A LIVING OR SMART BUILDING: THE GUANGZHOU TOWER

By

Gary C. Hart[1]
Anurag Jain[2]
Chukwuma G. Ekwueme[3]

SUMMARY

This paper presents the evolution of the structural design of one of the tallest structures in the world. The architectural design was developed by Mehrdad Yazdani at Cannon Design Group.

1. INTRODUCTION

A great structural design starts with an architectural vision and the desire by the structural engineer to make this vision become a reality with consideration of the environmental and human loading acting on the structure during its life. The basic architectural vision of the Guangzhou Tower is three twisting interconnected legs; see Figure 1, taken from our computer model of the structure. The architectural plan at each level of the tower rotates and twists. Also the diamond pattern provides visual elegance and structural stability. As a structural engineer we always try to develop our own vision of the best structural system that compliments the architectural vision. The basic structural vision here is of a Living Structure that can be adapted and improved from a structural engineering perspective as new high-tech products become available, as our understanding of the forces of nature improves using ground and aerial instrumentation, and as we improve the accuracy of our structural modeling to estimate structural response to wind and earthquake loading. The architectural vision is clearly great, innovative and imaginative. This paper focuses on the structural engineering vision and the evolution of the design to best implement both the architectural and structural visions.

The Guangzhou Tower is a structural system of three triangular spirally twisting steel legs interconnected at intermediate levels by floors of observation decks, see Figures 2 and 3, taken from our computer model of the structure. The footprint of Guangzhou Tower is an equilateral triangle with 100m long sides and a total height of 540m above grade. Each leg of the tower also has a triangular footprint with 33.33m long sides (1/3 of total size). With increasing elevation, the plan of each leg reduces in size, twists counter-clockwise and rotates counter-clockwise about the center of the tower. The smallest size of the tower is at an elevation of 420m, with each leg being an equilateral triangle with sides 11.11m long (1/3 of base dimension). From elevation of 420m to 540m, the tower flares out/ opens up, with

[1] Managing Principal
[2] Principal
[3] Associate Principal

each leg consisting of equilateral triangles with 17.46m long. At the top, each leg of the tower has rotated 120 degrees counterclockwise about the center of the tower. The three legs of the tower are connected to each other by floor diaphragms at elevations of 90, 100, 110, 120, 420, 430, 440, and 450m, where each diaphragm connects only two alternating legs of the tower. A woven tensioned steel wire mesh will be wrapped around the structural framing.

The steel part of the structural system of the tower consists of (see Figure 4):

- Spines: Members connecting the vertices of the triangles of each leg of the tower, in an upward but twisting direction.
- Exterior Diagonals: Members that make up the diamond shape on the façade of each leg of the tower.
- Exterior Ties: Horizontal members going around the interior of each face of the leg of the tower every 60m.
- Floor Diaphragms: Observation floors at elevations of 90, 100, 110, 420, 430, and 440m, where each diaphragm connects two alternating legs of the tower. Diaphragms at elevations of 120 and 450m form the roofs of the observation levels at elevations 110 and 440m, respectively.
- Connections: Moment connections between members. The members framing into each joint have a slightly different orientation, making each connection's geometry unique and complex.

The Foundation will be a system of pile caps under each leg, supported on piles, with deep grade beams connecting the pile caps.

The structural performance of the tower is enhanced and the design greatly improved by making it a Living or Smart building as will be discussed in more detail in the following section. The key structural elements in this living structural system are Supplemental Taylor Viscous Damping Devices. These devices – dampers – are energy dissipating devices that absorb and dissipate the energy going into the structure from the environmental loads. These devices help reduce the wind load on the structure (by changing/offsetting the characteristics of the tower), and the displacements and floor accelerations (by improving structural integrity). The strength, serviceability and human comfort criteria can then be satisfied with smaller members in the structure. These active dampers have a control mechanism and a feedback system that are tuned to the actual displacement of the structure. A computer processes the data from the feedback system in real time and sets the controls of the dampers to produce the desired effect. Also the feedback system can also be used for visual entertainment, wherein the actual movement of the tower could be graphically projected on TV consoles and viewed by visitors. It is self evident that in this electronic age that will only become more beneficial to structural engineers during the life of this structure which will certainly exceed the currently assumed 50 years for code structural design criteria that both the hardware and software of this damping feedback system will make great advances and be upgraded with time. Just as future medical advances will improve our quality of life this technology advances are planned for in the design and will improve the quality of life for the Living or Smart building.

