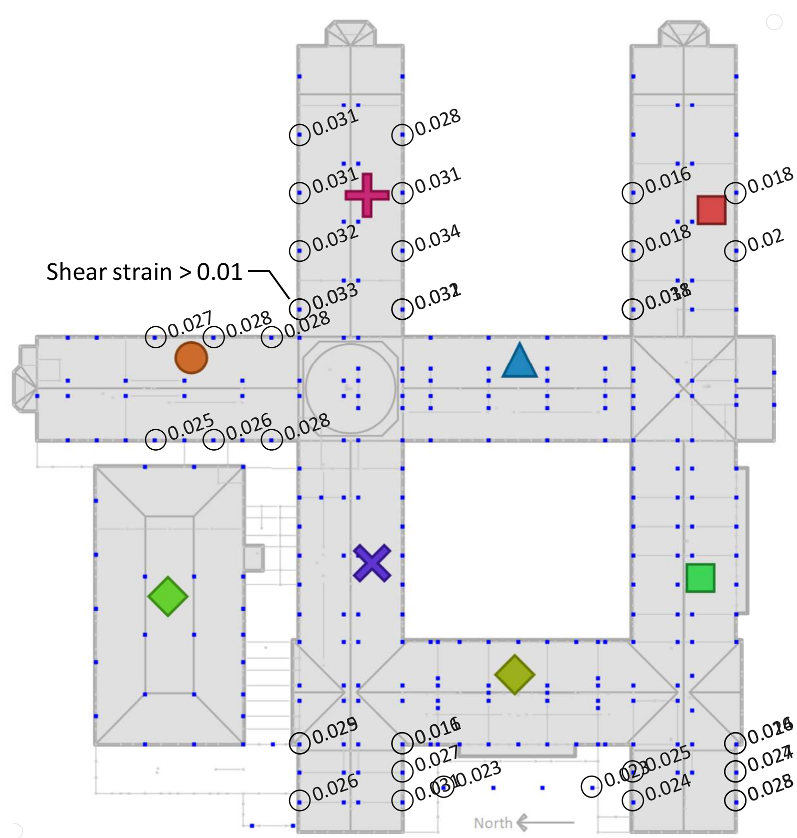
Column Shear Strain, BSE-2N



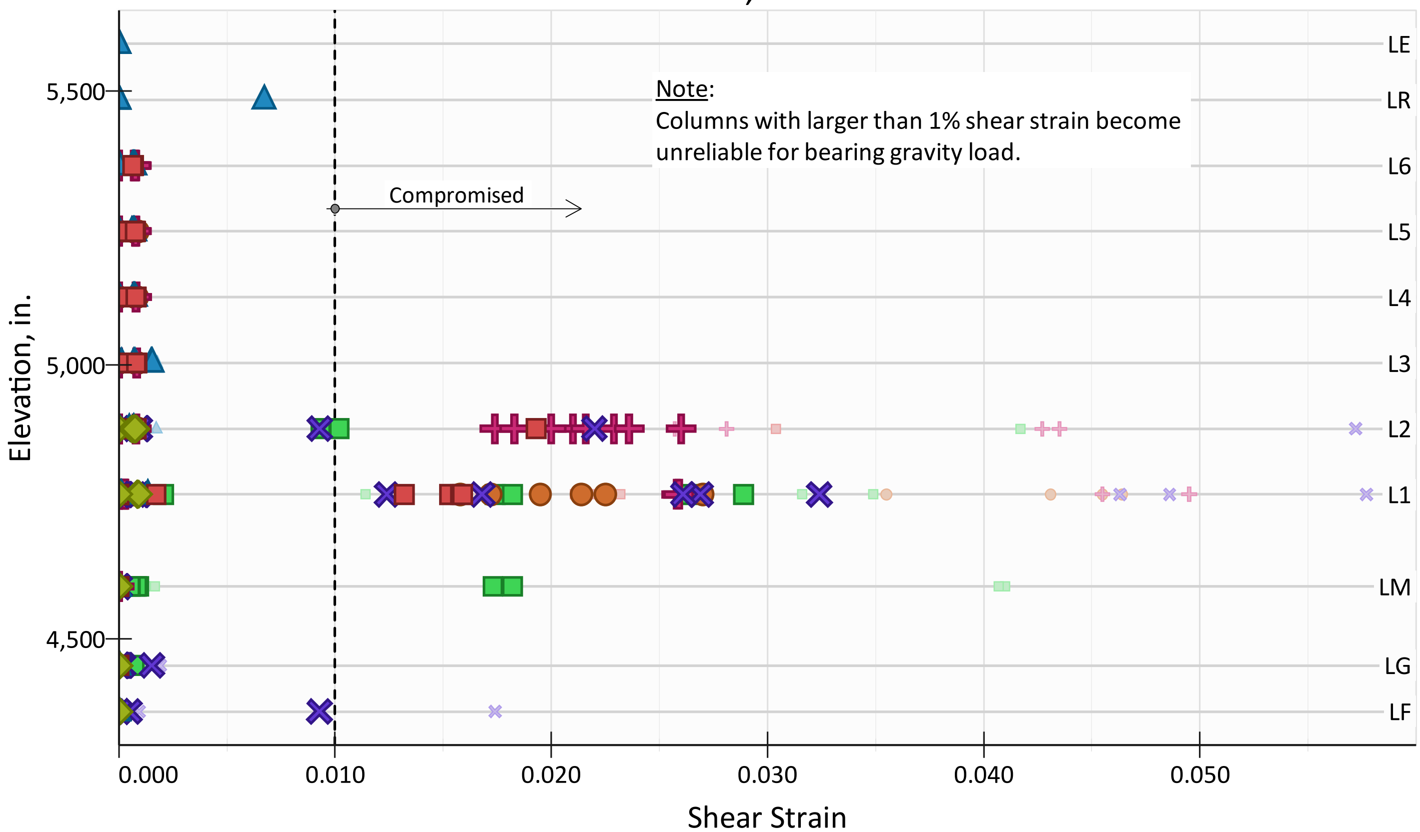


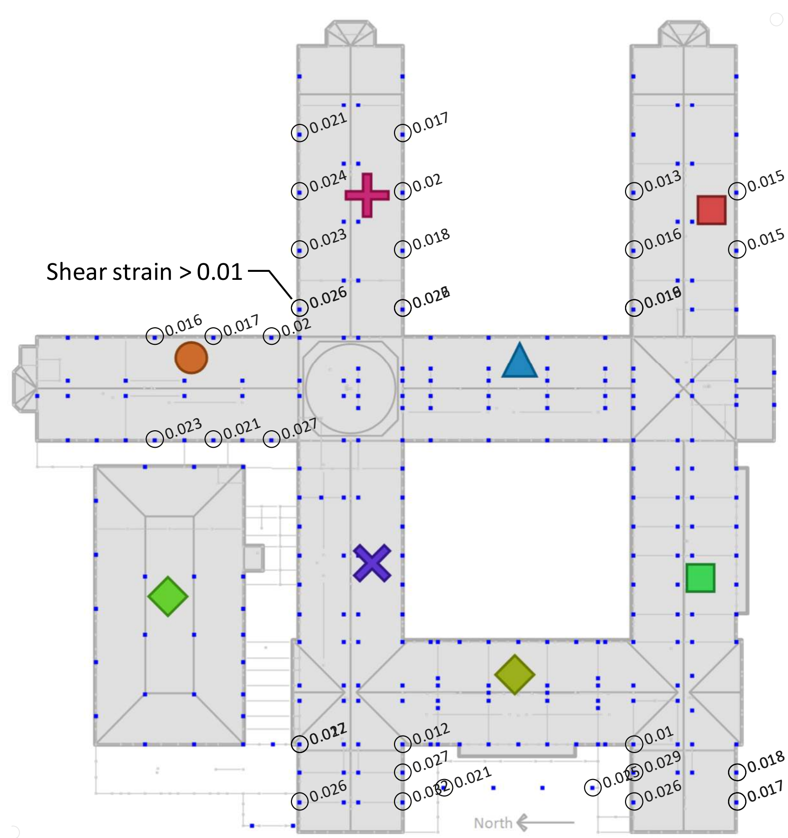
Model Variation 1: Column Shear Strain, BSE-2N

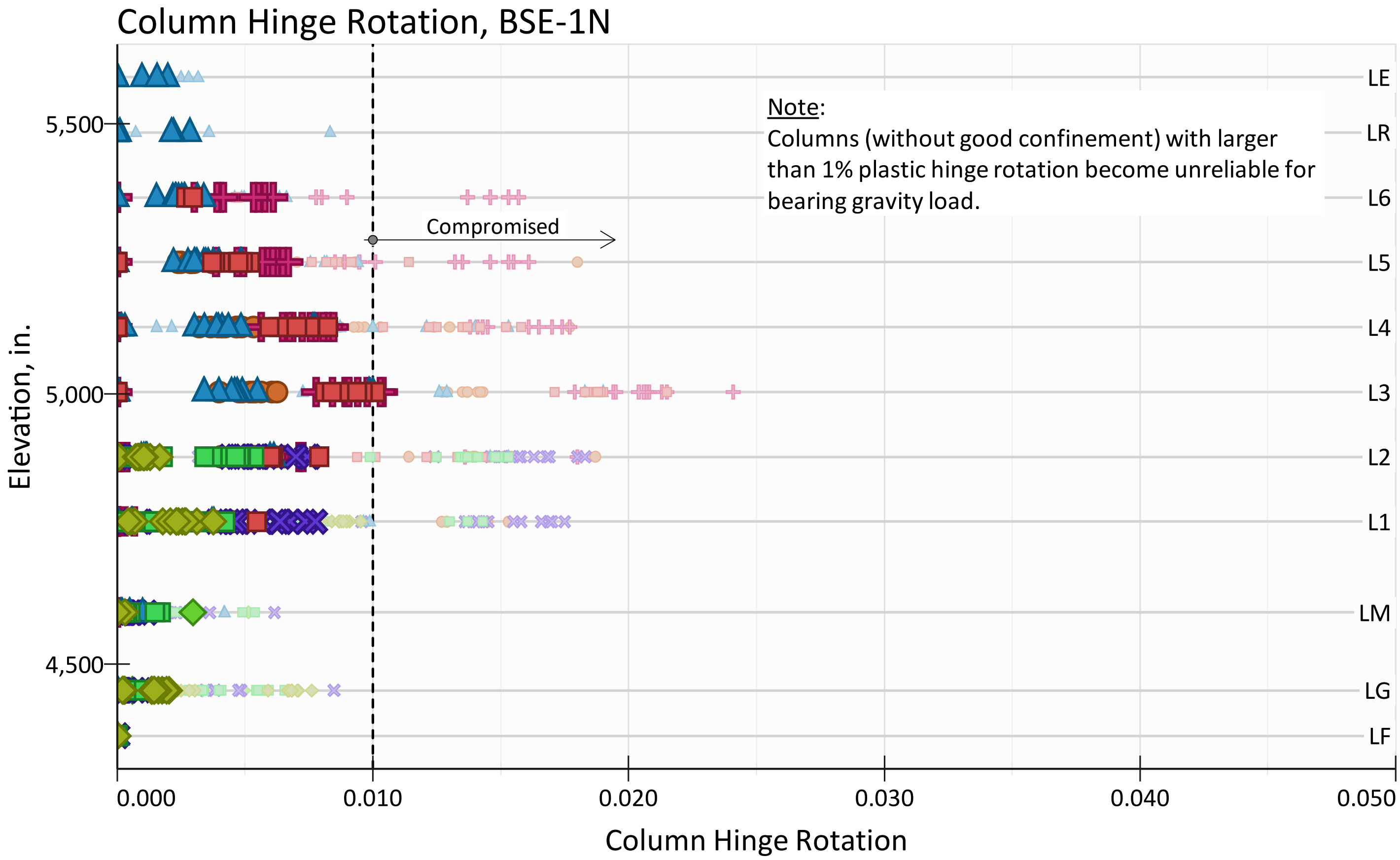


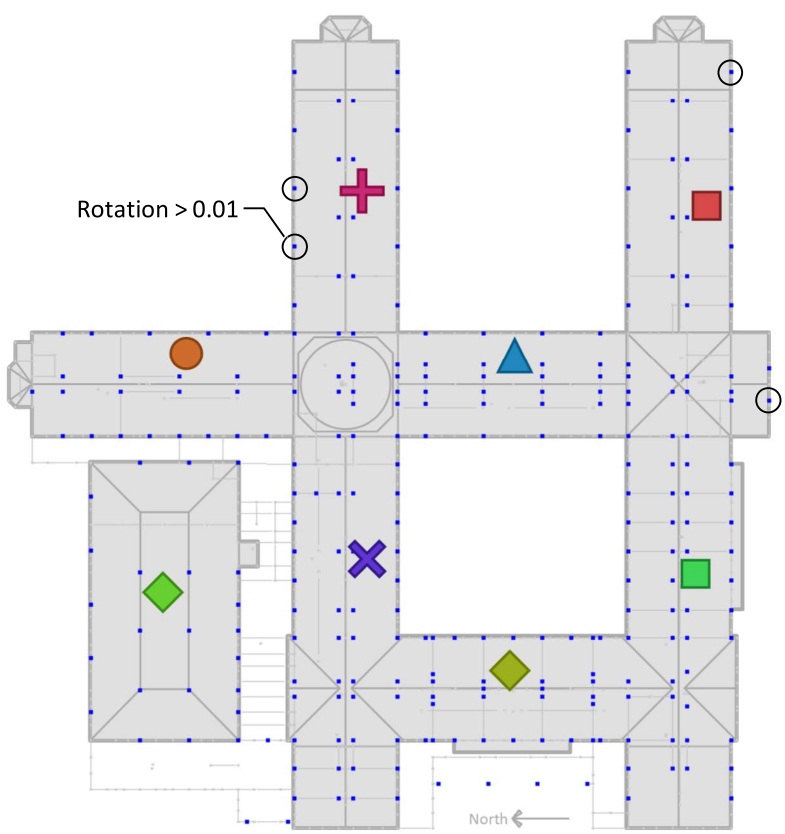


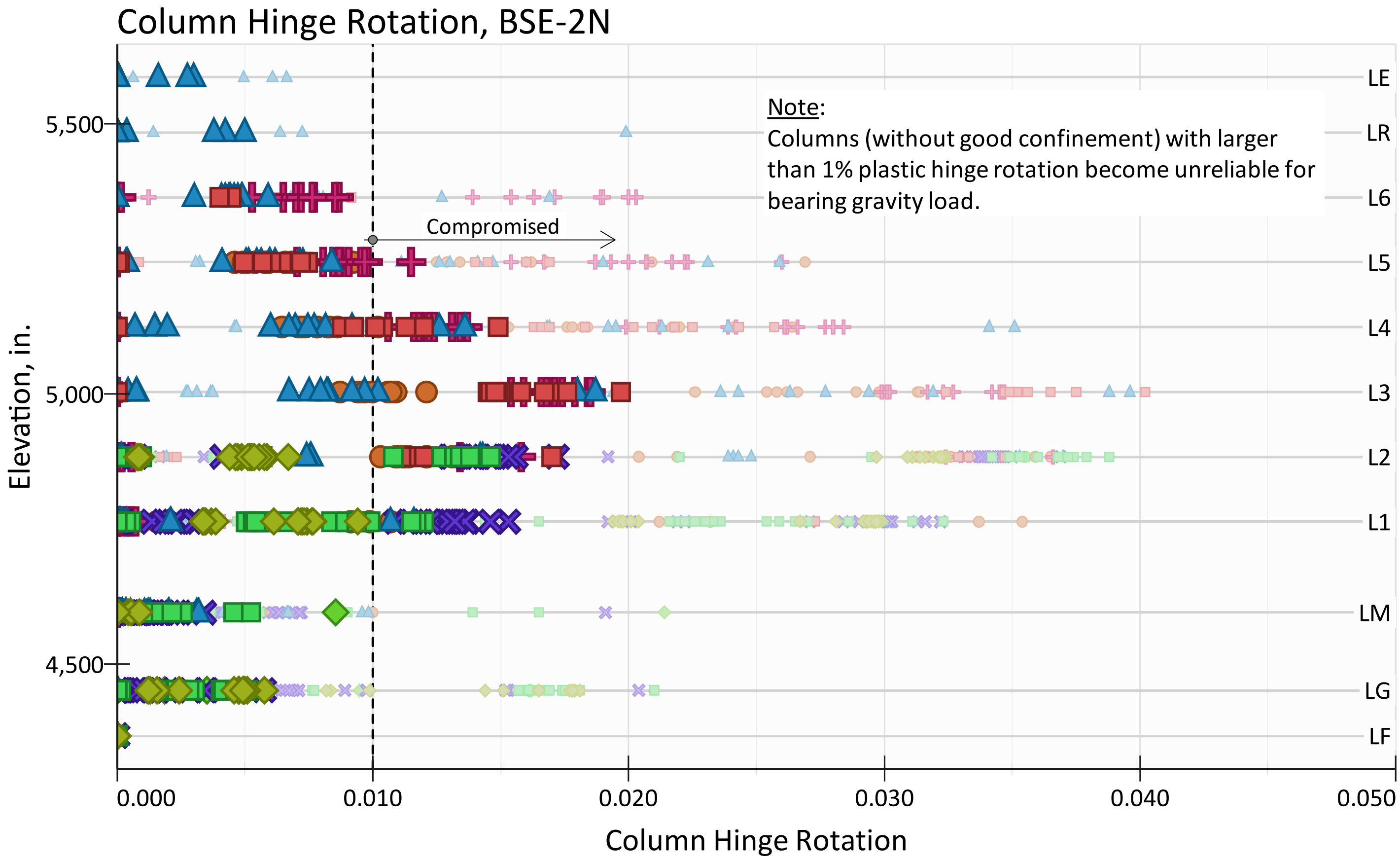
Model Variation 2: Column Shear Strain, BSE-2N

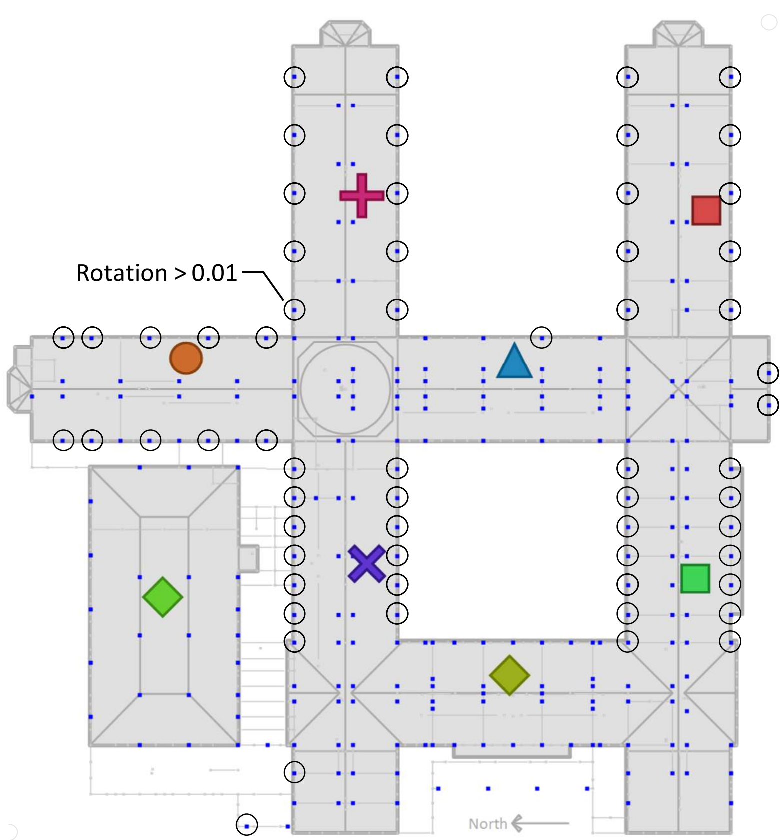


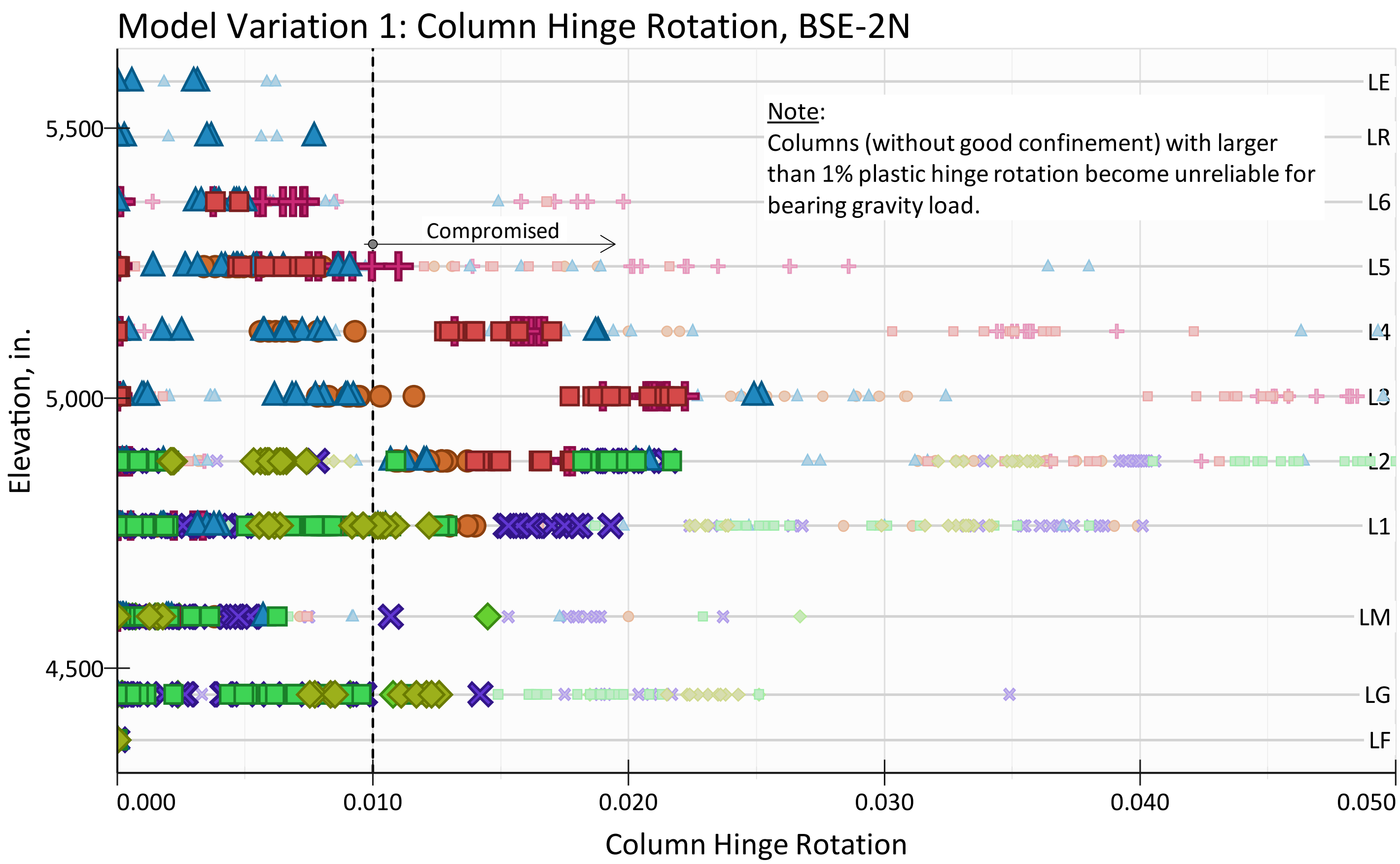


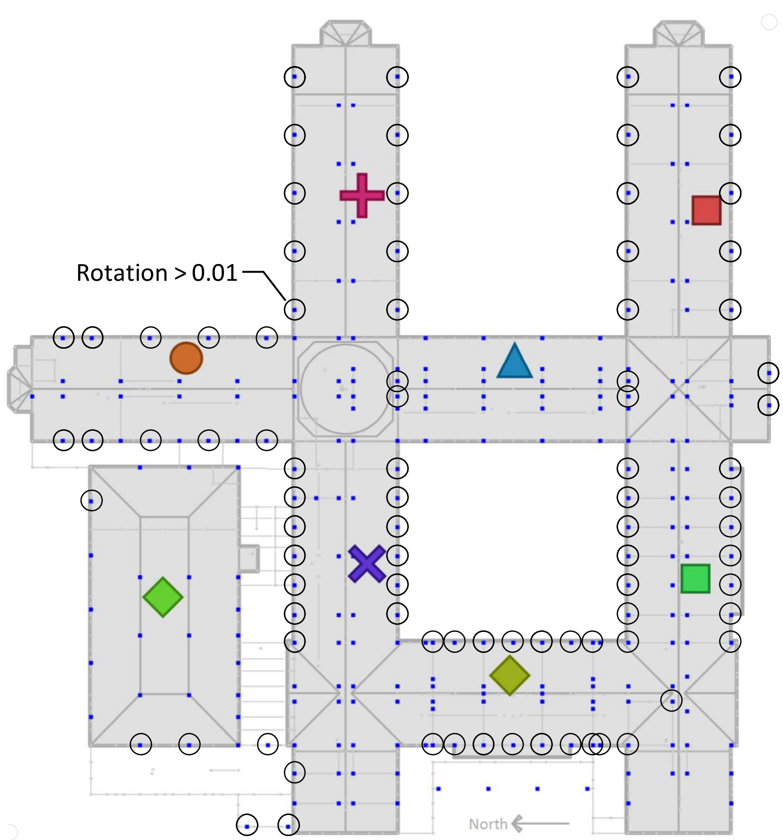

Column Hinge Rotation, BSE-1N
Elevation, in.
5,500
5,000
4,500
LE
LR
L6
L5
L4
L3
L2
L1
LM
LG
LF
Compromised
Note:
Columns (without good confinement) with larger than 1% plastic hinge rotation become unreliable for bearing gravity load.
0.000
0.010
0.020
0.030
0.040
0.050
Column Hinge Rotation


