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Structural Peer Review Statement

This structural peer review and report is complete for the foundation and superstructure submissions.

Structural peer reviewer name: Benjamin Pimentel

Structural peer reviewer address: 519 Eighth Avenue, 20 Floor, New York, NY

Project address: 532 Neptune Avenue, Brooklyn, New York, 11224

Department application number for structural work: NB # 320622714

Structural Peer Reviewer Statement

I, Benjamin Pimentel, am a qualified and independent NYS licensed and registered engineer in accordance with BC Section 1627.4, and I have reviewed the structural plans, specifications, and supplemental reports for 532 Neptune Avenue, Brooklyn, New York, 11224 (NB #320622714) and found that the structural design shown on the plans and specifications generally conforms to the foundation and structural requirements of Title 28 of the Administrative Code and the 2008 NYC Construction Codes. The structural Peer Review Report is attached.

New York State Registered Design Professional

Name: Benjamin Pimentel

Date: November 14, 2016

CC: Madison 30 31 LLC
Stephen Desimone

532 Neptune Avenue
Brooklyn, NY

Peer-review Report

Rosenwasser/Grossman Consulting Engineers, P.C.

November 14, 2016

Prepared for

Cammeby's International

45 Broadway, New York, NY

Prepared by

Chandra Dinata, PE

Bernard Taub, PE

Ben Pimentel, PE

I hereby certifies that I have performed the peer review in accordance with the New York City Building Code and requirements set forth therein.

Name: Ben Pimentel

License No.: 086645

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1. Project Introduction and Executive Summary

A 42-story residential reinforced concrete high-rise tower and a 4-story enlarged shopping center podium are proposed to be built at 532 Neptune Avenue, Brooklyn. The height of the tower is approximately 473 feet above the street level (the average slenderness ratio is 7) and it will become the tallest building in the Coney Island neighborhood. The building is located on the city block bordered by Neptune Avenue to the north, West 5th street to the east, a permanent sewer easement to the south, and West 6th street to the west, which includes an active New York City Transit Authority (NYCT) elevated train structure. Currently the site is occupied by an on-grade parking lot and one-to three-story buildings.

Rosenwasser/Grossman Consulting Engineers P.C. was retained by the owner, **Cammeby's International**, to provide a peer review for the residential tower portion on the basis of the 2014 New York City building Code Section BC 1617. The adequacy of the estimated design loads, the selected design criteria, appropriate interpretation of geo-technical engineering report are reiterated in this report for completeness. The structural stiffness, the primary structural members design and the serviceability of the building are reviewed. However, special design requirements for the extreme loading conditions above and beyond the code-prescribed loads (such as blast design), which may be requested by the owner or a third party will not be included in our general code-prescribed peer review discussed herein.

The structural finite element analysis model which was originally provided by the Engineer of Record was verified. To better understand and review the superstructure performance, ETABS structural analysis models were independently developed by RGCE based on the latest available geometry and structural drawings provided by the Engineer of Record. In order to efficiently make the building structure design code-compliant, we have constant discussions with the Engineer of Record whenever we are of the opinion that the structural design can be improved upon. Please note that this peer-review report is only based on the documentations available to RGCE.

Below is the list of documents Rosenwasser/Grossman Consulting Engineers P.C. received from the Engineer of Record:

1) Structural design drawings – Peer Review Set dated August 18th, 2016
2) Structural design drawings – Phase 2 CD submission dated October 14th, 2016
3) Architectural drawings – DOB Set dated September 2nd, 2016
4) Geotechnical report prepared by LANGAN dated May 22nd , 2015
5) Structural Revit model (provided by DeSimone on April 8th, 2016)
6) Interim wind tunnel report for structural loads prepared by CPP Inc, dated June 9th,2016

The peer-reviewed items of the building by this office are summarized as follows:

- Accumulated axial loads for columns and shear wall piers are independently computed and checked;
- The seismic design loads and wind design loads used in the structural design are verified;
- Overall behavior of the structure was reviewed and compared with code criteria;
- The representative structural members were spot-checked using the results from our independent analysis;

Based on all of the above, it is our opinion that the current superstructure design is in general conformance with the structural design provisions of the NYCBC 2014. The Code Compliance of the design according to NYCBC 2014 section BC 1617.5.1 is summarized in the checklist (See appendix A)

It shall be noted that Rosenwasser/Grossman Consulting Engineers P.C reports its own opinion and functions solely as a peer reviewer regarding the design by the Engineer of Record (DeSimone Consulting Engineers). The structural Engineer of Record shall retain sole responsibility for the structural design of the entire building.

2. Design Parameters and Building System

2.1 Design Codes and References

- 2014 New York City Building Code
- ACI 318-11 Building Code Requirements for Structural Concrete and Commentary
- ASCE 7-2005 Minimum Design Loads for Buildings and Other Structures

2.2 Material Properties

The following materials are specified in the structural drawings.

- Concrete shear walls and columns : 10,000 psi to 6,000 psi
- Concrete Slabs : 8,000 psi to 5,000 psi
- Reinforcing bars: Grade 80 for #14 and larger, Grade 75 for #10 and #11, Grade 60 for #9 and smaller

2.3 Design loads

2.3.1 Floor Uniform Gravity Loads (based on the load maps in S-013 to S-018 drawing)

	Uniform Dead Load (*psf*)	Superimposed Dead Load (*psf*)	Live Load (*psf*)
Typical floors for Residential units	100	15	40
Typical floor Balconies	100	15	60
5th Floor Outdoor Terrace	175	Varies (Paver area and lawn area)	100
Main Roof	150	180	100
Typical mechanical floors	150	30	150
Residential Amenities	150	Varies	100

The façade load for typical floors is 0.2 *kip/ft*.

The gravity load values are code compliant and in conformance with conventional practice.

2.3.2 Snow Loads

The ground snow loads, Pg, is 25 psf and the flat roof snow loads, Pf, is 20 psf minimum. This designation is different from the snow design data indicated on the drawing. However, since the live load of the roof is significantly higher than the snow loads, this discrepancy will have no effect on the structural design.

2.3.3 Wind Loads

Wind loads are estimated from the wind tunnel testing by CPP Inc. Herein the interim wind tunnel load is used in our independent wind load analysis.

- Basic Wind Speed for New York City: 98 *mph* 3-second gust speed at 33 feet above the ground in wind exposure C (based on local wind climate with annual probability with 0.02, 50 year mean recurrence interval)
- Importance Factor: $I_W = 1.0$ (Structural Occupancy Category II)
- Exposure: C
- Assumed damping ratio: 2% for estimation of structural loads
- Design wind loads for 50 years recurrence wind (wind tunnel testing)
  - Maximum wind load in N-S Direction: 1839 kips (Wind load case 2)
  - Maximum wind load in E-W Direction: 3660 kips (Wind load case 4)
- Design wind loads for 10 years recurrence wind (wind tunnel testing)
  - Maximum wind load in N-S Direction: 909 kips (Wind load case 2)
  - Maximum wind load in E-W Direction: 2207 kips (Wind load case 3)

We have communicated with the E.O.R. that the building periods used for the structural analysis and the wind tunnel study are different by approximately 15% and we believe that this discrepancy would affect the magnitude of the static wind loads provided by the wind tunnel testing. In response to this, the E.O.R has obtained the wind load scale factor from CPP which will be used for the final structural design.

2.3.4 Flood Loads

The ground floor slab is raised to elevation +12'-0" which is above the current Base Flood Elevation of 11'-0" and therefore the flood loads are not applicable to the above-ground structure.

2.3.5 Seismic loads

- Site: New York City ($S_s = 0.281\ g$, $S_1 = 0.073\ g$). According to the geotechnical report by LANGAN, a Site-Specific Seismic Study for the site at 532 Neptune Avenue was carried out. In the peer review we use the parameters specified in the structural drawings by the Engineer of Record.
- Seismic Use Group I (Occupancy category II):
- Site Class: D ($F_a = 1.57$ & $F_v = 2.4$)
- Importance Factor: $I_E = 1.0$ (Seismic use group I)
- Load Resisting System: "Bearing Wall System with Ordinary Reinforced Concrete Shear Wall"
- Response Modification Factor: $R = 4.0$
- System Over-strength Factor: $\Omega_0 = 2.5$
- Deflection Amplification Factor: $C_d = 4.0$
- Seismic Design Category: B
- Seismic Base Shear: 121,000 *kips* × 0.013 = 1,573 *kips*
  - Seismic Response Coefficient: $C_s = 0.013$ (As per ASCE 7-05, Sect. 12.8.1.1)
  - Building Effective Weight: 121,000 *kips* (approximately)
- Seismic force is significantly less than the wind force and therefore it does not govern the structural design.

2.4 Structural System

2.4.1 Gravity Load Resisting System

An 8-inch/12 inch thick flat slab supported by cast-in-place concrete columns and shear walls are utilized to resist the gravity loads.

2.4.2 Lateral Load Resisting System

It is confirmed by the Engineer of Record that only the shear walls with link beams are utilized to resist the wind and seismic loads. Frames are not considered to participate in resisting the lateral loads.

2.5 Foundation System

A pile foundation was recommended by Langan and is used in the building foundation design by the Engineer of Record. The 18" diameter Auger Cast-In-Place piles with 250 ton allowable axial compression capacity and 75 ton allowable uplift capacity are used in the building tower portion.

3. Building Analysis

3.1 Building periods

In our independent ETABS analysis models, the section properties with cracking effect for shear walls and link beams are assigned based on ACI 318 suggestions and our structural design practice as necessary. The concrete slabs are not considered as part of the lateral system and therefore in our model, we neglected the contribution of the slab by reducing their stiffness into 0.01.

