606 WEST 57TH STREET, New York

Design of Structurally-linked Multiple Tower Structures

By Sunghwa Han, P.E., S.E., LEED AP

The behavior of structurally-linked multiple tower structures is largely affected by the effectiveness of the rigid links made between towers at the bases or at the tops. These links have a strong influence on the distribution of wind and earthquake loads. Depending on the interactive dynamic behavior of the adjacent building parts with one another, a separation may or may not be needed. If the links are rigid enough, then two linked towers can be considered to be one integral structure. But if the links cannot be made strong enough, one might consider placing a separation joint.

Project Description

The project consists of a shared base with two levels below grade and two residential towers (1.2 million square feet of gross building area holding 1,028 apartments): a 42-story tall tower on the east and a 17-story tall tower on the west side of the site. The two towers are connected by a shared podium below the 7th floor, and the podium footprint is enlarged below the 2nd floor. These two towers are structurally separate above the 7th floor. However, the occupants of the two towers require access between the two at every floor. Therefore, there is a separate, 10-story tall, 7-foot wide and 30-foot long cast-in-place concrete "bridge" structure (the west bridge) between two towers. This bridge structure will provide a continuous means of egress for each floor of the 17-story west tower.

Figure 1. Building perspective (south elevation).

The 42-story east tower is comprised of three sections (*Figure 1*). Two towers emanate from the 2nd floor, and each rises up to the 28th floor on the east side of the site. On top of these two towers, there is an additional 13-story tall building section rigidly connecting the two towers above the 28th floor. The two eastern towers are 30 feet apart below the 28th floor and have another seven-foot wide

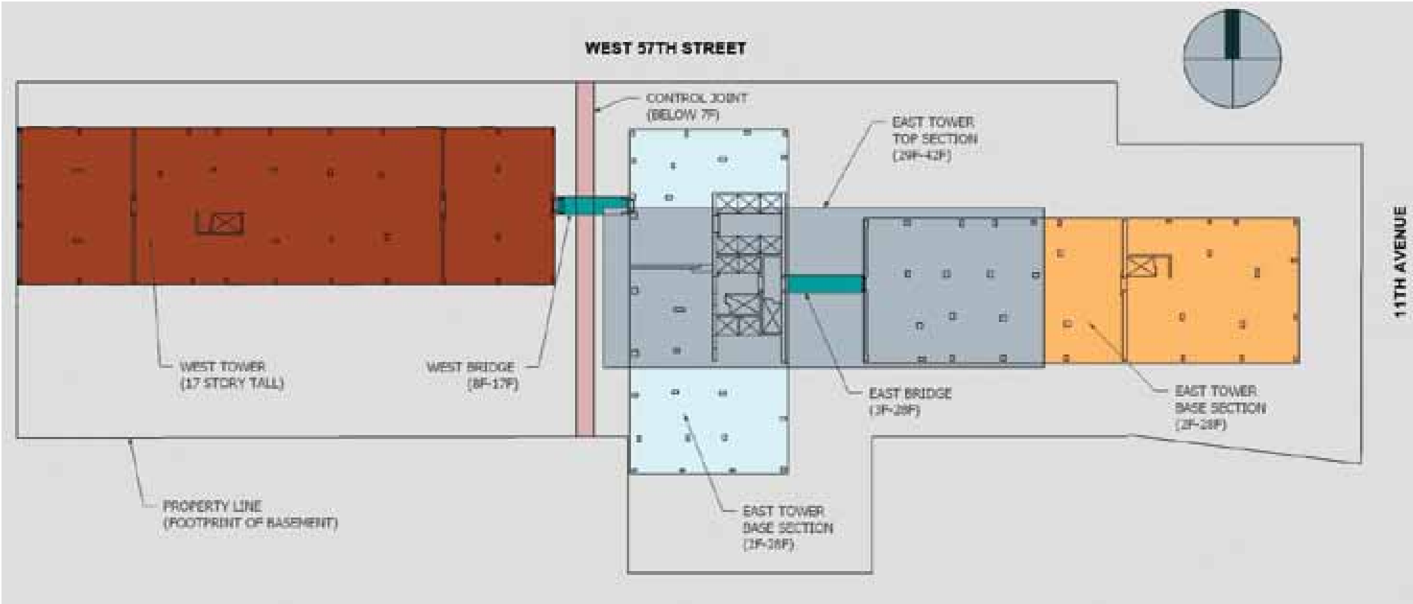

Figure 2. Site plan.

and 14-inch thick concrete slab pedestrian corridor-bridge (the east bridge) connecting them at every floor from the 3rd floor to the 28th floor. The corridor bridges are fixed to one tower but rest on sliding bearing pads on the other tower. This allows the two eastern towers to sway independently of one another from the 3rd floor to the 28th floor.