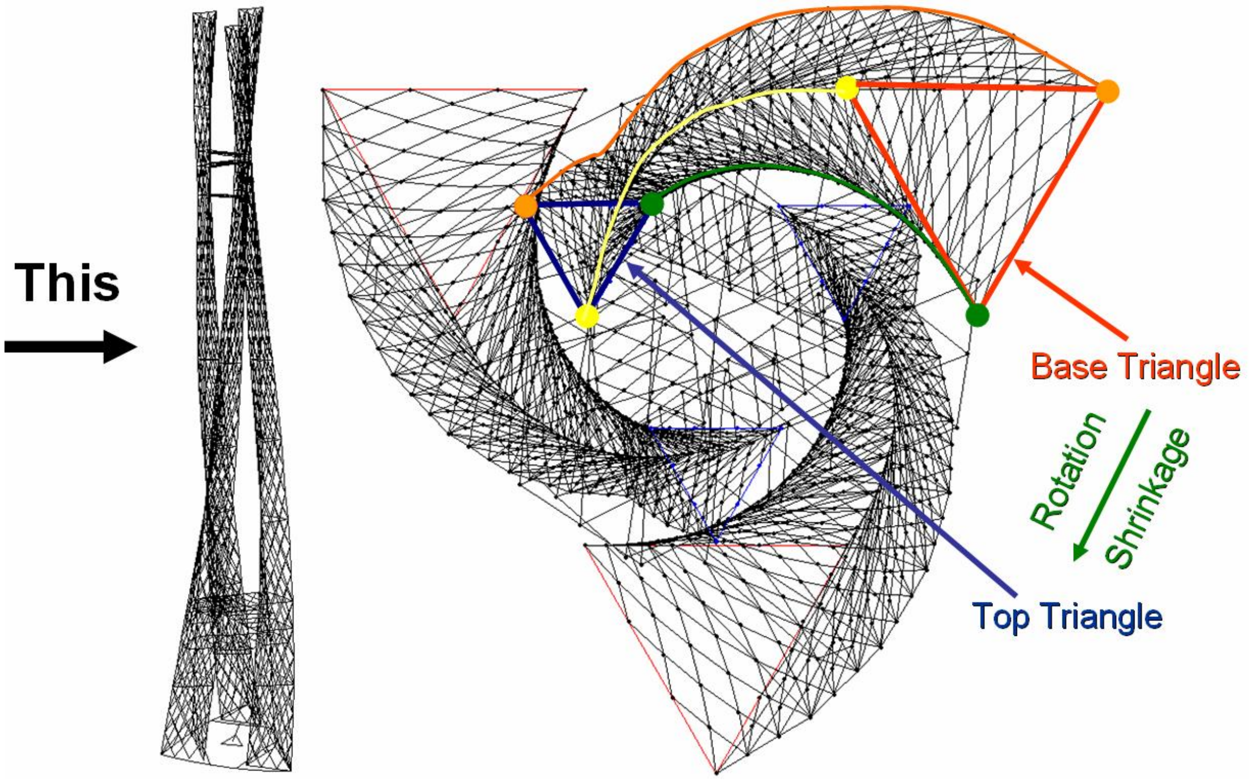


Figure 1 Rotating and Shrinking Triangular Plan

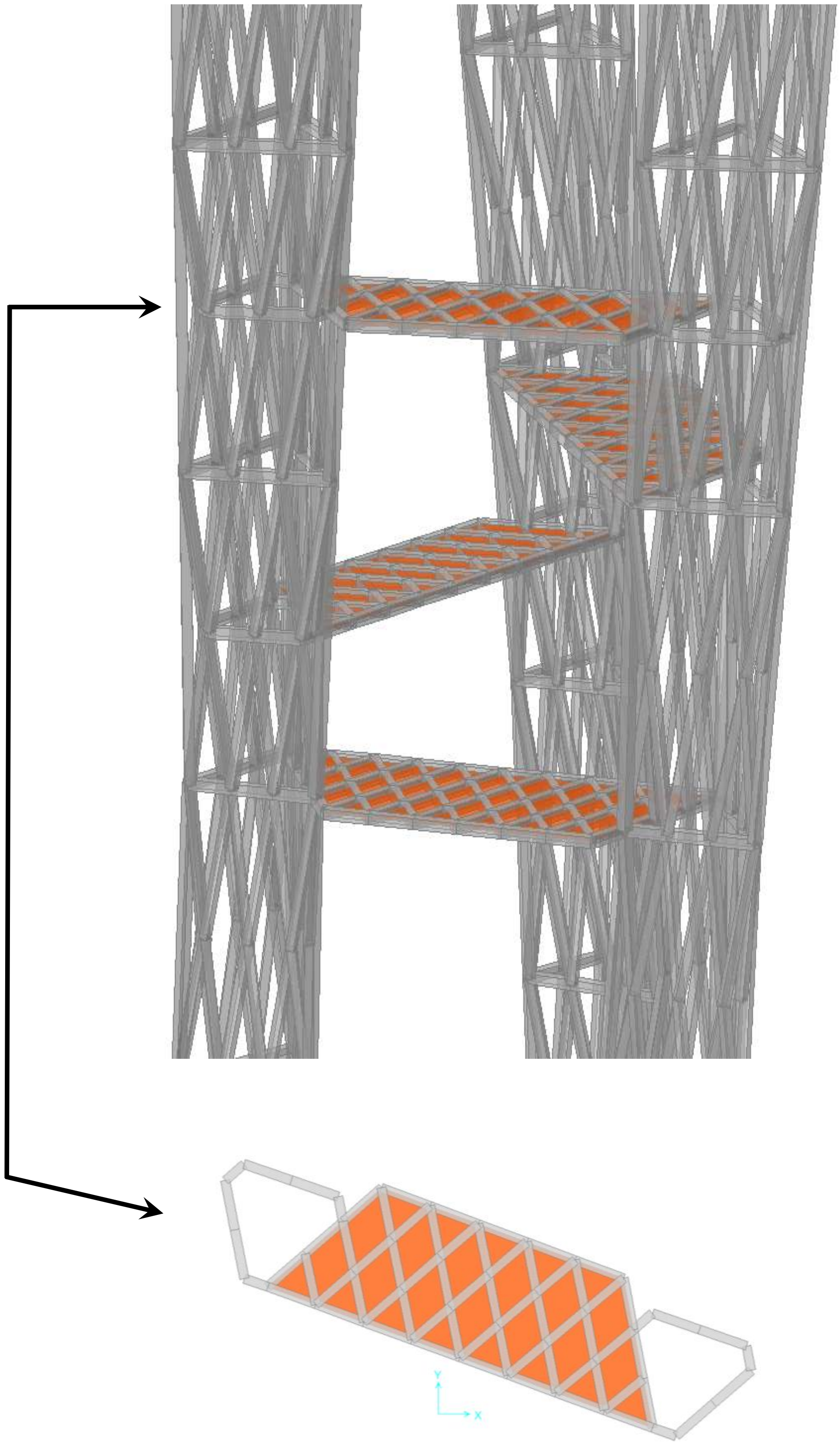


Figure 2 Close-up of Upper Platforms Connecting the Legs of the Tower

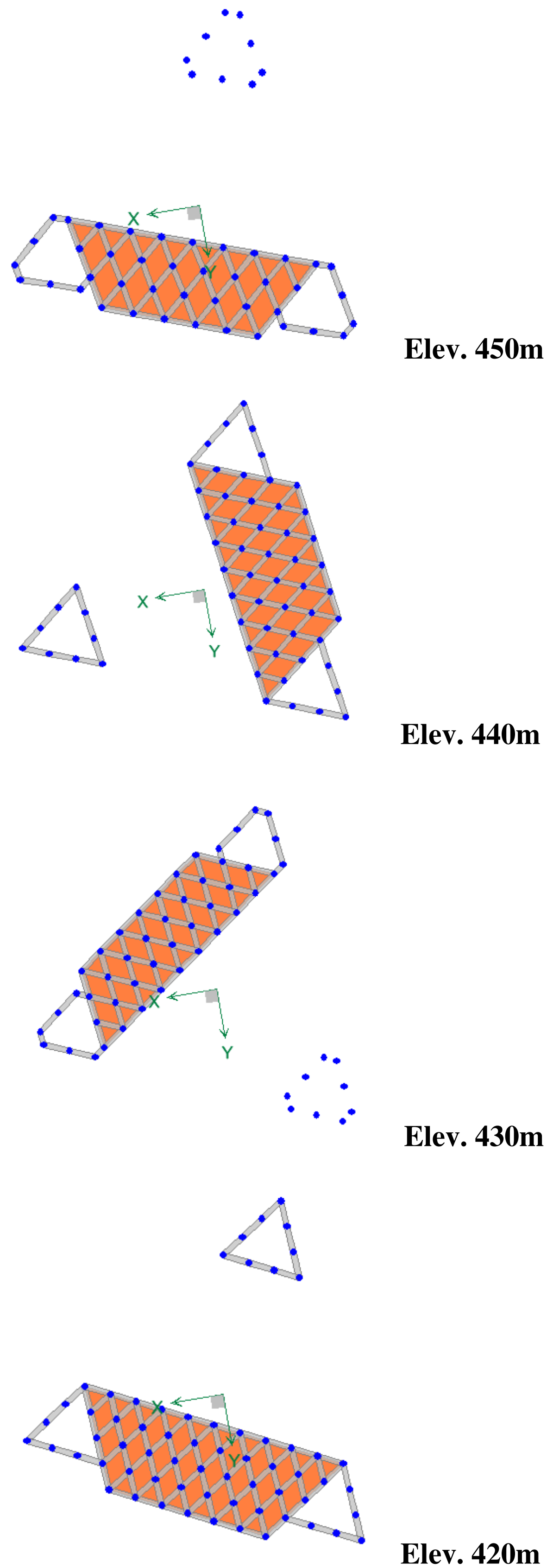


Figure 3 Layouts of Upper Platforms

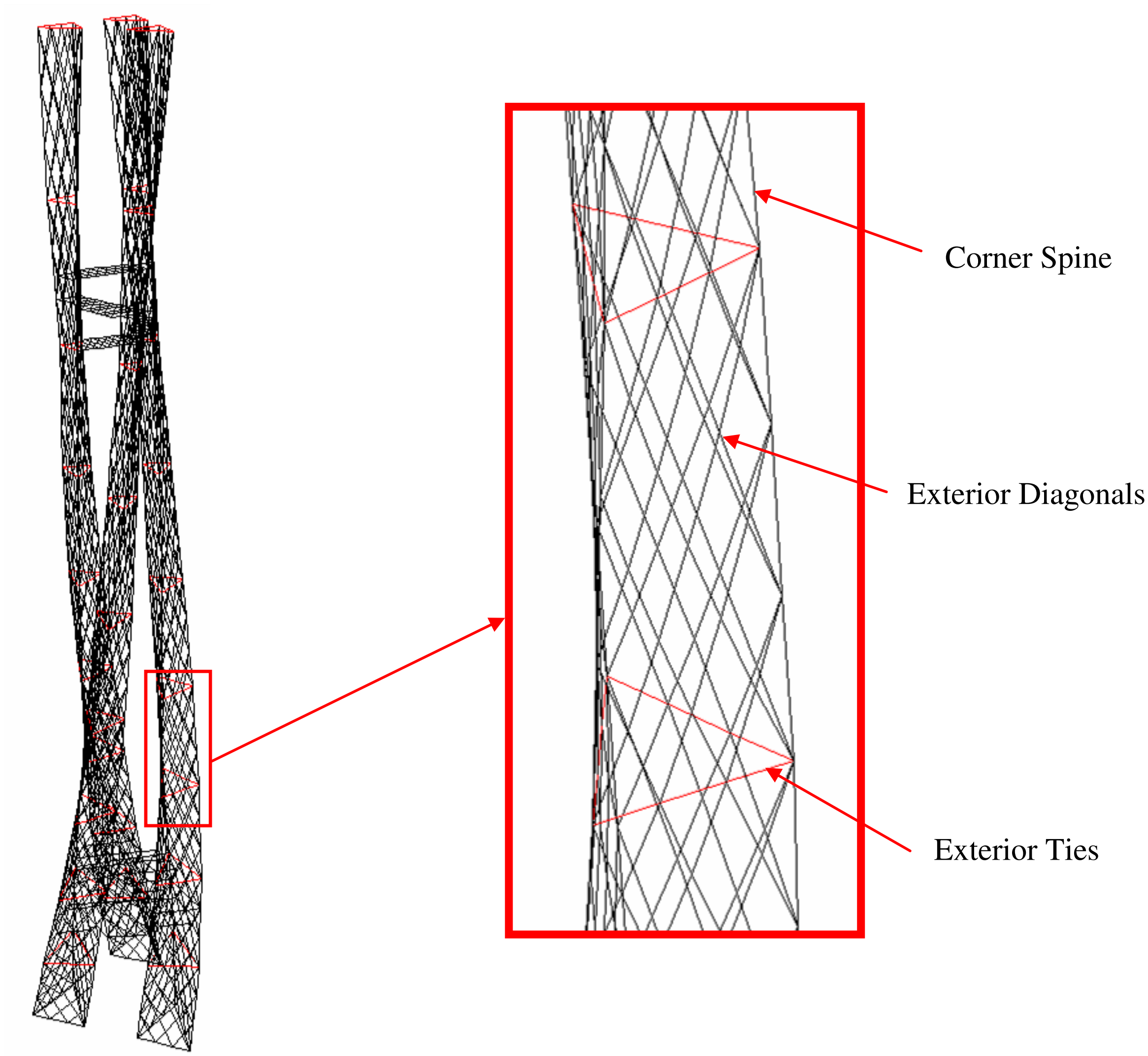


Figure 4 Structural System of Guangzhou Tower

2. THE STRUCTURAL VISION OF A LIVING STRUCTURE

Unfortunately, most owners and architects are only interested in minimum first cost buildings and do not consider reality as it exists in structural engineering. The structural design of a Living or Smart Building provides a structural system that satisfies current minimum code design criteria, meets the architect's vision, and provides the owner with an optimized structure that continues to fulfill its needs well into the future. The building's lateral force resisting system is designed by the structural engineer with the knowledge that the Taylor dampers in the structure can be easily modified to incorporate the expected high technology advances in computers and instrumentation and therefore an expected increase in both building performance and confidence in that performance to natural environmental loading as technology advances.

We believe that this and other landmark structures are more than an inanimate metal and glass. It is like a child – a child that is conceived with a passionate vision of its form, structure and purpose; nurtured through the schematic design phase and the development of construction documents; and cared for during the labor pains of plan check corrections, requests for information, shop drawing review, and construction observation. Like a smart child, this building structure will mature, perform necessary functions during its life, and eventually grow old and die. Our building structural design will control from a structural engineering perspective the performance or *quality of life* that this building experiences during its existence.

A Living or Smart structural design will involve more in-depth and sophisticated structural engineering analyses so as to more accurately define the expected performance of the structure during future earthquakes and severe winds. This extra effort can only be done by a few existing building design firms, but if it is done will result in a reduction in the construction cost of the structure as well as an increase in the confidence that the structure will not experience human discomfort beyond acceptable performance standards and will not collapse, in part or in total, in a major earthquake or severe wind. These analyses consider credible scenarios of future earthquakes and severe winds, including hurricanes, during the building life and calculate for each the expected damage to the structural and nonstructural systems and the building contents. In addition to the optimal design based on these advanced analyses, the building is also designed with the recognition that we are beginning a century of extreme technological advancement. This recognition is an essential part of the design of a Living or Smart Building because it recognizes that it is possible that the building's lateral force resisting system with its Taylor dampers can easily be modified during the life of the building.

A Living or Smart structural design is consistent with many other aspects of our lives where when we purchase a quality item, we recognize that the item must be able to accommodate future changes. For example, the purchase of a Rolex watch because of its ability to accommodate fashion changes. Another example is that a computer is designed to have the memory capacity upgraded with newer and better cards. A third example is that a retirement plan for a person must be designed to accommodate the uncertainty expected to occur in the typical 20-year retirement period.

Our Living or Smart Building design recognizes and incorporates in its design viscous Taylor dampers that anticipate changes in technology that are expected to occur in the over 100 year design life of the building. This technology development takes two forms: Research and New Products.

Research in the area of structural engineering continuously advances the basic accuracy with which structural engineer's model, using mathematical equations, the behavior of buildings under everyday loads and also loads caused by extreme and rare environmental events. As modeling techniques advance, structural design procedures become more accurate and optimal.

None of us likes to be sick. In a similar way, an important building does not like to suffer damage when less than major levels of earthquake or severe wind loading occur. Unfortunately, the realities of life are that a code designed building that is not a Living or Smart building will be sick, and will experience significant levels of earthquake ground motion or severe winds that produce damage. For example, the building code sets the exposure time or design life for a Non-Living or Smart building to be 50 years. This is not addressing reality for an important building which can be expected to exist for 100 or more years. Therefore, the important building can be expected to experience various sized wind induced loads. When these forces occur, the building can expect damage; however, collapse of the structure is not expected because collapse prevention is a basic mandatory design criterion. Most often building damage is to the nonstructural system; damage to the structural lateral force resisting system typically occurs only during the most severe loading. A Living or Smart building design calculates the expected life cycle damage for different exposure windows and then in final design selects the final Taylor damper properties to best meet the owner's objections.

3. INITIAL CONCEPTUAL DESIGN

A good structural design incorporates substructures that are repeated throughout the structure and this is, construction wise, cost effective. Therefore, it is very important to focus special attention on the typical substructure. Figure 5 shows an elevation of the structure and the substructure that we will now address. We started our study with a study of a two dimensional (2-D) substructure of the complete structure. The substructure under consideration is defined as one face of a triangle of one leg of the overall structure. The basic configuration of the substructure consists of a 60m diamond-shaped lattice work, signifying the bottom $1/9^{th}$ of the overall height of the tower. (The tower was architecturally conceptualized as consisting of 9 sections, each 60m high). Figures 6(a) and (b) illustrate the setup and basic lattice work of the substructure. The substructure itself is divided into three levels, each 20m high, while the nodes comprising of member intersections are defined at each 10m height. The members are assumed to be hollow rectangular sections of 1m x 1m x 0.1m.