The first three preliminary building natural periods are obtained as:

- 1st Mode: 5.21 sec (Primary North-South direction, X direction)
- 2nd Mode: 4.6 sec (Primary Torsion)
- 3rd Mode: 4.33 sec (Primary East-West direction, Y direction)

Note that the first natural mode shape is along the north-south direction even though the dimension of the building along this direction is larger. The reason for the above noted is that the shear wall stiffness along the east-west direction is designed to be larger than the stiffness along the north-south direction.

3.2 Maximum Drift

A. Wind loads:

The maximum wind total-drift based on 10-year wind load

- North-South direction: 4.8 inch (H/1110)
- East-West direction: 7.7 inch (H/690)

The maximum wind story-drift based on 10-year wind load

- North-South direction: 1/887 at the 20th floor
- East-West direction: 1/412 at the 30th floor

The story drift values are in conformity with the value 1/400, which is the current common design industry engineering practice of allowable story drift ratio for high-rise building.

B. Seismic loads

The maximum seismic elastic inter-story drift

- North-South direction: 0.41 inch (h/340)
- East-West direction: 0.47 inch (h/295)

The amplified inter-story drift with $C_d = 4.0$ is less than the allowable maximum drift of (h/50).

3.3 Column and Shear Wall Long-term Shortening

Based on the computed gravity loads for columns and shear wall piers, the long-term shortening for the vertical members are estimated by RGCE based on the suggestion in ACI 209.2R-08. It is found that the maximum long-term column shortening is about 5.6” at the top of the building. According to our engineering experience, the effect of the column shortening on building façade and elevators can be effectively controlled if they are considered in the building design.

3.4 Human Perception and Occupant Comfort

A high-rise building tends to constantly move under the wind loads and excessive occupant discomfort shall be avoided by limiting the peak accelerations (peak torsional velocities) at the topmost occupied floor of the building. Some design codes specify an approximate formula to estimate the building peak accelerations, but the results are not reliable due to the complexity and uncertainty of wind loading. In current design practice the peak acceleration value based on wind tunnel testing is typically used to check if the human discomfort can be controlled. The interim wind tunnel testing report indicates that the 10-year peak acceleration using a damping ratio of 0.02 is 17 mg. This acceleration is acceptable for the residential building.

4. Structural Members Design and Discussion

The design for the main types of structural members in this building are reviewed and discussed in this section. Note that the discussion is based on the structural drawings dated August 18, 2016 and the updated structural drawings dated October 14, 2016 with the understanding that the structural design is still underway.

4.1 Pile Foundations

- Pile foundations supporting columns

  The gravity loads for the columns are computed and compared with the foundation plan in the current structural drawings. In general, the pile foundations have sufficient capacities to support the columns. However, the pile cap reinforcement, for example TPC-6, does not have sufficient capacity and needs to be increased.

  We have communicated with the E.O.R. and the pile cap reinforcements have been updated in the latest structural drawings.

- Pile foundations supporting shear walls

  Since the building is designed on the assumption that the shear walls and link beams resist all the lateral loads, uplift forces are expected to occur in the shear wall piles.

  We have communicated with the E.O.R. that our ETABS analysis indicates that there are significant uplift forces under the shear walls that need to be addressed. In response to this, on the letter dated October 28, 2016, the E.O.R. has elaborated their approach in designing the mat foundation for our review. The E.O.R utilized a finite element SAFE model to analyze the mat foundation. The concrete mat and the pile supports with the spring constants were entirely modeled and the shear wall and column loads from the ETABS and the spreadsheet were applied to the mat. The ultimate load combinations were run in order to get the mat reinforcement. We believe that this approach is acceptable and can be applied. We also reviewed the SAFE model of mat foundation supporting SW-3 and SW-4 provided by E.O.R. and the computed reinforcement was compared to the reinforcement shown in the structural drawing. The reinforcement is generally reasonable.

4.2 Shear Walls and Link Beams

The design of shear walls and link beams were spot-checked on the basis of the August 18th, 2016 structural drawings. Below are some of our findings:

1) SW4 supporting 6th-15th floor requires more longitudinal reinforcement than what is shown in the plan.
2) No longitudinal reinforcement is shown for SW3 and SW4 supporting 16th-20th floor.
3) SW1 supporting 21st-29th floor requires more longitudinal reinforcement than what is shown in the plan. In addition to that, the current reinforcement shown is approximately 4.2%, please consider using mechanical couplers at the rebar splice.
4) SW6 supporting 2nd-5th floor shows 3 layers #8@6"/#7@6" vertical. Can #10@6" E.F. or #9@6" E.F. be used instead for easier installation?
5) Link beams LB-03 from 10th- 25th floor are overstressed.

We have communicated with the E.O.R. and the entire shear walls and link beams have been redesigned in the latest October 14th, 2016 structural document. On the letter dated October 28th, 2016, the E.O.R. elaborated their methods of designing the shear wall which utilizing the combination of several commercial finite element software SAFE and ETABS and the in-house spreadsheet in order to get maximum forces. The S-concrete software is finally being used to design the reinforcement.

We believe that the method described above is acceptable and can be applied as long as all the steps are carefully followed. We also performed the final spot-check using our independent ETABS at the most critical SW-1, SW-2, SW-3 and SW-4 at various floors to get the approximate required reinforcement at the boundary zone and found that the reinforcement provided on the design document were very high (approximately 5%-8% reinforcement ratio).

We recommend that the E.O.R reviews the reinforcement ratio at all shear walls to confirm that it is below 8% and confirms that the mechanical coupler is being used for all shear walls with over 4% reinforcement ratio.

We also reviewed the link beam design sample calculation provided by E.O.R and the method used in the calculation is acceptable. The maximum moment and shear forces at the link beam LB-01 at 25th floor was also compared with the forces from our ETABS model and we found that the design forces used in the sample calculation is approximately 15% larger and therefore it is a conservative design.

4.3 Columns

The gravity loads for the columns are computed and the column capacities under gravity loads are checked. Generally, the tower column loads at the foundation level shown on the column schedule are 5%-10% higher than our independent calculations.

Some of our deficiencies that we found during the review were listed below:

1) The reinforcement is generally adequate for most columns. However when we spot-checked column #21 and #22, we found that these columns require more reinforcement than what is shown in the column schedule.
2) Column #14 between 4th-5th floor is a walking column and the design has not been done to address the moment force due to eccentricity.
3) Column #1 at 4th-5th floor is not consistent between plan and column schedule.

We have communicated with the E.O.R. and the column design has been rechecked and updated in the latest column schedule.

4.4 Flat Slabs

A. Two-way punching shear capacity

Two-way punching shear capacities under the load combination of $1.2DL + 1.6LL$ (the governing gravity load case) are checked for the typical floor and the 5th floor using finite element SAFE software. Owing to the close column layout and adequate column sizes, we find that all the slab-column connections in the typical floors and the 5th floor have appropriate punching shear capacities.

B. Flexural Design

The flat slab flexural design is carried out for the typical floors (6th floor~ 34th floor) and 3rd floor by using the SAFE software. The required reinforcing was calculated and compared to the reinforcing specified in the structural drawing. In general, the flexural design was reasonable and found to be in accordance with code requirements.

The top reinforcement length for the ground floor slab should be increased because the supports are pile caps instead of columns, while the reinforcement can be adjusted owing to smaller clear spans. We've communicated with the Engineer of Record and the revision is being done.

C. Serviceability

Slab deflection due to dead loads and live loads for typical floor is checked. Our computation indicates that:

1) The immediate deflection due to live load: $\Delta_{max,live} = 0.1''$
2) The total long-term deflection due to dead load and live load: $\Delta_{max,longterm} = 0.9''$

The deflection values are relatively small due to the close spans between the columns, and the slab deflection satisfies the code requirements.

5. Reviewer's Opinions

Rosenwasser/Grossman Consulting Engineers, P.C. has completed the peer review of the design documents prepared by the Engineer of Record (DeSimone Consulting Engineers). Based on our independent analysis and spot-checking the structural drawings available to us, we've concluded that:

1. The building design gravity loads and wind design loads used in the structural design are in conformance with the 2014 New York City Building Code and the ACI 318-2011.
2. The layout of the primary structural system is well distributed. There are no horizontal structural irregularities and vertical structural irregularities. The structural plans are generally consistent with the architectural drawings.
3. There are complete load paths for both gravity loads and lateral loads in the building structure.
4. The building has enough stiffness for both wind design loads and seismic design loads. The building maximum story drifts under both seismic and wind loads are within the code specified limits and the current design practical limits. The structural integrity provisions specified in the design codes are properly followed.
5. Local resistance method was performed by the Engineer of Record to carry out the key-element analysis for this building.
6. The current foundation design is in compliance with the recommendations by the geo-technical engineers.
7. The structural member designs are generally adequate to support the resist the applied forces.

Appendix A. Code Compliant Check List

Peer Review – Code Compliance Check List as per 2014 NYCBC section BC 1617.5.1 - Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
1. Design Loads					
1) Dead and Live loads	NYCBC BC 1607 Table 1607.1	Floor load maps on Drawing S-013 to S-018		√	
2) Snow loads	NYCBC BC 1608	Snow design data on Drawing S-010		√	The snow design criteria need to be revised. However, the snow loads do not govern the design.
3) Wind loads	NYCBC BC 1609	• Interim wind tunnel testing report by CPP dated June 9, 2016 • Supplemental 50-years wind tunnel scale factor by CPP received on September 21, 2016	The 10 years and 50 years recurrence design wind loads are provided by CPP from the wind tunnel testing. As a result of the continuous coordination, the building period has increased and therefore the supplemental scale factor was obtained in order to get more precise wind load.	√	The amplified wind loads will be used for the final structural design.
4) Soil lateral loads	NYCBC BC 1610			N/A	There are no foundation walls and retaining walls on the project.
5) Flood Loads	NYCBC BC 1612			√	The ground floor slab is raised to elevation +12'-0" which is above the current Base Flood Elevation of 11'-0" as noted on the geotechnical report.