The ground level gently slopes down to the west in the site. Thus, a portion of the ground floor is exposed above grade at the southwest corner of the site. Top of rock elevation gradually slopes to the west, but drops drastically at the northwest corner of the site. Spread footings or excavated piers resting on an allowable bearing capacity of 40 tons per square foot (tsf) bedrock was recommended in the eastern portion of the site. The site is surrounded by the existing low-rise buildings ranging in height from three to seven stories tall. Most of the existing buildings are planned to be underpinned except the relatively newer buildings at the west end of the site. These buildings were found to be supported by caissons. The structure will also have caissons in this western portion of the site, where the top of rock elevation is low.

Structural System of Towers

A combination of cast-in-place concrete (CIP) shear walls and 10-inch thick flat plates supported by CIP columns and shear walls is utilized to resist gravity loads, wind loads and earthquake loads. Some of the tower columns need to be transferred to accommodate a variation in the architectural layouts.

The main elevator core located in the middle of the site (one of two eastern towers – *Figure 2*) provides vertical transportation for occupants of other towers. This main core is constructed with concrete shear walls. It is the primary source of the overall building stiffness, resisting the lateral loads and reducing the calculated displacements of the east tower.

The seismic load resisting system of the buildings is defined to be a "Shear wall-Frame Interactive System with Ordinary Moment Frames and Ordinary Reinforced Concrete Shear walls" per the New York City Building Code (NYCBC) 2008. This system allows utilizing frame elements as a part of the seismic load resisting system for a building with a seismic design category B rating. It was beneficial to consider contribution of frames in the seismic load resisting system to reduce the calculated building displacements.

Structural System of Two Bridges and Separation Joints

Originally, both bridges (west and east) were designed as a concrete slab pedestrian corridor-bridge fixed to one tower and supported on a sliding connection resting on the other tower at every floor. The lateral displacements of each tower were calculated, and the largest relative displacement was used to determine the distance of the separation joint for these elements shared by the adjacent towers.

The displacements due to wind loads were calculated using the results from the wind tunnel test performed by Rowan Williams Davies & Irwin Inc. (RWDI). The elastic displacements due to the seismic loads were also calculated based on the elastic dynamic analysis (response spectrum analysis). These calculated displacements were magnified using the deflection amplification factor of $C_d = 5$ for Shear Wall-Frame Interactive System to compute the inelastic maximum displacements as per ASCE 7. These calculated inelastic maximum displacements were significantly larger than the displacements due to the wind loads.

The required distance of separation joints was calculated to be 2 inches at the sliding connection of the east bridge and approximately 9 inches at the sliding connection of the west bridge, assuming two towers can be moving in opposite directions at the same time. The 2 inches of movement can be easily accommodated, but the 9 inches seismic separation requirement became a critical issue to the cladding design.

Code Requirements for Building Separation

The building code allows the use of alternate methods to compute the distance of separation gaps which generally produce preferable results. When considering a seismic separation at the lot line, it is acceptable to set back structure from the property line by 1 inch for every 50 feet in height according to the NYCBC 2008 for structures assigned to seismic design category A, B or C (the project building is assigned to Seismic Design Category B). This section of the code may be applied to separate structures, assuming that the provided separation is sufficient enough to avoid significant damage to the adjoining structures.

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The design team has adopted a more advanced analysis method (Nonlinear Response History Analysis – NLRHA) to ensure the sufficient separation gap between adjacent structures. The structural system of the west bridge was modified to become a separate ten story tall structure consisting of a 14-inch thick flat plate and four corner columns supported by the transfer beams at the 7th floor. This independently supported west bridge structure, with separation gaps at both ends, would be able to accommodate larger differential movements of two adjacent towers than the previously considered system (a flat plate supported on a sliding connection at one end).

A 6-inch seismic gap (magnified by less than C_d) is tentatively provided at both ends of the west bridge. This will allow 3½ inches of movement of the buildings, after the compressible material in the gap joints is fully compressed. This net 3½ inches of separation has been verified through the recent NLRHA.

Development of Ground Motion Time Histories

Initially, the site-specific seismic design spectra were developed for design of the structures. For NLRHA, three sets of ground motion time histories consisting of two horizontal components were provided by Amec Foster Wheeler, Environment & Infrastructure, Inc. Three sets of seed time history (1999 Chi-Chi Earthquake) were selected. Then, these time histories were scaled to match their response spectra with the site-specific design response spectrum as per Section 16.1.3.2 of ASCE 7-10 Chapter 16. This design spectrum (*Figure 3*) represents the Maximum Considered Earthquake (MCE) with a mean recurrence interval of 2475 years (2% probability of exceedance in 50 years).