Rotation > 0.01
North

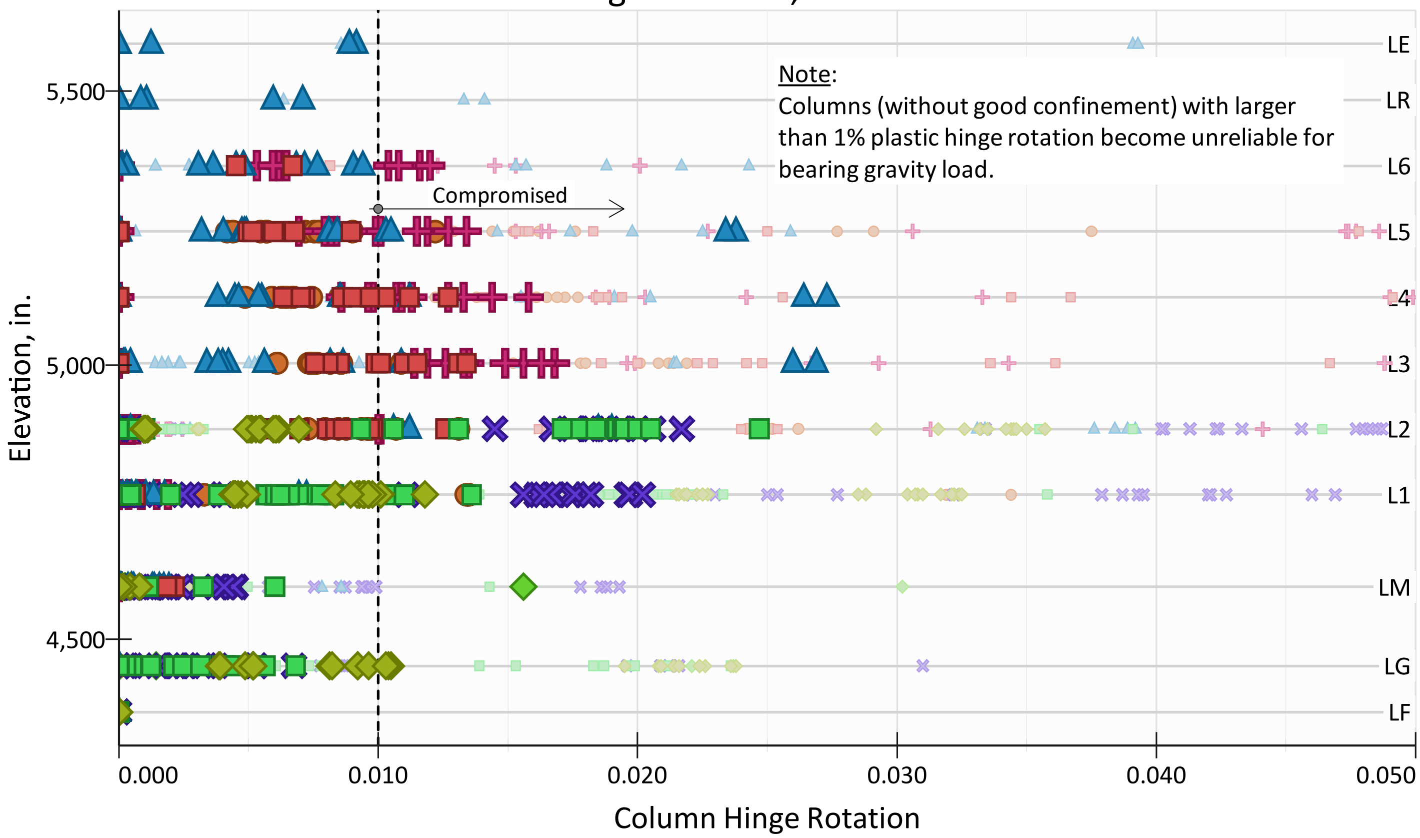

Column Hinge Rotation, BSE-2N
Elevation, in.
5,500
5,000
4,500
LE
LR
L6
L5
L4
L3
L2
L1
LM
LG
LF
Note:
Columns (without good confinement) with larger than 1% plastic hinge rotation become unreliable for bearing gravity load.
Compromised
0.000
0.010
0.020
0.030
0.040
0.050
Column Hinge Rotation

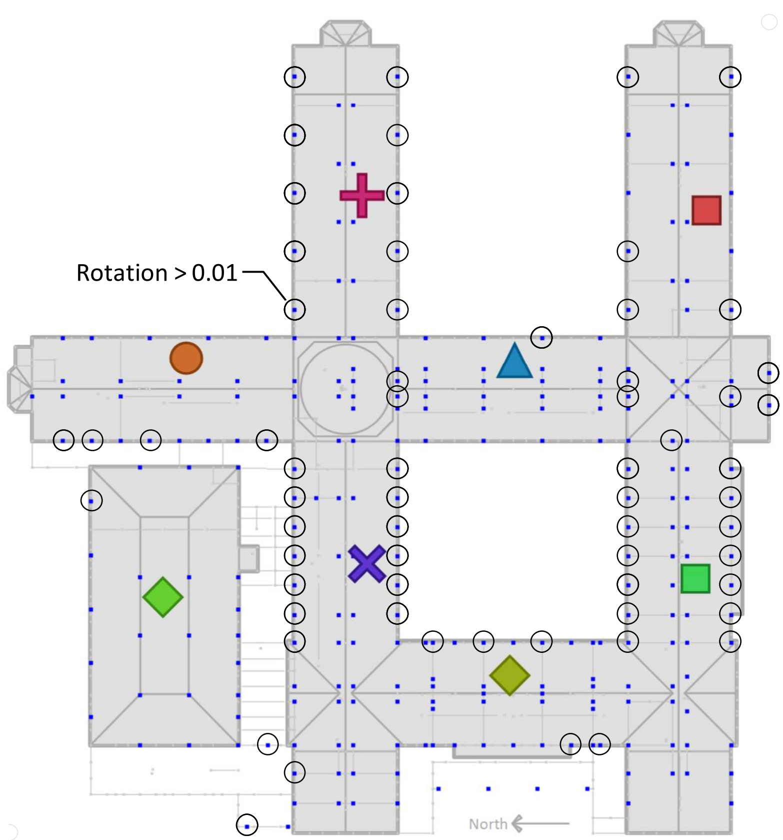

Rotation > 0.01
North

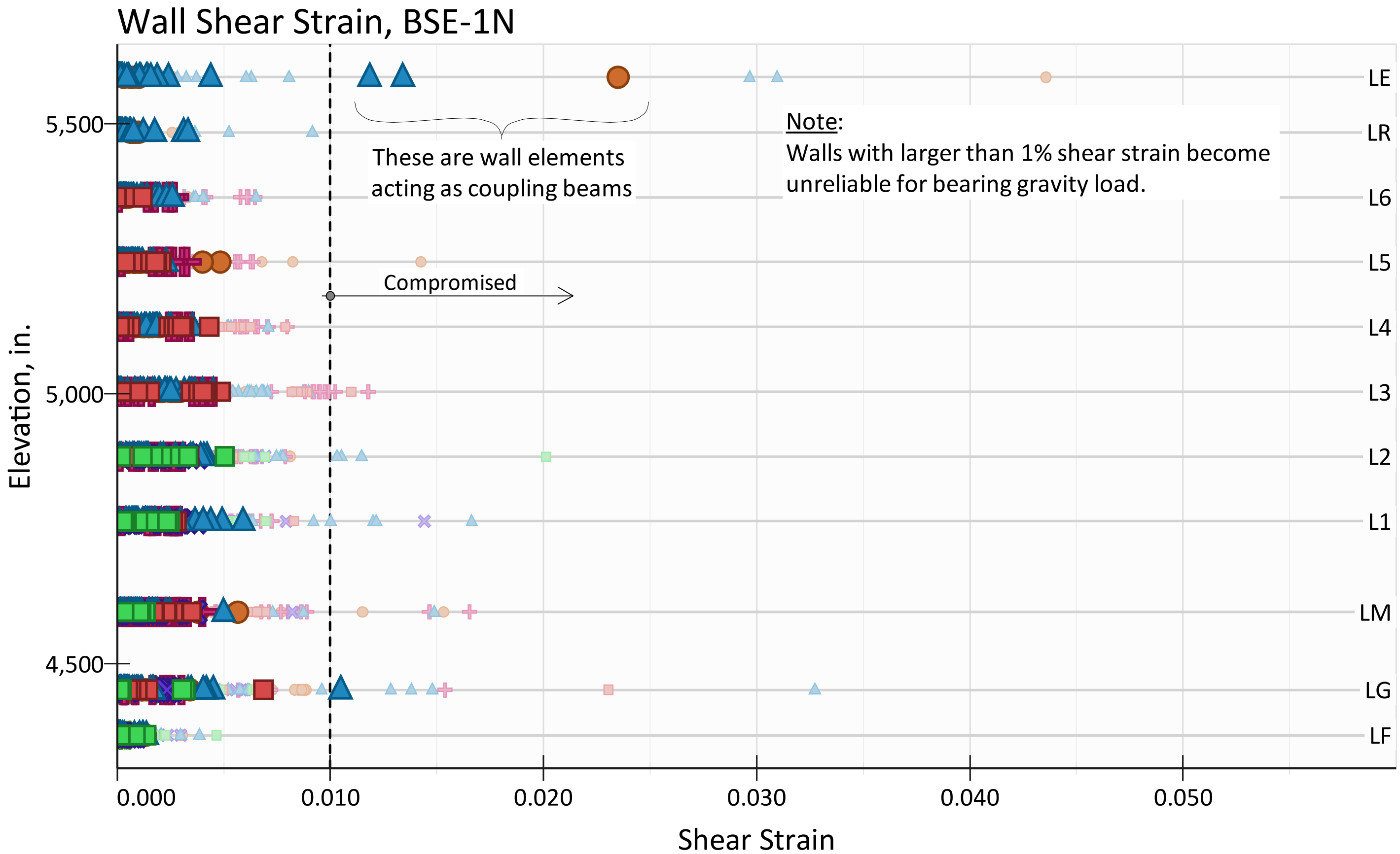

Model Variation 1: Column Hinge Rotation, BSE-2N
Note:
Columns (without good confinement) with larger than 1% plastic hinge rotation become unreliable for bearing gravity load.
Compromised
LE
LR
L6
L5
L4
L3
L2
L1
LM
LG
LF
5,500
5,000
4,500
Elevation, in.
0.000
0.010
0.020
0.030
0.040
0.050
Column Hinge Rotation

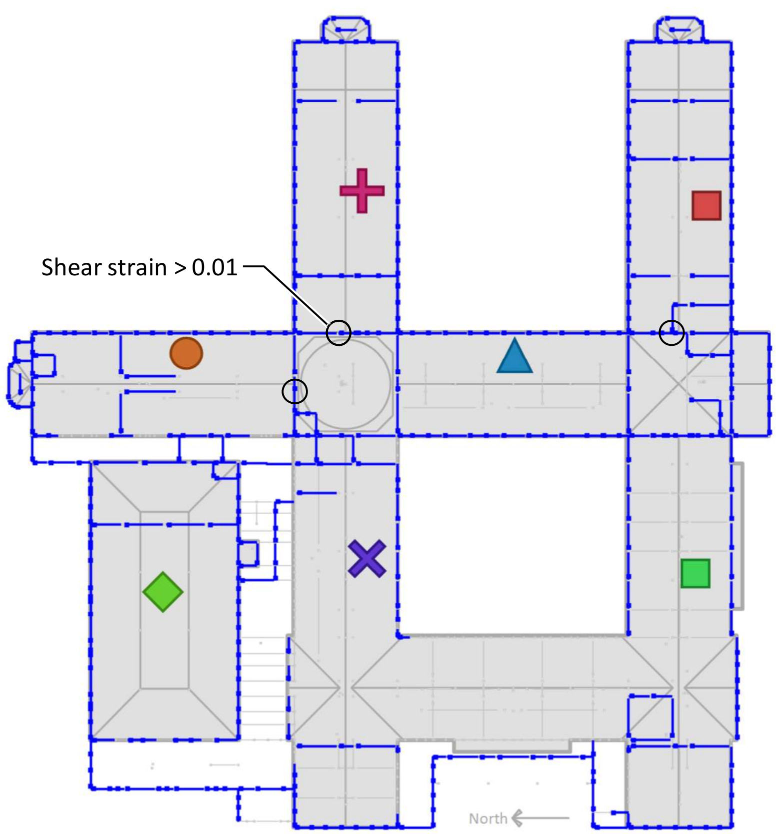

Rotation > 0.01
North

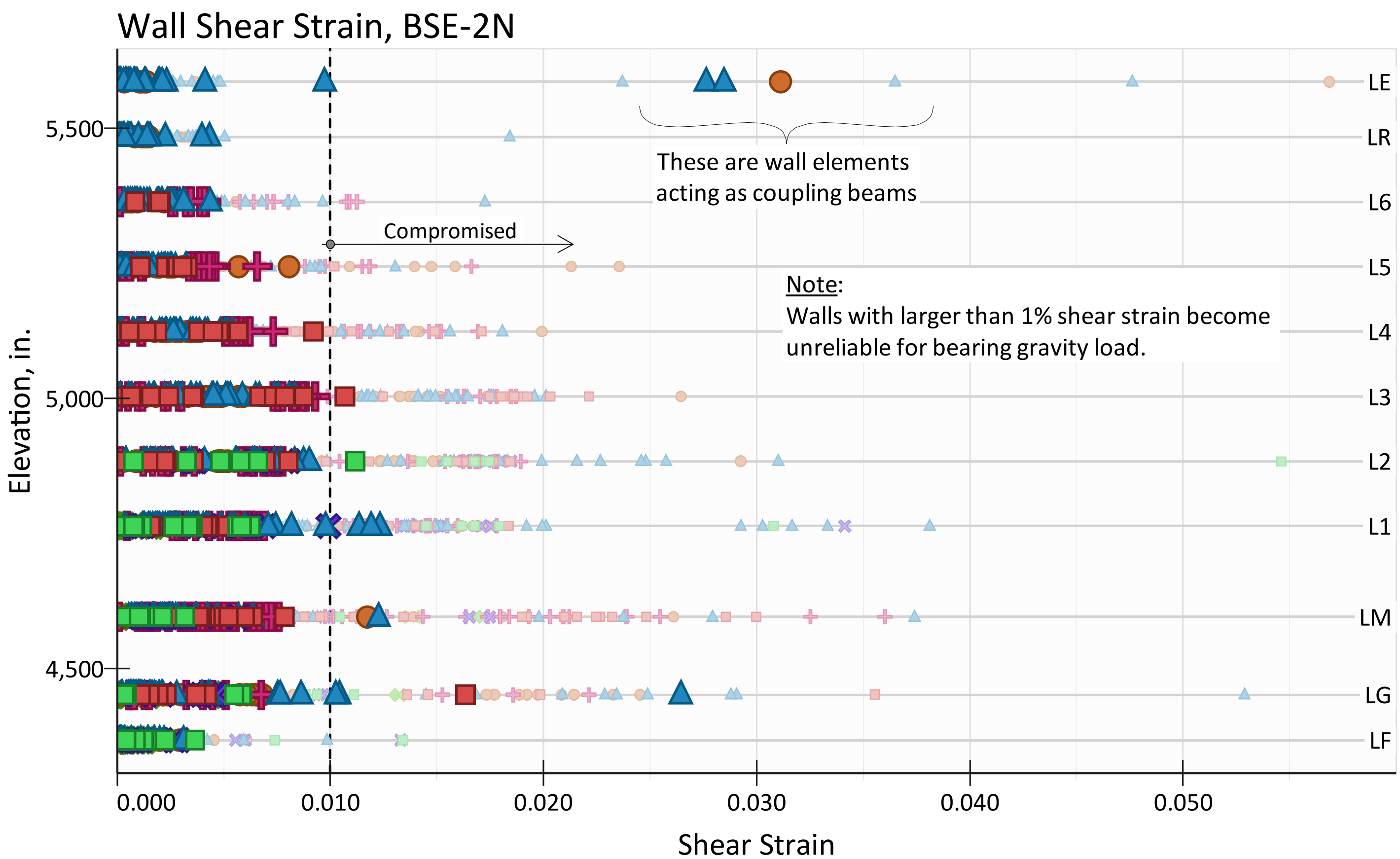
Model Variation 2: Column Hinge Rotation, BSE-2N

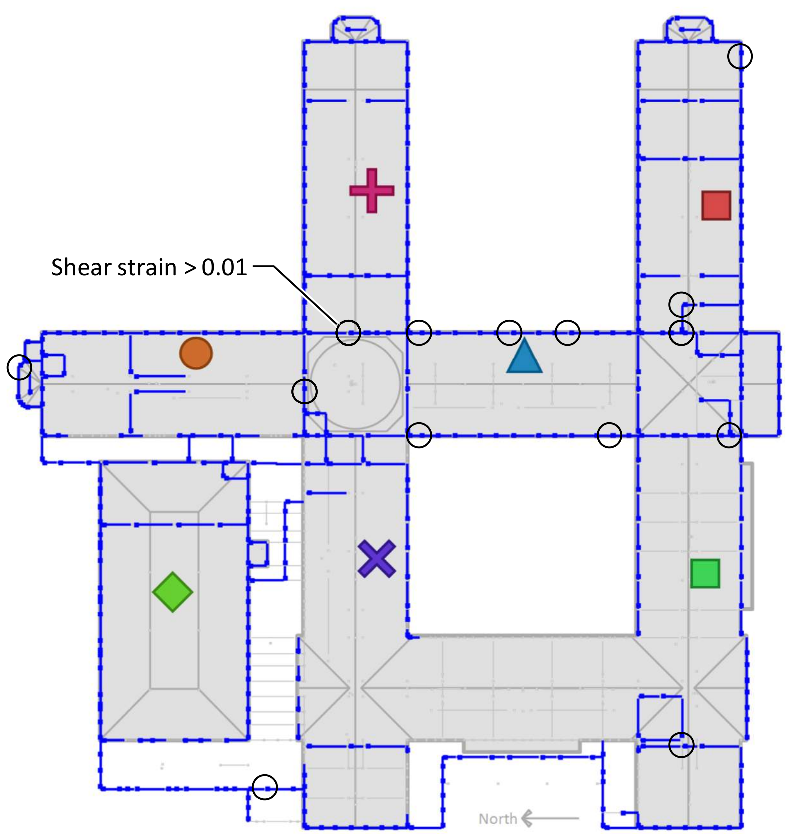


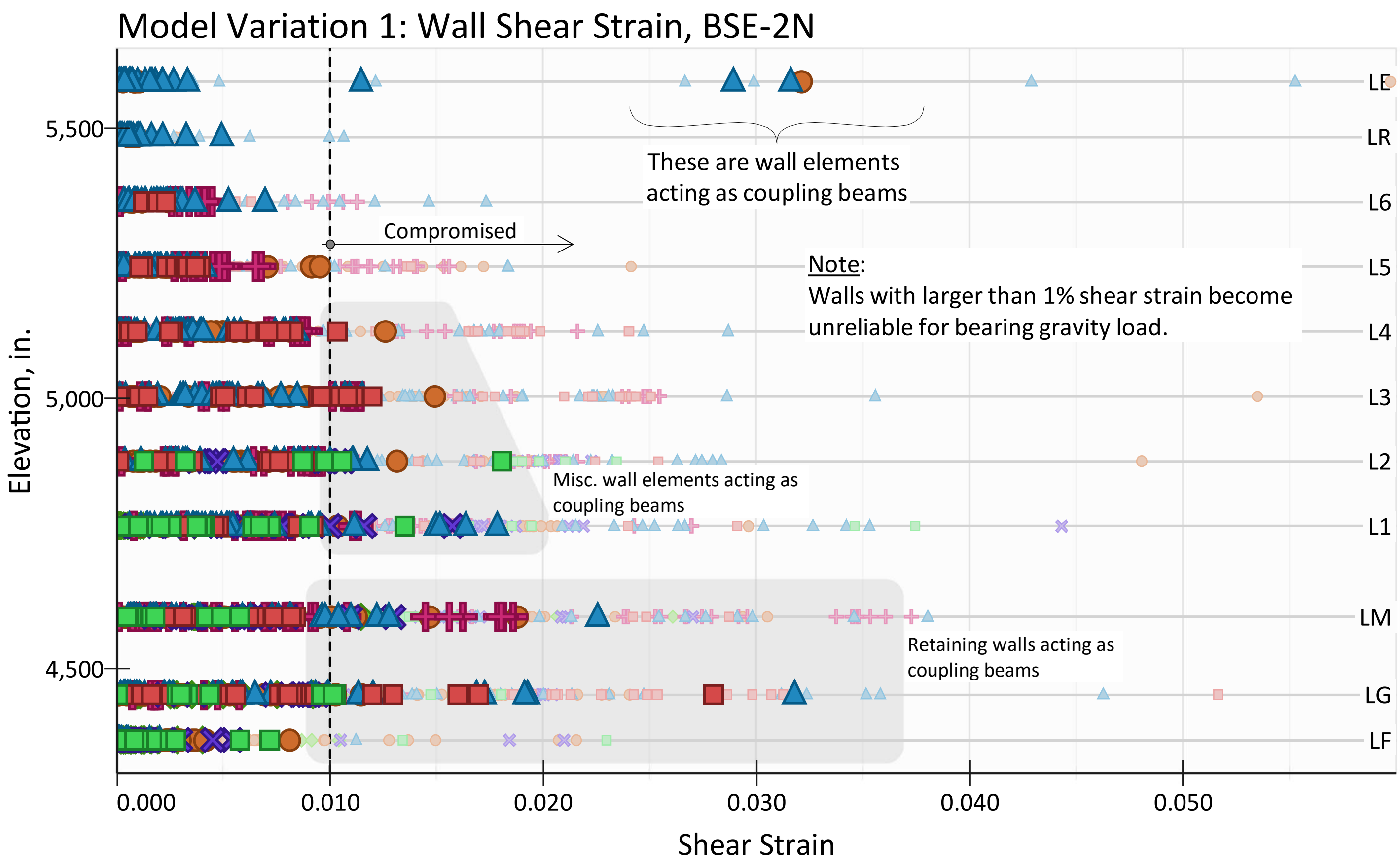


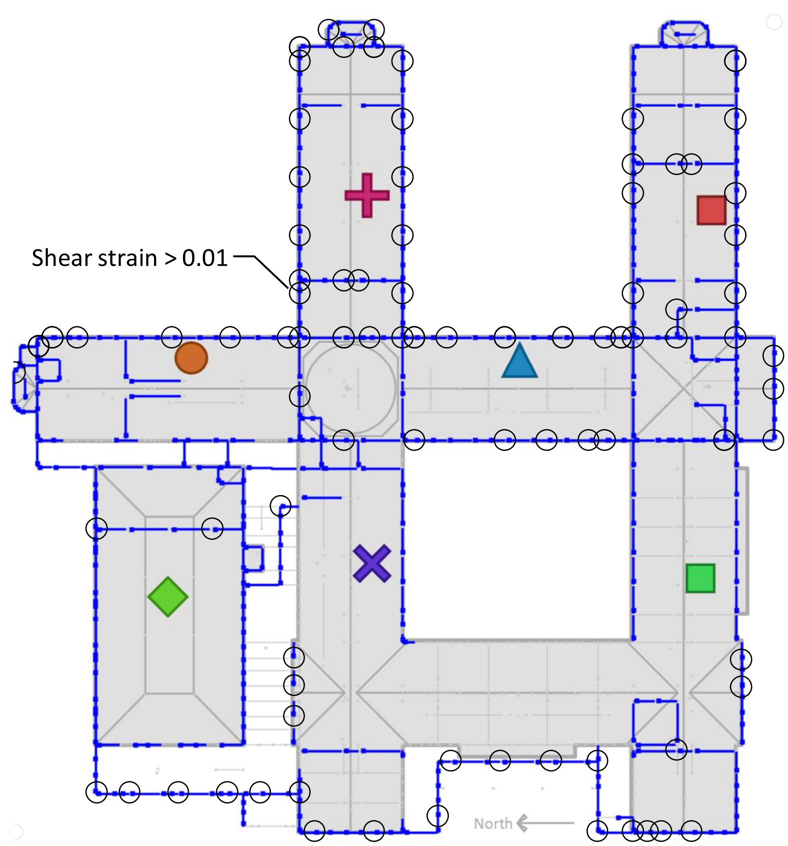

Wall Shear Strain, BSE-1N
Elevation, in.
5,500
5,000
4,500
LE
LR
L6
L5
L4
L3
L2
L1
LM
LG
LF
These are wall elements acting as coupling beams
Note:
Walls with larger than 1% shear strain become unreliable for bearing gravity load.
Compromised
0.000
0.010
0.020
0.030
0.040
0.050
Shear Strain