Peer Review – Code Compliance Check List as per 2014 NYCBC section BC 1617.5.1 - Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
6) Seismic loads	NYCBC BC 1613	Seismic design data on Drawing S-010		√	The seismic load does not govern the structural design
2. Structural Design Criteria and Assumptions					
1) Serviceability					
A. Lateral displacement	NYCBC BC 1604.3 and ASCE 7-10 Section 12.12	Structural drawings	• Story drift due to wind loads: As confirmed by the E.O.R, 10-year recurrence wind loads were used to estimate story drift for evaluation of serviceability. According to our study, story drift at the critical floor deems acceptable. • Inter-story drift due to earthquake loads: less than 0.02 x hn (the maximum allowable inter-story drift for seismic use group I).	√	Story drift criteria (hn/400 at 10 year recurrence wind) used for design can be acceptable, as long as non-structural elements such as cladding and components, partitions and mechanical equipment are properly designed to accommodate this estimated building movement.
B. Perception to motion	ISO criteria (Selected by the wind tunnel testing lab)	Structural drawings	• 10 year wind tunnel results based on 2% of critical damping indicated an acceptable acceleration. • The E.O.R. has coordinated with the wind tunnel consultant to	√	

Peer Review – Code Compliance Check List as per 2014 NYCBC section BC 1617.5.1 - Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
			check that the change in the building period will not result in the acceleration issue.		
2) Analysis	NYCBC BC 1604.4	Structural drawings	• An independent structural analysis model was generated to analyze and design the primary structural members. • The overall behavior of the structure and internal forces at members were reviewed and compared with the original design.	√	
3) Anchorage to foundation	NYCBC BC 1604.8	Foundation drawings (FO-series)	• Axial loads at columns were independently calculated from base (foundation) to top (main roof). All columns are designed only for gravity loads. • Axial loads at shear walls were independently calculated from base (foundation) to top (main roof). The shear walls are designed to resist both the gravity and the lateral loads.	√	The peer review report clarifies the approach used by E.O.R to design the mat foundation.

Peer Review – Code Compliance Check List as per 2014 NYCBC section BC 1617.5.1 - Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
3. Conformity of structural design with engineering investigation					
1) Geo-technical engineering report		• Structural drawings • Geotechnical report dated May 22, 2015 • Correspondences between E.O.R and geotechnical engineer	• It is recommended to use 18 inch diameter Auger-Cast-In-Place (ACIP) drilled pile to support the shear walls and columns. • The groundwater level was found at about el. +1 and the Base Flood Elevation is at el. +11. • The ACIP piles will be utilized to resist the compression and tension axial forces.	√	
2) Wind tunnel testing report		• Interim wind tunnel testing report by CPP dated June 9, 2016 • Supplemental 50-years wind tunnel scale factor by CPP received on September 21, 2016	• Wind forces and moments are based on a 50 year/10 year recurrence wind. • 10 load cases in consideration of directionality of wind and the structural dynamic properties of the building are provided • The wind tunnel testing report indicated that the 10-year peak accelerations with a 2% of critical damping ratio is within the commonly acceptable range for	√	

Peer Review – Code Compliance Check List as per 2014 NYCBC section BC 1617.5.1 - Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
			residential buildings.		
4. Review the structural frame and the load supporting parts of floors, roofs, walls, foundations.		Structural drawings		√	Other secondary structural items are excluded.
5. Complete load path					
1) Gravity loads		Structural drawings	• Gravity loads are resisted by cast-in-place flat plate (horizontal elements) and cast-in-place columns and shear walls (vertical elements).	√	Load path for the gravity loads is complete
2) Wind loads		Structural drawings	• Wind loads are transferred to shear walls by rigid diaphragm (typically 8”/12” thick flat plate) • Lateral load resisting system consists of the reinforced concrete shear walls with link beams. • Ground floor was assumed to be the base for the lateral loads	√	Load path for the wind loads is complete
3) Seismic loads		Structural drawings	• Seismic loads are transferred to shear walls by rigid diaphragm	√	Load path for the seismic loads is complete

Peer Review – Code Compliance Check List as per 2014 NYCBC section BC 1617.5.1 - Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
			(typically 8"/12" thick flat plate) • Lateral load resisting system consists of the reinforced concrete shear walls with link beams. • Ground floor was assumed to be the base for the lateral loads		
6. Design of members	NYCBC BC 1617.5.2	Structural drawings	Representative structural elements (flat plate at one typical floor, shear walls, columns, link beams, pile caps, mat foundation) to be checked based on the results from our analysis.	√	
1) Flat plate		• Typical floor framing plans	• Adequacy of slab thickness and reinforcing is reviewed. • Slab reinforcing due to gravity load and the punching shear ratio are checked using the software SAFE.	√	
2) Shear wall		• Shear wall rebar plans	• Shear wall reinforcing at various floors are spot-checked.	√	The amplified wind load will be used for the final structural design.
3) Columns		• Column schedule	• Column reinforcing at various floors are spot-checked.	√	

Peer Review – Code Compliance Check List as per 2014 NYCBC section BC 1617.5.1 - Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
4) Link beams		• Link beam schedule	• Design of link beams (cast-in-place concrete and embedded steel beam) is reviewed.	√	The amplified wind load will be used for the final structural design.
5) Pile cap supporting shear walls and columns		Foundation drawings (FO-series)	• Adequacy of pile layout is reviewed. • Adequacy of the specified pile capacity is reviewed. • Adequacy of thickness and reinforcement of pile cap is reviewed.	√	
7. Performance-specified structural components					
1) Supplementary damping system					Supplementary damping is not required.
2) Cladding					Cladding design and performance and their connections are excluded in this review.
8. Structural Integrity					
1) Prescriptive requirement	NYCBC BC 1615				
A. Continuity and ties	NYCBC BC 1615.2				

Peer Review – Code Compliance Check List as per 2014 NYCBC section BC 1617.5.1 - Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
• Slab reinforcing	NYCBC BC 1916.2.1	Structural drawings	Continuous mat of bottom reinforcement is provided in two perpendicular directions at all levels.	√	
• Peripheral ties	NYCBC BC 1916.2.2	Structural drawings			The continuous perimeter tie reinforcement is not clearly indicated in the structural drawing.
• Horizontal ties	NYCBC BC 1916.2.3	Structural drawings		√	The column integrity bottom bars are indicated on plan.
• Vertical ties	NYCBC BC 1916.2.4	Structural drawings		√	The columns are tied continuously from foundation to the roof.
B. Lateral bracing	NYCBC BC 1615.3	Structural drawings	Floor slabs at each floor are connected to the columns and shear walls	√	
C. Vehicular impact	NYCBC BC 1615.5			N/A	
9. General conformance of structural plans with architectural plans		• Structural drawing – Peer Review Set dated August 18th, 2016 • Architectural set dated September 2nd, 2016		√	In general, the geometry, the size and the location of the primary structural members are consistent with the architectural drawing.
10. Major mechanical items					

Peer Review – Code Compliance Check List as per 2014 NYCBC section BC 1617.5.1 - Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
1) Water tank				√	The weight of the water tank is indicated on the load maps S-018, but the supporting structural elements have not been designed.
2) Emergency generator					Not indicated in the structural drawing.
3) Cooling tower					Not indicated in the structural drawing.
4) Fuel oil tank					Not indicated in the structural drawing.
5) Supplementary damping system					Supplementary damping is not required.
11. General completeness of structural drawings		Structural Drawing – Peer Review Set dated August 18th, 2016		√	The structural design is generally completed with the exception of few items noted in the peer review report.

Appendix B. Log of Structural Findings

Date Received	Peer Review Comments from RGCE	Date Responded	DeSimone's Response	Location
11-Apr-16	Based on the ETABS model you provided, the framing effect of the flat slab on the entire structure is not considered, so the seismic force resisting system for this project should be "Bearing wall systems: ordinary reinforced concrete shear walls", instead of "Building Frame systems: ordinary reinforce concrete shear walls". As a result, the Response Modification Factor should be 4.0 instead of 5.0 as you specified. This will change the seismic base shear. Please confirm/revise the associated parameters (DWG. S-010.01).	20-Oct-16	This has been updated	See S-010
11-Apr-16	We can read the coordinates of columns and shear walls from your Revit model for our review. For the construction documents comprehensiveness, please mark all the coordinates on your drawings.	20-Oct-16	This has been updated	See 200 Series drawings
11-Apr-16	On the drawing S-342, there is a cantilever slab next to the column 117. Please review the deflection and the top reinforcing for the cantilever slab.	20-Oct-16	This has been updated	See S-342
11-Apr-16	On the drawing S-360, the columns 21 and column 22 shifted at the 35th floor, please review the shear capacities.	20-Oct-16	This column walks below, it has been updated	See S-401-S402 and S-361
25-Apr-16	Pile foundation capacity for the column 7 is not sufficient. The loads for the column 7 at lower floors are not correct.	20-Oct-16	This has been updated	See FO-103
25-Apr-16	It looks like that the pile foundation capacities for the columns 5, 6 and 11 are not enough (100~200 kips less). Could you please double check the loads at the foundation level?	20-Oct-16	This has been updated	See FO-103