Various configurations of the 2-D substructure were analyzed to study their stability and performance. These configurations differ from each other in the type of connections between members and the presence/ absence of ties and corner spines. The connections defined in a substructure model could be all pinned or all moment connections or a combination of both. The models could also have external ties at the top only, i.e. 60m height, or every 20m. Furthermore, the model could have vertical members/ spines at the two sides. Figures 7 through 13 illustrate a total of eight different substructure models.

The substructure models were subjected to the self-weight of the structural members and the dead load of the tower above the 60m level, which was applied as point/ concentrated loads at the top nodes. The resulting deflections and stresses were utilized as the performance yardsticks in the assessment of their structural behavior. The deflections at a node are given in the horizontal (U1) and vertical (U3) directions, while the stresses in the structural members are a combination of axial and bi-axial bending. Table 1 presents the maximum deflections and stresses obtained from each substructure model under the loads described above. The deflections tabulated are for nodes X.060-1 and X.040-1 (refer to Figure 6 for nomenclature) which typically exhibit the largest resultant displacement in the substructure. The stresses are the maximum values occurring in any structural member of the substructure. Included in the table are descriptions of the models and connections used and a remark on the stability of the structure. The results presents in Table 1 are illustrated in Figures 7 through 13, which depict the undeformed and deformed shape of each model.

The results of the study indicate that pinned connections with the basic diamond-lattice structure (Figure 7 - Models B-2D-1 and B-2D-2) results in an unstable structure. The part of the structure bounded by a triangle extending upward from the outer supports is stable. However, the lattice work assembled on top of this triangular base does not have enough support points to be stable. A deformed shape would typically show those members heaping around the triangular base. Adding a tie at 60m high, as in Model B-2D-2, does not provide enough constraints/ support for stability.

The largest deflections in a stable structure were obtained for Models B-2D-6 and B-2D-8. The deformed shape of Model B-2D-6 depicts members flaring out excessively at the top under the applied loads, while Model B-2D-8 exhibits its maximum deformation at level 40 (Node X.040-1), as the horizontal tie at the top restrains the flaring out observed in model B-2D-6. Both these models induced extremely high stress on the members. In contrast, Models B-2D-5 and B-2D-4 exhibit the smallest deflections and stresses, which are attributed to the restraining actions of the horizontal ties and/ or spines. However, comparison of the deflection values and the stresses indicates that, from a structural engineering point of view, the use of horizontal ties every 20m (Model B-2D-5) is a more efficient use of resources. This is further illustrated by a comparison of Models B-2D-3 and B-2D-4, which reveals that the addition of a horizontal tie at the top level reduces the deflections by 71% and 35% at the top and 40 m levels, respectively. Furthermore, providing moment connections in lieu of horizontal ties and/ or spines as in Model B-2D-7 was found not to be as effective. It demonstrated, however, that design of the architecturally desirable open-lattice structure is feasible on the condition that the relatively high stresses (91 ksi) and deflections (resultant displacements of 60mm and 156 mm) could be reduced to manageable levels with a change in member section properties.

The three legs of the tower rotate about the centre and twist on their axis as they progress upwards. Each leg of the tower ends up rotating 120 degrees with an offset of more than 50m from its footprint at the base. Therefore, P-Delta effects might play an important role in the structural behavior of the tower, which the 2-D substructure models do not capture. On the other hand, each leg of the tower has three faces interconnected to each other at the edges, which enhances the structural performance of the leg significantly. Each leg of the tower could be considered analogous to a hollow, triangular section similar to a pipe section. Furthermore, the three legs of the tower are connected to each other by floor diaphragms at certain elevations, which induce framing action among the three legs and consequently reduce deflections and stresses in the members. Therefore, to study the outcome of these opposing effects a global 3-D model of the whole structure need to be studied.

The three-dimensional (3-D) study was focused on fine-tuning the behavior of the open-lattice structure (Model B-2D-7) described above. This diamond-shape lattice work was favored for architectural reasons and it was desired to preserve it as much as possible without compromising the structural integrity of the tower. Various 3-D configurations were analyzed to assess their structural performance and suitability. These 3D configurations are global models of the whole tower (540m high), which include the three triangular legs, with all connections assumed to be moment connections.

The models/ configurations differ in the number and type of links between the three legs. The links between the three legs could be every 60 m, or at 2 or 3 selected heights, or none at all. There are three types of links assumed in the different models:

1. Diaphragm constraint of the nodes on the inside face of each triangle in the three legs at the link level (see Figure 14(a)).
2. Axially rigid (rod) link between adjacent vertices of the triangles and horizontal members connecting the nodes on the inside face of each triangle at the link level (see Figure 14(b)).
3. Horizontal members connecting the adjacent vertices of the triangles and horizontal members connecting the nodes on the inside face of each triangle at the link level (see Figure 14(c)). These members are all of the same size.

The models also differ in that some of them might (or might not) have members along the corners of the triangles (like a spine). All members in the models are assumed to be hollow rectangular sections of 1m x 1m x 0.1m (same as in 2-D models).

For analysis and assessment of structural performance of the different configurations, static dead load and modal analyses were performed. The dead load consists solely of the self-weight of the members of the structure. Structural performance measures include the fundamental period of the structure, the deflection at the top (540m height) and the maximum stress (computed as a combination of the axial and bi-axial bending) occurring in any member of the tower.

Table 2 summarizes the results obtained for the 10 different 3-D configurations of the tower. The longest period, 28.8 sec, is obtained for Model B-3D-1, which does not have corner members/ spines or links or horizontal members. The structure is very flexible with horizontal and vertical displacements of 44.6m and 4.9m, respectively, at the top. The arrangement of the members in the diamond-shape lattice work - without any ties or links – essentially allows the structure to move/ behave like an accordion. The resulting stresses are also very high, with a maximum of 130 ksi. These large deflection values and extremely high stresses could be reduced by adding diaphragms at certain levels to link and frame the three legs of the tower. Model B-3D-2 exhibits such a configuration, with nine diaphragm/ Type 1 constraints for all nodes (every 60m), which yields deflections of 0.7m and 0.3m, in the horizontal and vertical directions, respectively. The stresses are also significantly less, with a maximum of 28 ksi, while the fundamental period was reduced to 22.5 sec. In Model B-3D-3, the links are substituted by Type 2 links, resulting in more flexible structure with a fundamental period of 23.9 sec. The horizontal and vertical deflections at the top are 13.4m and 2.0m, while the maximum stress increases to 62 ksi. Adding corner members/ spines to the model – as in Model B-3D-4 – yields a structure with the shortest fundamental period of 11.9 sec. The addition of these corner members also helps to reduce the deflections to 3.8m and 0.6m (reduction of 71% and 67%) in the horizontal and vertical directions, respectively. The maximum stresses obtained were 31 ksi on the spines and 25 ksi on other members. It was found that substituting the Type 2 links with Type 3 links every 60m - Model B-3D-10 - with spines/ corner members present does not alter the structural behavior of the tower significantly. The fundamental period obtained was the same, while there were slight increases in the defections and stresses. These results and observations indicate that Type 1 links – diaphragm constraints – rather than Type 2 or Type 3 links, in combination with corner spines, are more efficient in enhancing structural performance of the tower. However, architectural considerations/ constraints preclude the provision of diaphragms/ links every 60m and hence, the structural behavior of the tower with fewer diaphragms or links has to be analyzed.

As a point of departure, Model B-3D-6 was constructed with no corner spines and two diaphragm constraints at 180m and 420m heights. As expected, this yields a relatively large fundamental period of 27.5 sec. The defections are also relatively large with 3.8m and 1.0 m in the horizontal and vertical directions, respectively, while the maximum stress obtained was 46 ksi. Adding a third diaphragm at level 540m – Model B-3D-7 – does not change the fundamental period and the maximum stress obtained. However, it reduced the top deflections by 78% and 46%, to 0.8m and 0.6m, in the horizontal and vertical directions, respectively. This configuration was not implemented as architectural considerations precluded the provision of a diaphragm at the 540m level.

Another point of departure was to model the tower without any links/ diaphragm constraints but with corner members/ spines. This Model, B-3D-5, yields a relatively small fundamental period of 12.1 sec – compared to models B-3D-6 and 7. The stresses were also comparable, with maximums of 42 ksi and 32 ksi on the spines and other members, respectively. However, the horizontal deflection increased to 7.1m – an increase of 187% (compared to Model B-3D-6) - due to the loss of framing action of the diaphragm constraints, while the vertical displacement was reduced to 0.8m (reduction of 27% compared to Model B-3D-6) due to the vertical resistance of the spines/ corner members.

Model B-3D-8 was constructed with corner spines and two diaphragm constraints (Type 1) at level 180m and 420m, as a combination of models B-3D-6 and B-3D-5. This configuration yields a relatively short fundamental period of 12.2 sec, and the stresses were reduced to 30 ksi on the spines and 20 ksi on other members. The deflections obtained at the top were 1.9m and 0.4m in the horizontal and vertical directions, respectively. It is noted that the horizontal displacement is 1/280 of the height, which is larger than the 1/500 limit (1.08m for 540m) usually used in preliminary design of tall structures. To try to reduce this excessive displacement, Model B-3D-9 was created with horizontal ties around the triangles of each leg of the tower at the diaphragm levels (180m and 420m). This configuration does not change the fundamental period and the maximum stresses on the members. There was a slight decrease in the maximum stress on the spines – a reduction of 10% to 27 ksi compared to Model B-3D-8 - due the hoop action of the horizontal tie members, while the defections were reduced to 1.0m and 0.3m (reduction of 45% and 27% compared to Model B-3D-8). The horizontal displacement has a ratio of 1/515, satisfactorily less than the desired limit 1/500.