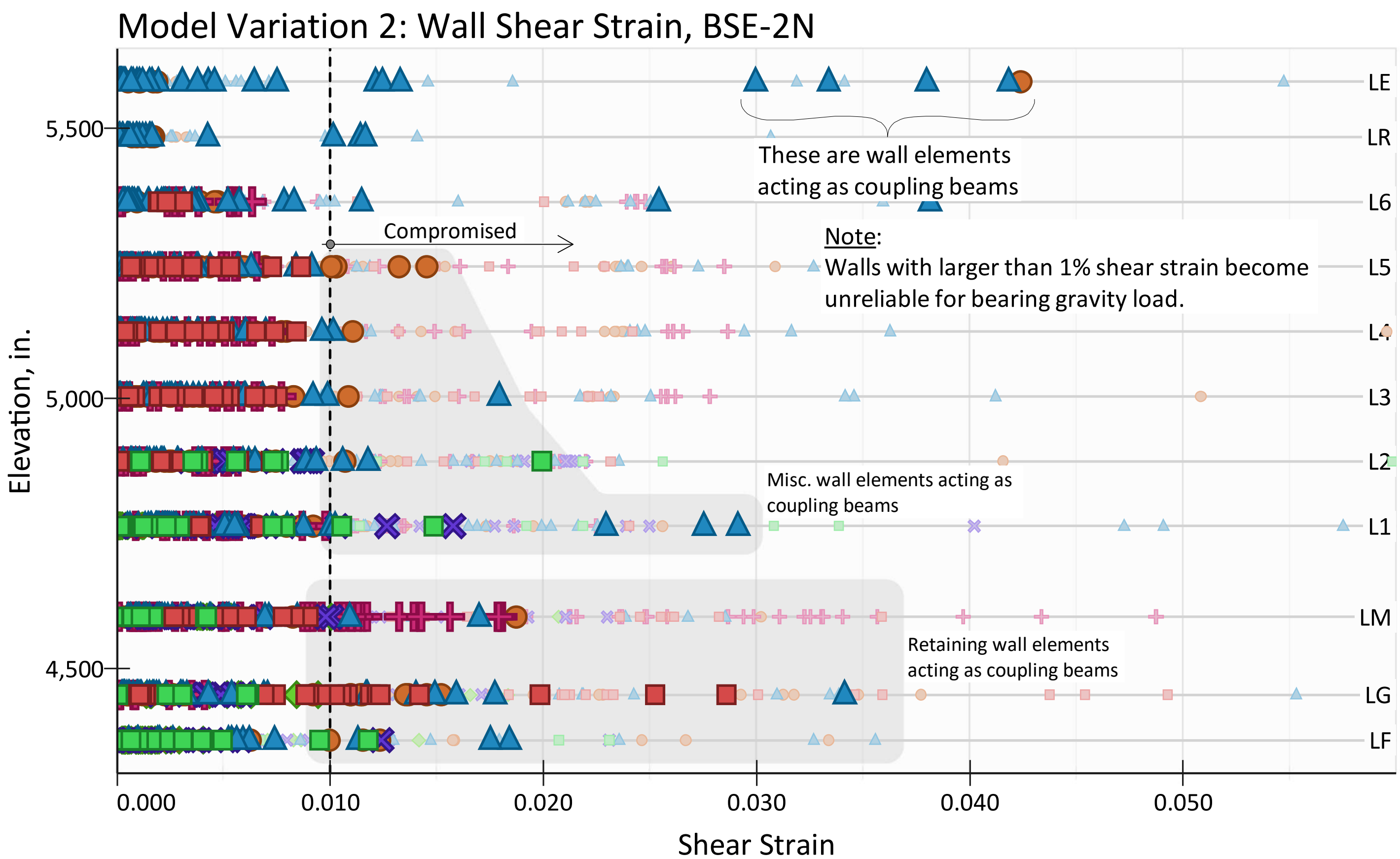

Shear strain > 0.01
North

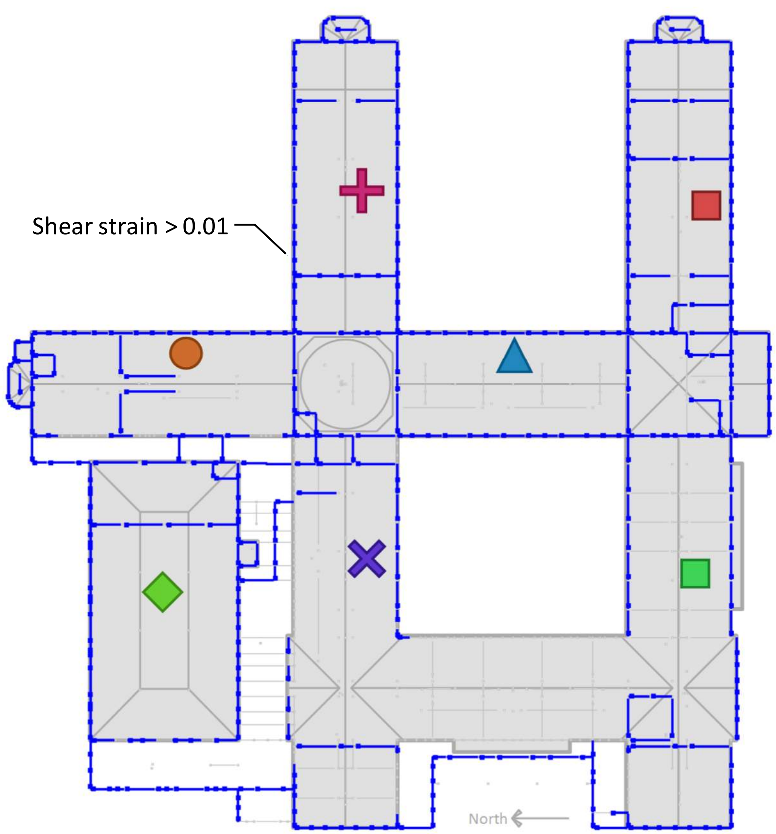

Wall Shear Strain, BSE-2N
Elevation, in.
5,500
5,000
4,500
Shear Strain
0.000
0.010
0.020
0.030
0.040
0.050
LE
LR
L6
L5
L4
L3
L2
L1
LM
LG
LF
These are wall elements acting as coupling beams
Compromised
Note:
Walls with larger than 1% shear strain become unreliable for bearing gravity load.

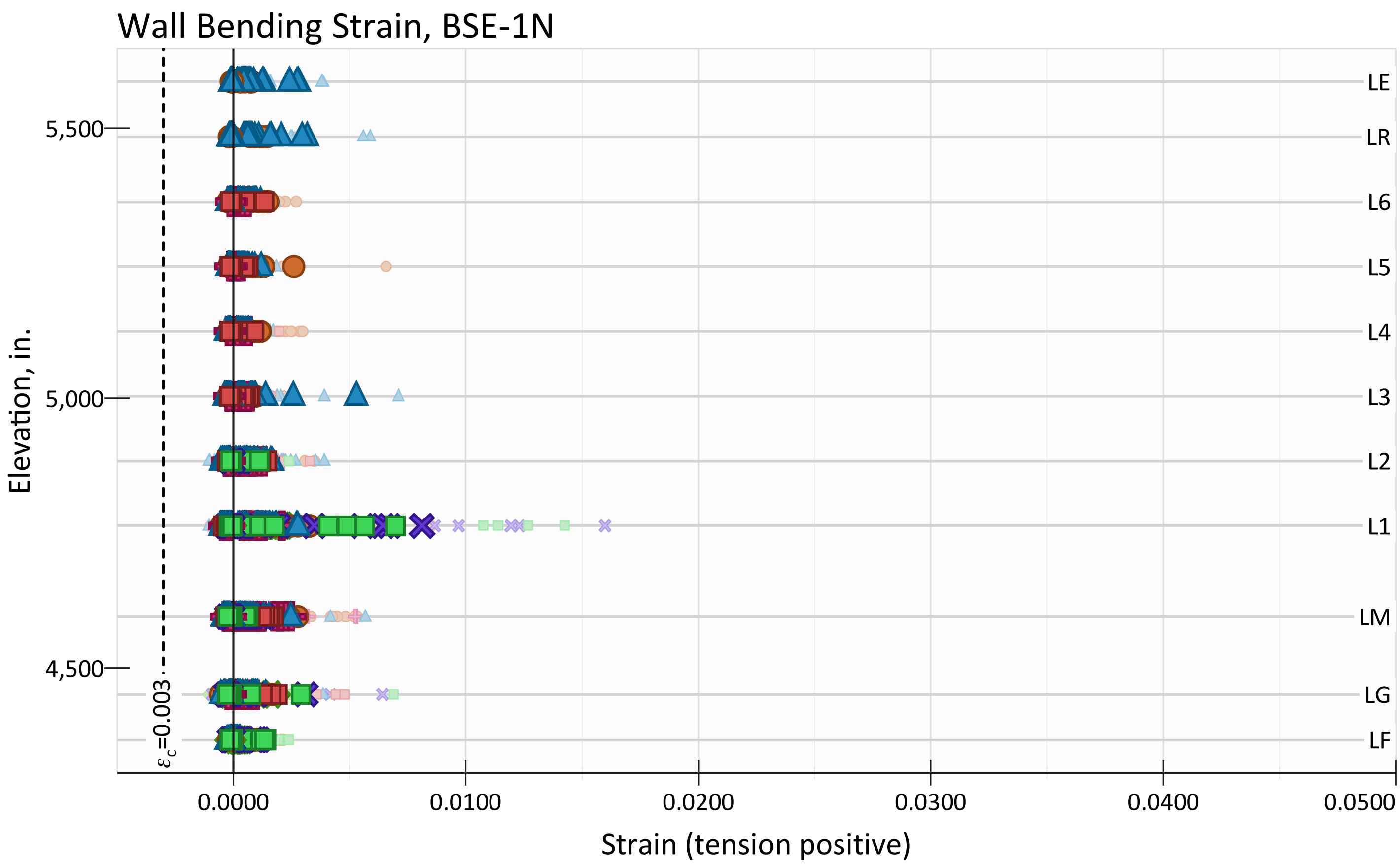

Shear strain > 0.01
North

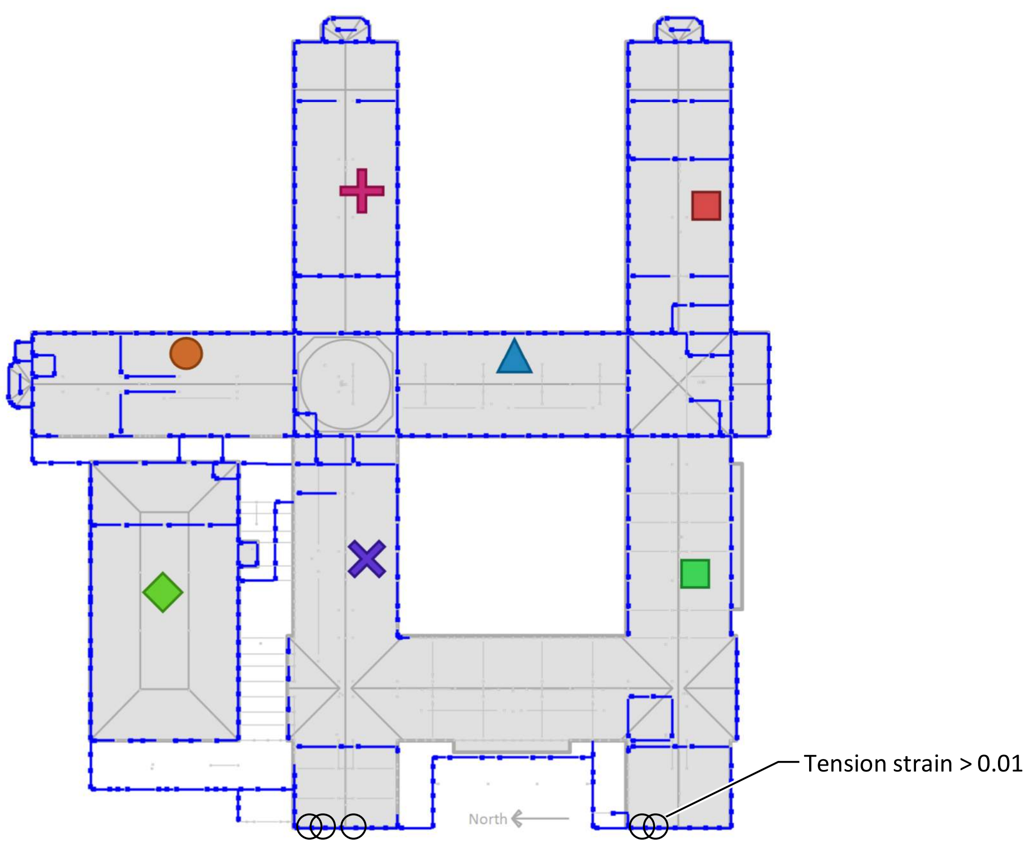

Model Variation 1: Wall Shear Strain, BSE-2N
Elevation, in.
5,500
5,000
4,500
LE
LR
L6
L5
L4
L3
L2
L1
LM
LG
LF
These are wall elements acting as coupling beams
Compromised
Note:
Walls with larger than 1% shear strain become unreliable for bearing gravity load.
Misc. wall elements acting as coupling beams
Retaining walls acting as coupling beams
0.000
0.010
0.020
0.030
0.040
0.050
Shear Strain

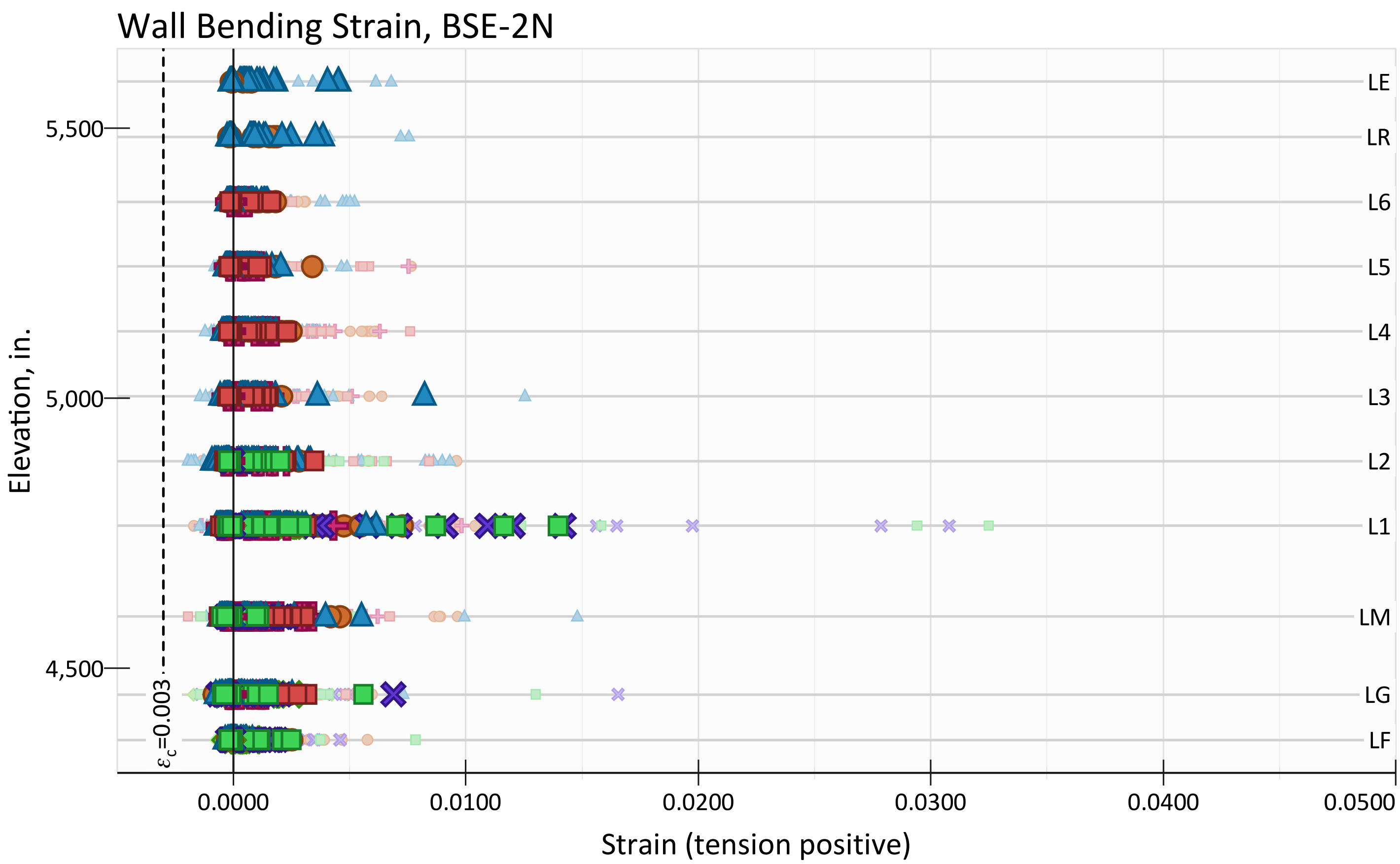

Shear strain > 0.01
North

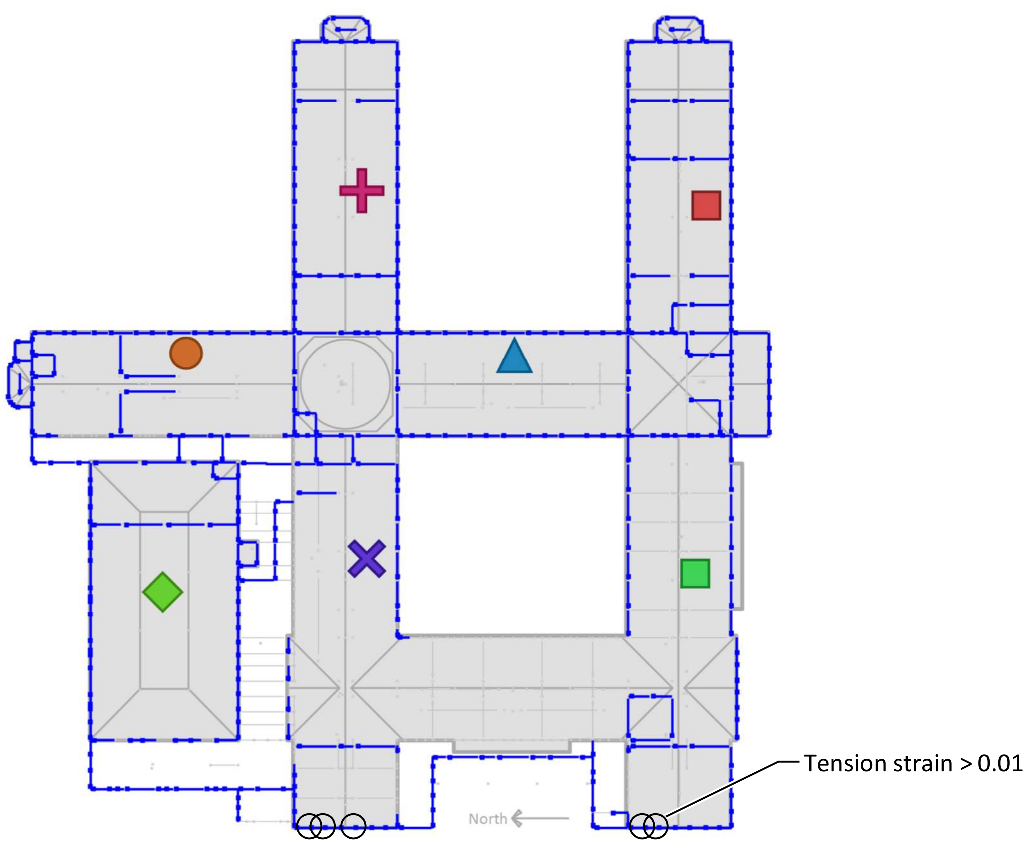

Model Variation 2: Wall Shear Strain, BSE-2N
Elevation, in.
5,500
5,000
4,500
Shear Strain
0.000
0.010
0.020
0.030
0.040
0.050
LE
LR
L6
L5
L4
L3
L2
L1
LM
LG
LF
These are wall elements acting as coupling beams
Compromised
Note:
Walls with larger than 1% shear strain become unreliable for bearing gravity load.
Misc. wall elements acting as coupling beams
Retaining wall elements acting as coupling beams

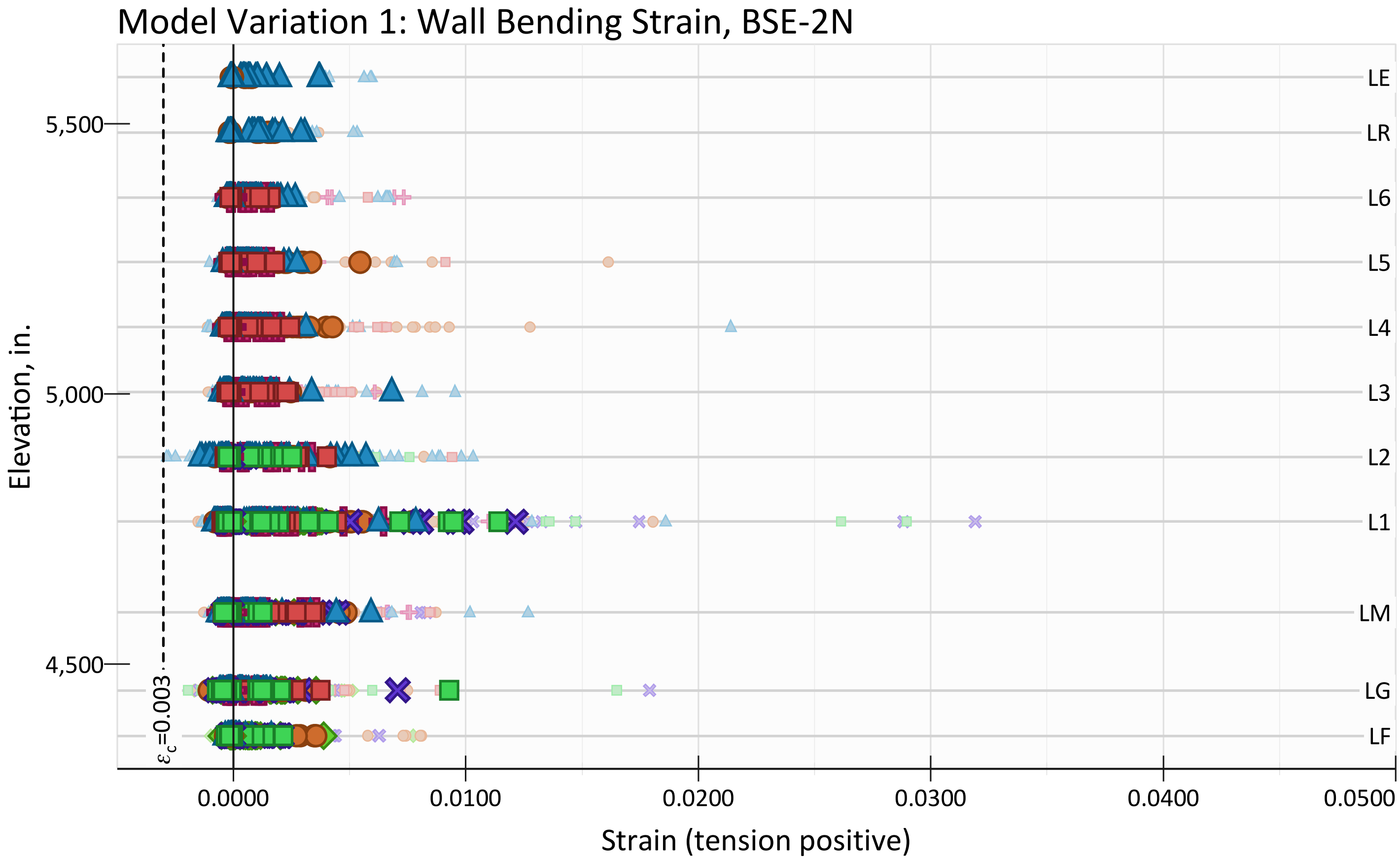

Shear strain > 0.01
North

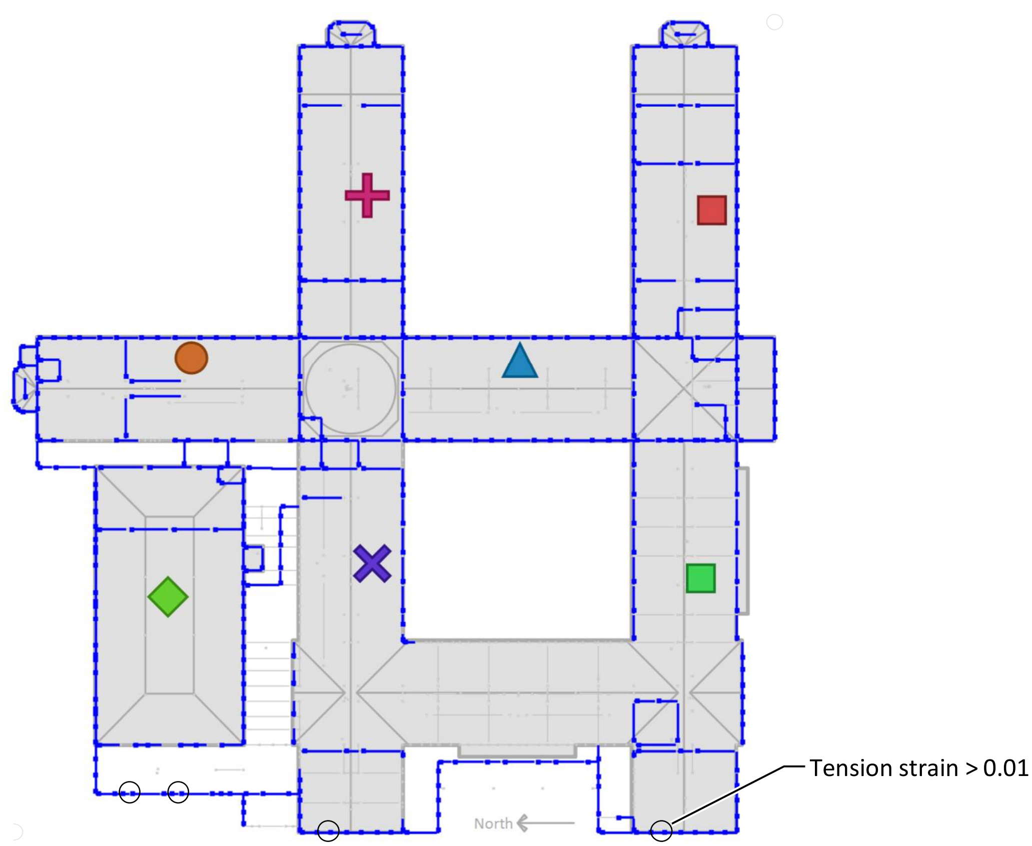

Wall Bending Strain, BSE-1N
Elevation, in.
5,500
5,000
4,500
ε_c=0.003
LE
LR
L6
L5
L4
L3
L2
L1
LM
LG
LF
0.0000
0.0100
0.0200
0.0300
0.0400
0.0500
Strain (tension positive)