25-Apr-16	Some of the slab top rebar lengths are less than the minimum requirements specified by ACI-318 (For example, column 9 at the typical floors). Please double check top rebar lengths.	20-Oct-16	All slabs have been redesigned	See 300 Series drawings
25-Apr-16	We noticed that the reinforcements for columns above the 5th floor are typically 16-#6, which make the most of the column design conservative. However, some columns needs more reinforcements than 16-#6 (For example, column 21 at 26th floor).	20-Oct-16	All columns have been redesigned	See S-401 and S-402
13-Sep-16	We modified our ETABS model based on the latest structural drawing dated August 18, 2016 and the analysis indicates that the 1st mode building period is 5.2 sec (E-W direction) and the 2nd mode building period is 4.6 sec (N-S direction). We also received the latest CPP wind tunnel report dated June 9, 2016 which indicates the E-W building period is 4.42 sec (f=0.2262 Hz) and the N-S building period is 4.34 sec (f=0.2304 Hz). The periods used at the structural analysis and the wind tunnel study are different by 15%, We believe that this high difference would affect the wind tunnel study in determining the static wind loads.	21-Sep-16	There were several changes in the shear walls due to the continuous coordination with the architect after we sent the structural properties for wind tunnel testing. The changes results in a change of the 1st mode period of 5.6 sec. and 2nd mode of 5.0s. Adjustments of shear wall thickness in the elevator cores. - Change in orientations of the end bulbs of the shear walls. - Additional Mechanical level added above main roof and the overall building height increased 11'. - Superimposed dead load has been updated in the upper floor to account for landscape, mechanical equipment and water tanks etc - Modifications in the extent of the slabs in top of the building above main roof - Overall building weight increased by approx. 5% due to the above 3 points	See email from Alex on Septebmer 21, 2016

13-Sep-16	The structural drawing F0-109 and FO-110 shows very minimal tension piles. Our analysis indicates that under the 0.6(DL+SDL+FAÇADE)+1.0 (50 YRS WIND 1-10) combination, there are significant uplift forces that need to be addressed.	23-Sep-16	As we discussed on the phone, our office used a SAFE model to analyze the mat foundation for the elevator core and other shear walls. The model includes the entire mat foundation on top of spring supports which has stiffnesses provided by the geotechnical engineer. The compression piles are defined as "compression only" springs. Two springs are defined for tension and compression at the locations of tension piles. Forces from shear walls are extracted from our Etabs model and applied as line loads to the mat. The loads from the columns are applied as point loads. Please see attached pdf file for the dead load and wind load case WT04 (worst case for uplift in north side) applied and the pile reactions with the combination "0.6 DEAD + 1.0 WIND". As shown in the last page, overturning moment of the building is resisted by the reactions from the compression piles in the south side and the tie down force from the tension piles in the north. All the pile reactions are below the design capacity of the piles. We are current in the process of optimizing the design and will add tension piles in back in the south side. Please let us know if you have any comment on our design approach.	See email from Alex on September 23, 2016
13-Sep-16	Drawing S-411, no dimension at the SW3/SW4 wall opening and no information for the link beam above it?	21-Sep-16	Dimension is marked up for drafting. Spandrel currently being designed.	See S-411
13-Sep-16	Drawing S-411, no information for the link beam at SW2?	21-Sep-16	Spandrel currently being designed.	See S-411
13-Sep-16	Drawing S-412, no information for the beam at SW-1?	21-Sep-16	No beam there.	See S-412
13-Sep-16	Drawing S-413, lines and dimensions drafting errors.	21-Sep-16	Marked up for drafting.	See S-413

13-Sep-16	Drawing S-417, no dimension at the length of SW1 and SW2?	21-Sep-16	Marked up for drafting.	See S-417
13-Sep-16	It appears that there is a wall opening at SW-1 between 4th and 5th floor, however there is no information at the shear wall plan S-414. Please confirm.	21-Sep-16	Marked up for drafting.	See S-430 for Shear Wall Elevation
13-Sep-16	Drawing S-342 indicates 14" thick podium slab, but the drawing S-016 indicates 150 psf slab weight. Please confirm.	21-Sep-16	The slab weight is 175 psf, it is marked up for drafting.	See S-016
13-Sep-16	Drawing S-343 – It appears that the drawing is incomplete and the floor loading is not available.	21-Sep-16	The mezzanine is being coordinated with the architect and MEP engineers and currently being designed.	See S-343
13-Sep-16	The size of LB-01 is not consistent between plan S-361 and detail S-405. Please confirm.	21-Sep-16	S-405 has the most correct sizes, S-361 is marked up for drafting.	See S-405
13-Sep-16	The size of LB-04 is not consistent between plan S-360/S-361 and detail S-405. Please confirm.	21-Sep-16	S-405 has the most correct sizes, S-361 is marked up for drafting.	See S-405
15-Sep-16	We started to randomly review the column design and please see the attached pdf for comments. Please send us the column design calculation for our review.	21-Sep-16		Sent as attachment with email from Alex on September 21, 2016
15-Sep-16	Can you please confirm that the SDL of main roof slab is 180 psf as indicated on S-018?	21-Sep-16	SDL of 180psf was taken as an assumption based on incomplete info from the architect and landscaping loads from previous projects. The main roof shall be updated as 68 psf (paver area) or 118 psf (lawn area) based on latest drawings and layout from landscape architect Todd Rader+Amy Crews received on 09/08/2016.	See email from Alex on September 21, 2016
15-Sep-16	Looking at the load map S-017 and architectural drawing A-110, how do you justify/apply the floor loading for the 5th floor outdoor terrace?	21-Sep-16	Calculation breakdown for the superimposed dead loads for the 5th floor terrace are attached.	See email from Alex on September 21, 2016

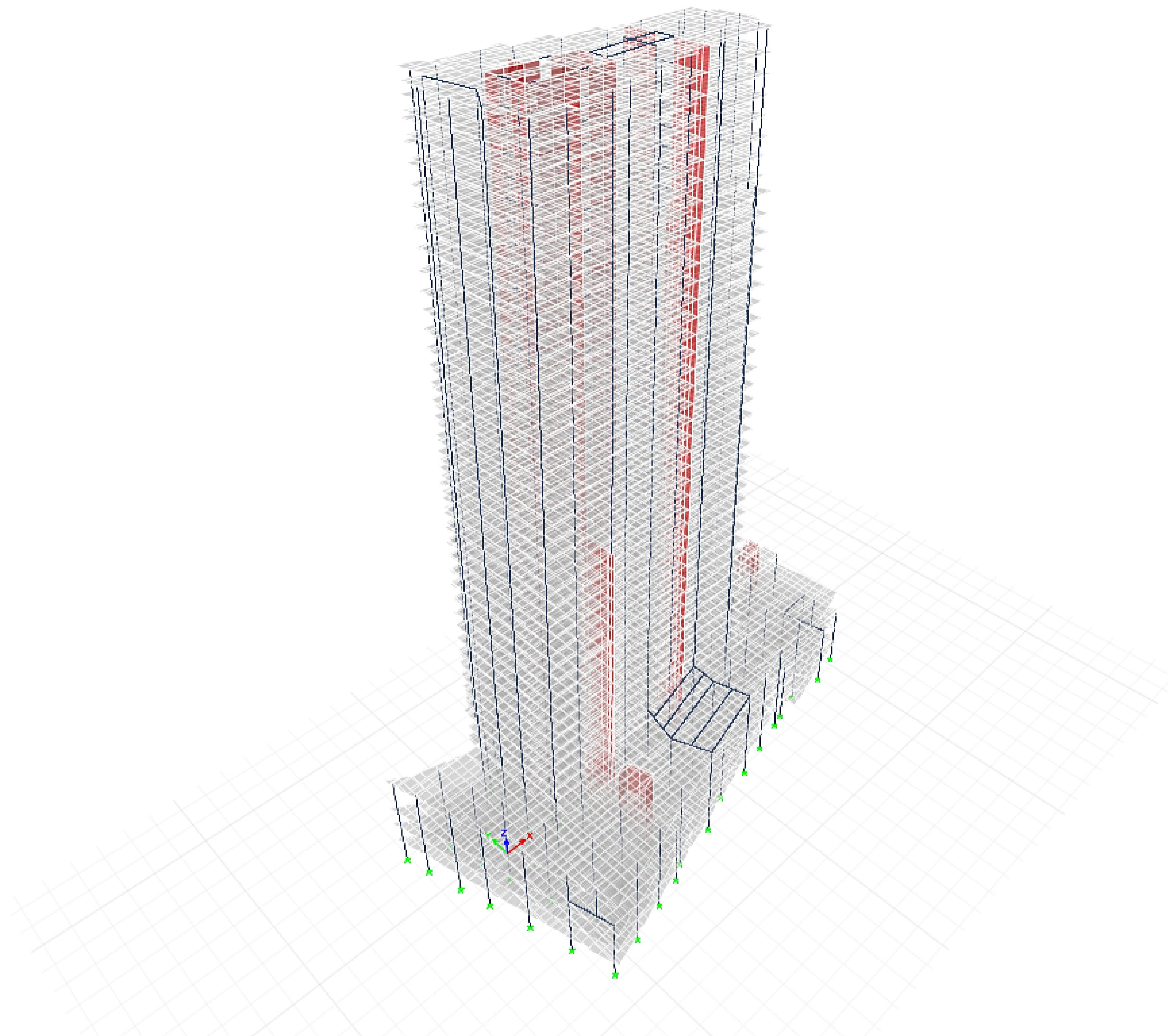
19-Sep-16	We did a quick check for the pile cap reinforcement and please see attached pdf. Please send us the sample of pile cap calculation for review.	20-Oct-16	Pile cap reinforcing has been updated	See FO-106
19-Sep-16	Column #14 at 4th – 5th floor appears to be a walking column and it seems that the design has not been done to address the moment force due to the eccentricity. Please confirm.	21-Sep-16	Column walk is happening. Mid-depth bars have been added to account for the moment due to the eccentricity. See attached 4th and 5th floor reinforcement plans.	See S-342 and S-352
19-Sep-16	Column #1 at 4th-5th floor is not consistent between plan and column schedule. Please clarify.	21-Sep-16	Column schedule has been updated to reflect the changes in the plan.	See S-401 and S-402
19-Sep-16	Is column #24 intentionally sitting on the eccentric pile cap TPC-4? Please clarify.	21-Sep-16	It is a drafting error. The pile cap should be concentric with the column	See FO-103
21-Sep-16	With the new building period, please confirm with CPP that there is no acceleration issue. (We are assuming that all the increased mass, particularly at the top of the building, have been accounted for in your analysis)	23-Sep-16	We have discussed with CPP about the change in building period and have sent the updated building properties for them to run the acceleration check of the building. We will update you once we receive the new acceleration from CPP. We have looked into the loads at the top of the building in our Etabs model and were able to reduce some of the weight. The main roof SDL and the EMR SDL (water tanks weight) assumptions were too high in our previous model V73. Also, the link beams stiffness was adjusted to account for the built-up steel members embedded in the link beams in most of the floors. After the above adjustments, the current 1st mode period is 5.46s.	See email from Alex on Septebmer 23, 2016