Estimating Required Seismic Separation

The Nonlinear Time History Analyses were performed using Perform 3D, a commercial analysis software which considers material nonlinearity. The analysis model on Perform 3D includes the cellar floor excluding foundation walls. The flat plates were modelled as slab-beam elements with an effective width reduced from a full panel width as recommended in ASCE 41. Stiffness of slab-beam elements and columns was further reduced in consideration of nonlinear behavior of structural members. Shear walls were modelled using fiber sections, and link beams are modelled using beam elements with an inelastic hinge at both ends of the link beams.

Wind Loads on Structurally Linked Structures

In general, for a typical single tower, the wind tunnel testing results are combined with the local climate data and the dynamic properties of the structure to generate the design wind loads. Wind loads are provided in the form of static point loads applied at each floor, along with the multiple load cases representing loading that imposes the maximum effect on the structural members. Each set of wind loads is represented by the corresponding load combination factors for each component (X, Y and Torsion) which are applied simultaneously to the tower to determine the internal forces acting in each member of the lateral load resisting system.

For buildings with rigid links or shared components, additional loading scenarios must be considered. Load cases maximizing their effects on each tower will be separately generated first. When the foundation, podium or the linked portions such as a bridge between

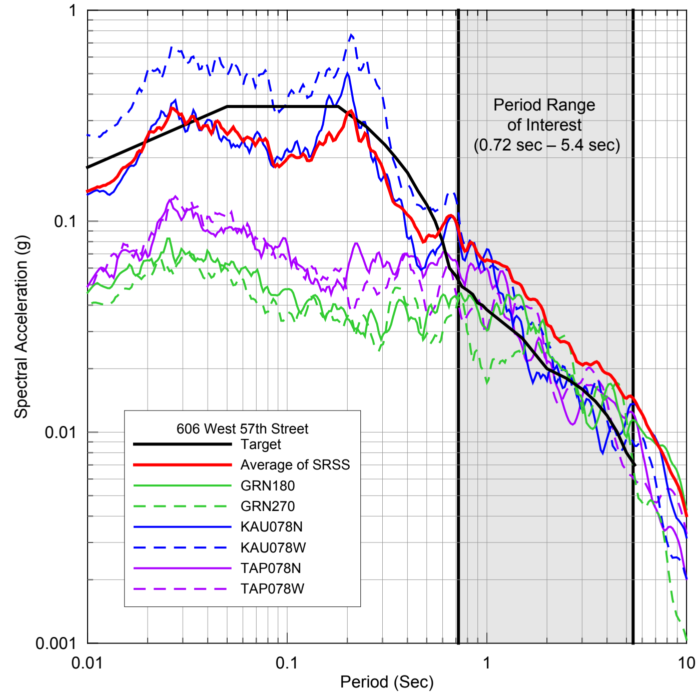

Figure 3. Comparison of response spectra and 3 sets of scaled time histories prepared by Amec.

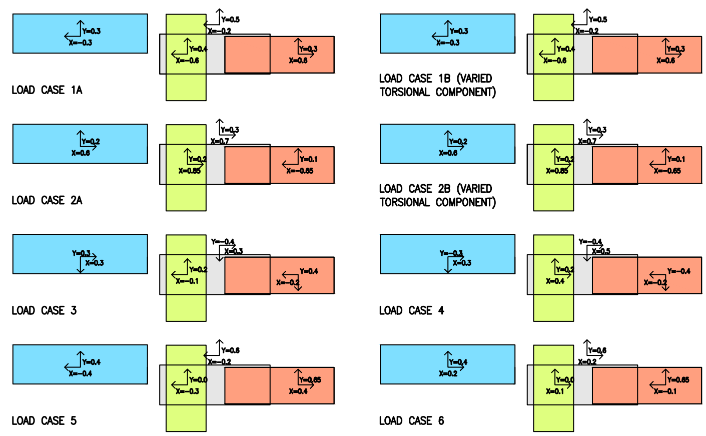

Figure 4. Wind load cases maximizing differential loading between towers.