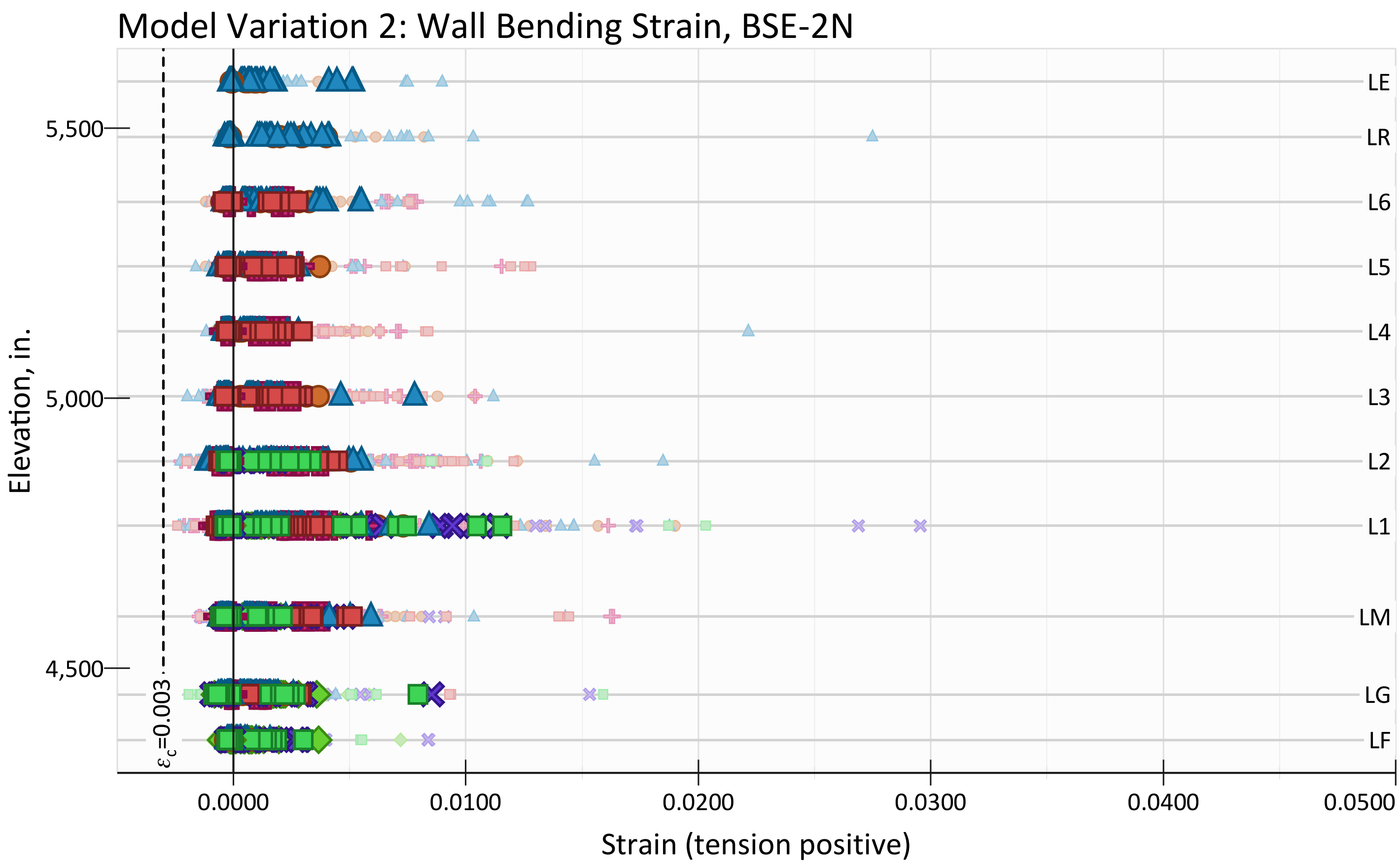

North
Tension strain > 0.01

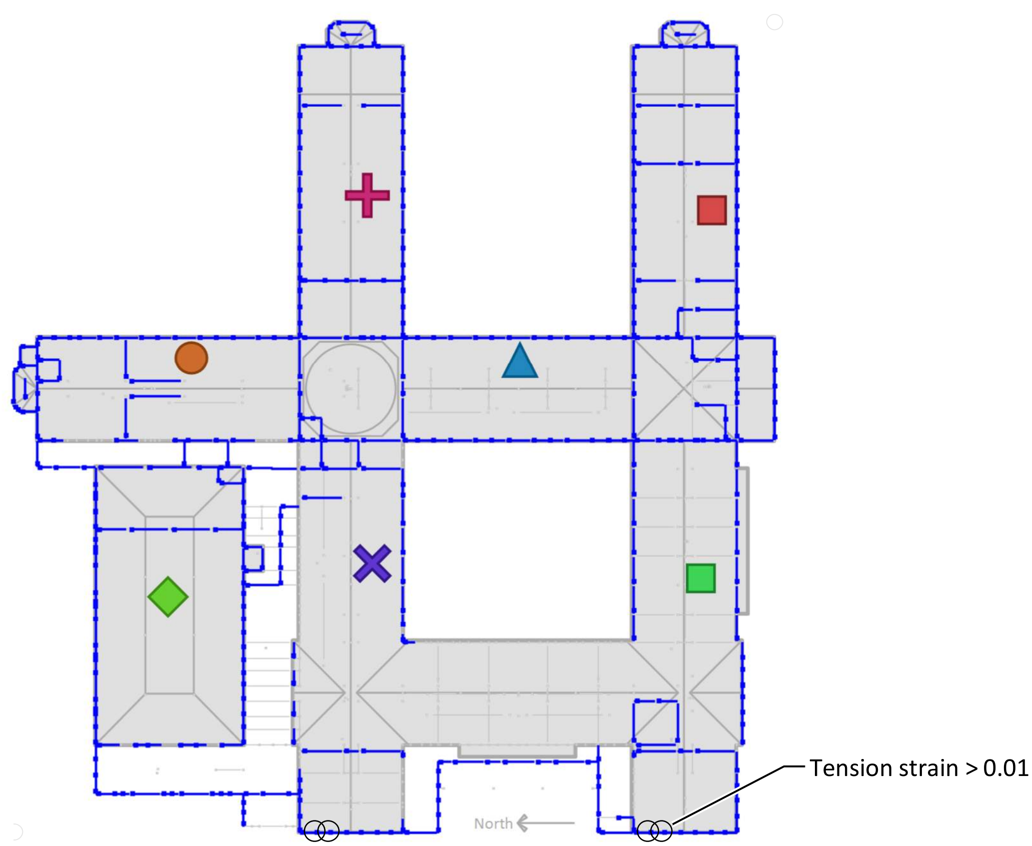

Wall Bending Strain, BSE-2N
Elevation, in.
5,500
5,000
4,500
ε_c=0.003
LE
LR
L6
L5
L4
L3
L2
L1
LM
LG
LF
0.0000
0.0100
0.0200
0.0300
0.0400
0.0500
Strain (tension positive)


Tension strain > 0.01
North

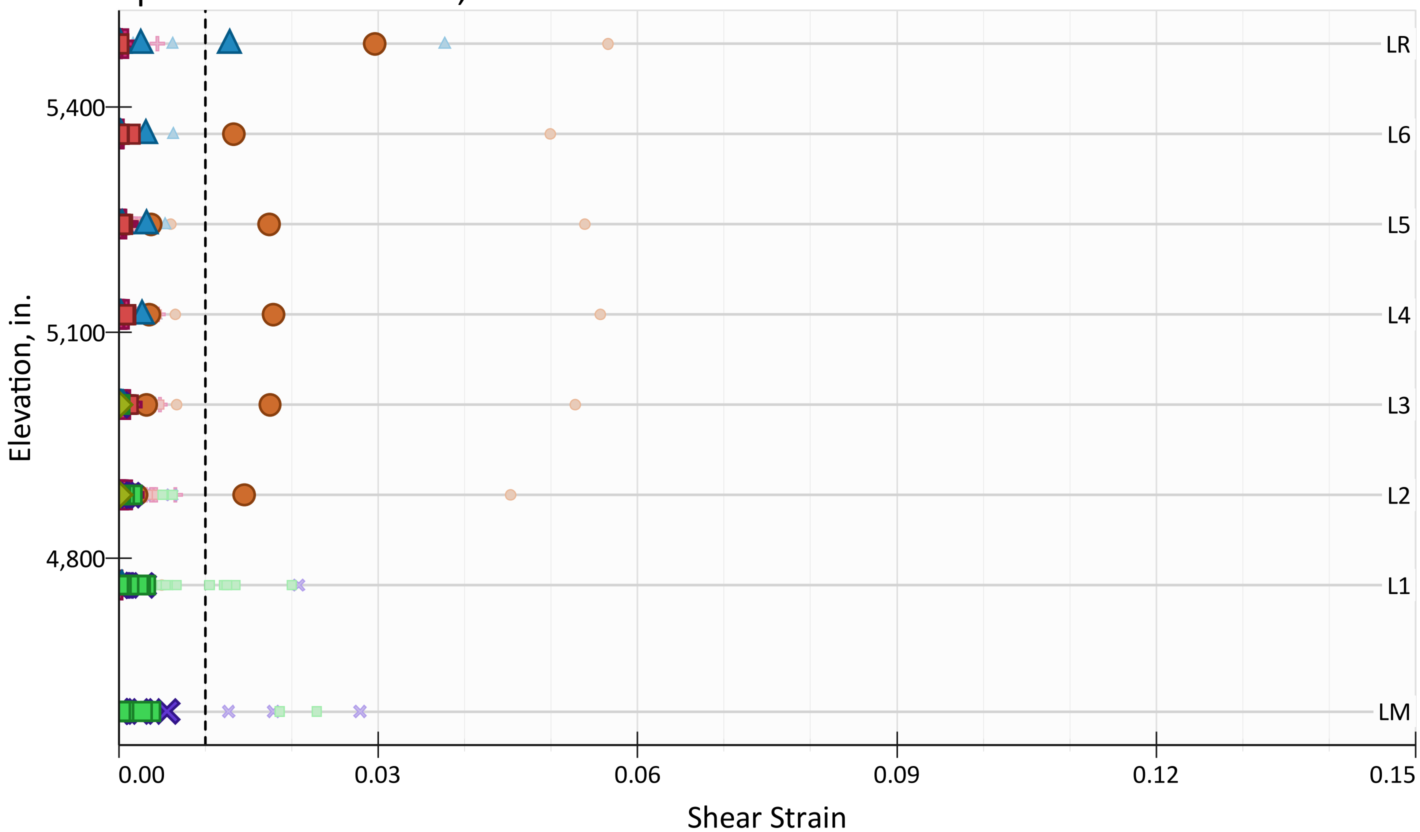

Model Variation 1: Wall Bending Strain, BSE-2N
Elevation, in.
5,500
5,000
4,500
ε_c=0.003
LE
LR
L6
L5
L4
L3
L2
L1
LM
LG
LF
0.0000
0.0100
0.0200
0.0300
0.0400
0.0500
Strain (tension positive)

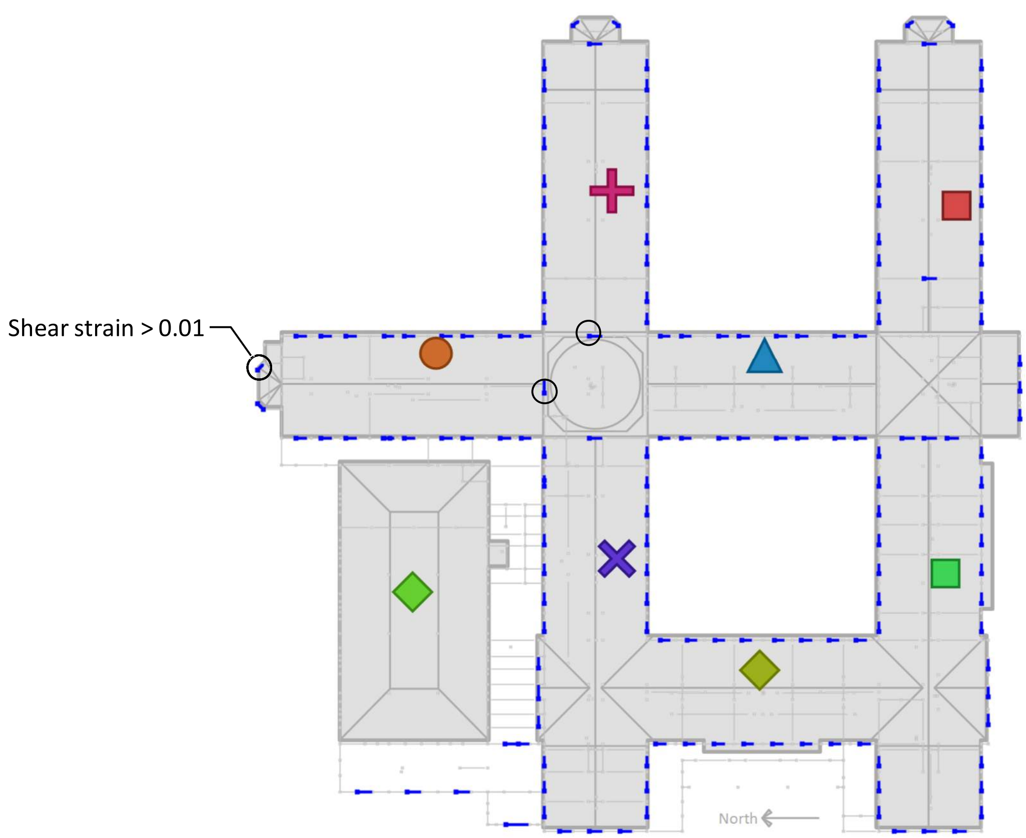

North
Tension strain > 0.01

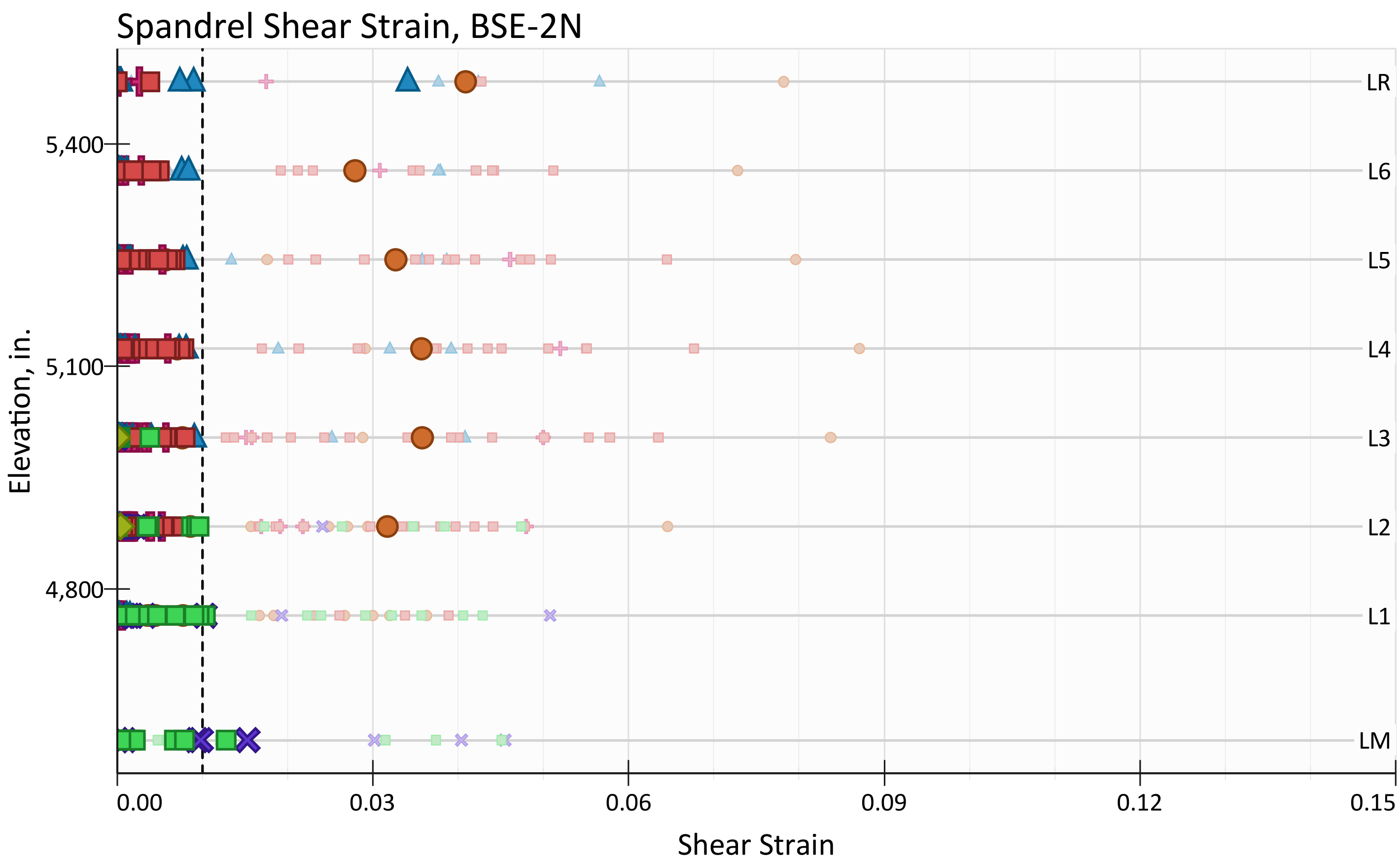

Model Variation 2: Wall Bending Strain, BSE-2N
Elevation, in.
5,500
5,000
4,500
ε_c=0.003
LE
LR
L6
L5
L4
L3
L2
L1
LM
LG
LF
0.0000
0.0100
0.0200
0.0300
0.0400
0.0500
Strain (tension positive)