21-Sep-16	The May 22, 2015 geotechnical report did not specifically mention the pile tension capacity and it says that the reinforcement pending additional compression, tension, and lateral analysis. Do you have the final design recommendation from the geotech engineer?	23-Sep-16	There has not been any formal report issued by the geotechnical engineer since the May 22, 2015 report. The tension capacity of the 18" ACIP piles, as well as their spring stiffness (compression and tension) were sent to us via email. Please see attached.	See email from Alex on Septebmer 23, 2016
4-Oct-16	SW4 supporting 6th-15th floor requires more longitudinal reinforcement than what is shown in the plan.	20-Oct-16	All Shear Walls have been redesigned	See 400 series drawings
4-Oct-16	No longitudinal reinforcement is shown for SW3 and SW4 supporting 16th-20th floor.	20-Oct-16	All Shear Walls have been redesigned	See 400 series drawings
4-Oct-16	SW1 supporting 21st-29th floor requires more longitudinal reinforcement than what is shown in the plan. In addition to that, the current reinforcement shown is approximately 4.2%, please consider using mechanical couplers at the rebar splice.	20-Oct-16	All Shear Walls have been redesigned	See 400 series drawings
4-Oct-16	SW6 supporting 2nd-5th floor shows 3 layers #8@6"/#7@6" vertical. Can #10@6" E.F. or #9@6" E.F. be used instead for easier installation?	20-Oct-16	All Shear Walls have been redesigned	See 400 series drawings
4-Oct-16	Link beams LB-03 from 10th- 25th floor are overstressed.	20-Oct-16	All Link Beams have been redesigned	See S-405

Appendix C. Sample of ETABS Output

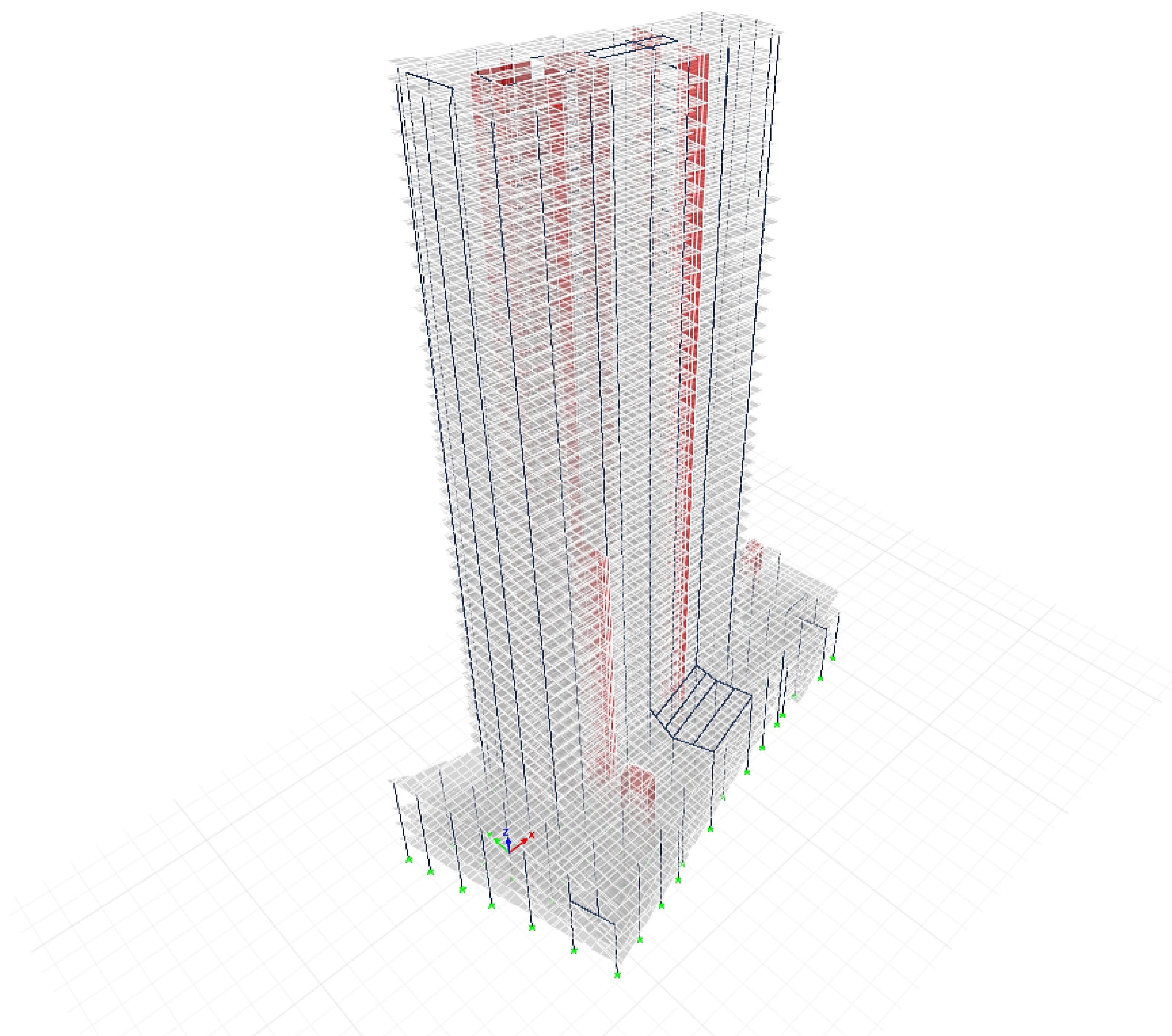
3-D View Mode Shape (Modal) - Mode 1 - Period 5.209