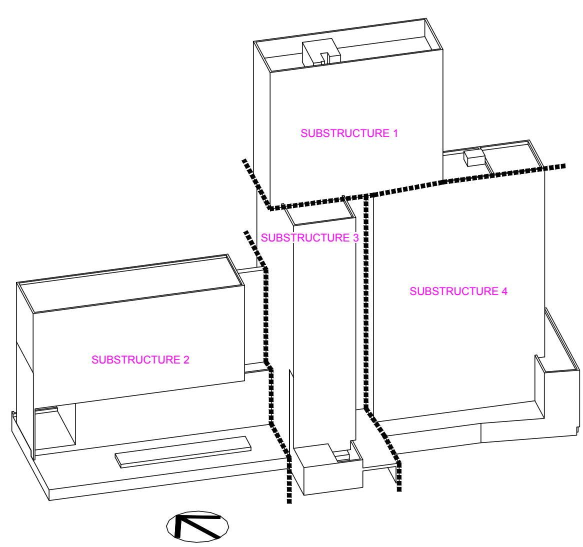

Figure 5. Multi-blocks (substructures) approach used in wind tunnel study.

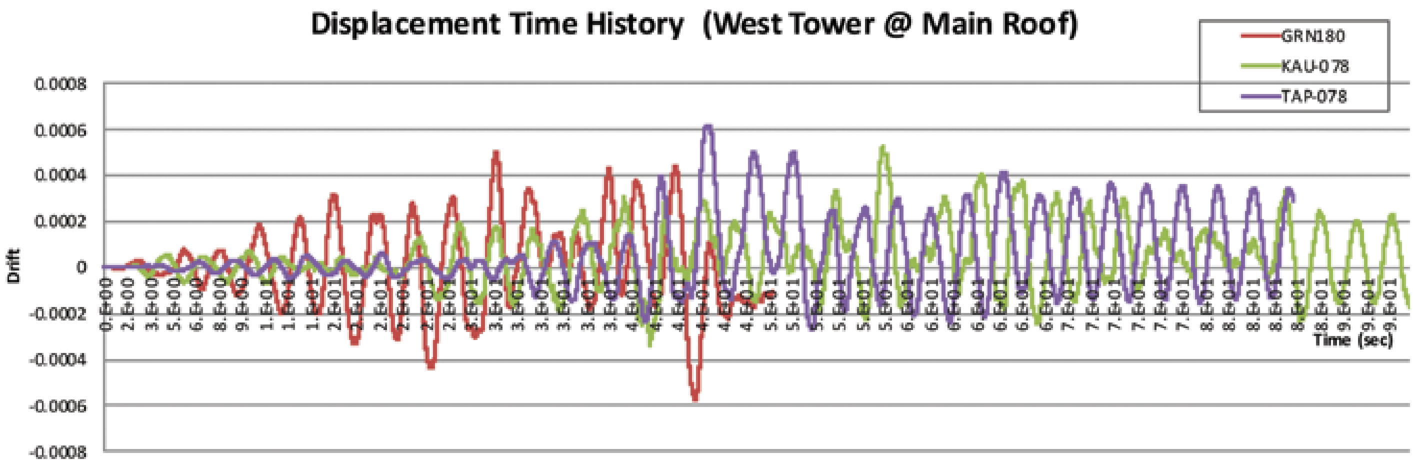

Figure 6. Sample displacement time history (at top of the west tower in E-W direction).

towers are involved, additional simultaneous loading conditions maximizing differential loading between linked towers can be generated. These load cases (*Figure 4*) were used to design the two bridges and the 29th floor slab, which is the lowest rigid link of the top 13-story tall section spanning the two towers rising up to the 28th floor in the east tower. The differential movements of the adjacent towers were calculated using these load cases, and compared with the maximum calculated displacements for the seismic loads to finalize the distance of separation gaps.

For an appropriate analysis of the wind tunnel data and accurate assessment of wind-induced responses, the entire building was "divided" into five blocks or "sub-structures" as shown in *Figure 5* and presented in the RWDI's wind study report. Diaphragms assigned in the analysis model, using ETABS®, are matched with this substructure configuration.

Miscellaneous

The site is approximately 550 feet in the east-west direction by 200 feet in the north-south direction. The base of the towers is designed as one single structure without any expansion joint. In order to minimize shrinkage cracks, one 4-foot wide control strip is placed between the west tower and the east tower in the podium floors. It is located at approximately ⅔ of the building length in east-west direction (*Figure 2*).

Project Team

Owner: TF Cornerstone Inc.
Structural Engineer: Rosenwasser/Grossman Consulting Engineers, P.C.
Design Architect: Arquitectonica
Architect of Record: SLCE Architects, LLP
Geo-technical Engineering Consultant: RA Consultants LLC
Geo-seismic Engineering Consultant: Amec Foster Wheeler, Environment & Infrastructures, Inc.
Wind Engineering Consultant: Rowan Williams Davies & Irwin Inc. (RWDI)

Currently, the foundation is under construction and the super-structure is scheduled to be completed by 2017. Danny Jadeja, P.E., Senior Associate, was the project manager and Ben Pimentel, P.E., President, was the engineer of record.▪

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