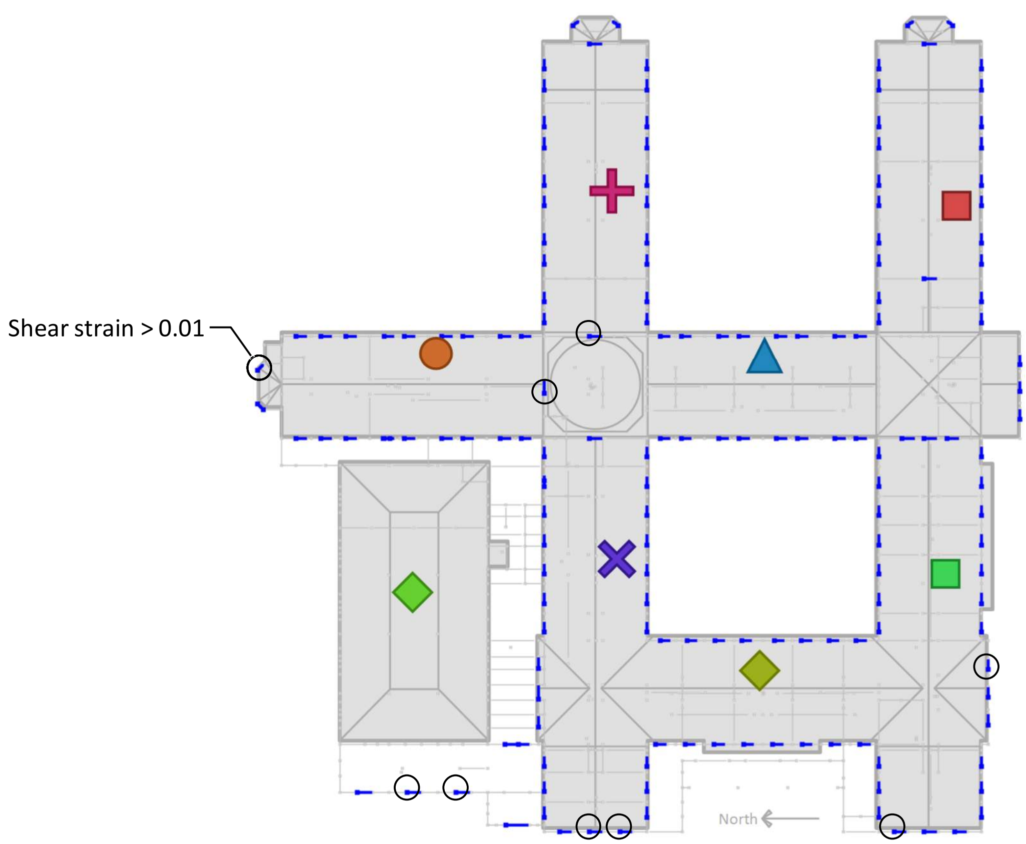

North
Tension strain > 0.01

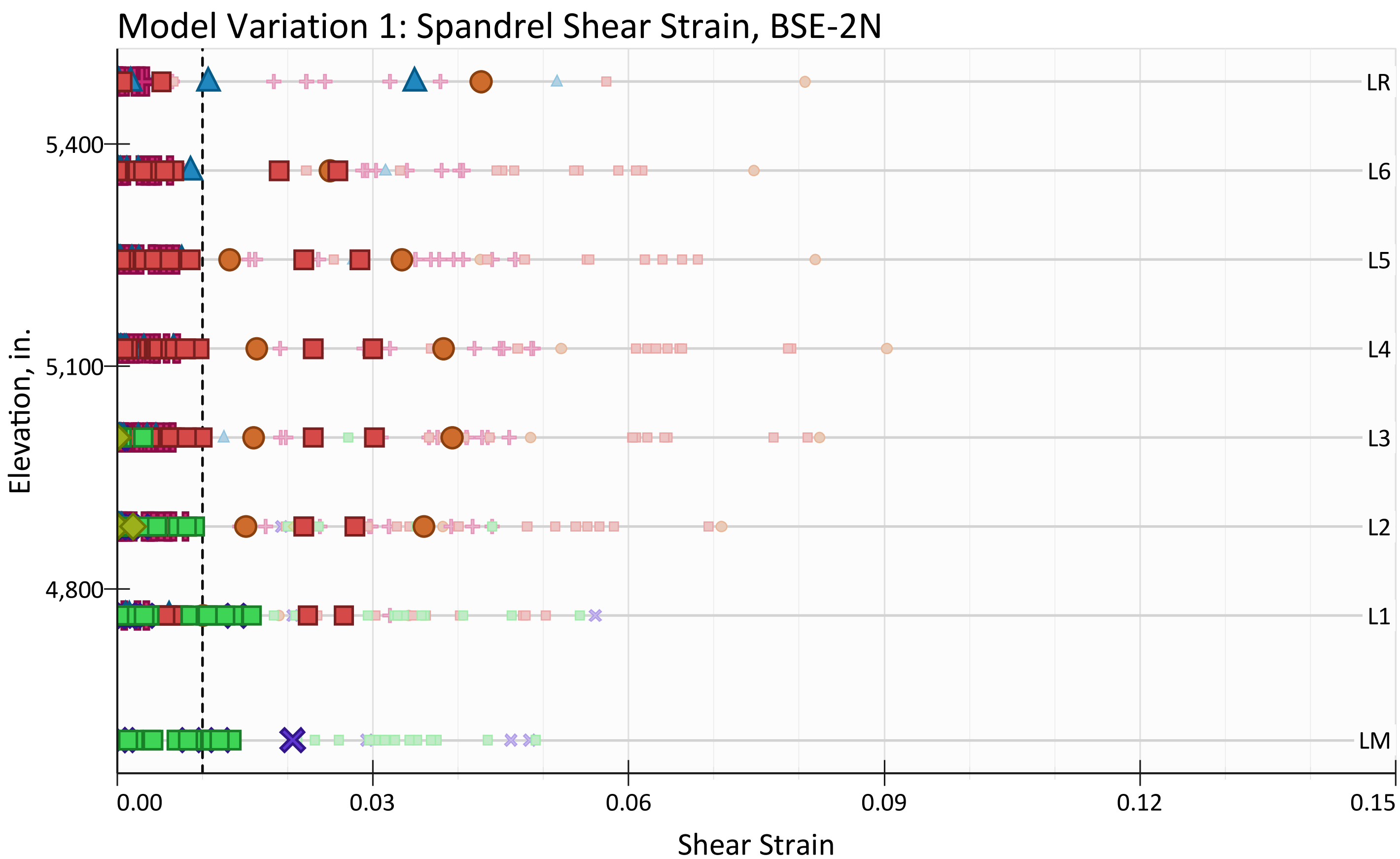
Spandrel Shear Strain, BSE-1N

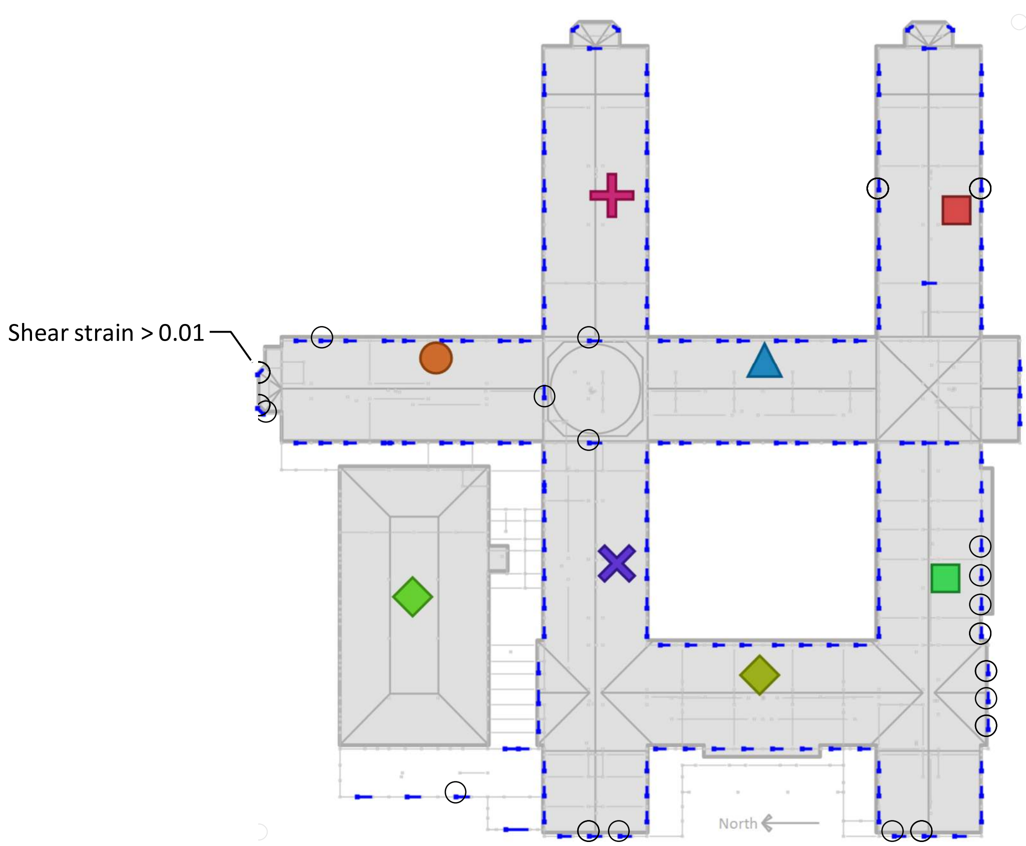


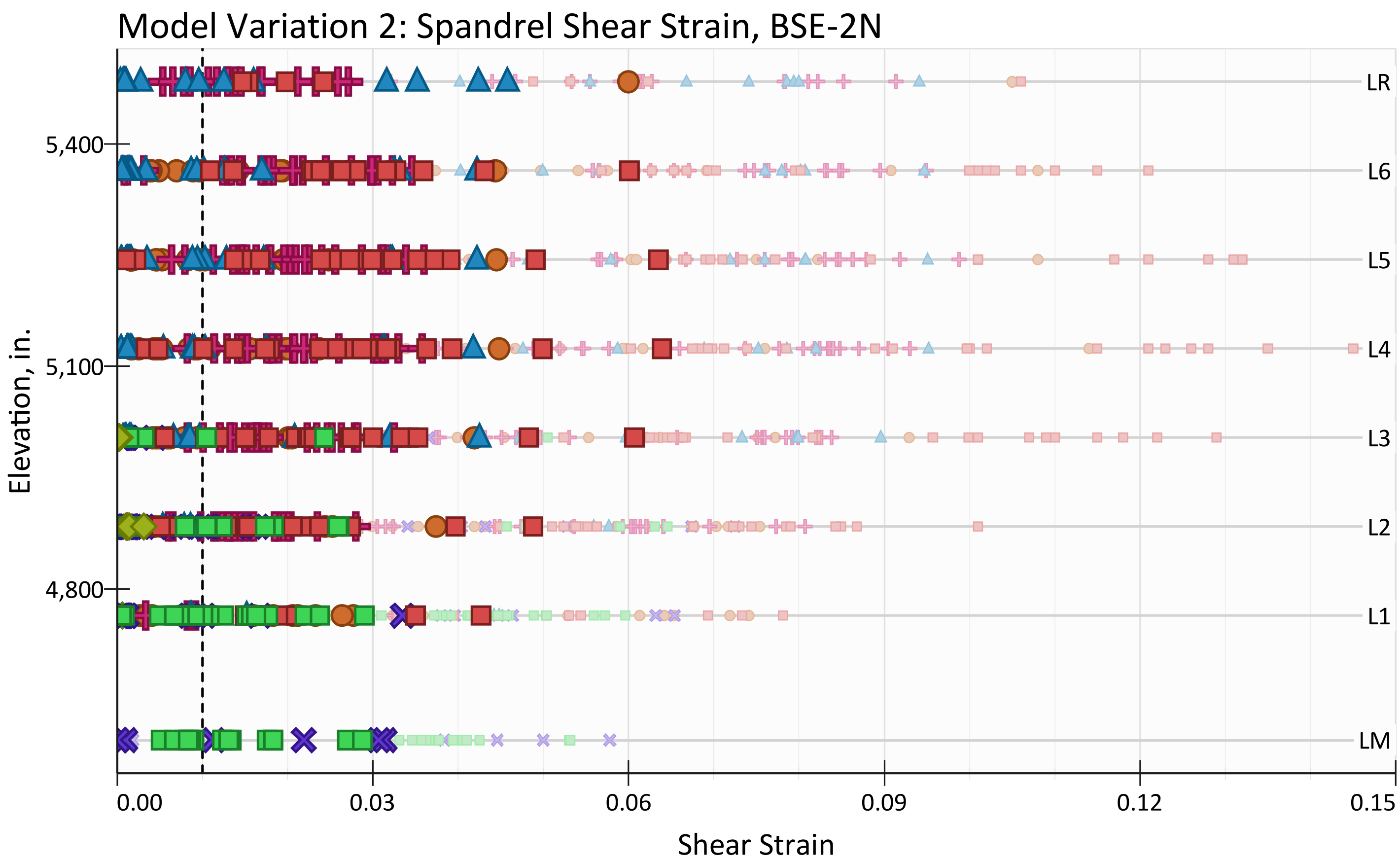


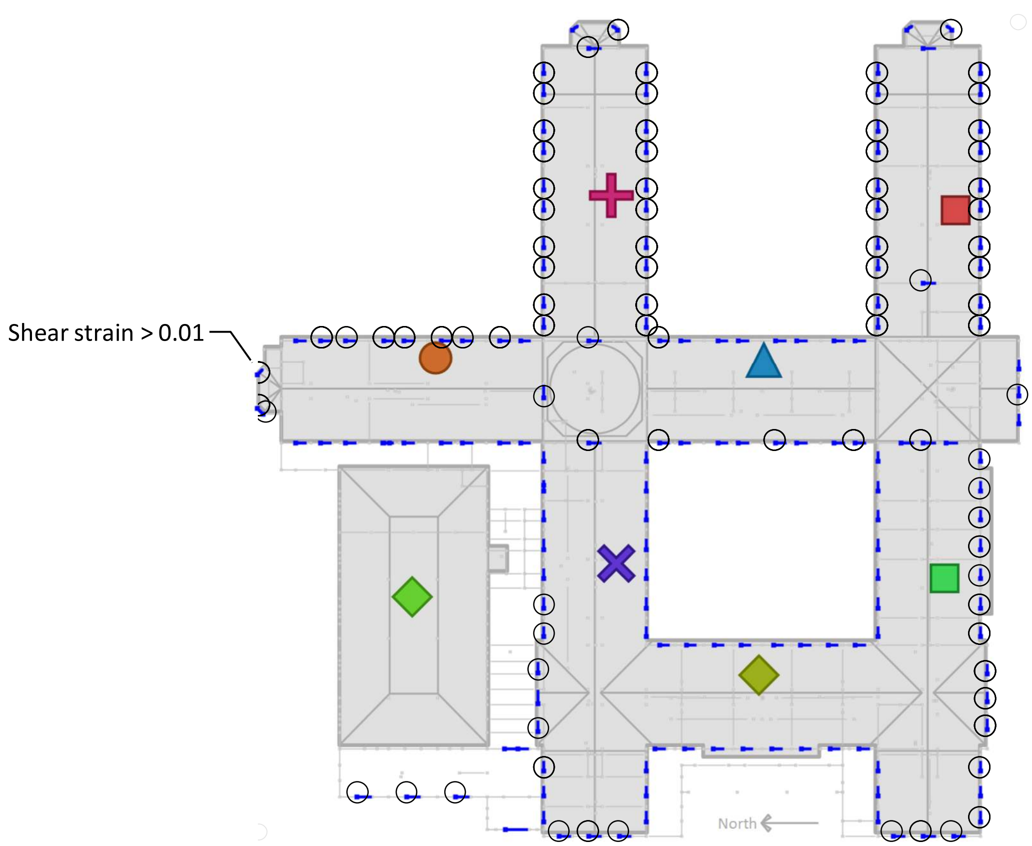

Spandrel Shear Strain, BSE-2N
Elevation, in.
5,400
5,100
4,800
LR
L6
L5
L4
L3
L2
L1
LM
0.00
0.03
0.06
0.09
0.12
0.15
Shear Strain

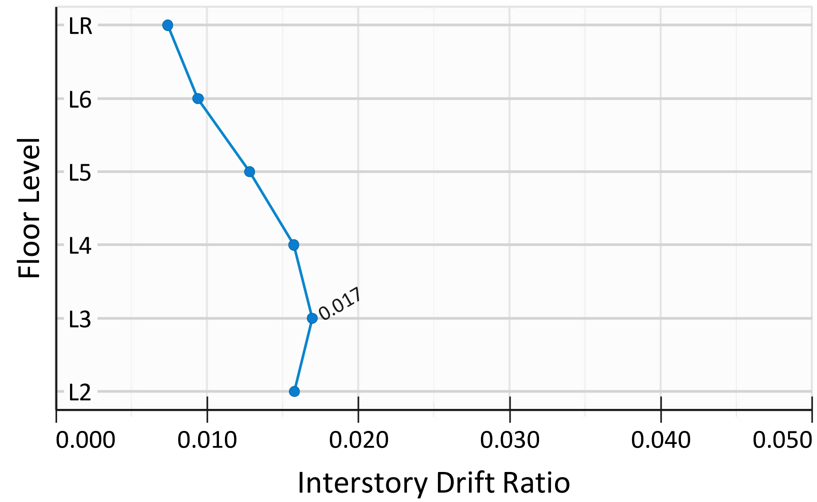

Shear strain > 0.01
North

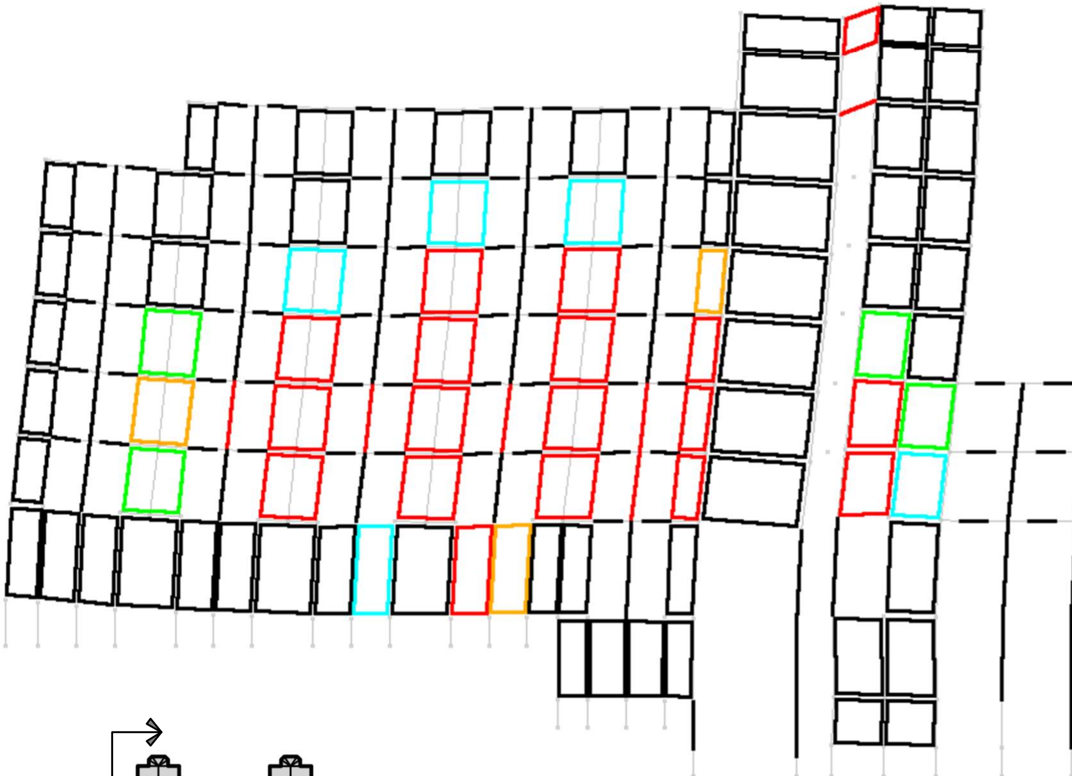
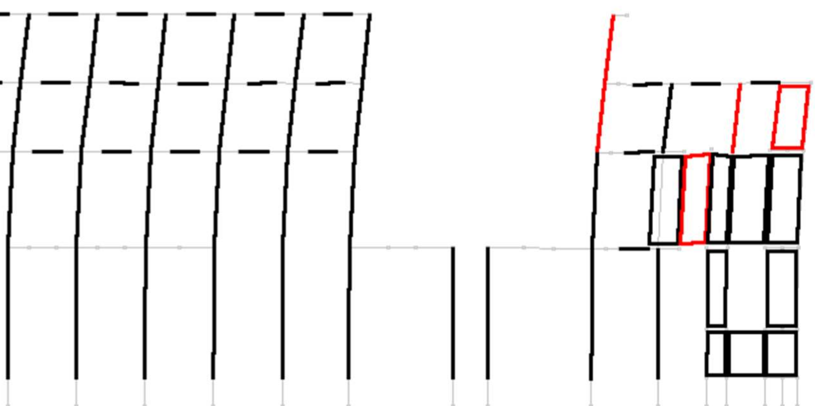

Model Variation 1: Spandrel Shear Strain, BSE-2N
Elevation, in.
5,400
5,100
4,800
LR
L6
L5
L4
L3
L2
L1
LM
0.00
0.03
0.06
0.09
0.12
0.15
Shear Strain

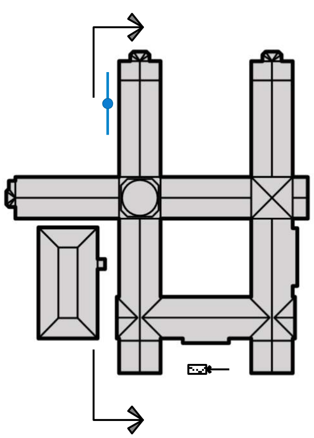

Shear strain > 0.01
North


Model Variation 2: Spandrel Shear Strain, BSE-2N
Elevation, in.
5,400
5,100
4,800
LR
L6
L5
L4
L3
L2
L1
LM
0.00
0.03
0.06
0.09
0.12
0.15
Shear Strain


Shear strain > 0.01
North





Displacement Scale = 10

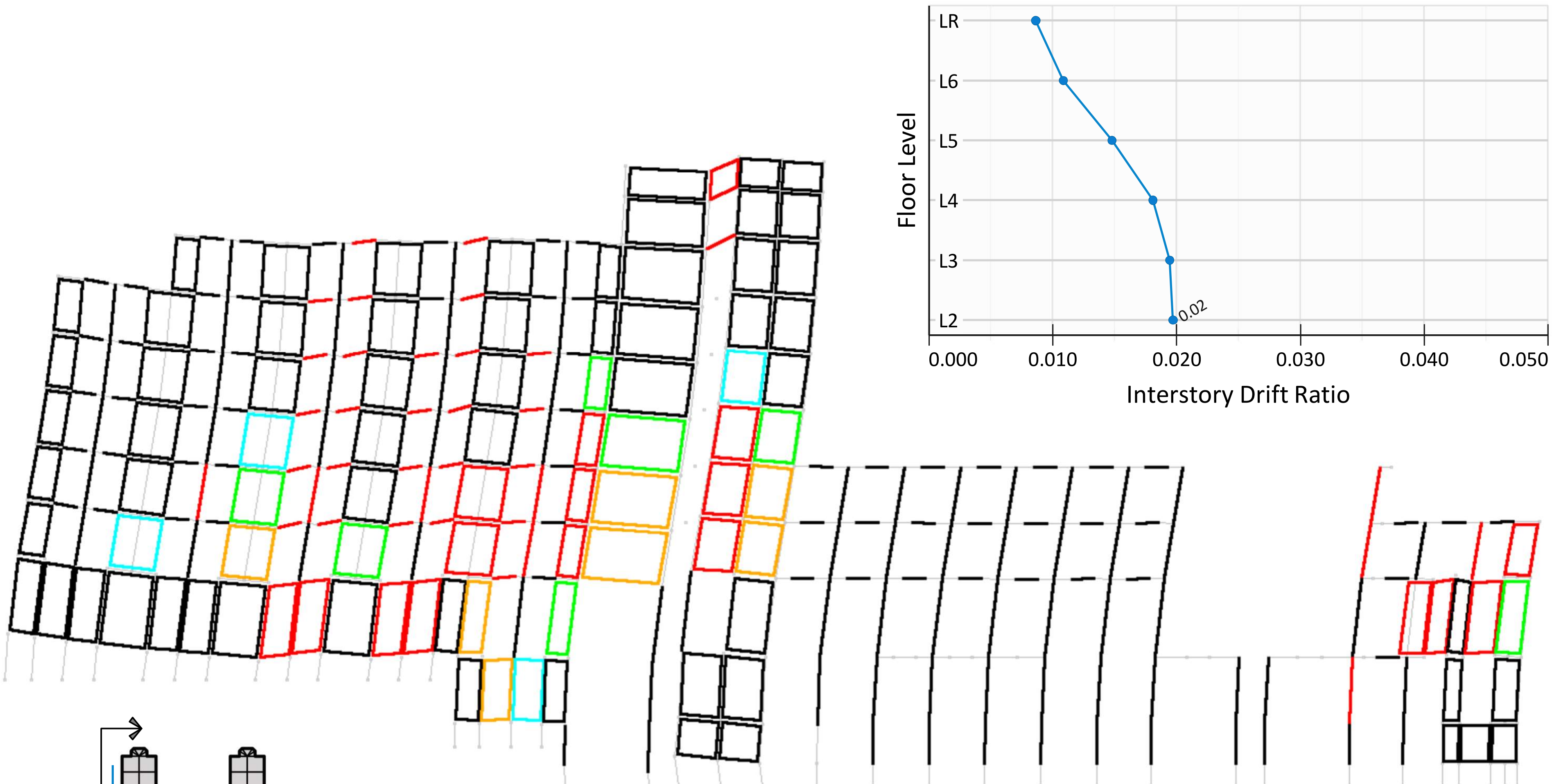
Shear Strain (%): 0.7 0.8 0.9 1

Base Model:

- No horizontal soil springs (fixed horizontally at terraced levels)
- Shear strain at strength loss, $DL = 0.007$ for columns, walls, and spandrels
- Elastic Modulus
  - Columns: $0.5\,E_c$
  - Spandrels: $0.35\,E_c$
  - Walls: $1\,E_c$ (cracking in fiber sections)

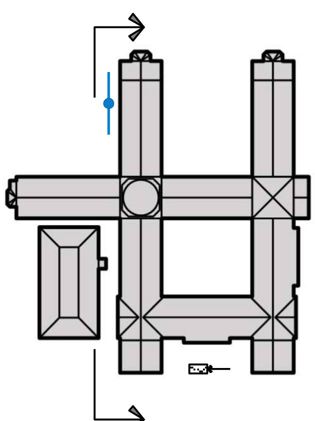
Base Model

BSE-2N Record 5 at time = 5.4 sec



Variation 1 = Base Model plus

- Horizontal Soil Springs
- Shear strain at strength loss, *DL* = 0.003 for columns and walls

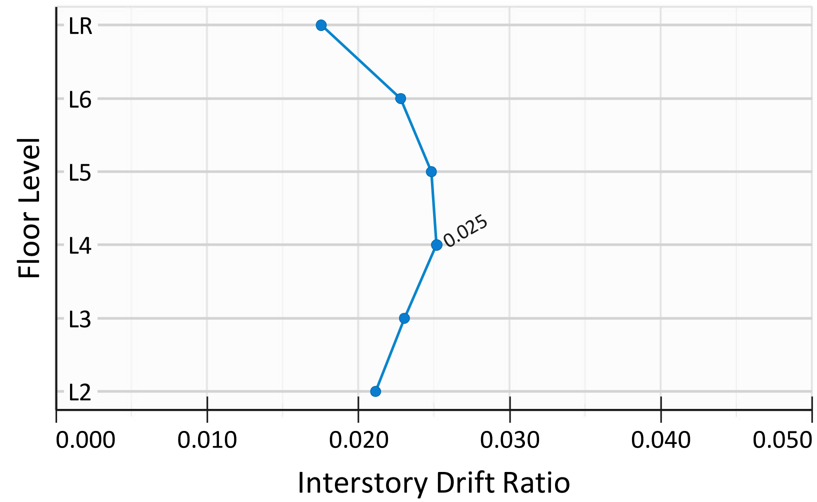


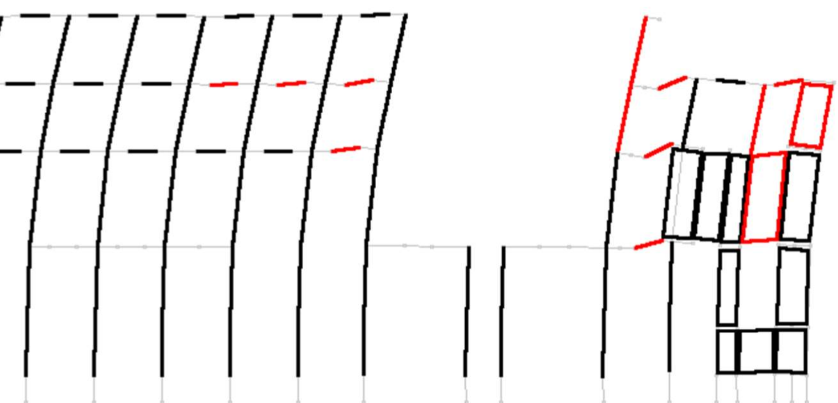
Displacement Scale = 10

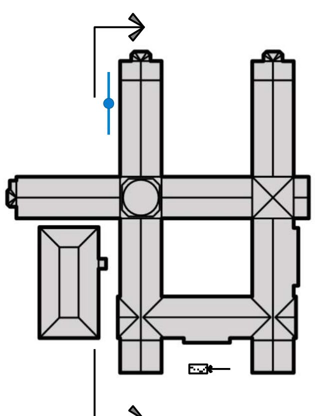
Shear Strain (%): 0.7 0.8 0.9 1

Model Variation 1

BSE-2N Record 5 at time = 5.4 sec





Displacement Scale = 10

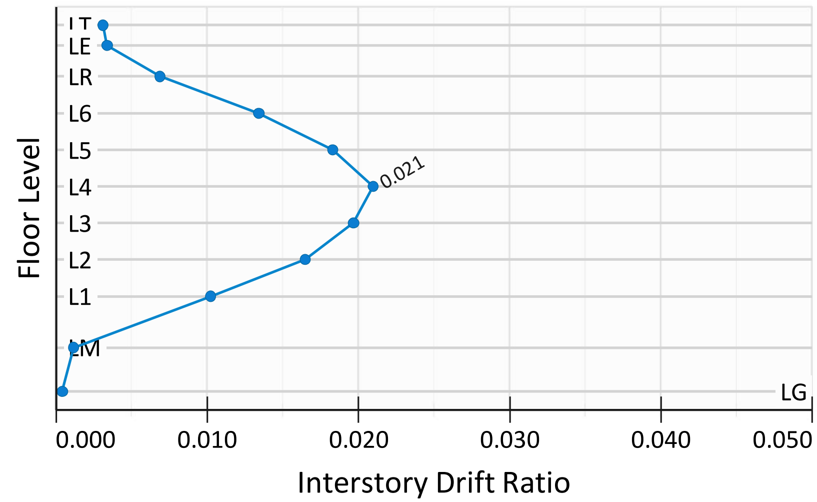
Shear Strain (%): 0.7 0.8 0.9 1

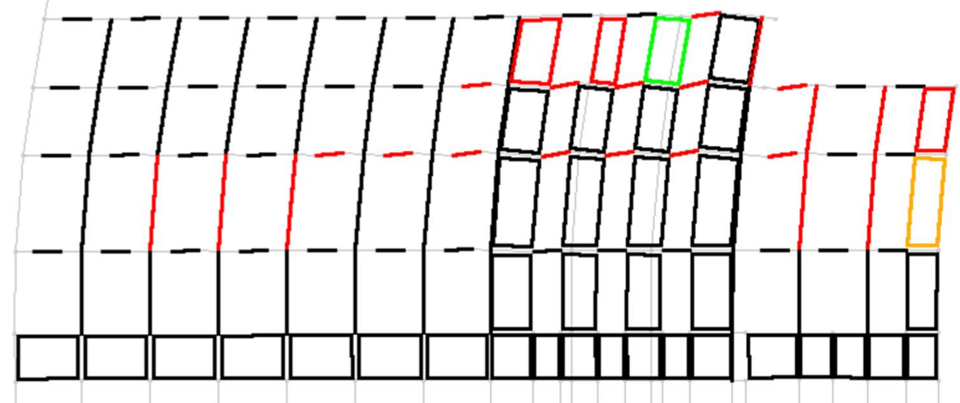
Variation 2 = Variation 1 plus

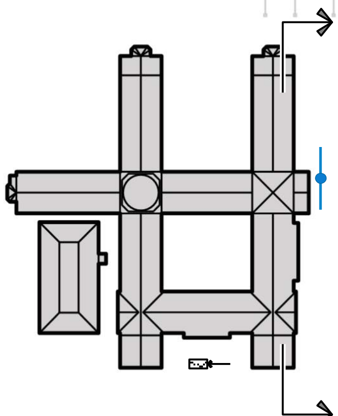
- Columns, $E_{eff} = 0.35 E_c$
- Spandrels:
  - $E_{eff} = 0.2 E_c$
  - Shear hinge changed to Trilinear (was EPP) with $FY = 2 \cdot \sqrt{f'_c} \cdot b_w \cdot d$ and $DL = 0.003$

Model Variation 2

BSE-2N Record 5 at time = 5.4 sec

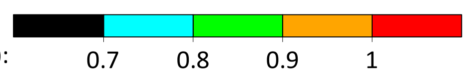






Displacement Scale = 10

Shear Strain (%):



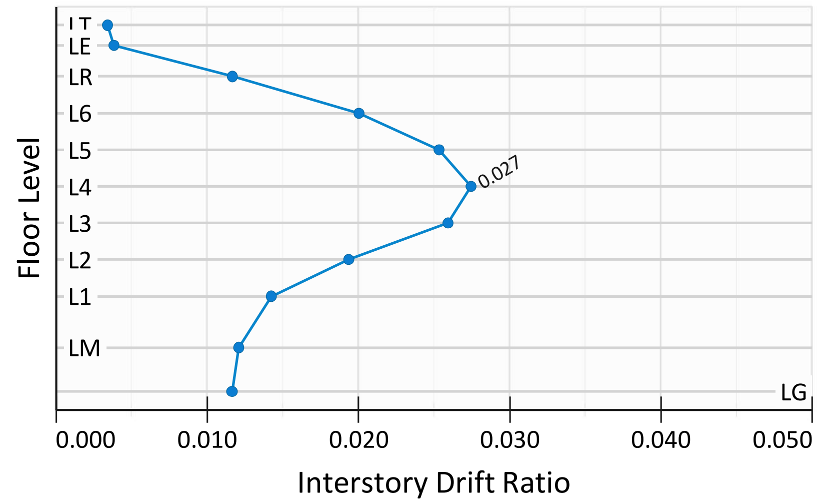
Base Model:

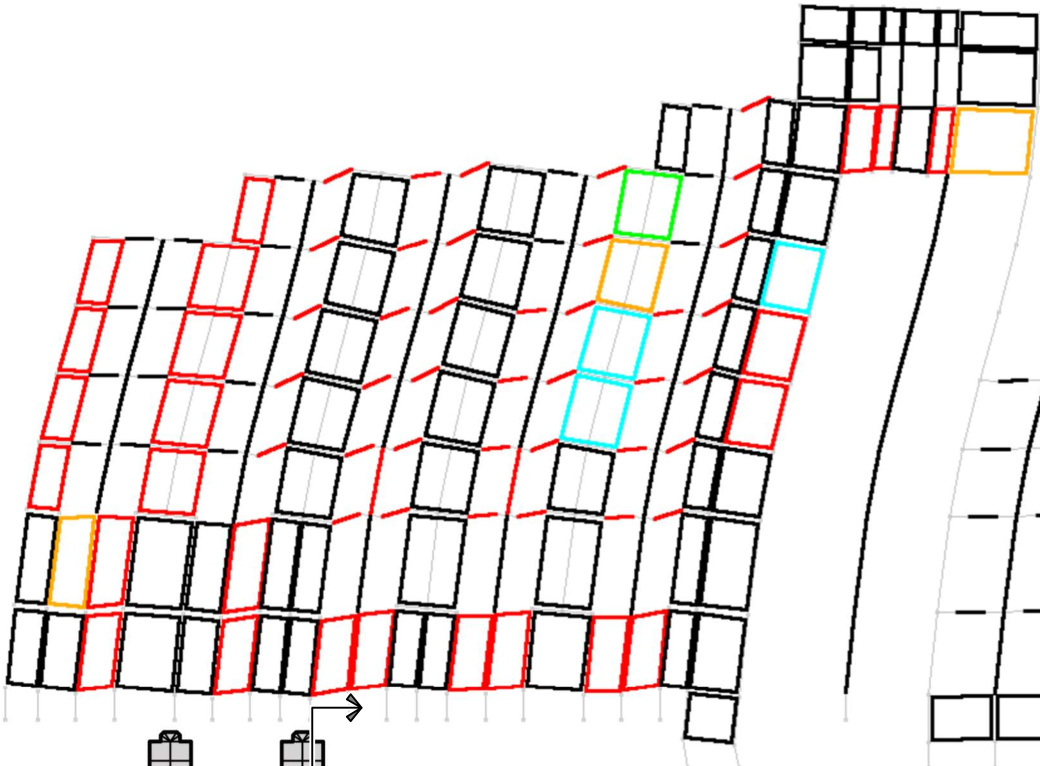
- No horizontal soil springs (fixed horizontally at terraced levels)
- Shear strain at strength loss, $DL = 0.007$ for columns, walls, and spandrels
- Elastic Modulus
  - Columns: $0.5\,E_c$
  - Spandrels: $0.35\,E_c$
  - Walls: $1\,E_c$ (cracking in fiber sections)

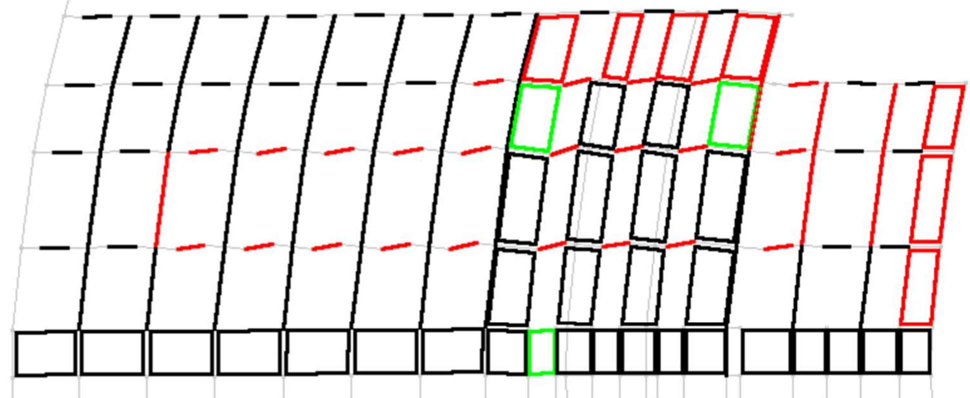
Base Model

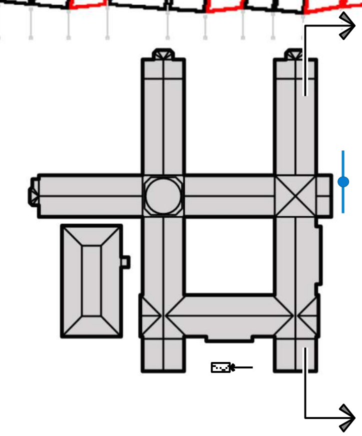
BSE-2N Record 5 at time = 5.4 sec

TIPPING
STRUCTURAL ENGINEERS









Displacement Scale = 10

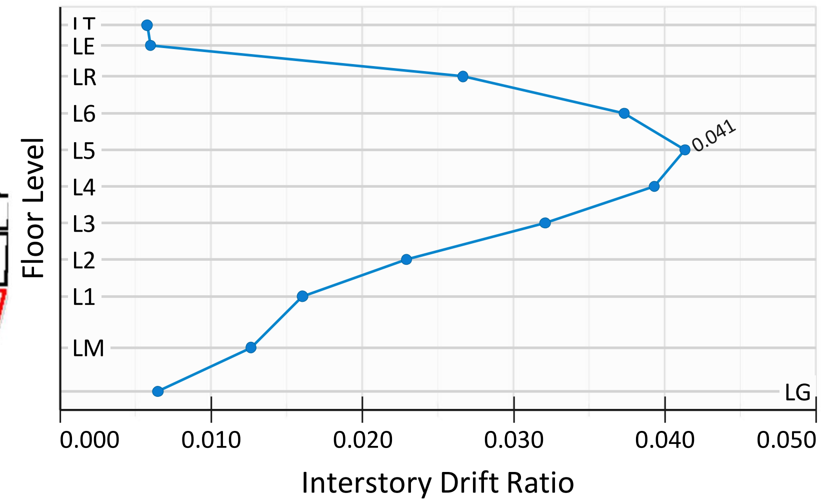
Shear Strain (%): 0.7 0.8 0.9 1

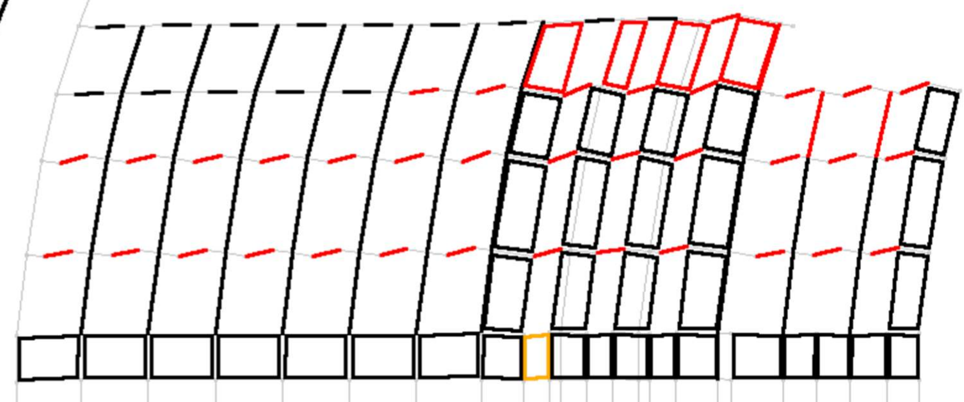
Variation 1 = Base Model plus
- Horizontal Soil Springs
- Shear strain at strength loss, *DL* = 0.003 for columns and walls

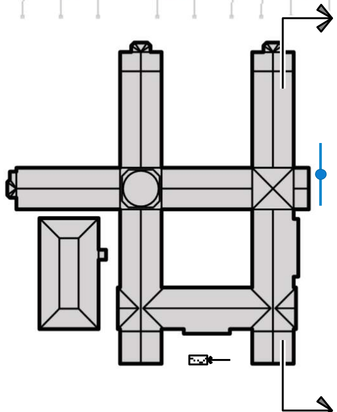
Model Variation 1
BSE-2N Record 5 at time = 5.4 sec

1906 SHATTUCK AVENUE | BERKELEY, CA 94704
510 549-1906 WWW.TIPPINGSTRUCTURAL.COM

Author: korolyk
Date: 2018-11-12, 12:44
File name: 18044 - NRHA results 2018-10-02.chu
Layout: Snapshots

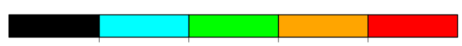






Displacement Scale = 10

Shear Strain (%):



Variation 2 = Variation 1 plus

- Columns, $E_{eff} = 0.35\,E_c$
- Spandrels:
  - $E_{eff} = 0.2\,E_c$
  - Shear hinge changed to Trilinear (was EPP) with $FY = 2 \cdot \sqrt{f'_c} \cdot b_w \cdot d$ and $DL = 0.003$

Model Variation 2

BSE-2N Record 5 at time = 5.4 sec

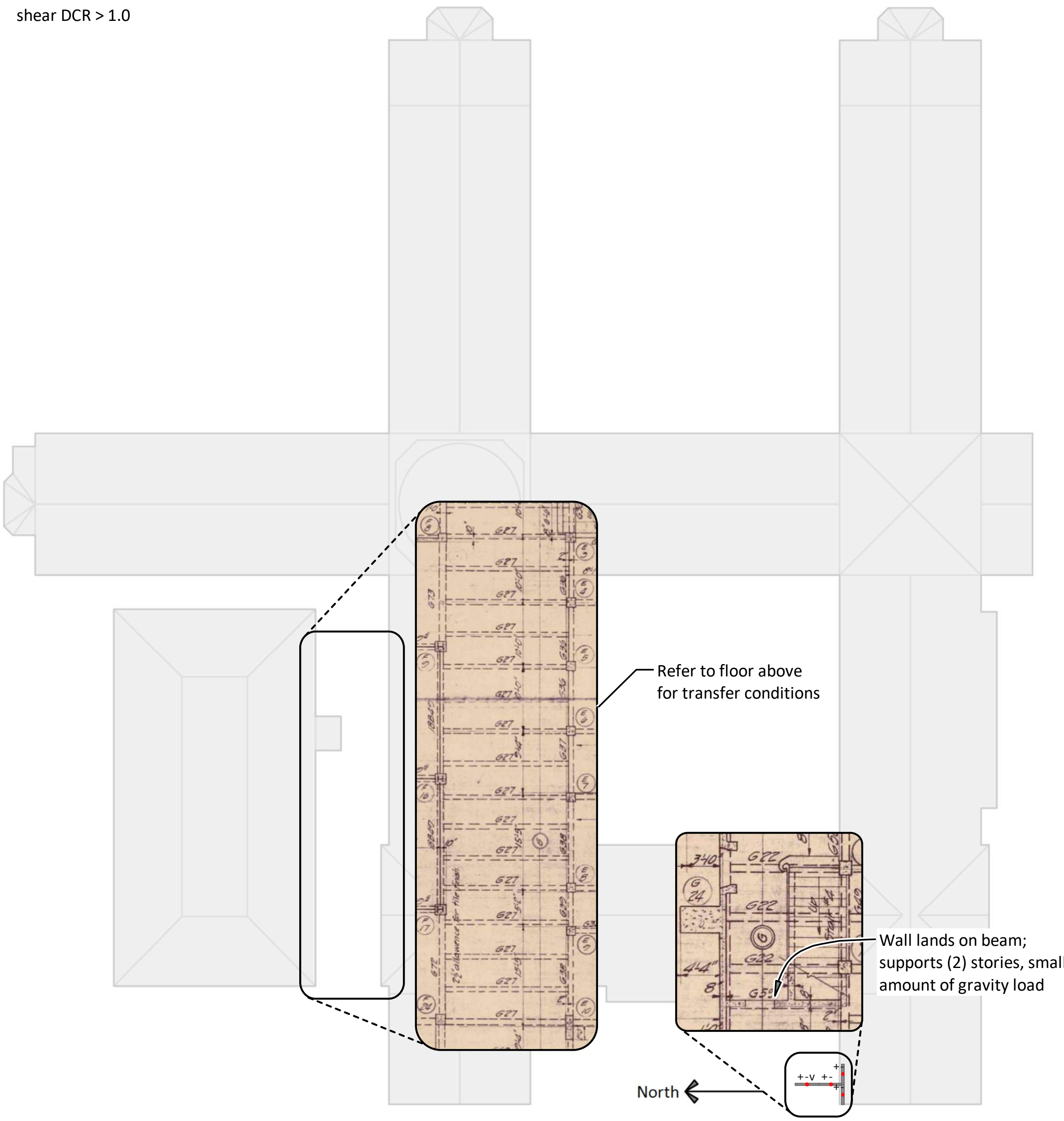
Beam Shear Mechanisms

Notes:

1. Most gravity beams are not modeled explicitly.
2. Beams that transfer loads from wall elements are modeled as elastic frame elements.
3. Shear and moment demands are recovered from the elastic elements and compared with estimated capacities.
4. In the illustrations below, elements with shear or moment demands exceeding capacities are the highlighted, but those with excess shear are of greater concern.

Legend:

- ● indicates DCR is greater than 1.0
- \+ positive moment DCR > 1.0
- \- negative moment DCR > 1.0
- v shear DCR > 1.0



Ground Floor

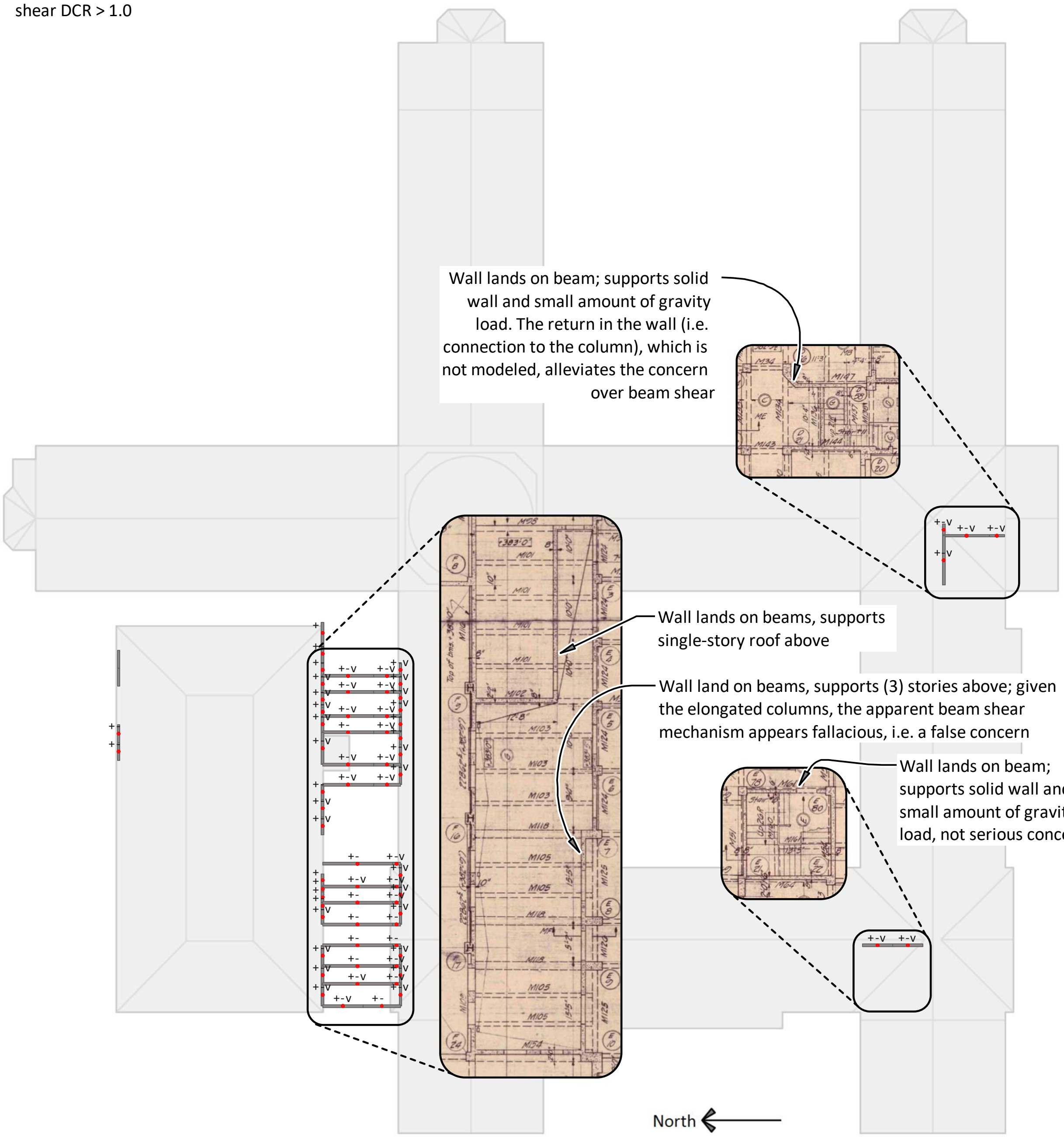
Beam Shear Mechanisms

Notes:

1. Most gravity beams are not modeled explicitly.
2. Beams that transfer loads from wall elements are modeled as elastic frame elements.
3. Shear and moment demands are recovered from the elastic elements and compared with estimated capacities.
4. In the illustrations below, elements with shear or moment demands exceeding capacities are the highlighted, but those with excess shear are of greater concern.

Legend:

- ● indicates DCR is greater than 1.0
- \+ positive moment DCR > 1.0
- \- negative moment DCR > 1.0
- v shear DCR > 1.0



Mezzanine Floor

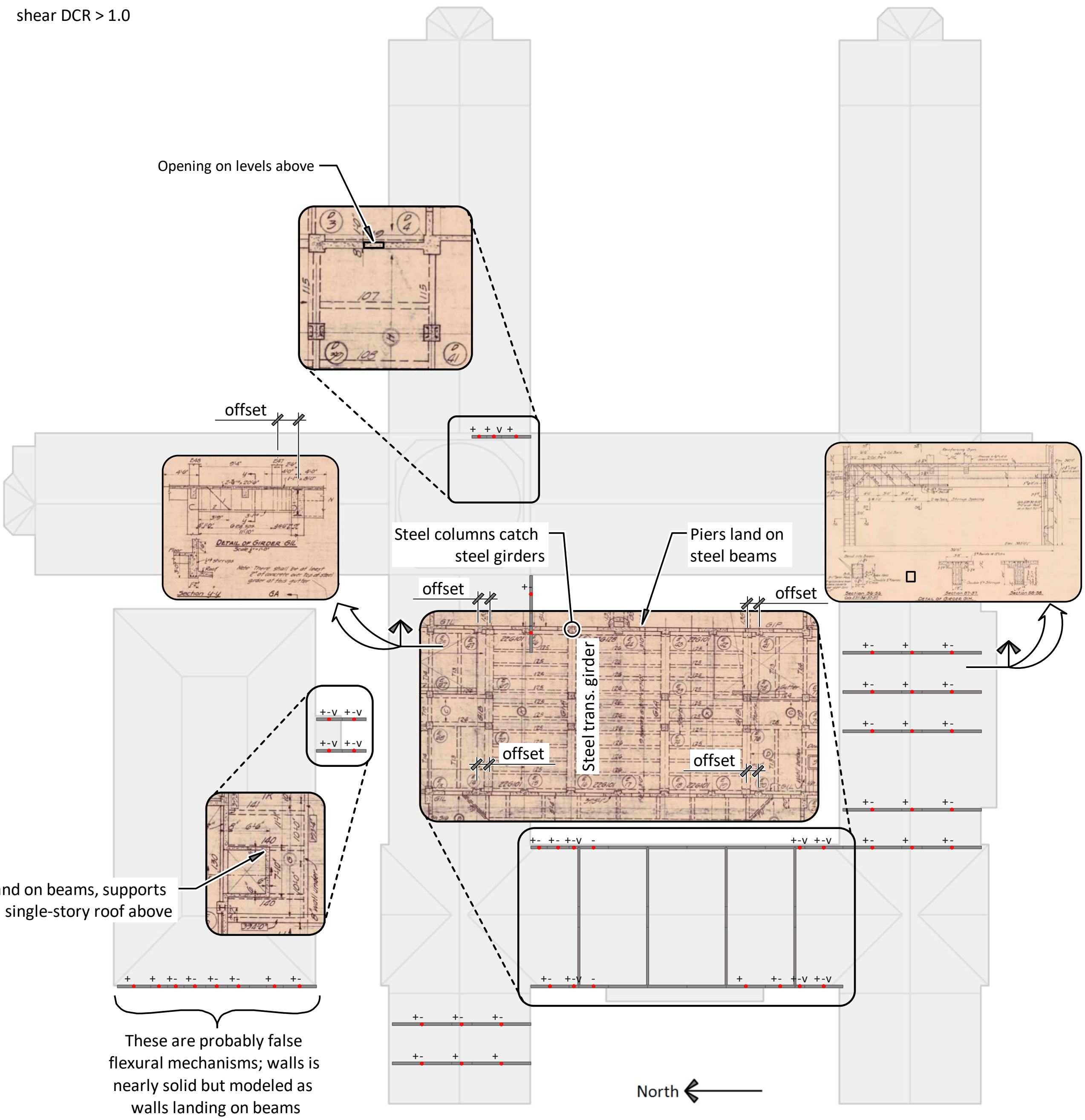
Beam Shear Mechanisms

Notes:

1. Most gravity beams are not modeled explicitly.
2. Beams that transfer loads from wall elements are modeled as elastic frame elements.
3. Shear and moment demands are recovered from the elastic elements and compared with estimated capacities.
4. In the illustrations below, elements with shear or moment demands exceeding capacities are the highlighted, but those with excess shear are of greater concern.

Legend:

- ● indicates DCR is greater than 1.0
- \+ positive moment DCR > 1.0
- \- negative moment DCR > 1.0
- v shear DCR > 1.0



First Floor

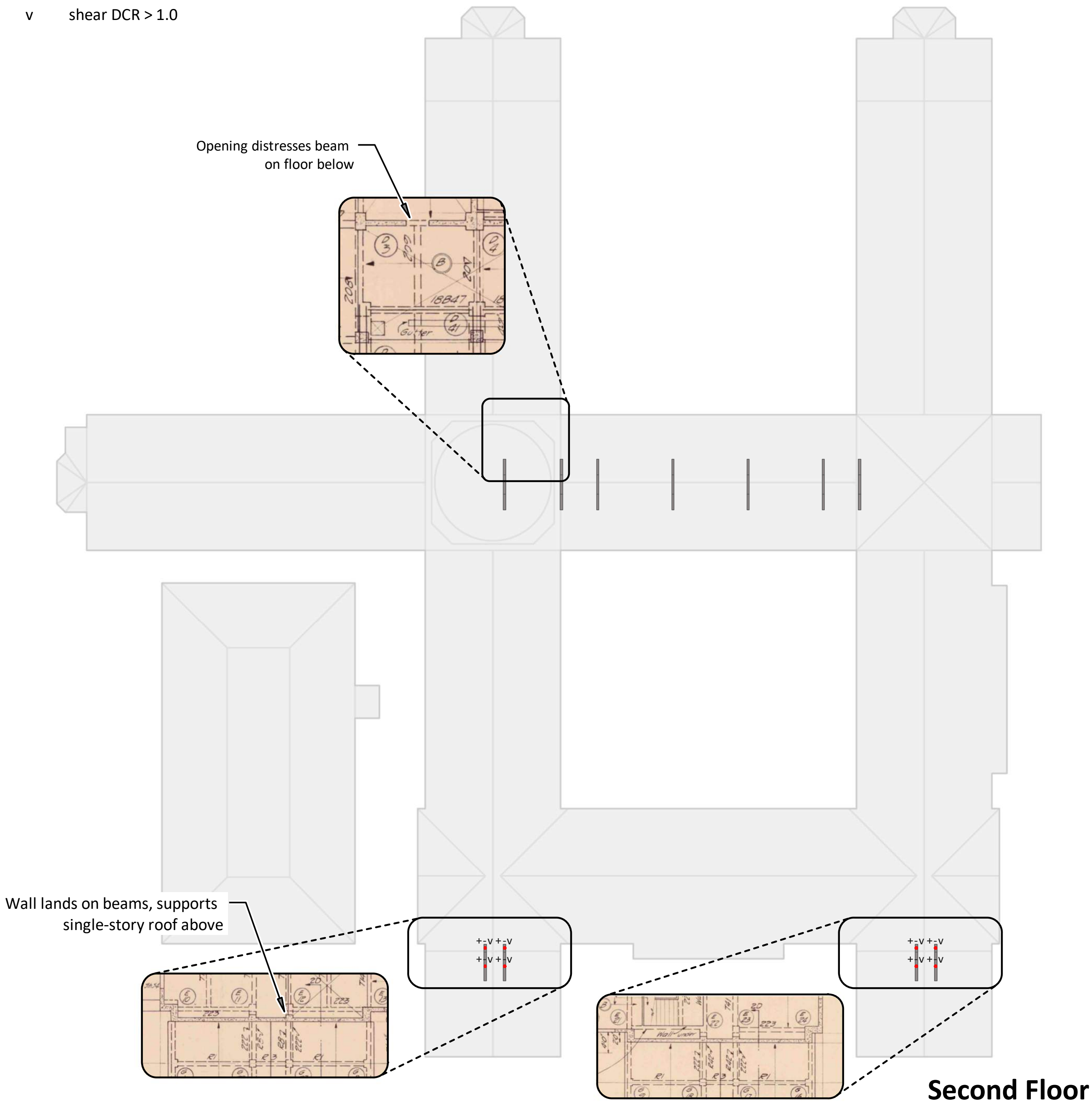
Beam Shear Mechanisms

Notes:

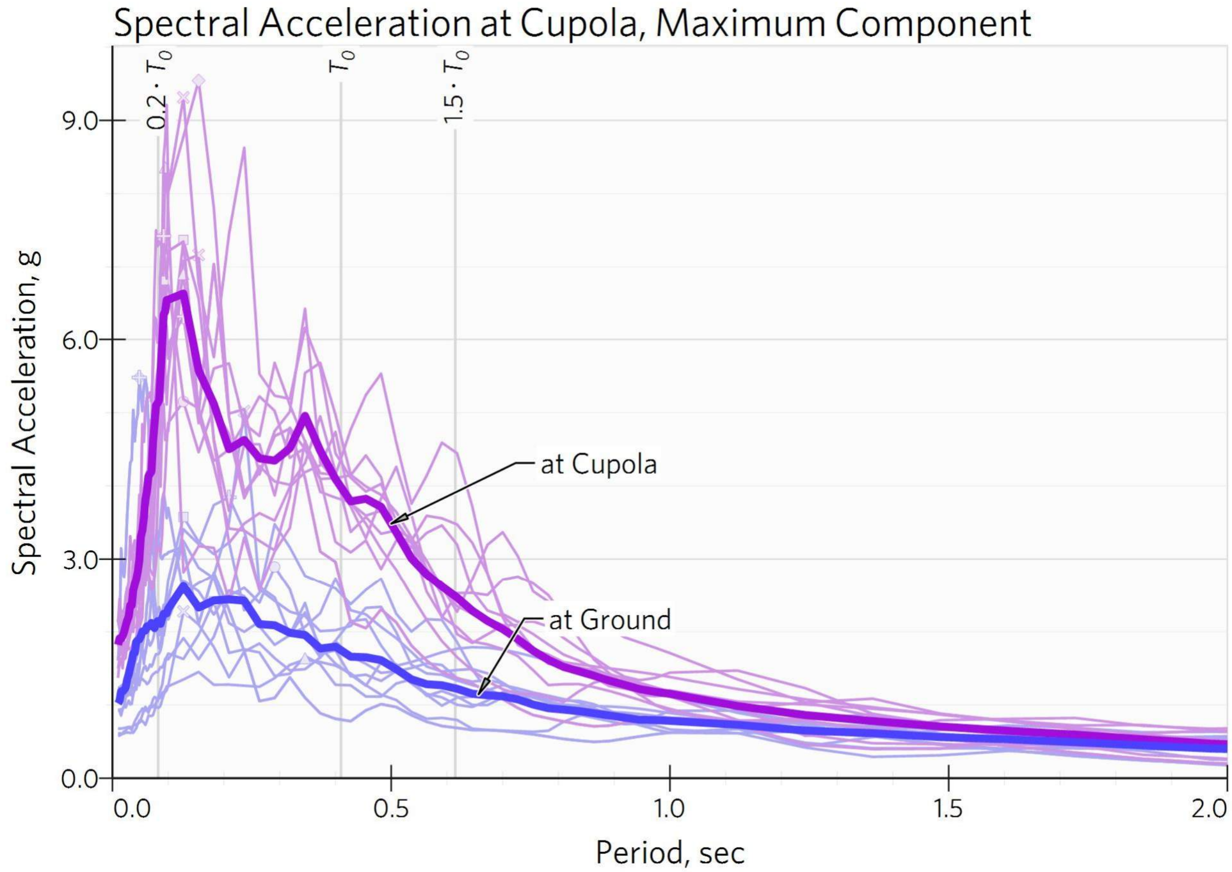
1. Most gravity beams are not modeled explicitly.
2. Beams that transfer loads from wall elements are modeled as elastic frame elements.
3. Shear and moment demands are recovered from the elastic elements and compared with estimated capacities.
4. In the illustrations below, elements with shear or moment demands exceeding capacities are the highlighted, but those with excess shear are of greater concern.