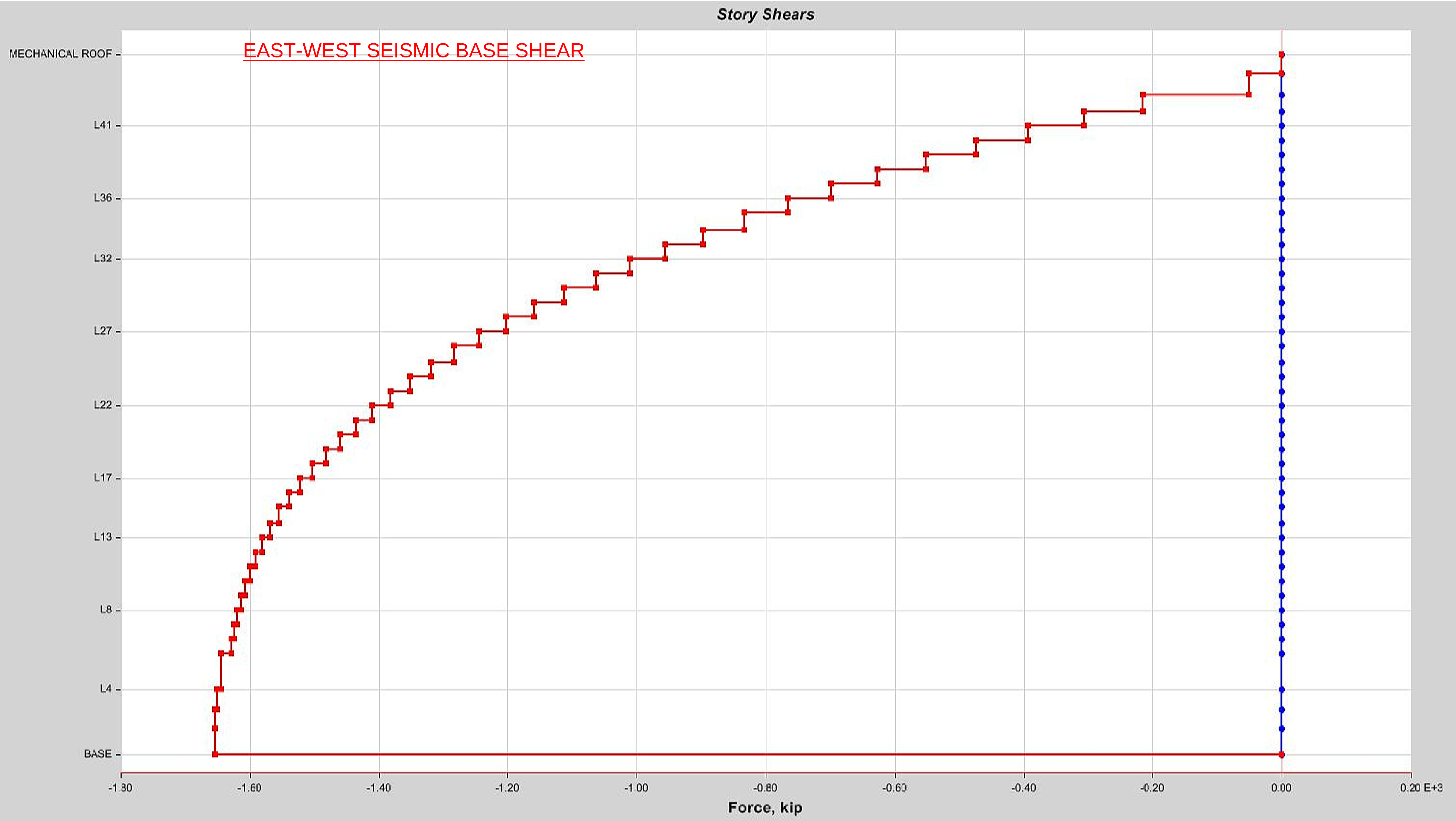
FIRST MODE PERIOD = 5.21 S

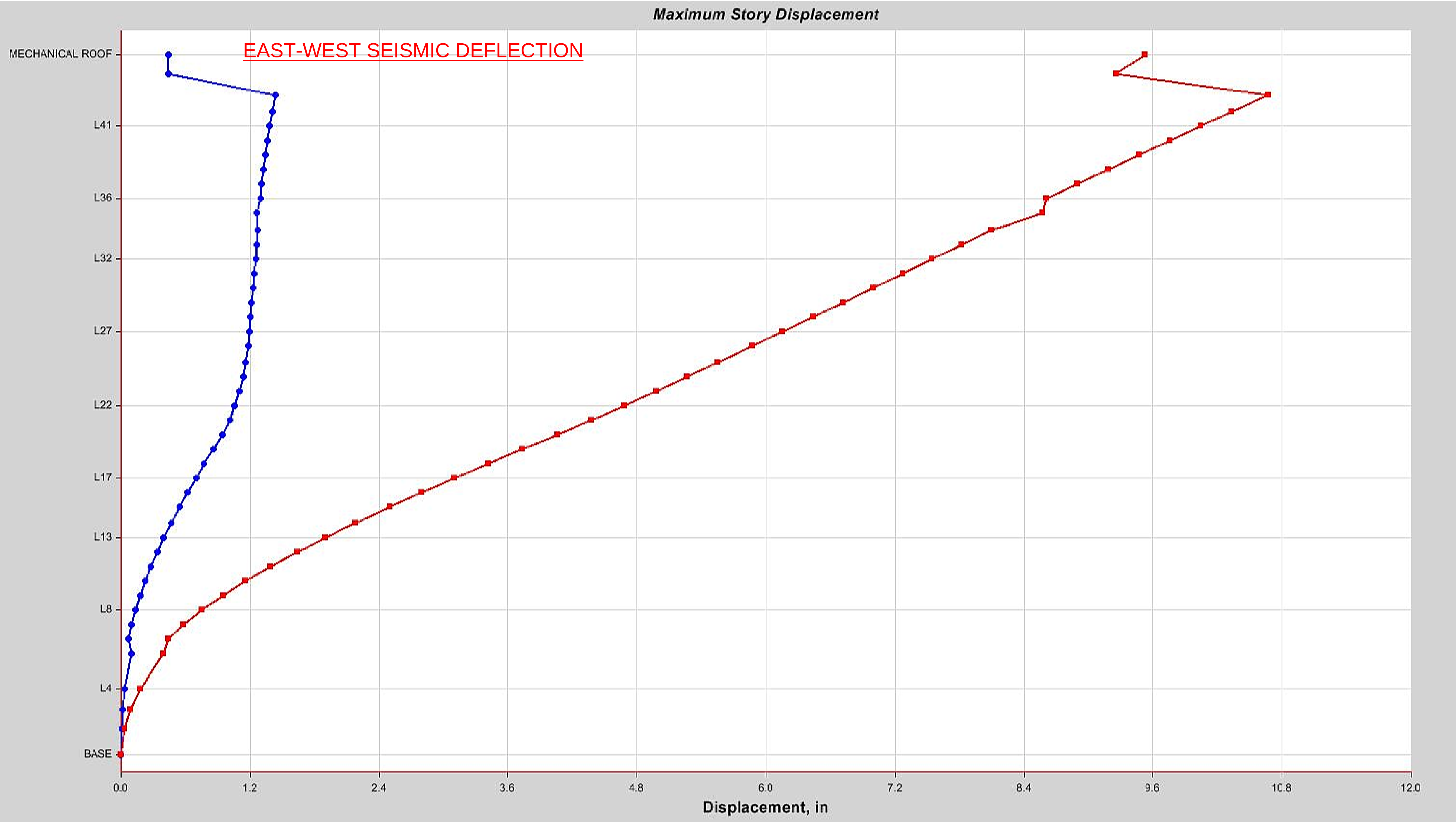


3-D View Mode Shape (Modal) - Mode 2 - Period 4.7 | Story Response

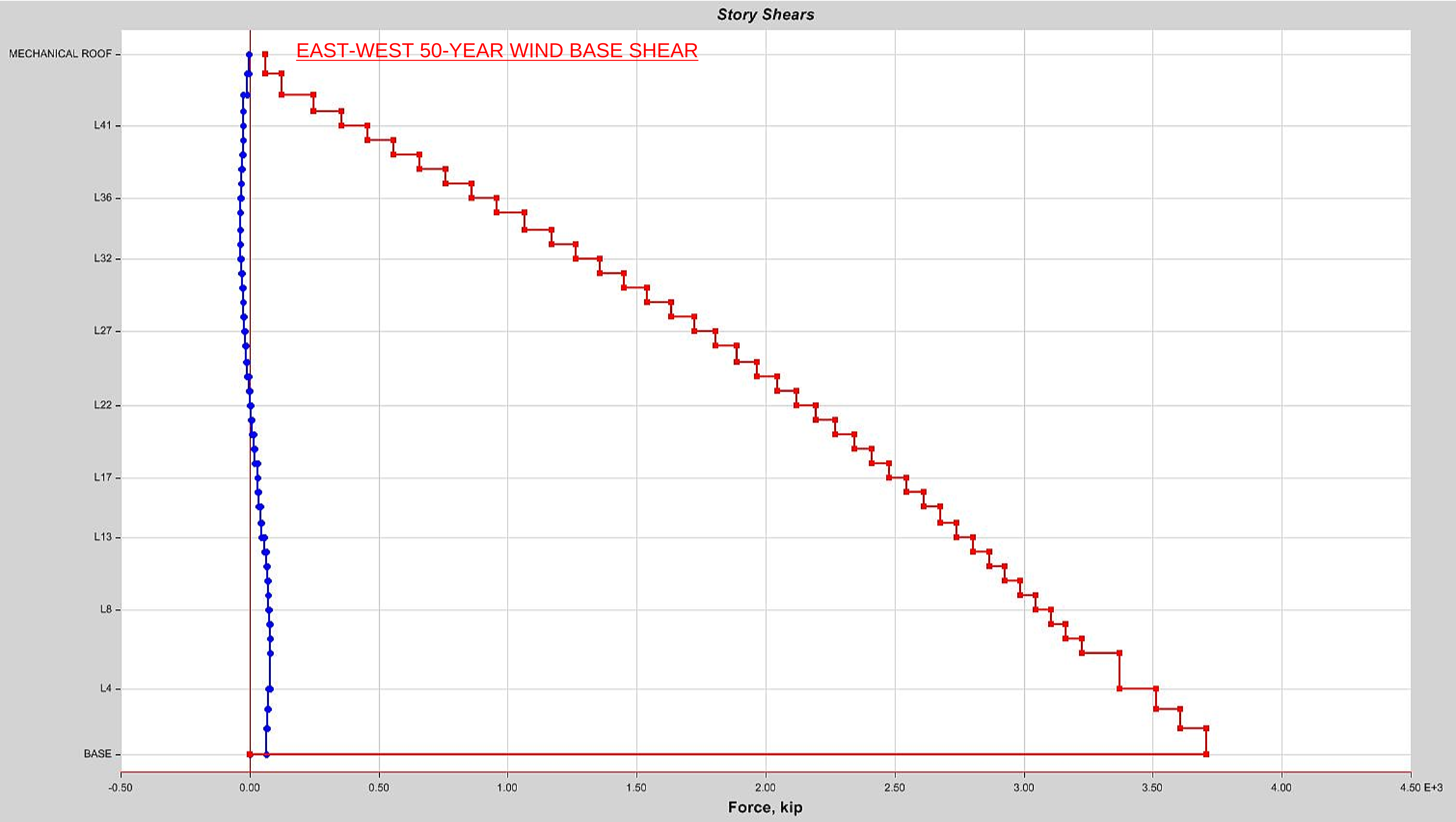
<u>SECOND MODE PERIOD = 4.7 S</u>



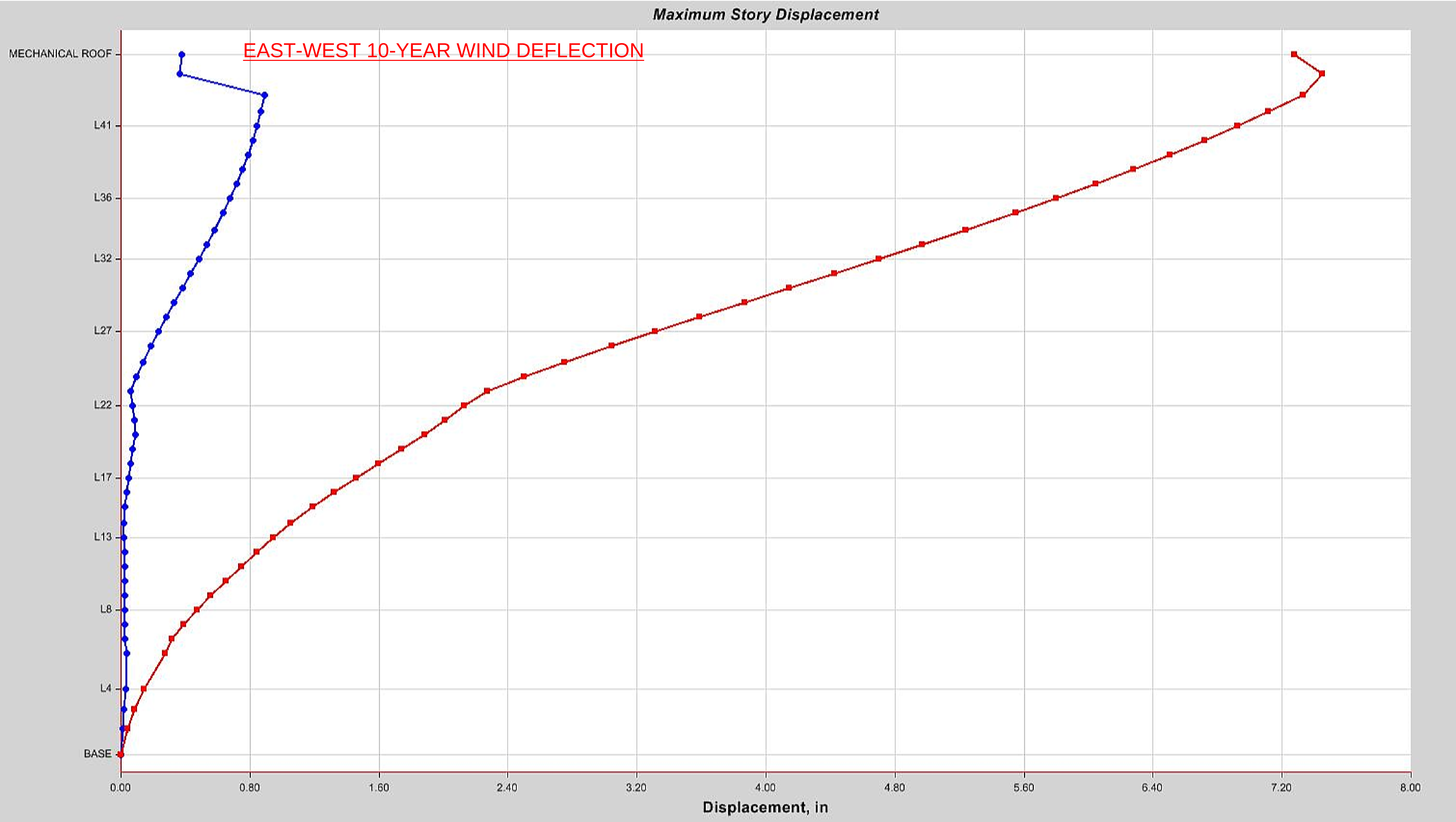

Story Shears
EAST-WEST SEISMIC BASE SHEAR
MECHANICAL ROOF
L41
L36
L32
L27
L22
L17
L13
L8
L4
BASE
-1.80
-1.60
-1.40
-1.20
-1.00
-0.80
-0.60
-0.40
-0.20
0.00