Table 1 Two-Dimensional Substructure Models

Model	Description	Connections	Remarks		
			Stability	Deflection[mm][*]	Max. Stress[ksi]
B-2D-1	2-D Original Lattice – Diamond Shape as Basic Structure	Pinned	Unstable	x	x
B-2D-2	2-D Diamond Shapes - with Horizontal Members every 60 m	Pinned	Unstable	x	x
B-2D-3	2-D Diamond Shapes - with Vertical Members at the Corner/ Edge	Pinned	Stable	Node X.060-1: U1 = 96.7 U3 = 54.1 Node X.040-1: U1 = 105.9 U3 = 45.4	39.9
B-2D-4	2-D Diamond Shapes - with Vertical Members at the Corner/ Edge and a Horizontal at 60 m	Pinned	Stable	Node X.060-1: U1 = 3.4 U3 = 31.8 Node X.040-1: U1 = 68.6 U3 = 29.5	30.8
B-2D-5	2-D “X” Shapes - with Horizontal Members every 20 m	Pinned	Stable	Node X.060-1: U1 = 3.7 U3 = 36.7 Node X.040-1: U1 = 4.3 U3 = 19.3	10.1
B-2D-6	2-D Original Lattice – Diamond Shape as Basic Structure	Moment / Continuous	Stable	Node X.060-1: U1 = 1141.6 U3 = 668.9 Node X.040-1: U1 = 300.8 U3 = 177.8	265.0
B-2D-7	2-D Diamond Shapes - with Horizontal Members every 60 m	Moment / Continuous	Stable	Node X.060-1: U1 = 4.9 U3 = 59.6 Node X.040-1: U1 = 135.7 U3 = 76.9	91.1
B-2D-8	2-D “X” Shapes - with Horizontal Members every 60 m	Moment / Continuous	Stable	Node X.060-1: U1 = 6.6 U3 = 88.3 Node X.040-1: U1 = 930.6 U3 = 498.3	209.4

[*] Deflections are given in absolute values: U1 = horizontal, U3 = Vertical

Table 2 Three-Dimensional Substructure Models

Model	Description	Connections	Period [sec]	Remarks		
				Stability	Deflection [mm]*	Max. Stress [ksi]
B-3D-1	Original diamond lattice; No links, no corner/ spine members	Moment / Continuous	28.8	Stable	U1 = 44,611 U3 = 4,947	130
B-3D-2	Original diamond lattice with diaphragm constraint (Type 1 link) every 60m	Moment / Continuous	22.5	Stable	U1 = 700 U3 = 269	28
B-3D-3	Original diamond lattice with Type 2 links every 60m	Moment / Continuous	23.9	Stable	U1 = 13,419 U3 = 1,950	62
B-3D-4	Original diamond lattice with Type 2 links every 60m (B-3D-3) with corner/ spine members	Moment / Continuous	11.9	Stable	U1 = 3,849 U3 = 642	31 (on spines) 25 (on others)
B-3D-5	Original diamond lattice with corner / spine members but no links	Moment / Continuous	12.1	Stable	U1 = 7,091 U3 = 750	42 (on spines) 32 (on others)
B-3D-6	Original diamond lattice with diaphragm (Type 1 link) at 180 and 420m only, but no spine/ corner members	Moment / Continuous	27.5	Stable	U1 = 3,788 U3 = 1,028	46
B-3D-7	Original diamond lattice with diaphragm (Type 1 link) at 180, 420 and 540m only, but no spine/ corner members	Moment / Continuous	27.5	Stable	U1 = 829 U3 = 554	46
B-3D-8	Original diamond lattice with diaphragm (Type 1 link) at 180 and 420m only, (B-3D-6) with corner / spine members	Moment / Continuous	12.2	Stable	U1 = 1,918 U3 = 421	30 (on spines) 20 (on others)
B-3D-9	(B-3D-8) with ties around each triangle at 180 and 420m	Moment / Continuous	12.2	Stable	U1 = 1,048 U3 = 309	27 (on spines) 20 (on others)
B-3D-10	Original diamond lattice with corner/ spine members (B-3D-4) and type 3 links every 60m	Moment / Continuous	11.9	Stable	U1 = 3,951 U3 = 659	31 (on spines) 26 (on others)

* Deflections are for a node @ 540 m (top of tower) given in absolute values: U1 = horizontal, U3 = Vertical
For 540 m height: 1/500 deflection = 1,080 mm

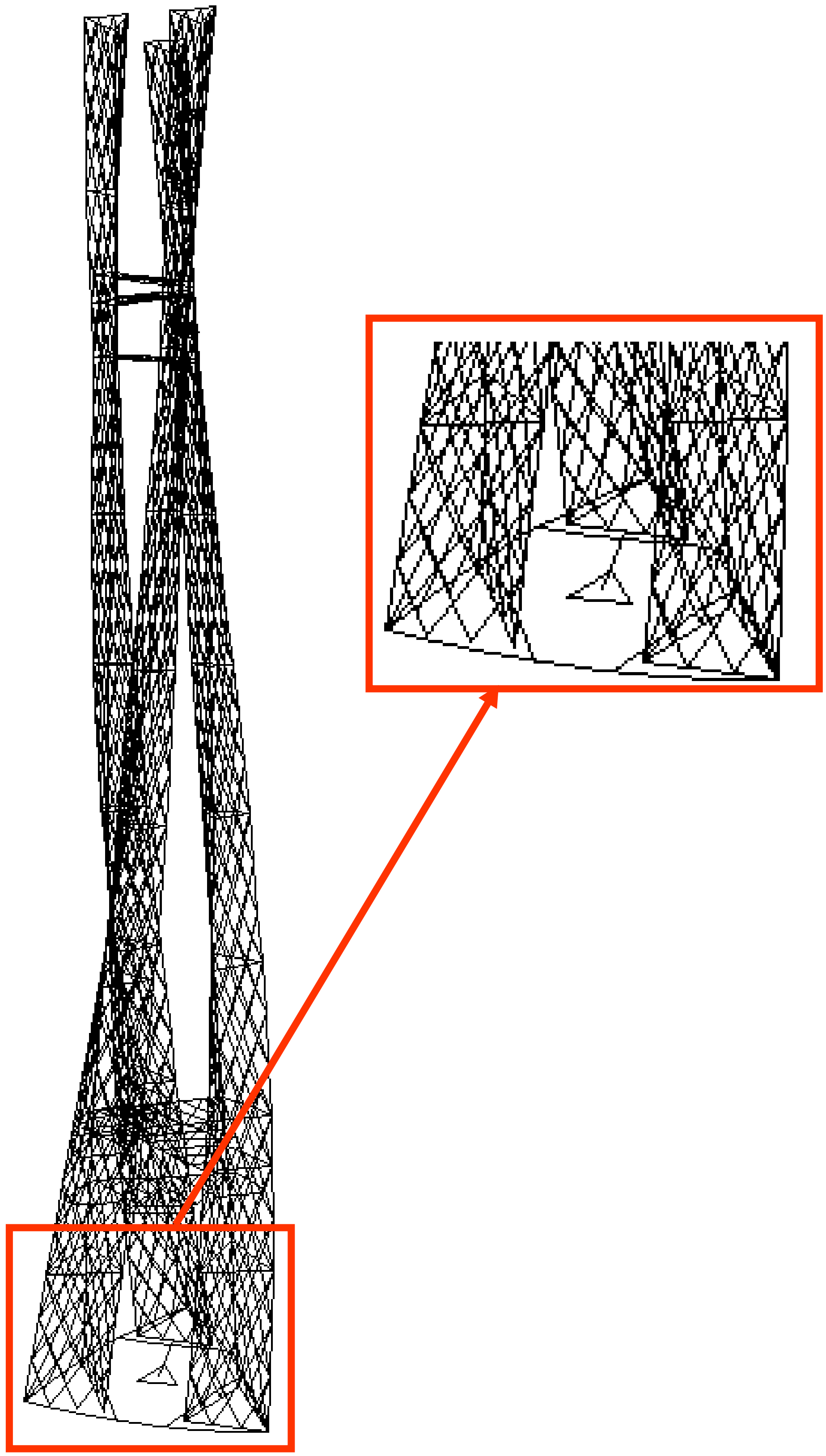
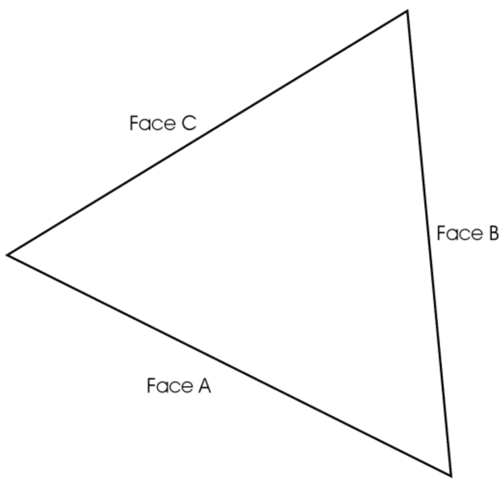
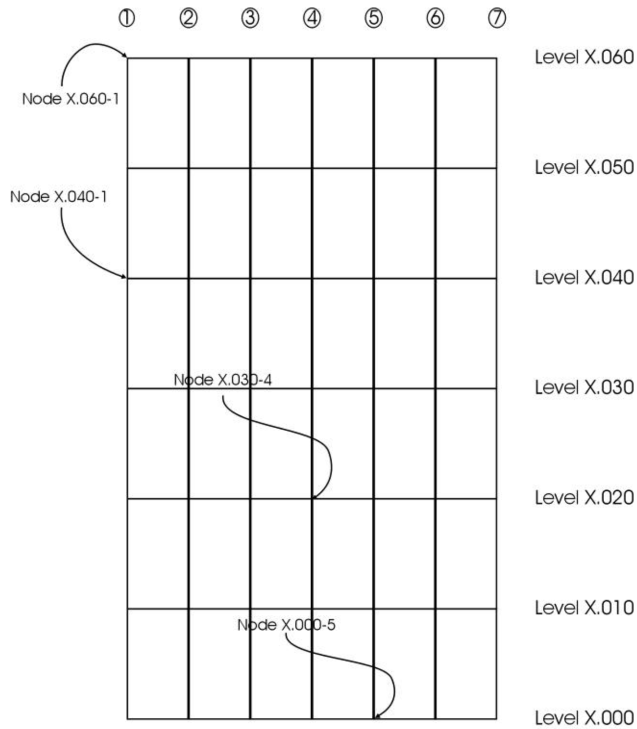


Figure 5 Substructure Structural Analysis

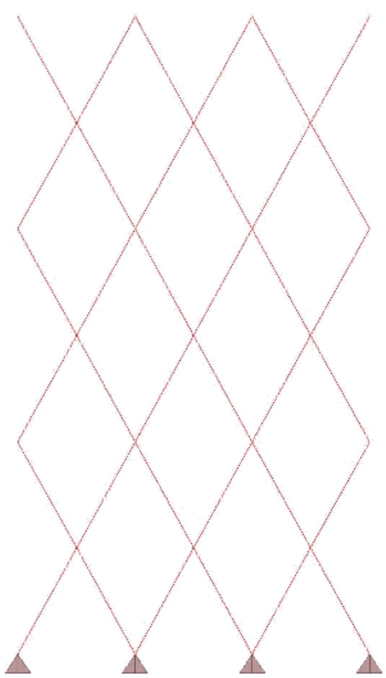


(a) Triangle at Level "Z"

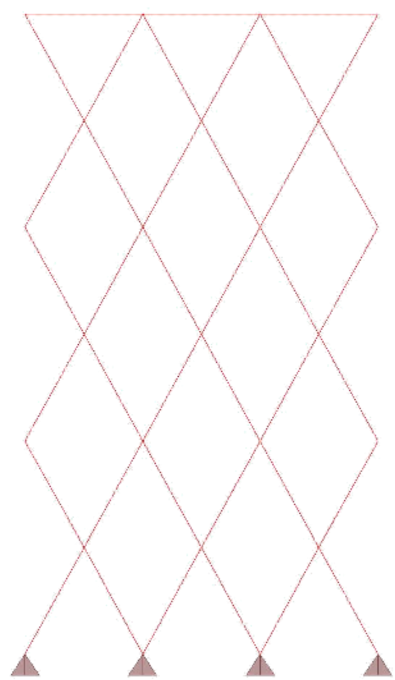


(b) Grid Setup on Face "X"