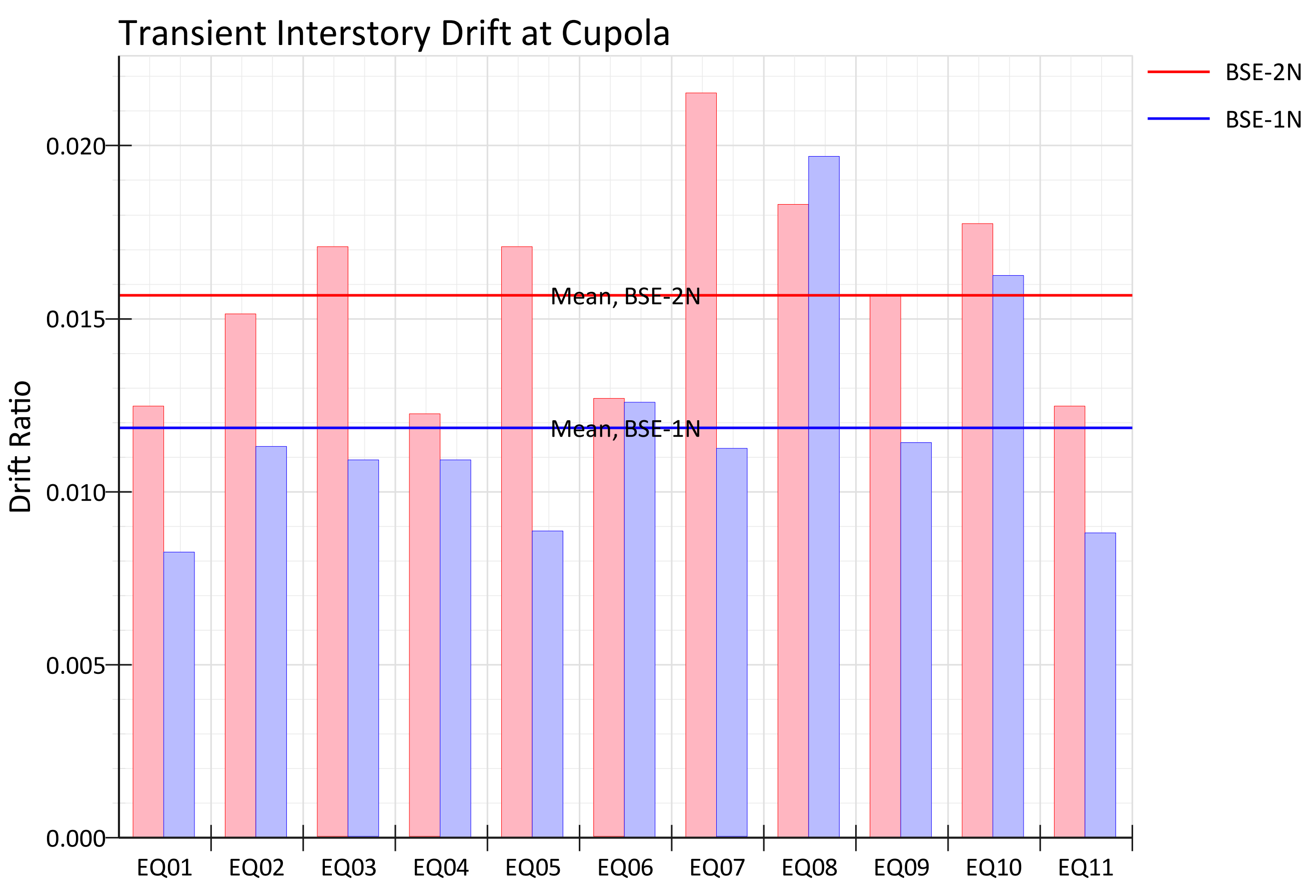
Legend:

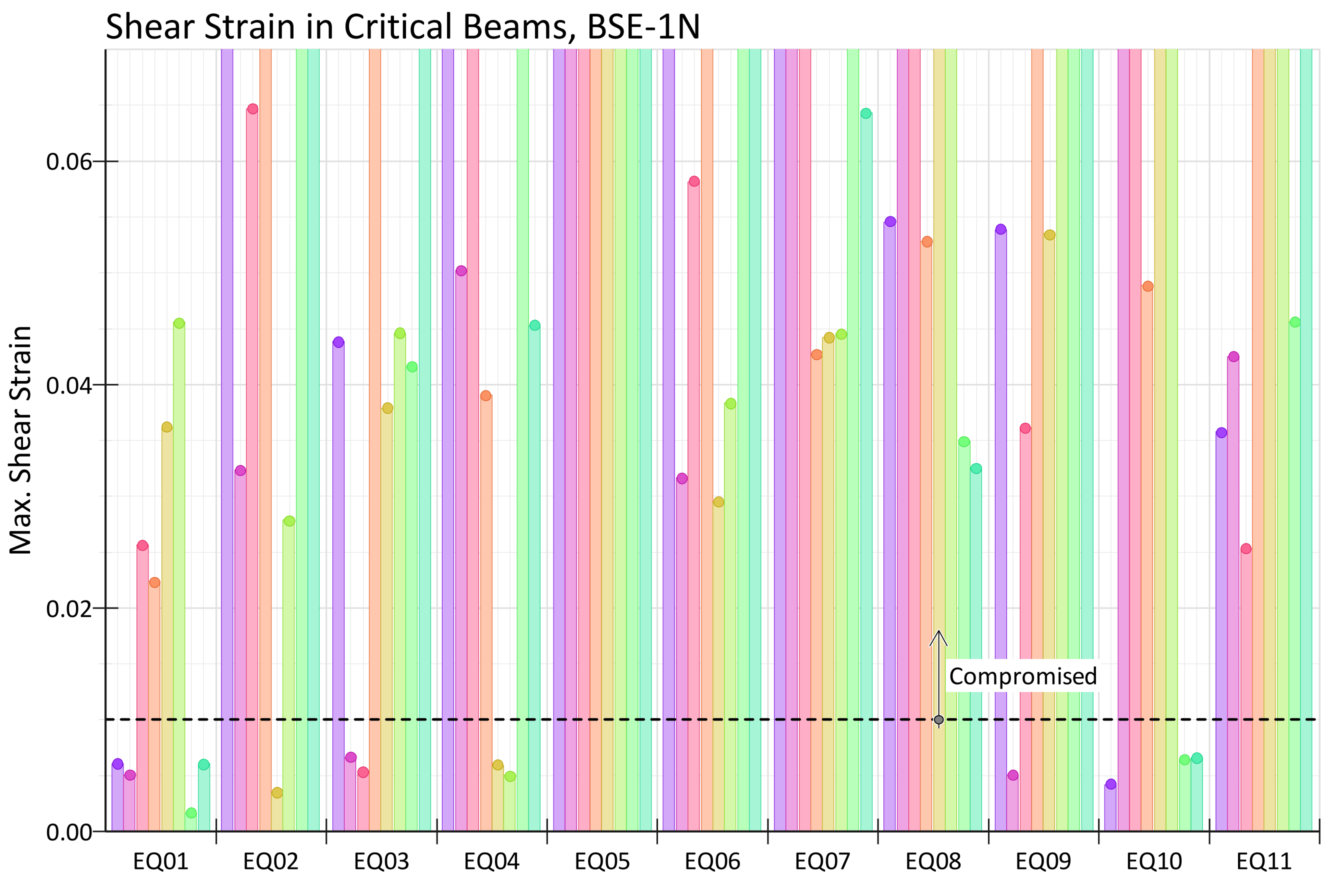
- ● indicates DCR is greater than 1.0
- \+ positive moment DCR > 1.0
- \- negative moment DCR > 1.0
- v shear DCR > 1.0

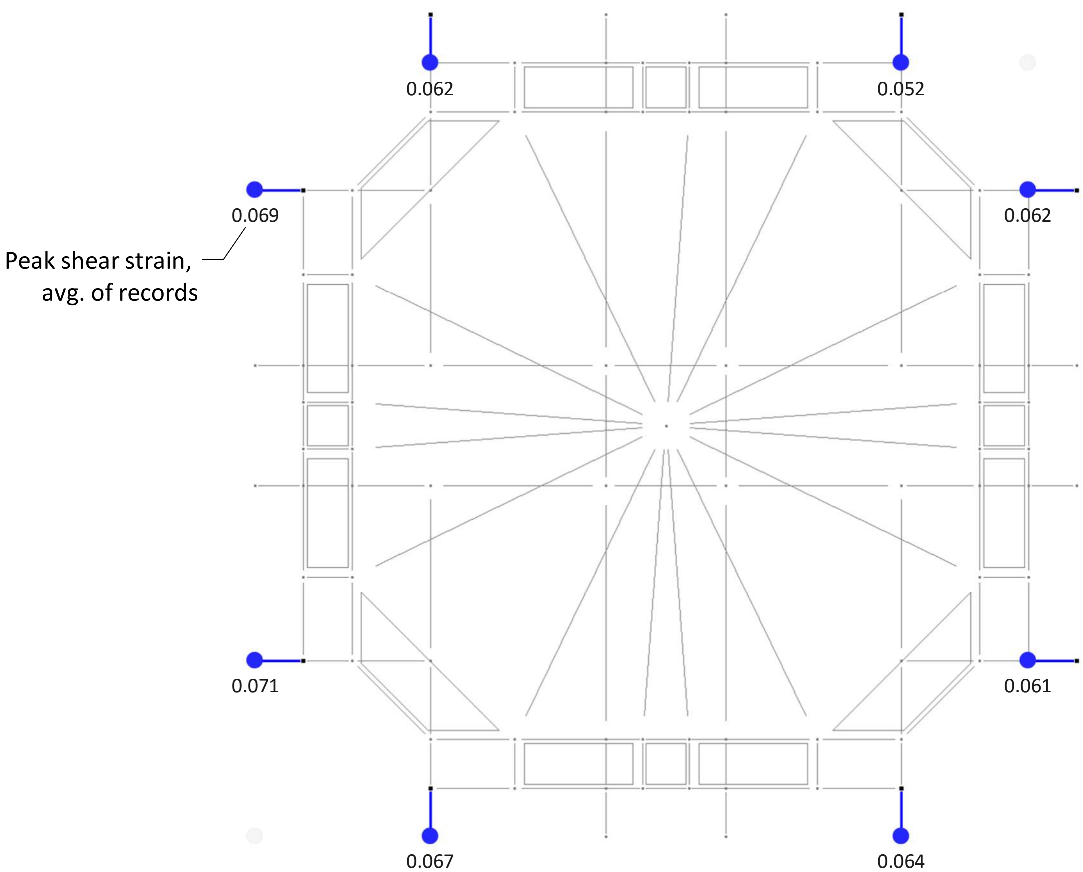


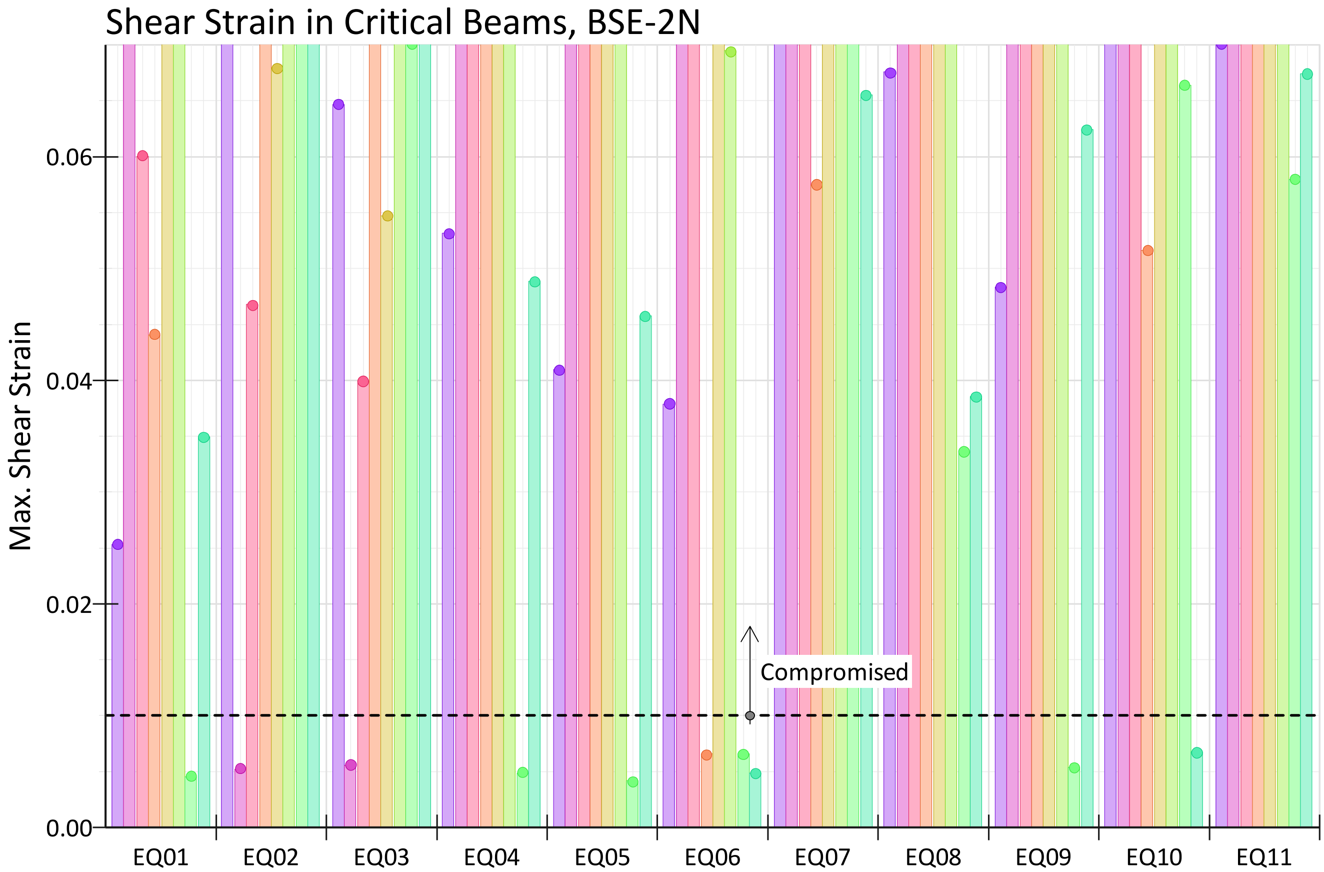
Second Floor

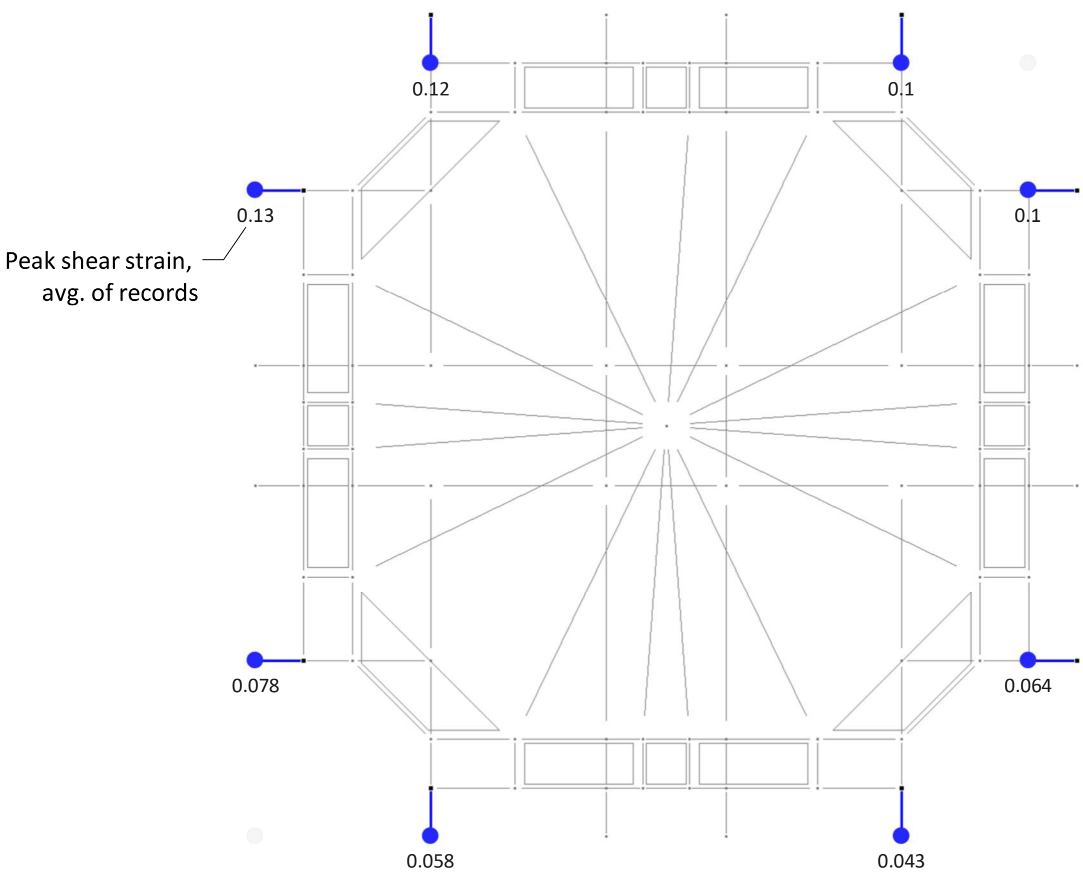

Spectral Acceleration at Cupola, Maximum Component
0.2 · T_0
T_0
1.5 · T_0
at Cupola
at Ground
Spectral Acceleration, g
9.0
6.0
3.0
0.0
0.0
0.5
1.0
1.5
2.0
Period, sec

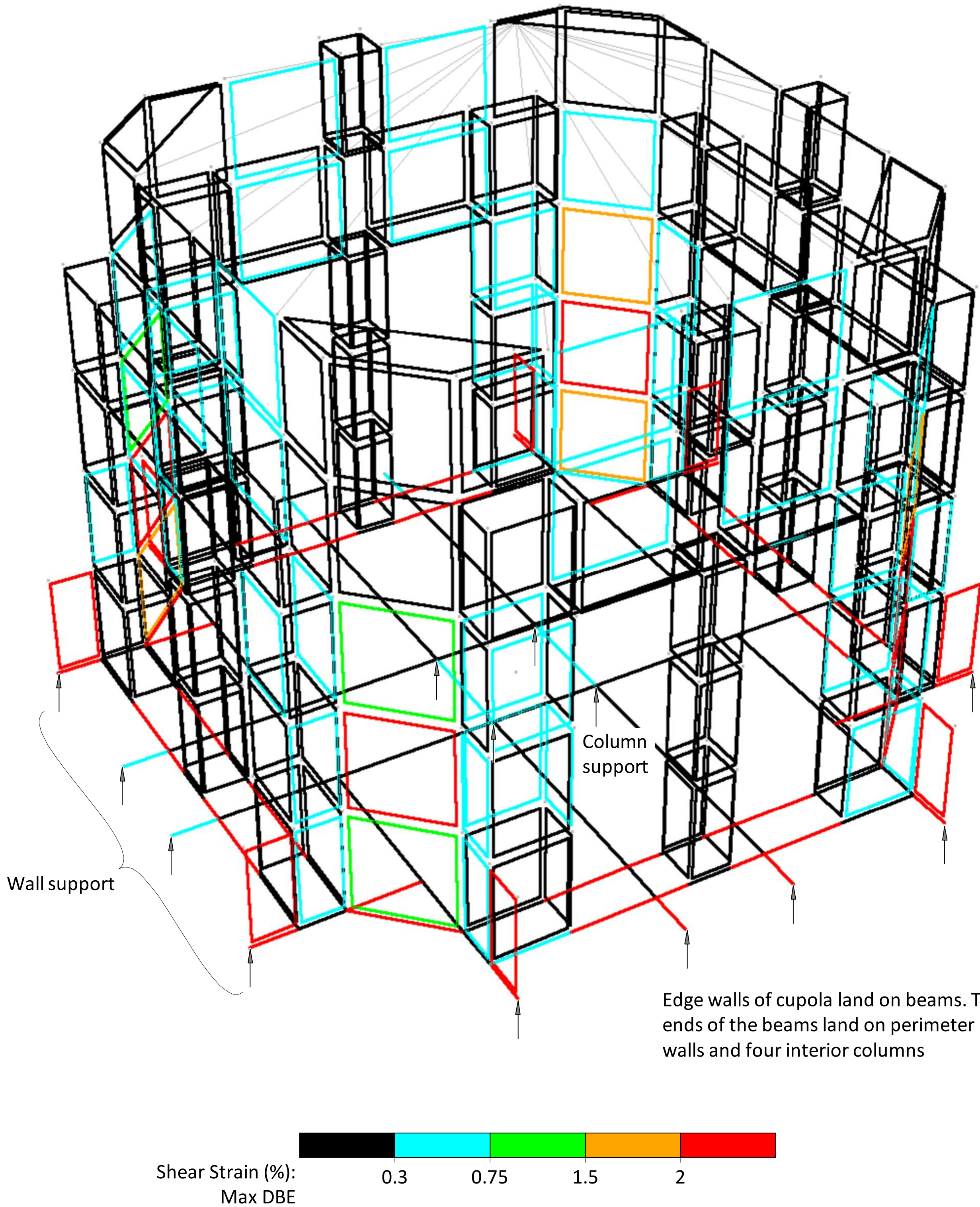

Transient Interstory Drift at Cupola
BSE-2N
BSE-1N
Mean, BSE-2N
Mean, BSE-1N
Drift Ratio
0.020
0.015
0.010
0.005
0.000
EQ01
EQ02
EQ03
EQ04
EQ05
EQ06
EQ07
EQ08
EQ09
EQ10
EQ11