0.20 E+3
Force, kip

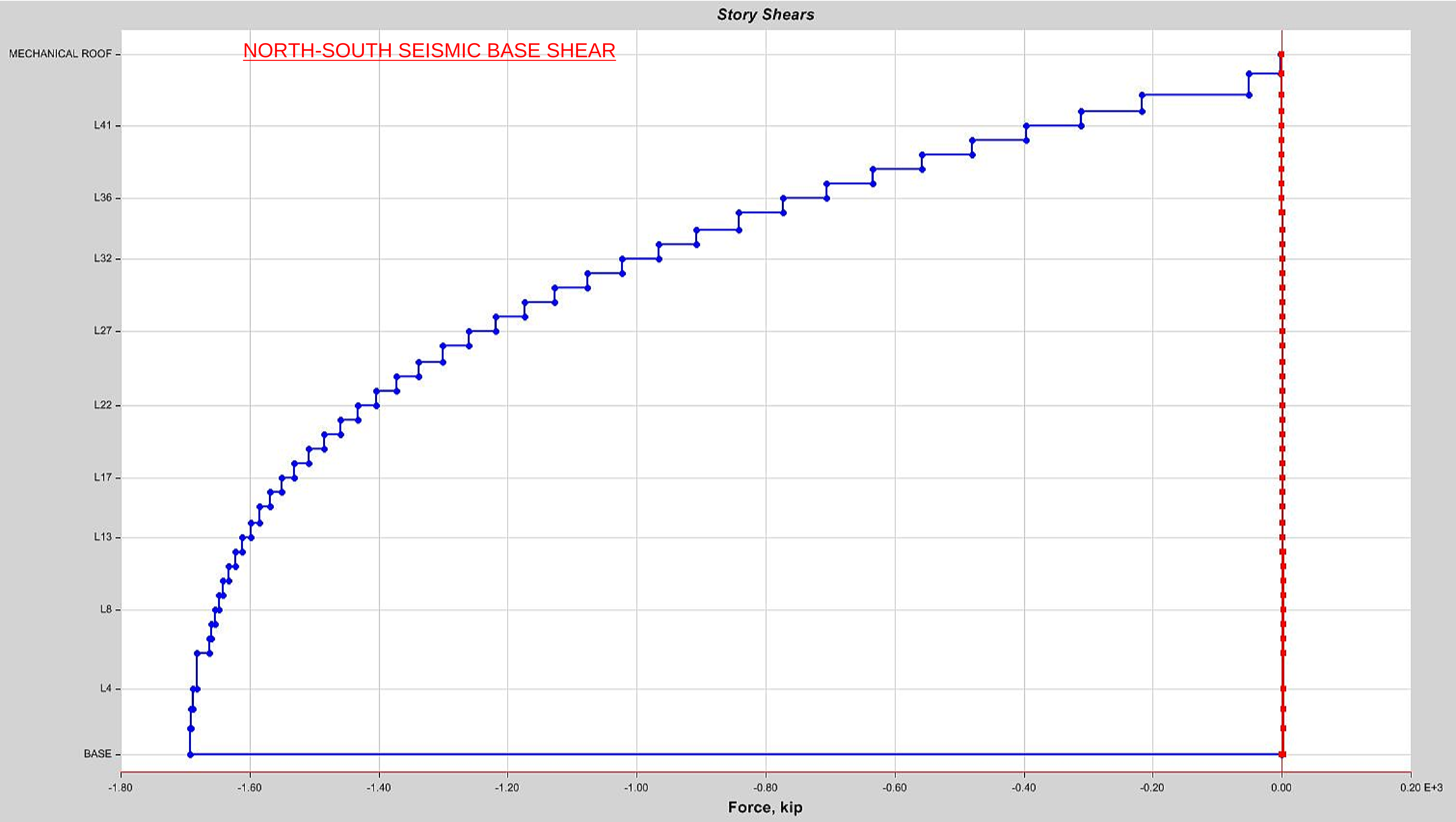


Maximum Story Displacement
EAST-WEST SEISMIC DEFLECTION
MECHANICAL ROOF
L41
L36
L32
L27
L22
L17
L13
L8
L4
BASE
0.0
1.2
2.4
3.6
4.8
6.0
7.2
8.4
9.6
10.8
12.0
Displacement, in

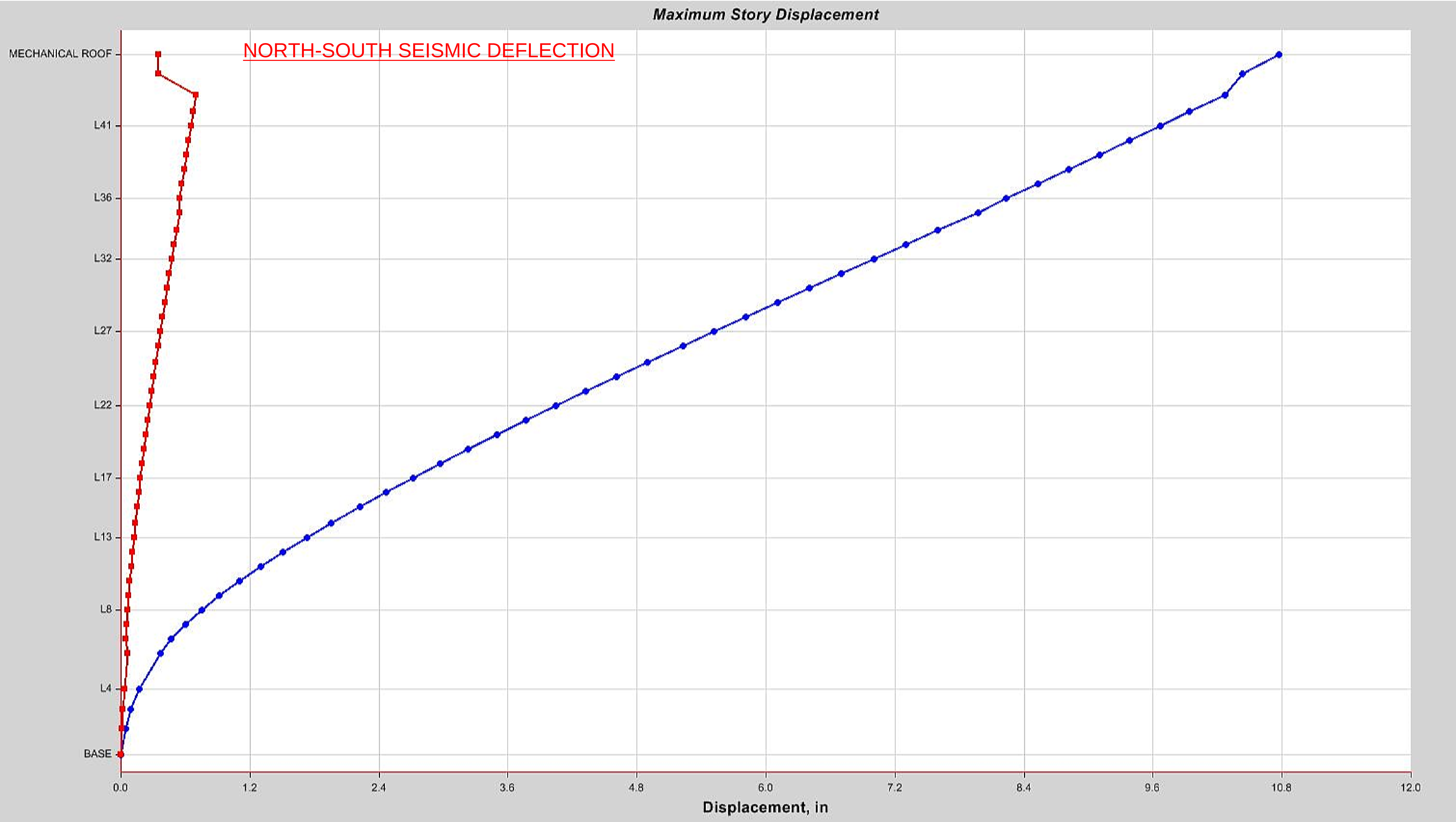


Story Shears
EAST-WEST 50-YEAR WIND BASE SHEAR
MECHANICAL ROOF
L41
L36
L32
L27
L22
L17
L13
L8
L4
BASE
-0.50
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50 E+3
Force, kip