Figure 6 Two Dimensional Substructure

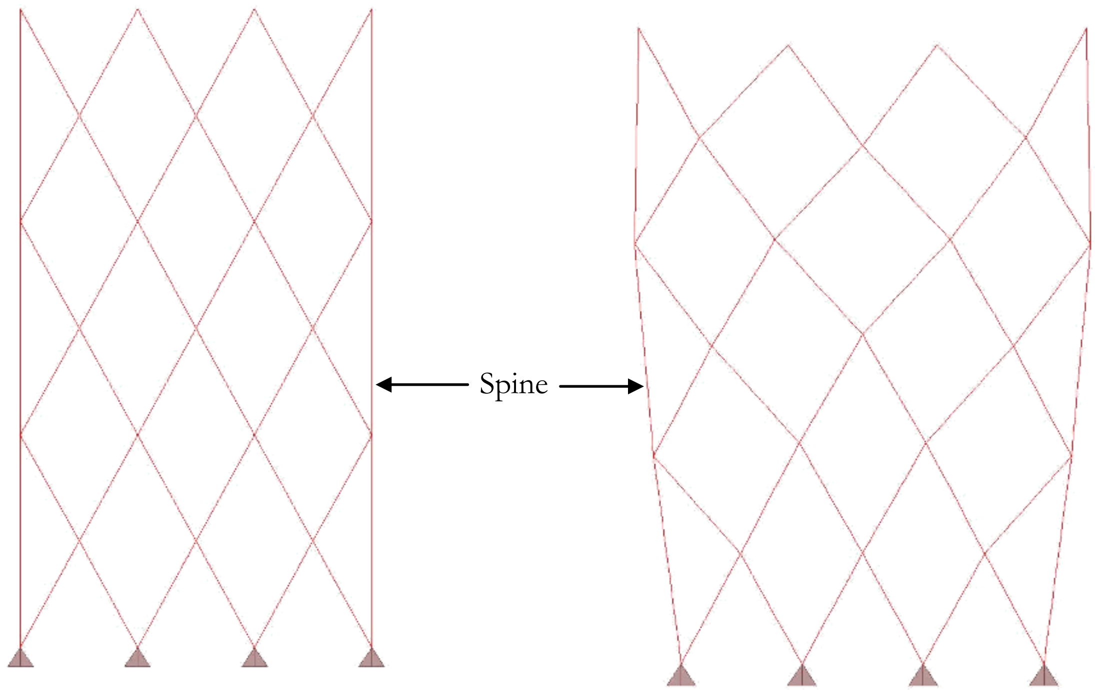


(a) Model B-2D-1: Original Lattice



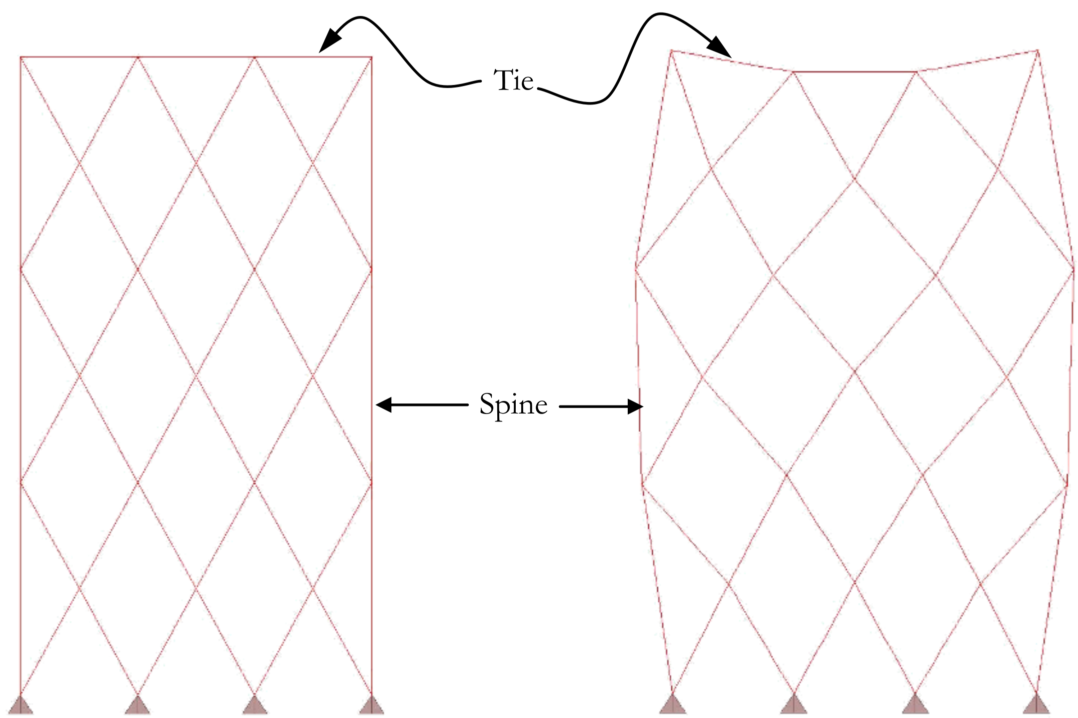
(b) Model B-2D-2

Figure 7 Unstable Models (Pinned Connections)



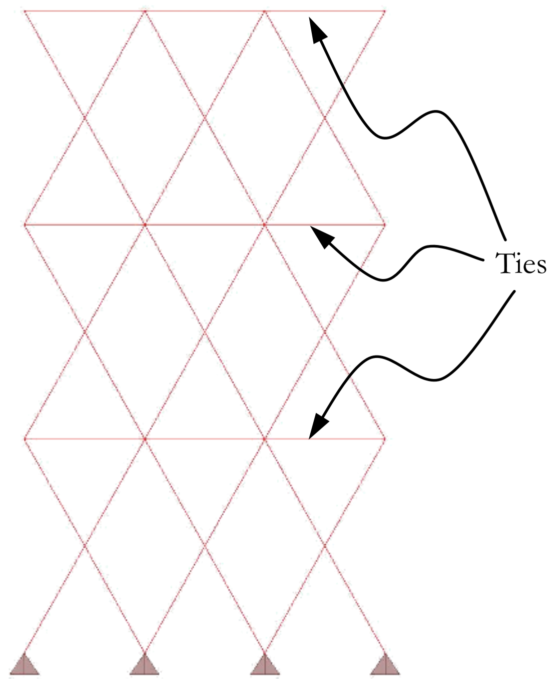
a. Undeformed Shape b. Deformed Shape

Figure 8 Model B-2D-3 (Pinned Connections plus Spine)

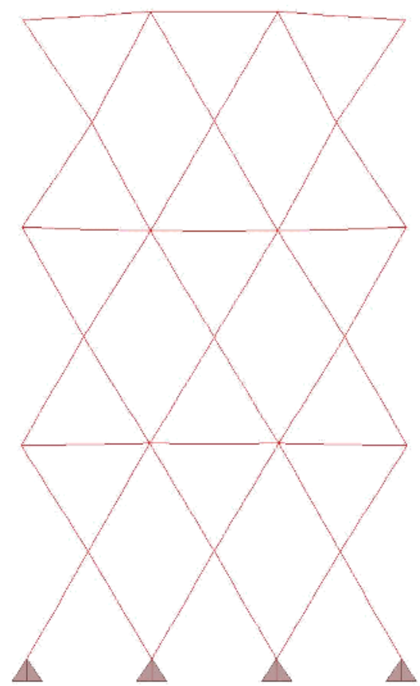


a. Undeformed Shape b. Deformed Shape

Figure 9 Model B-2D-4 (Pinned Connections plus Spine and Tie)

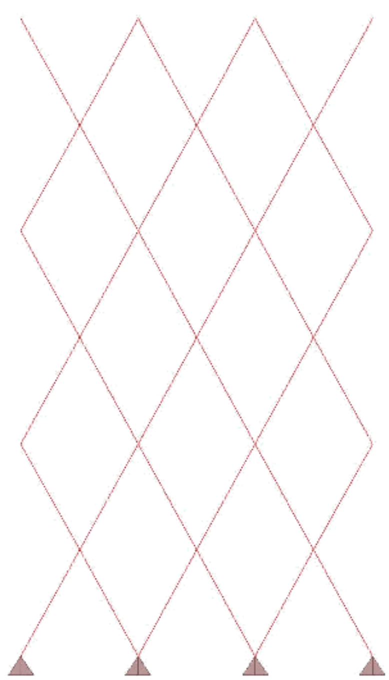


a. Undeformed Shape

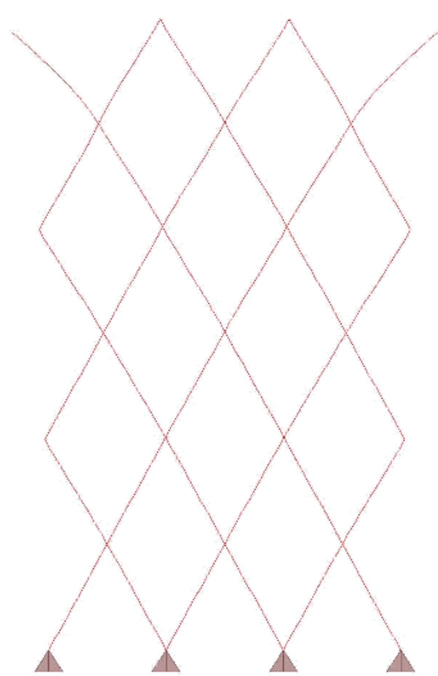


b. Deformed Shape

Figure 10 Model B-2D-5 (Pinned Connections plus Multiple Ties)