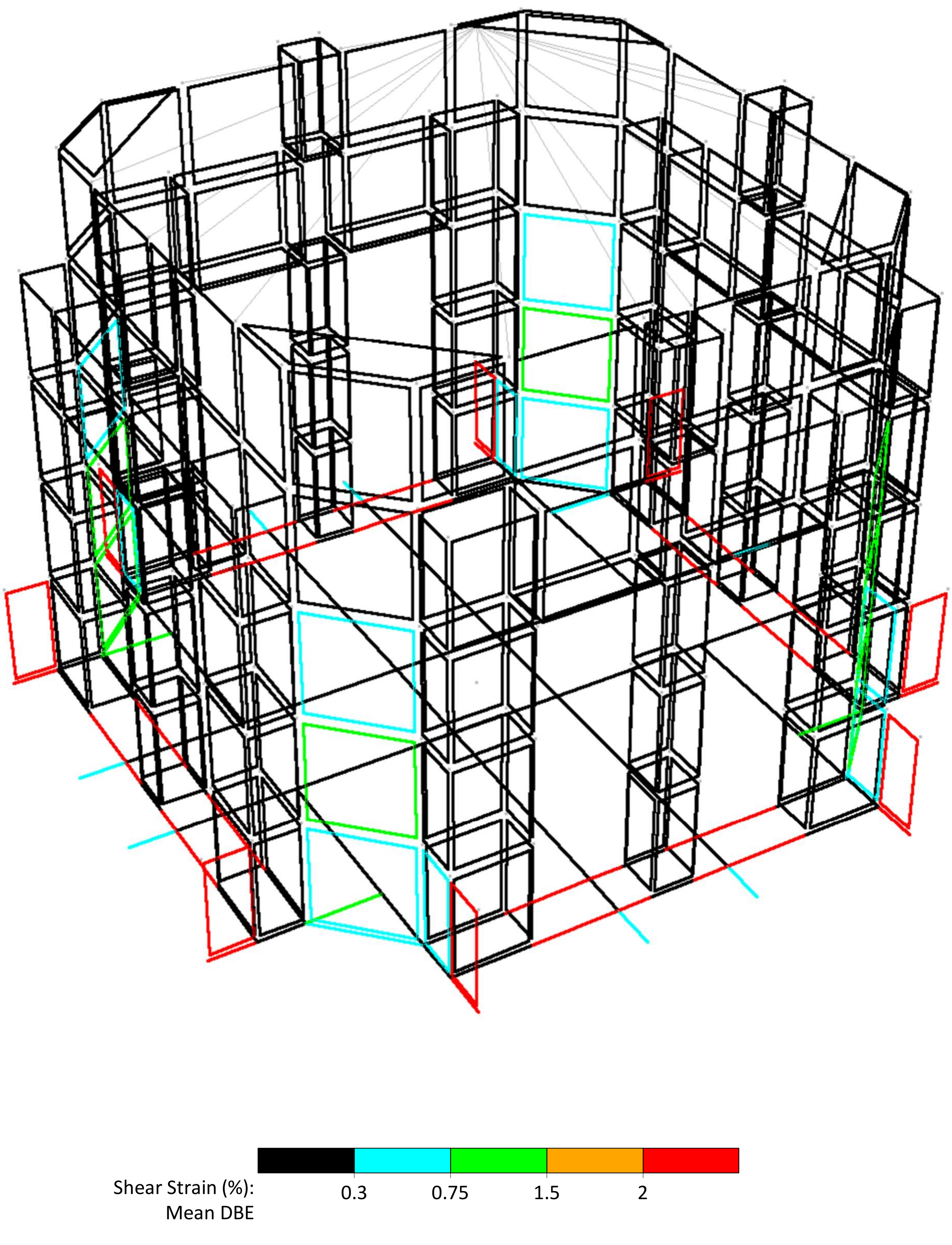

Shear Strain in Critical Beams, BSE-1N
Max. Shear Strain
0.06
0.04
0.02
0.00
Compromised
EQ01
EQ02
EQ03
EQ04
EQ05
EQ06
EQ07
EQ08
EQ09
EQ10
EQ11

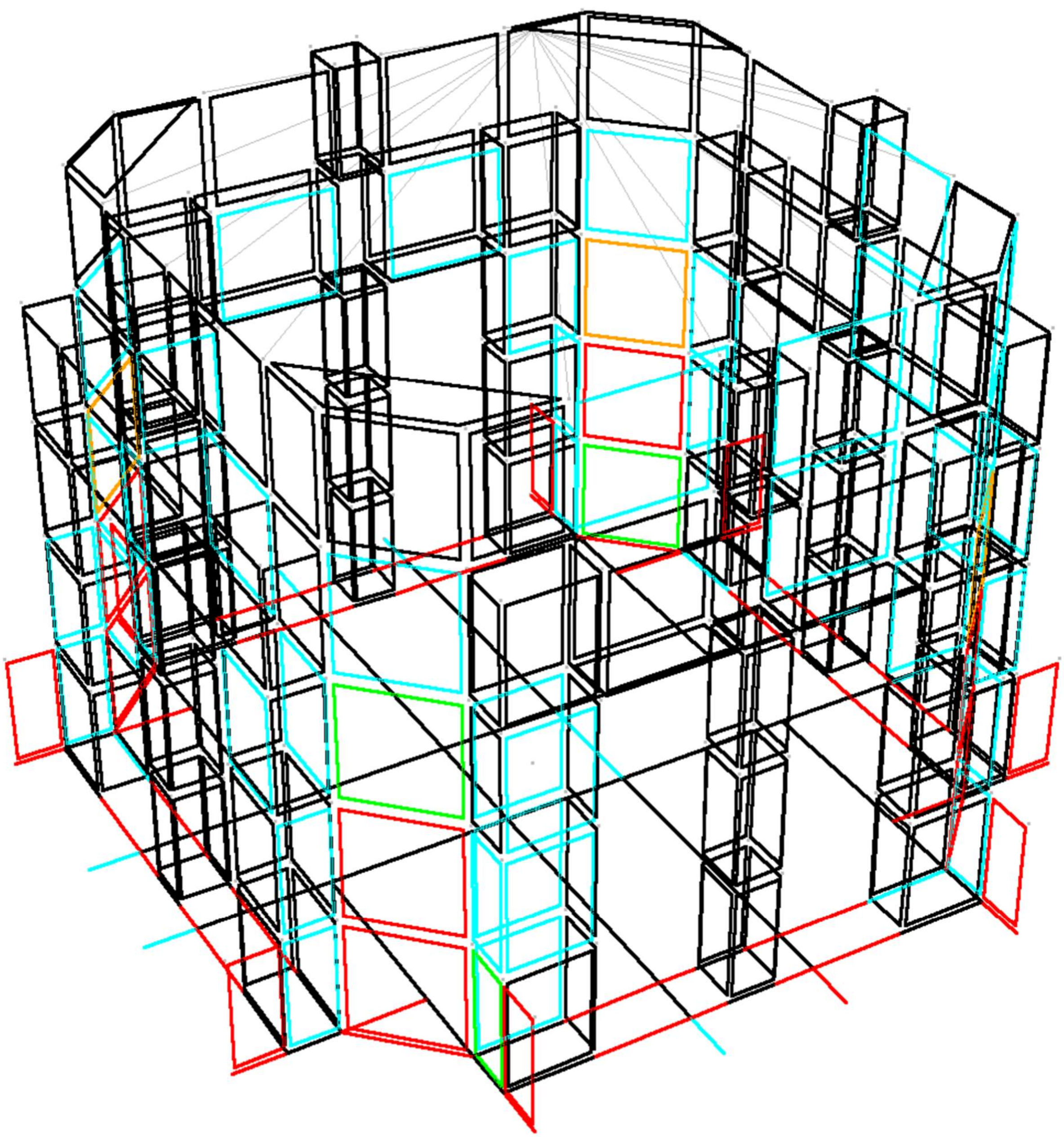

0.062
0.052
0.069
0.062
Peak shear strain,
avg. of records
0.071
0.061
0.067
0.064

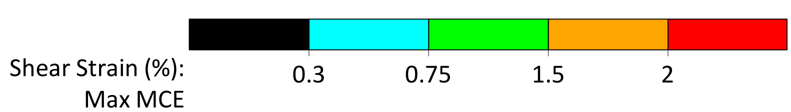

Shear Strain in Critical Beams, BSE-2N
Max. Shear Strain
0.06
0.04
0.02
0.00
Compromised
EQ01
EQ02
EQ03
EQ04
EQ05
EQ06
EQ07
EQ08
EQ09
EQ10
EQ11

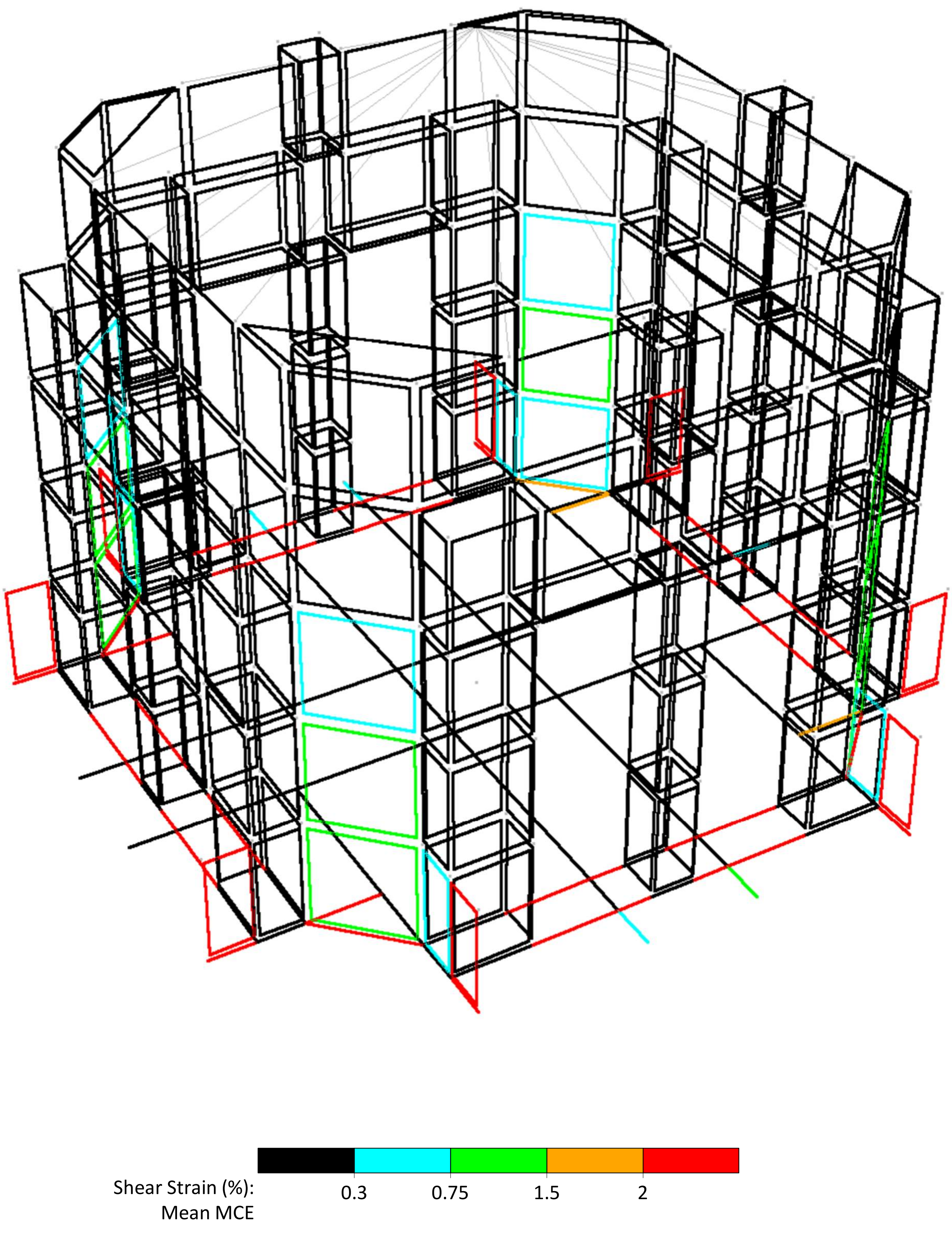

0.12
0.1
0.13
0.1
Peak shear strain,
avg. of records
0.078
0.064
0.058
0.043


Column support
Wall support
Edge walls of cupola land on beams. The ends of the beams land on perimeter walls and four interior columns
Shear Strain (%): Max DBE
0.3
0.75
1.5
2


Shear Strain (%):
Mean DBE
0.3
0.75
1.5
2




Shear Strain (%):
Max MCE
0.3
0.75
1.5
2


Shear Strain (%):
Mean MCE
0.3
0.75
1.5
2

UCB International House - Tier 1 Screening

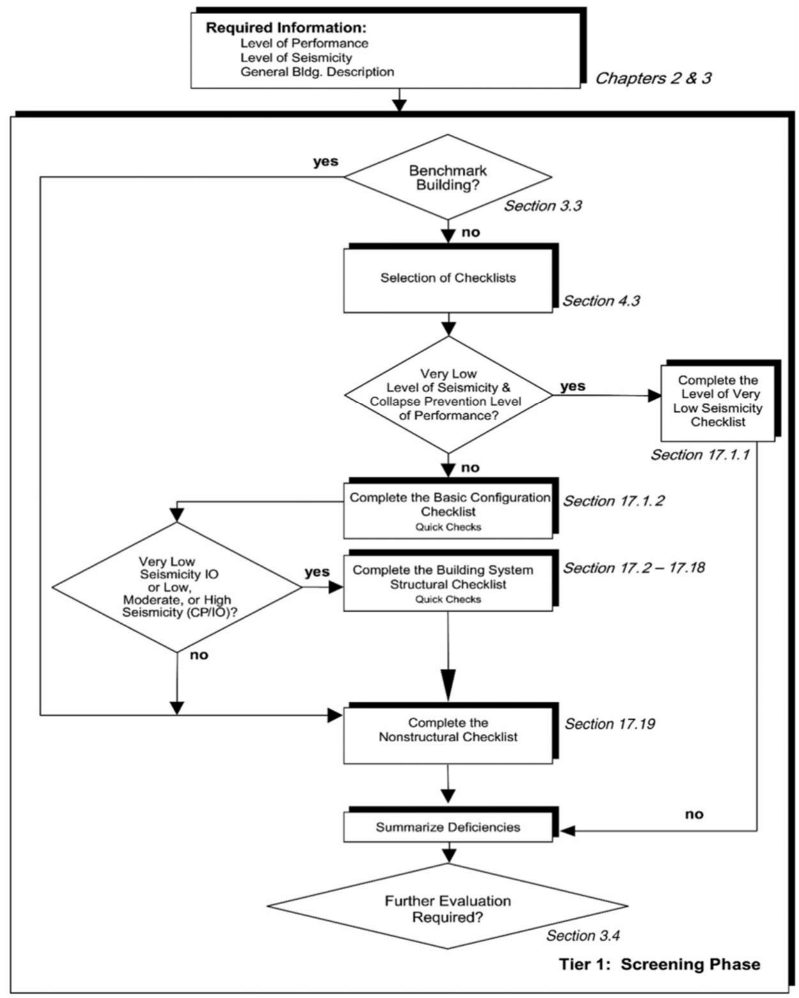


Figure 4-1. Tier 1 Evaluation Process
Note: IO = Immediate Occupancy, LS = Life Safety.

42 STANDARD ASCE/SEI 41-17

<u>Required Information:</u>

1. Level of Performance; 3-C at BSE-1E (Risk Category II)
2. Level of Seismicity; High
3. Building Type: C2 Concrete Shear Walls (Table 3-1)

<u>Selection of Checklists:</u>

- Basic Configuration Checklist (Sec. 17.1.2)
- Collapse Prevention Checklist (Sec. 17.2-17.17)
- Life Safety Nonstructural Checklist (Sec. 17.19)

Table 17-2 Collapse Prevention Basic Configuration Checklist:

tbl17|2

Compliance	*Item*	*Description*	*Notes*
C	Load Path	The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	
C	Adjacent Buildings	The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity.	
C	Mezzanines	Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	
C	Weak Story	The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.	
C	Soft Story	The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.	
NC	Vertical Irregularities	All vertical elements in the seismic-force-resisting system are continuous to the foundation.	Transfers girders present at walls over dining hall below pier and spandrel wall elements
NC	Geometry	There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines.	Change in N-S shear wall length of 63.6% from roof level to sixth level
NC	Mass	There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.	Concrete roof is not considered "light" and has a relatively large weight decrease from level 6
NA	Torsion	The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension.	Multiple diaphragms present at each level; torsion check is not applicable
U	Liquefaction	Liquefaction-susceptible saturated loose granular soils that;could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.	
U	Slope Failure	The building site is located away from potential earthquake-induced lope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.	
U	Surface Fault Rupture	Surface fault rupture and surface displacement at the building site are not anticipated.	
NC	Overturning	The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than 0.6Sa.	East wing end elevation is slender for overturning
C	Ties Between Foundation Elements	The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C.	

Supporting Calculations for Collapse Prevention Basic Configuration Checklist:

Mass Basic Configuration Checklist Item:

tblMassCheck

Floor	*W*	$\%_{diff,abv}$	$\%_{diff,blw}$
LR	1,630 *kips*	0%	88.8%
L5	3,078 *kips*	47%	5.87%
L4	3,259 *kips*	5.54%	49.3%
L3	4,866 *kips*	33%	23%
L2	5,986 *kips*	18.7%	17.7%
L1	7,046 *kips*	15%	0.26%
LM	7,064 *kips*	0.25%	23.5%
LG	5,402 *kips*	30.8%	0%

Geometry Basic Configuration Checklist Item:

tblShearWallGeometryCheck

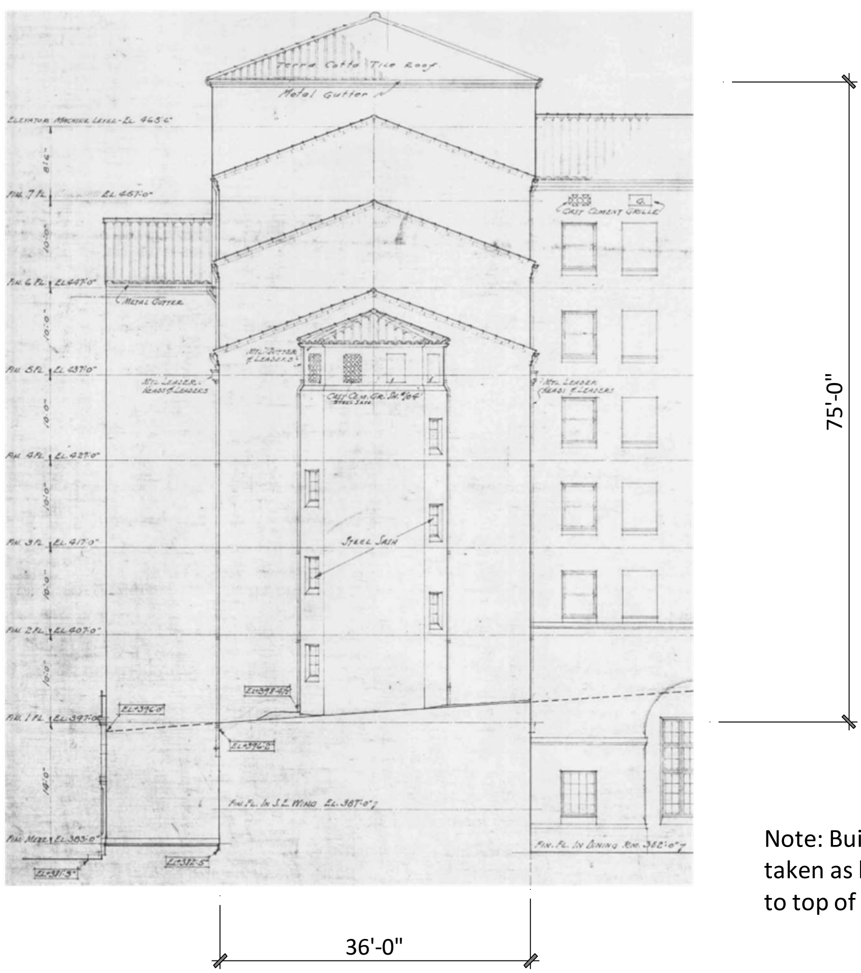
Floor	$A_{w,NS}$	$L_{w,NS}$	$\%_{diff,abv}$	$\%_{diff,blw}$
LR	$30{,}232\,in^2$	$315\,ft$	0%	63.6%
L6	$49{,}466\,in^2$	$515\,ft$	38.9%	8.27%
L5	$45{,}373\,in^2$	$473\,ft$	9.02%	1.45%
L4	$46{,}029\,in^2$	$479\,ft$	1.43%	23.9%
L3	$57{,}043\,in^2$	$594\,ft$	19.3%	0%

Note: EW walls and floors below L3 omitted for simplicity

Overturning Basic Configuration Checklist Item:

$0.6S_a \rightarrow 1.47$

$\frac{75\,ft}{36\,ft} \rightarrow 2.08$ (greater than $0.6S_a$)



Note: Building height for checklist item taken as height to base local roof, not to top of cupola

Table 17-24 Collapse Prevention Structural Checklist for Buliding Types C2 & C2a:

tbl17|24

Compliance	*Item*	*Description*	*Notes*
C	Complete Frames	Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system.	Pilasters present at walls to support concrete beams and girders
C	Redundancy	The number of lines of shear walls in each principal direction is greater than or equal to 2.	
NC	Shear Stress Check	The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 $\frac{lb}{in^2}$ (0.69 MPa) or $2\sqrt{f'_c}$	Wall shear stresses as high as 430 $\frac{lb}{in^2}$ at level 4
C	Reinforcing Steel	The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction.	Compliant for 8" walls
NA	Wall Anchorage at Flexible Diaphragms	Exterior concrete or masonry walls that are dependent on fexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	Flexible diaphragms not present; anchorage at flexible diaphragms check is not applicable
C	Transfer to Shear Walls	Diaphragms are connected for transfer of seismic forces to the shear walls.	Typical slab detailing indicates developed bars at slab to wall interface are present
C	Foundation Dowels	Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing directly above the foundation.	
NC	Deflection Compatibility	Secondary components have the shear capacity to develop the flexural strength of the components.	Approximately 1% drift expected; corridor beams failing in shear at 0.5% drift
NA	Flat Slabs	Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints.	Beams, girders & walls support all slabs (no flat slab to column connections present)
C	Coupling Beams	The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning.	
NC	Diaphragm Continuity	The diaphragms are not composed of split-level floors and do not have expansion joints.	Split levels present throughout mezzanine level
C	Openings at Shear Walls	Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	
NA	Uplift at Pile Caps	Pile caps have top reinforcement, and piles are anchored to the pile caps.	No pile caps present; pile cap uplift check is not applicable

Note: Flexible diaphragm checklist items not shown for clarity

Supporting Calculations for Collapse Prevention Structural Checklist:

Quick Check Procedure for Wall Shear (Sec. 4.4.3.3)

$$F_x = \frac{Wh}{Wh_{sum}} \cdot V$$

$$V = C \cdot S_a \cdot W_{sum}$$

$C = 1$ Modification Factor per Table 4-7

$$S_a = \frac{S_{X1}}{T}$$

$$S_{X1} = F_v \cdot S_1$$

$$F_v = 1.5$$

$$S_1 = 1.032$$

$$T = C_t \cdot \left(\frac{h_n}{ft}\right)^{\beta} \rightarrow 0.63$$

$$C_t = 0.02$$

$$h_n = max\left(tblStoryShearForces_h\right) \rightarrow 100\,ft$$

$$\beta = 0.75$$

Table Calculations:

$$v_{j,avg,NS} = \frac{1}{M_s} \cdot \left(\frac{V_j}{A_{w,NS}}\right) \quad \text{eqn. 4-8}$$

$M_s = 3$ system modification factor per Table 4-8

$$Wh_{sum} = \Sigma\left(tblStoryShearForces_{Wh}\right)$$

$$W_{sum} = \Sigma\left(tblStoryShearForces_W\right) \rightarrow 16{,}889\,kips$$

$$f'_c \cdot psi \rightarrow 3{,}000\,psi$$

$$2\sqrt{f'_c\,psi} \rightarrow 110\,psi$$

Note: Floors below L3 omitted for simplicity

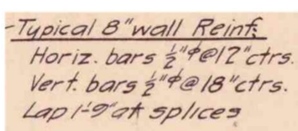
tblStoryShearForces

Floor	W	h	Wh	F_x	V_j	$A_{w,NS}$	$A_{w,EW}$	$v_{j,avg,NS}$	$v_{j,avg,EW}$
LT	938 kips	100 ft	93,826 kip · ft	3,569 kips	3,569 kips	15,016 in^2	13,796 in^2	0.079 ksi	0.086 ksi
LE	339 kips	94.5 ft	32,073 kip · ft	1,220 kips	4,789 kips	15,016 in^2	13,796 in^2	0.11 ksi	0.12 ksi
LR	1,630 kips	86 ft	140,176 kip · ft	5,332 kips	10,121 kips	30,232 in^2	22,755 in^2	0.11 ksi	0.15 ksi
L6	2,779 kips	76 ft	211,199 kip · ft	8,034 kips	18,155 kips	49,466 in^2	41,405 in^2	0.12 ksi	0.15 ksi
L5	3,078 kips	66 ft	203,153 kip · ft	7,728 kips	25,882 kips	45,373 in^2	38,509 in^2	0.19 ksi	0.22 ksi
L4	3,259 kips	56 ft	182,483 kip · ft	6,941 kips	32,824 kips	46,029 in^2	37,945 in^2	0.24 ksi	0.29 ksi
L3	4,866 kips	46 ft	223,839 kip · ft	8,514 kips	41,338 kips	57,043 in^2	55,701 in^2	0.24 ksi	0.25 ksi

Reinforcing Steel

$$\rho_v = \frac{barArea\left(4\right)}{8\,in \cdot 18\,in} \rightarrow 0.0014$$

$$\rho_h = \frac{barArea\left(4\right)}{8\,in \cdot 12\,in} \rightarrow 0.0021$$


Typical 8" wall Reinf.
Horiz. bars ½"ϕ @12" ctrs.
Vert. bars ½"ϕ @18" ctrs.
Lap 1'-9" at splices

TIPPING
STRUCTURAL ENGINEERS

Supporting Calculations for Collapse Prevention Structural Checklist (cont'd):

Deflection Compatibility

Push-over analysis of typical interior frame (Wing Framing detail on S15) indicates shear failure of T3 corridor gravity beams at 0.5% drift and flexural failure of adjacent T2 beams at 2% drift.

Shear failure of T3 corridor beam at 0.5% drift

Flexural failure of T2 beams at 2% drift