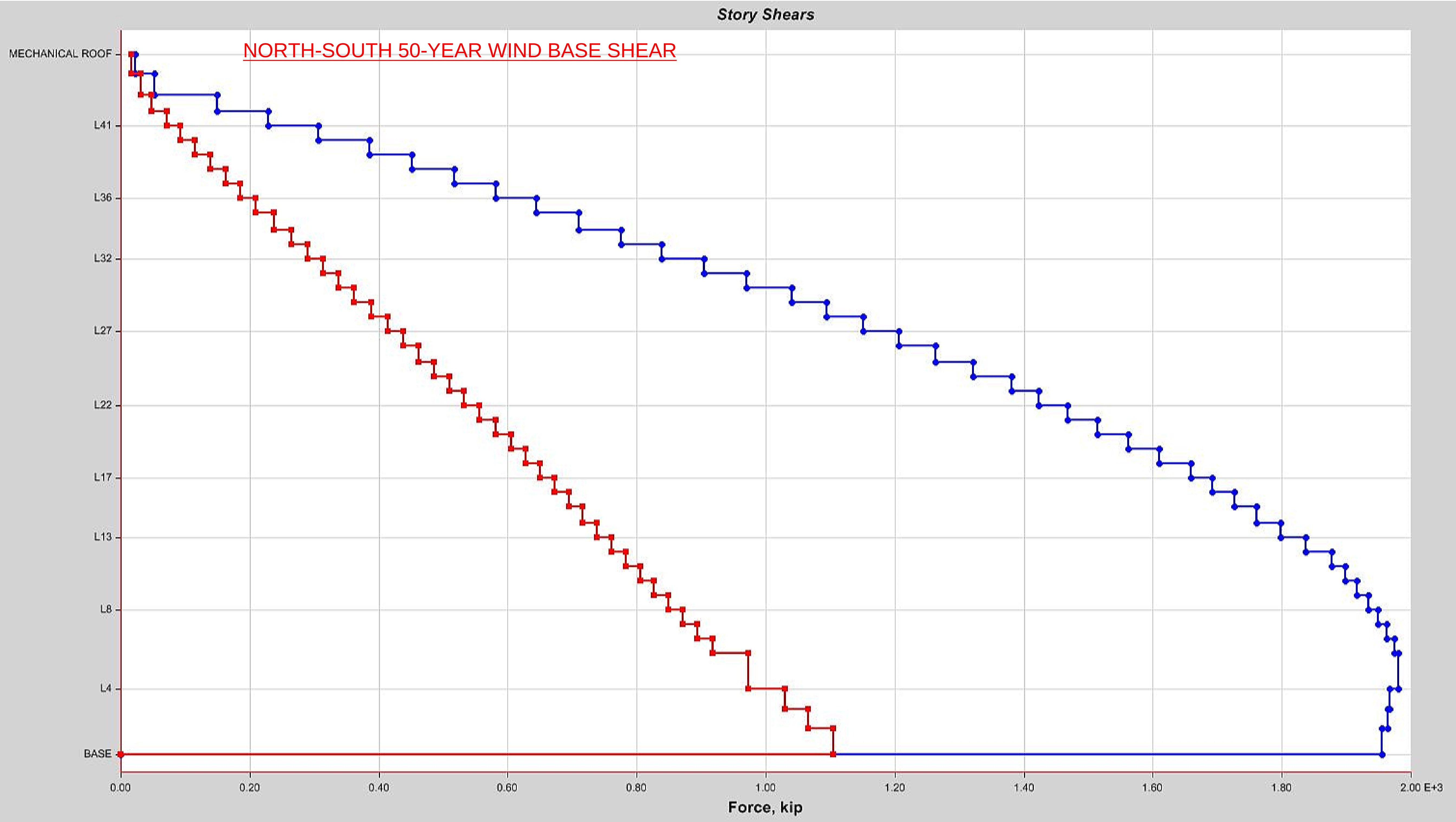

Maximum Story Displacement
EAST-WEST 10-YEAR WIND DEFLECTION
MECHANICAL ROOF
L41
L36
L32
L27
L22
L17
L13
L8
L4
BASE
0.00
0.80
1.60
2.40
3.20
4.00
4.80
5.60
6.40
7.20
8.00
Displacement, in

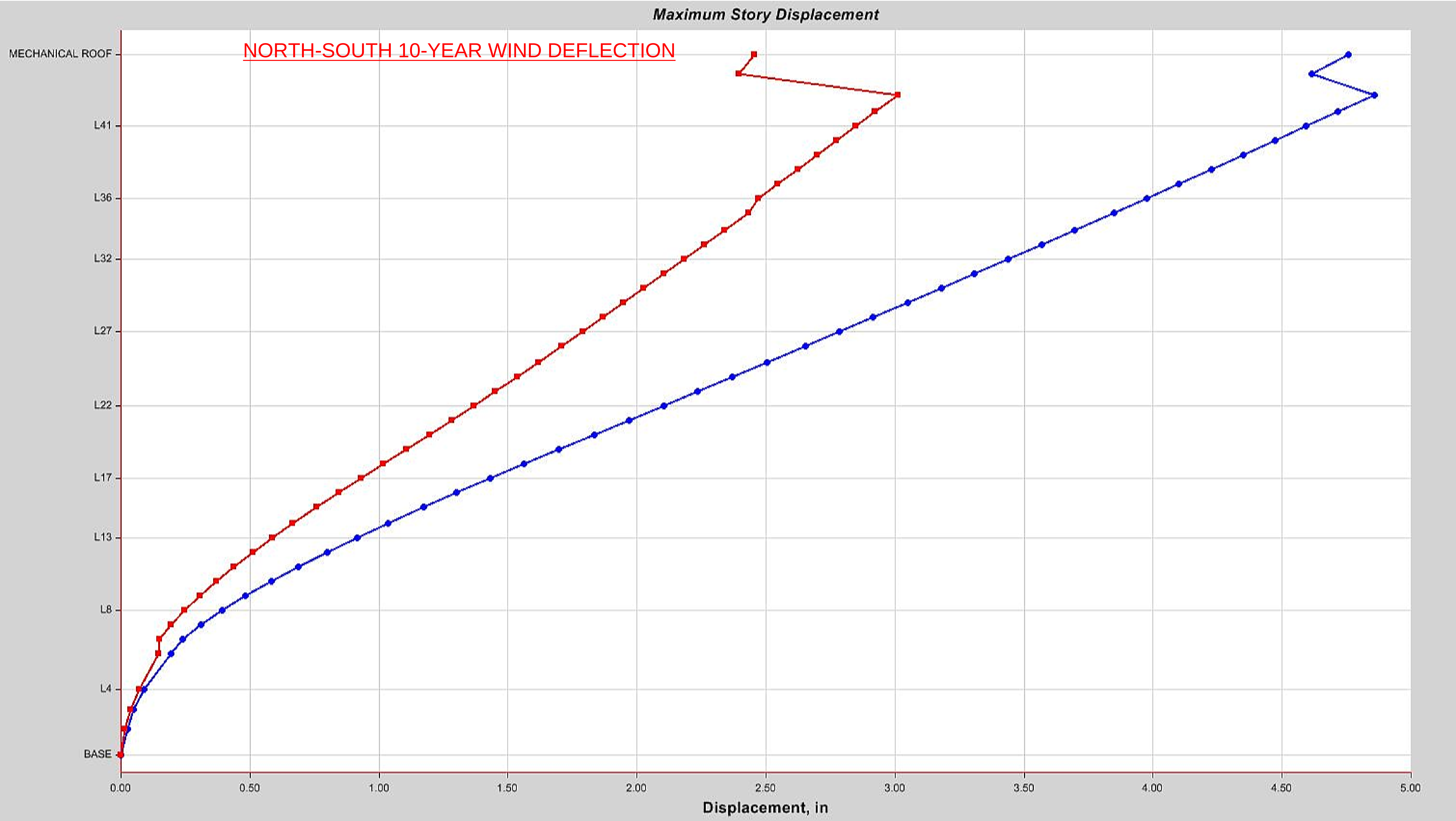


Story Shears
NORTH-SOUTH SEISMIC BASE SHEAR
MECHANICAL ROOF
L41
L36
L32
L27
L22
L17
L13
L8
L4
BASE
-1.80
-1.60
-1.40
-1.20
-1.00
-0.80
-0.60
-0.40
-0.20
0.00
0.20 E+3
Force, kip



Maximum Story Displacement
NORTH-SOUTH SEISMIC DEFLECTION
MECHANICAL ROOF
L41
L36
L32
L27
L22
L17
L13
L8
L4
BASE
0.0
1.2
2.4
3.6
4.8
6.0
7.2
8.4
9.6
10.8
12.0
Displacement, in


Story Shears
NORTH-SOUTH 50-YEAR WIND BASE SHEAR
MECHANICAL ROOF
L41
L36
L32
L27
L22
L17
L13
L8
L4
BASE
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00 E+3
Force, kip



Maximum Story Displacement
NORTH-SOUTH 10-YEAR WIND DEFLECTION
MECHANICAL ROOF
L41
L36
L32
L27
L22
L17
L13
L8
L4
BASE
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
Displacement, in