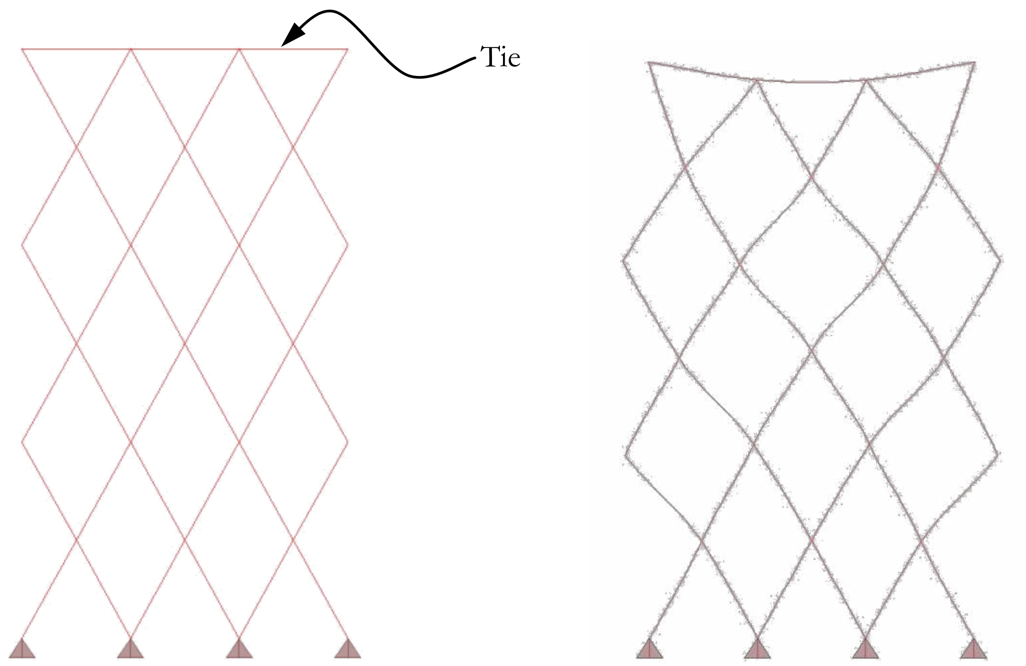


a. Undeformed Shape



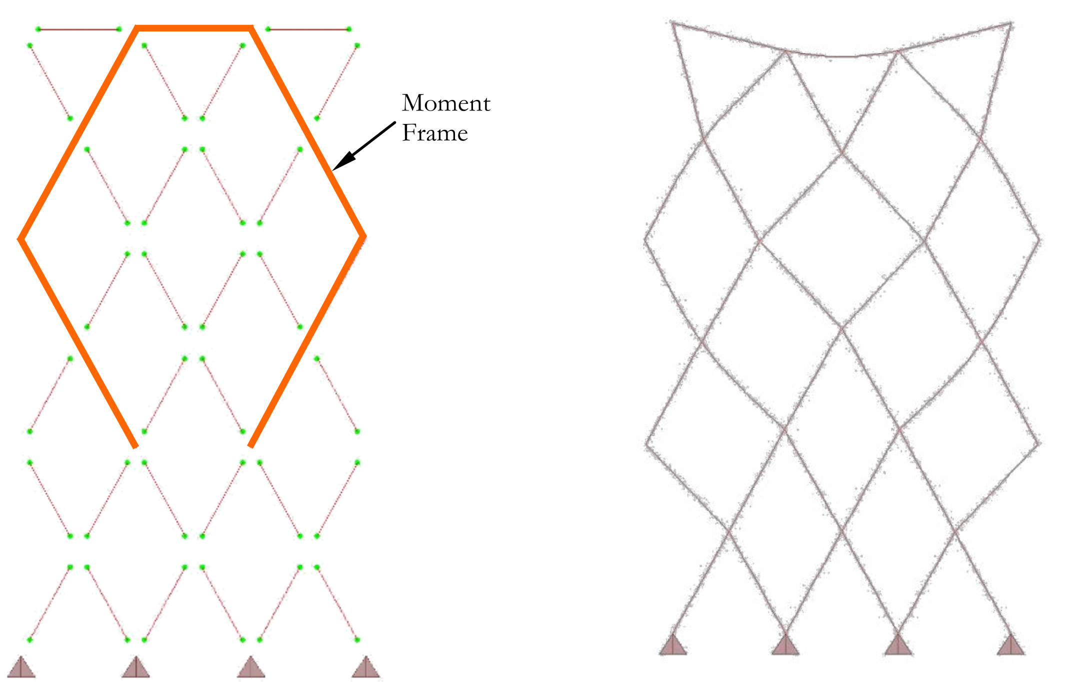
b. Deformed Shape

Figure 11 Model B-2D-6 (Moment Connections)



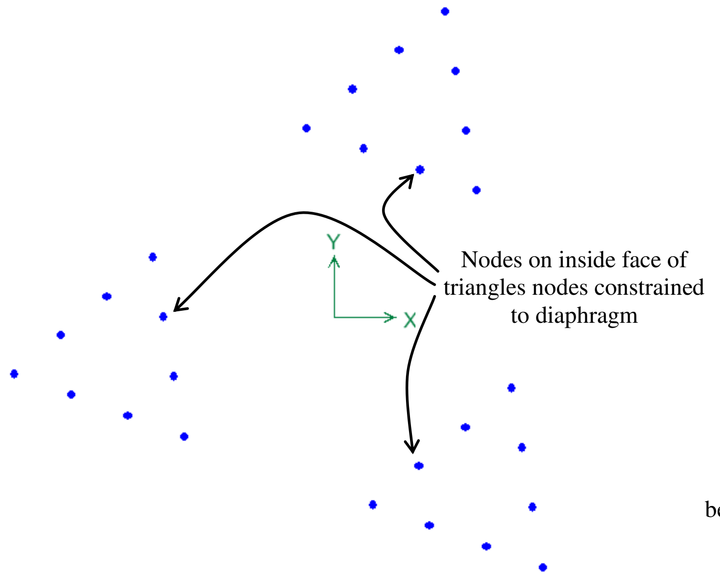
a. Undeformed Shape b. Deformed Shape

Figure 12 Model B-2D-7 (Moment Connections plus Tie)

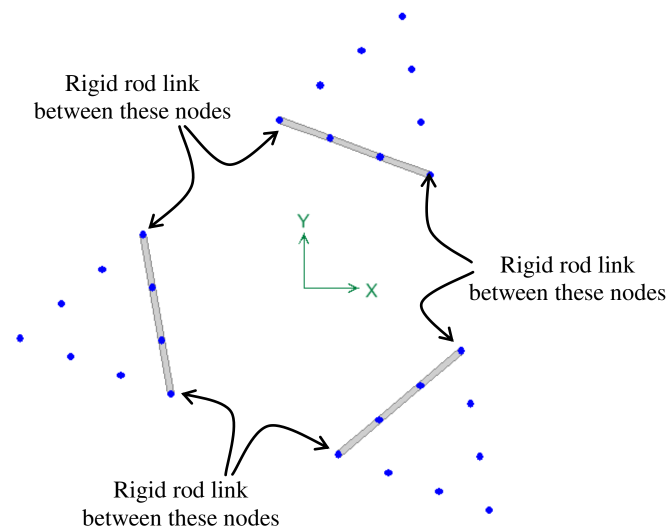


a. Undeformed Shape b. Deformed Shape

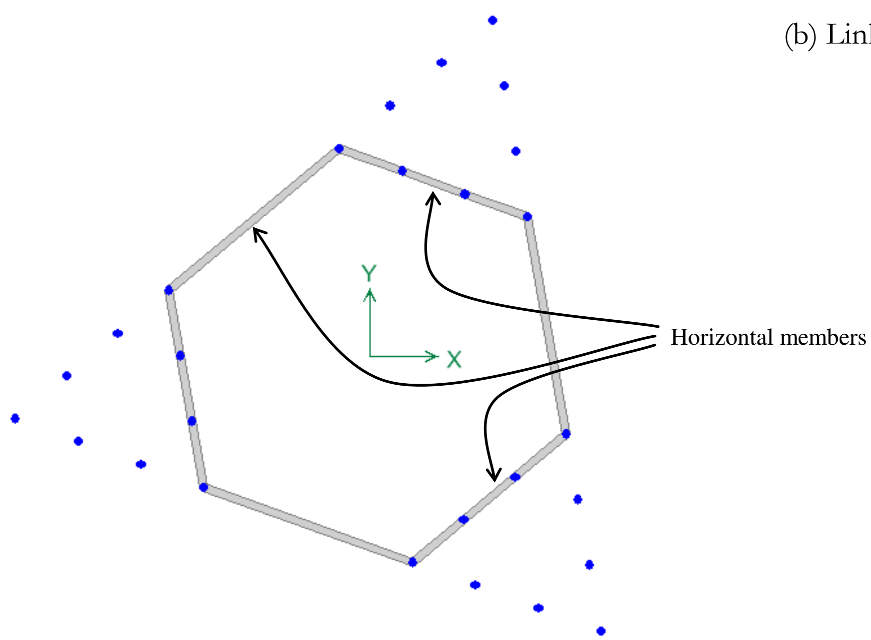
Figure 13 Model B-2D-8 (Triangle & Moment Frame)



(a) Link Type 1



(b) Link Type 2



(c) Link Type 3

Figure 14 Leg Link Connection at Level "Z"

4. STRUCTURAL LOADING, ANALYSIS, LOAD PATHS AND PERFORMANCE

The loads that the tower has to support / resist during its lifetime are:

- Dead loads: self-weight of the structure and any permanent fixtures, including elevators, antennas, etc.
- Live loads: loads from use and occupancy, excluding construction and environmental loads.
- Wind loads: loads due to pressure by wind pushing against the façade of the tower. The wind load depends on the geometry of the structure, structural damping, and the wind speed. The design wind speed is assumed to be 45m/s (100 mph) (3-sec gust wind speed, 50 year nominal return period) at a height of 10m above ground. The structural damping, which affects the dynamic interaction between the wind and structure, was taken to be 1%. This value of damping was derived from a comprehensive database of experimentally measured damping of tall structures, including towers, in Japan. The porosity of the mesh skin/ cladding and the geometry of the structure also affect the wind load, in which case, a force coefficient of 1.6 was estimated. This force coefficient and structural damping could be optimized by changing the shape and size of the opening in the mesh skin. A detailed wind tunnel evaluation of the loads will be performed in the design phase to account for the aero-elastic interaction of the structure with the wind.
- Seismic loads: loads induced on the structure by earthquake ground motion. A response spectrum for the site was created based on the seismic properties of the site and the resulting inertial loads applied to the structure.

These loads are combined according to specifications of the American Society of Civil Engineers (ASCE) standard, ASCE 7-02. The following load combinations were considered:

1. 1.4 D
2. 1.2 D + 1.0 L + 1.6W
3. 1.2 D + 1.0 L + 1.0 E

in which D = Dead load, L = Live loads, W = Wind load and E=Earthquake load. (For this tower, it was determined that the governing lateral load was wind, hence, the effects of wind load only were evaluated.

The structure was analyzed for gravity and wind loads, i.e. combinations 1 and 2 described above. The analyses carried out were nonlinear static analyses, which included the p-delta effects due to the twisting and leaning configuration of the legs of the tower. The members were checked according to specifications of American Institute of Steel Construction (AISC) standard, AISC-LRFD 2001 and sized accordingly. The process of iteration on design, analyses and checking converged on member sizes as follows:

- The spines range from 1.5m diameter and 150mm thick sections at the base to 600m diameter and 60mm thick sections at the top.

- The diagonals have the same outside dimensions as the spines, but have thinner sections.
- The horizontal ties, range from 0.9m diameter and 36mm thick sections at 60m height to 0.4m diameter and 16mm thick sections at the top.

Table 3 lists the different sizes of the members according to height. The steel beams supporting the floor diaphragms are assumed to have the same sections as the diagonal members at that height. The total weight of the structural members, including the weight of the floor diaphragms is estimated to be 60×10^6 Kg (133,000 kips).

The adequacy of the structure under progressive collapse due to extraordinary events (e.g. terrorist attacks) was scrutinized. These events are simulated by taking out one or two of the main load carrying structural members. The loads associated with progressive collapse are based on the US General Services Administration (GSA) guidelines on Progressive Collapse Analysis and Design, which specify that the structure be subjected to twice the dead load. It was found that the tower behaves satisfactorily under these extraordinary events and would be able to distribute the loads to the other undamaged members.

The fundamental period of the tower is 9.9 sec., while the second mode has a period of 9.8 sec. Both of these modes are translational, i.e. along an axis of the triangular base and parallel to one of the sides (orthogonal to the motion of the first mode). The third mode is torsional, i.e. rotation around the center of the tower, with a period of 4.9 sec. The static mode shapes for the first three modes are shown in Figure 15. The average horizontal displacement at the top of the tower due gravity loads (combination #1) is approximately 0.040m (0.007% drift), while for the design wind loads (combination #2) it is 11.0m (2.0% drift), which are within acceptable limits. Figure 16 shows the deflected shapes due to load combinations 1 and 2, respectively.

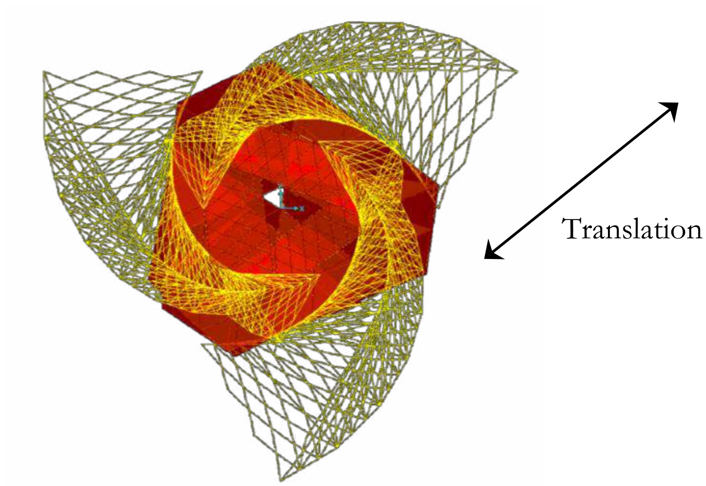
For human /occupant comfort at the observation levels, the horizontal floor accelerations induced by the wind loads was estimated using the deflections and structural properties obtained from the mathematical model of the tower. The criterion for human comfort was based on International Standardization Organization (ISO) code, ISO 6897, in conjunction with improvements recommended by Smith and Coull (1991)[4]. The computations show that the horizontal floor accelerations are within the acceptable limits for the nature of activities anticipated at the observation levels.

Figure 17 shows the load path for only a vertical gravity loading at the floor nearest the top of the tower. It is clear from this figure how the structural system distributes these loads to the tower's legs. Figure 18 shows the lateral load distribution over the tower legs for a wind loading applied in the direction from the bottom to the top of the page.

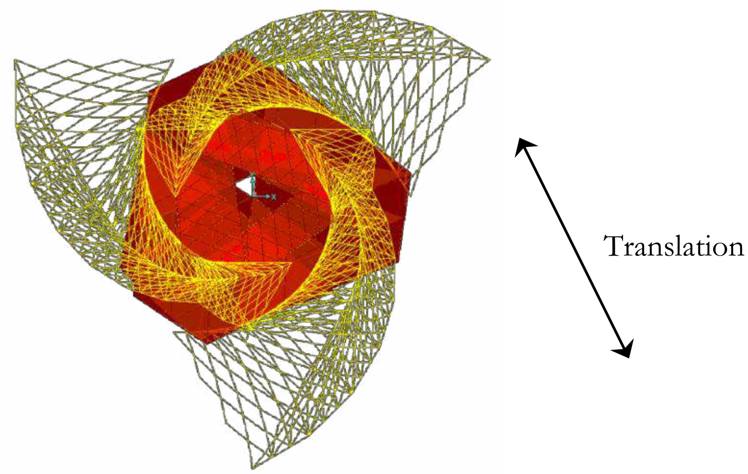
[4] B.S Smith and A. Coull (1991) "Tall Building Structures: Analysis and Design" John Wiley & Sons, Inc., New York, N Y: pp. 452-460

Table 3 Summary of member sizes

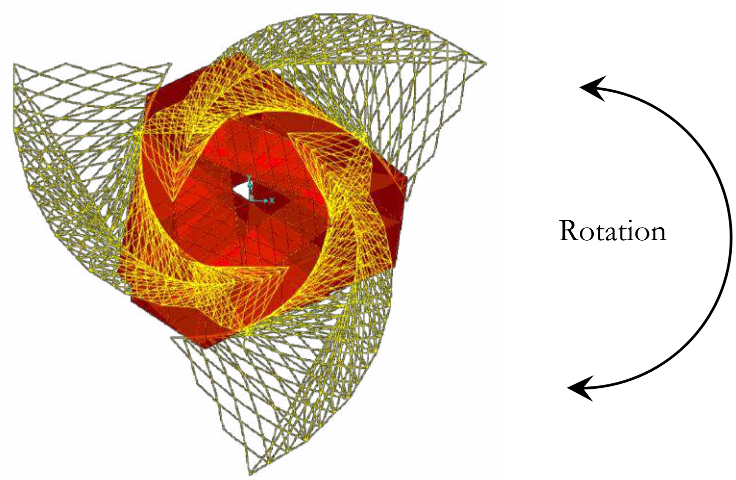
Height [m]	Member Sizes [m]		
	Spines	Diagonals	External Ties
0 - 60	$\phi = 1.50$ $t = 0.150$	$\phi = 1.50$ $t = 0.060$	$\phi = 0.90$ $t = 0.036$
120 – 240	$\phi = 1.25$ $t = 0.125$	$\phi = 1.25$ $t = 0.050$	$\phi = 0.80$ $t = 0.032$
240 – 360	$\phi = 1.00$ $t = 0.100$	$\phi = 1.00$ $t = 0.040$	$\phi = 0.60$ $t = 0.024$
360 – 480	$\phi = 0.75$ $t = 0.075$	$\phi = 0.75$ $t = 0.030$	$\phi = 0.50$ $t = 0.020$
480 - 540	$\phi = 0.60$ $t = 0.060$	$\phi = 0.60$ $t = 0.024$	$\phi = 0.40$ $t = 0.016$



(a) Mode 1

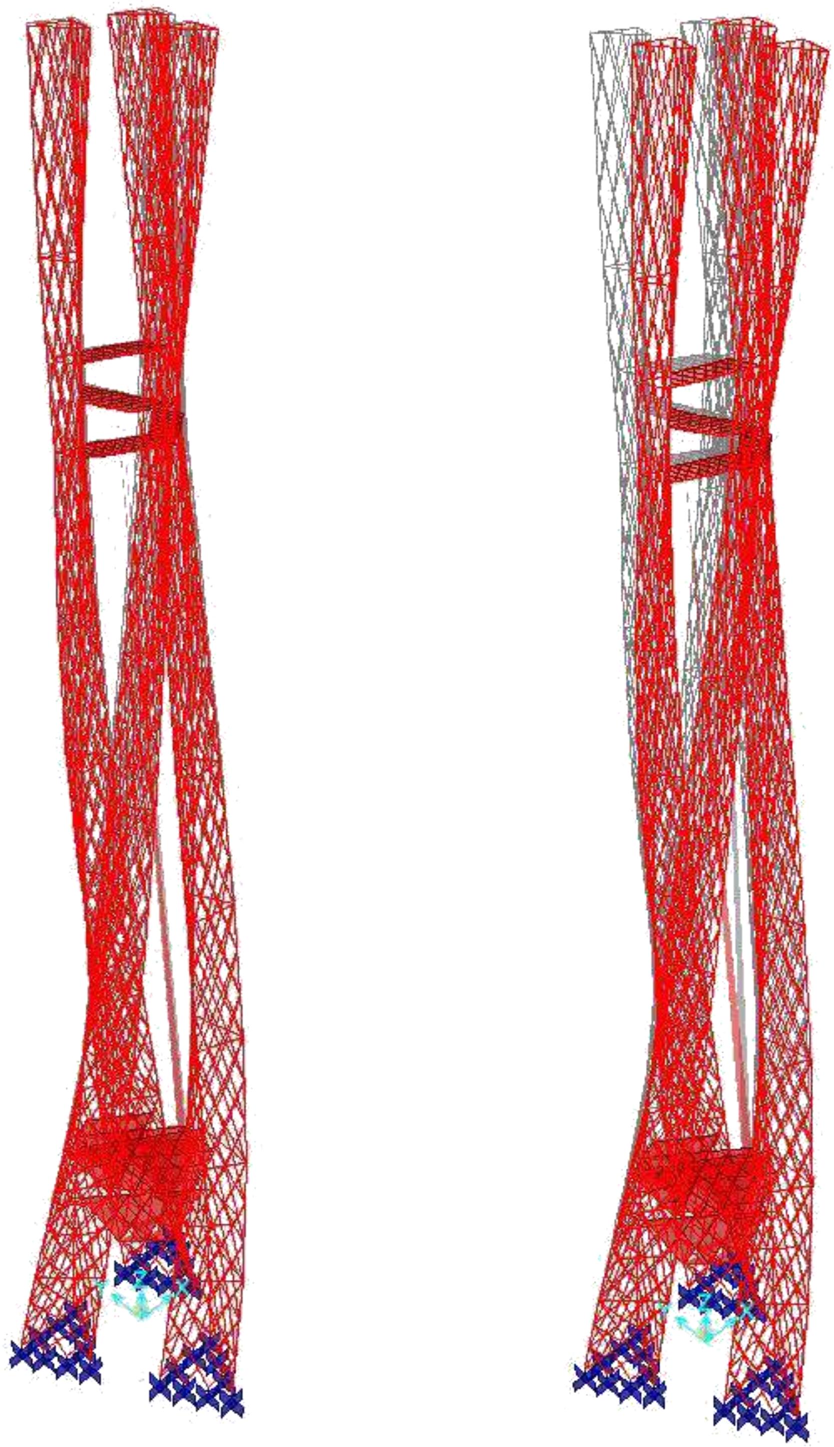


(b) Mode 2



(c) Mode 3

Figure 15 Modal Shapes of the Structure



a) Gravity Load (1.4 D) b) Gravity & Wind Loads (1.2 D + 1.6 W)

Figure 16 Deflected Shapes of Guangzhou Tower

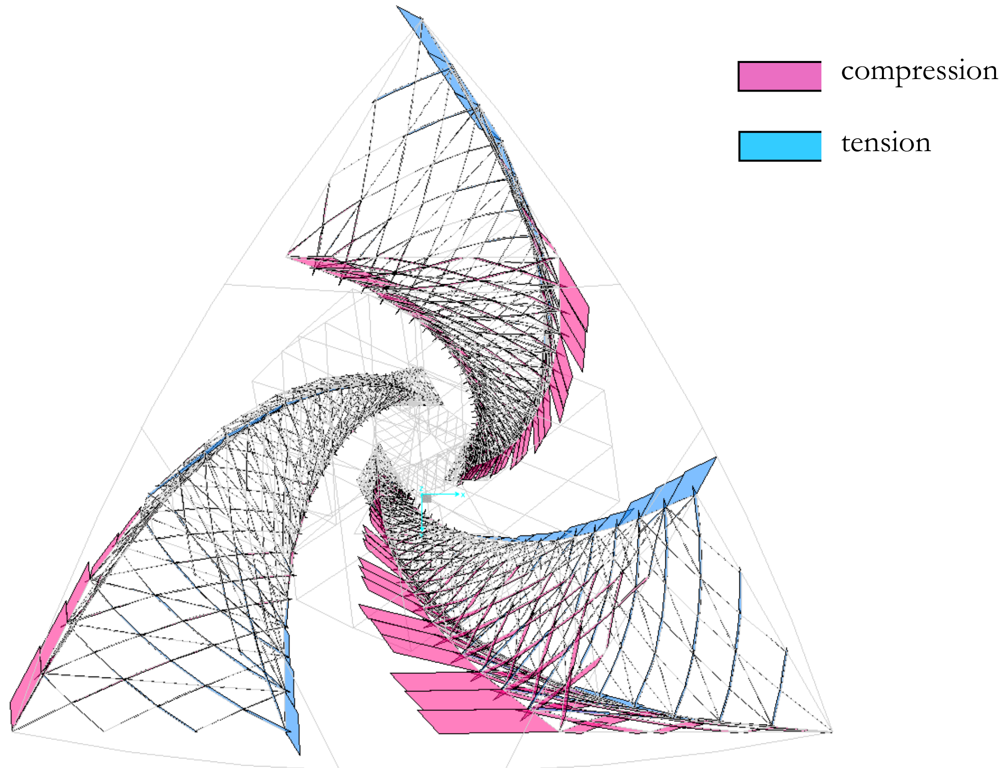


Figure 17 Load Path of Gravity Load on Platform

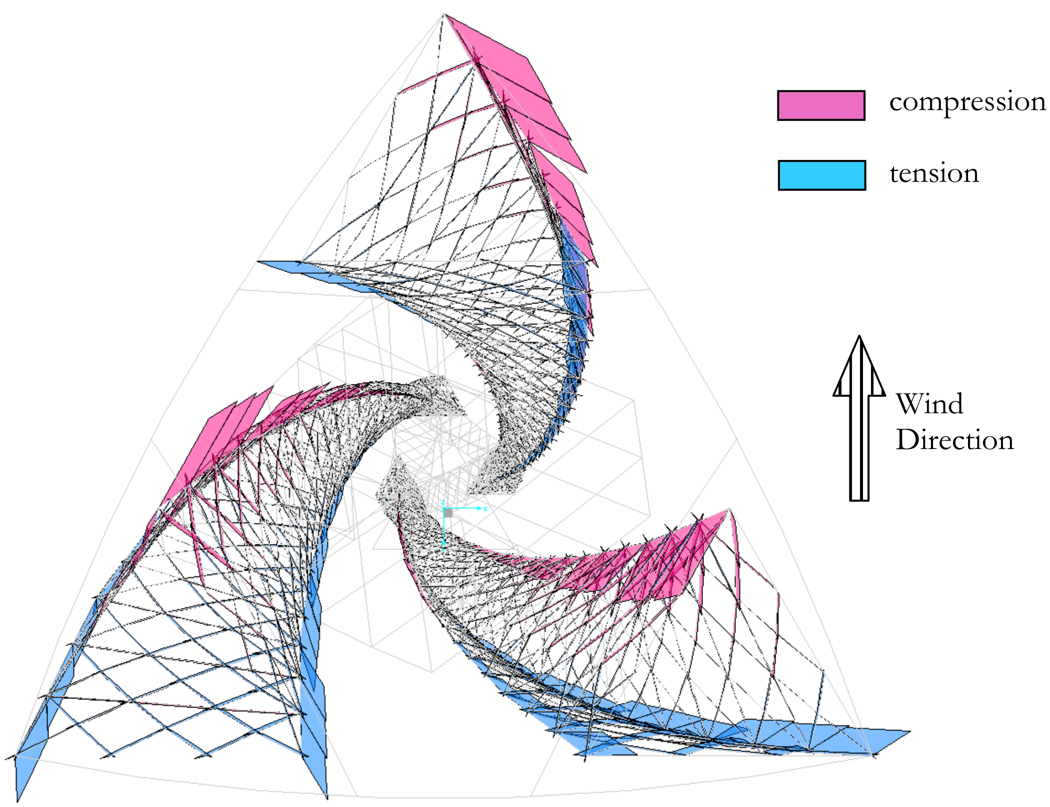


Figure 18 Load Path of Wind Load on Tower

5. THE LIVING ACTIVE TAYLOR DAMPER SYSTEM

Dampers are installed in buildings to "eat up" the energy that the wind or the earthquake imparts to a building. Figure 19 shows a Taylor viscous damper [1, 2]. Taylor dampers fall into one of three types which are passive, active, or semi-active. The properties of the dampers are selected to limit displacements and accelerators of the structure to code or professionally defined comfort or damage threshold limits. Passive dampers dissipate the wind or earthquake induced energy to the structural system by movement of the building and their mechanical properties are pre-defined. Prior or estimated future time varying signatures of wind or earthquake loads are used as the basis for the design of the characteristics of the damper. The efficiency of passive dampers is restricted because the properties of the dampers are fixed throughout their life span of the structures and also the time when the structure moves during the earthquake or hurricane and cannot be modified according to the actual motion the structure is experiencing. One benefit of these passive dampers is that they do not need a source of power to operate and their cost is relatively low since they are not accompanied by electronic devices or mechanical actuators.

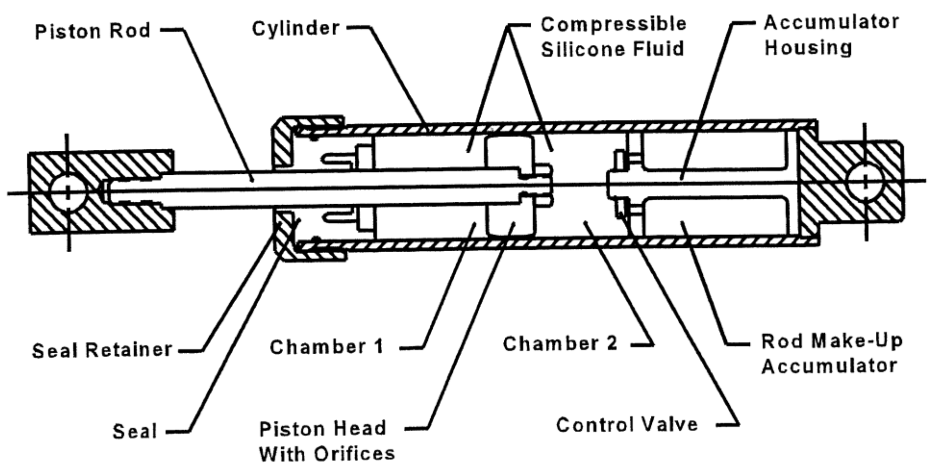


Figure 19 Typical Viscous Damper

The term active damper represents an active structural control system with several components. In a general sense, they are the building parallel to the control systems that are used in airplanes that operate an airplane when it is on autopilot. This active damper system in a building structure measures and monitors the motion of the structure in the earthquake or hurricane and then changes the properties of the damper to produce a predefined acceptable level of motion of the structure. The active damper control system consists of three main components: Monitors, Controller, and Actuators. Figure 20 is an illustration of the interaction between different components of an active damper control system [3].

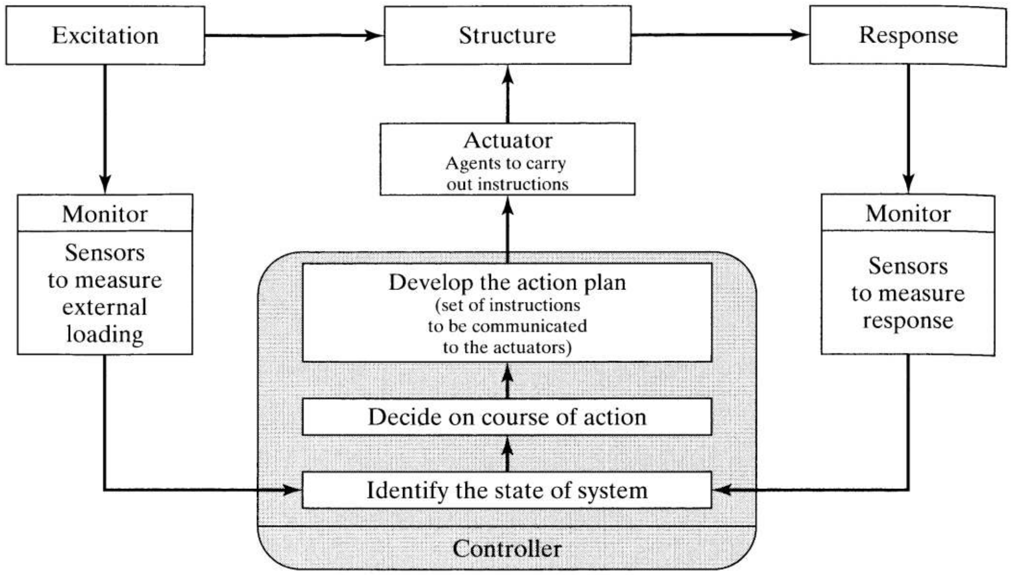


Figure 20 Active Control System [2]

Monitors collect data from the excitation source, i.e. the motion of the ground in the earthquake or the wind velocity at the site of the structure, and also the motion response (i.e. displacement or acceleration) of the structure. The controller module uses computer programs in their memory to calculate almost instantly using the theory of structural dynamics on a course of action to be taken to control the motion structures to acceptable levels. The actuators in an active damper system are the dampers that adjust their properties as directed by the controller module.

Active dampers are excellent for the limiting of the response of a structure to earthquake and wind loads because they can adjust their properties based on the actual motion of the structure and do not rely on pre-earthquake or hurricane estimates of what might happen. Active damper systems have been used in military applications for many years but they require care to ensure the operations are still relatively expensive and are not proven to be fail-safe.

A third type of damper system is called Semi-active dampers. In a sense this type of damper system provides a middle ground as the choice of dampers in the buildings. In semi-active dampers, the adjustment in the mechanical properties of the device is achieved in three ways. The most commonly used semi-active damper consists of a passive viscous damper with an external path for the fluid with a control valve as shown in Figure 21. The flow of liquid through the external pass can be regulated by changing the valve orifice opening size which in turn alters the mechanical properties of the damper. Two other types of semi-active dampers alternate the mechanical properties of the devices by exposing them to an electrical field (electro-faradic devices) or to a magnetic field (magneto-rheological devices). Semi-active dampers are less expensive than active damping systems and also do not require as much electrical power.

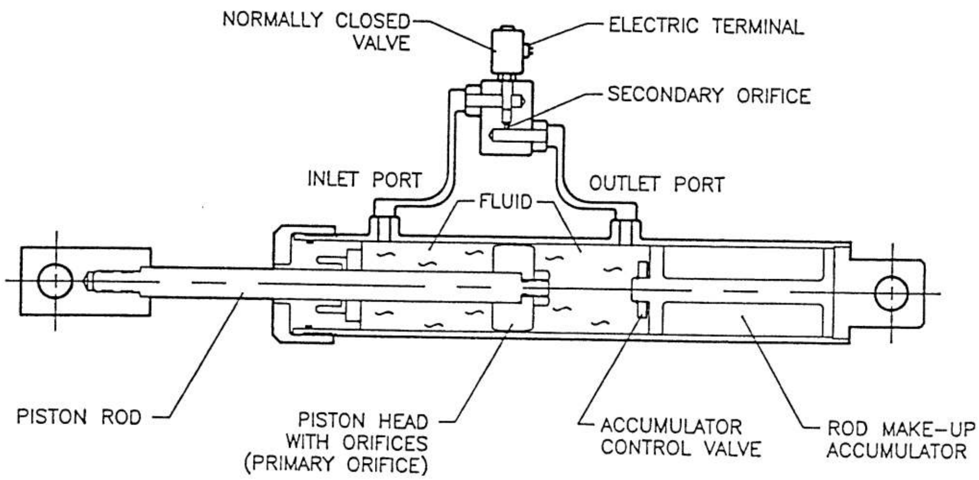


Figure 21 Semi-Active Damper

The equations of structural dynamics are used to design the properties of viscous dampers which are velocity-dependent energy dissipation devices. The damping equation for a linear damper is given by:

$$F = C\,V \tag{1}$$

Where F is the damping force and V is the velocity of motion in the direction of F. The coefficient C is the damping coefficient and depends on the mechanical properties of the damper. Linear viscous dampers were in effect the first generation of viscous dampers and have been replaced by non-linear viscous dampers where the damper force is given by

$$F = C\,V^{\alpha} \tag{2}$$

where α is the velocity exponent.

6. CONSTRUCTION

The construction sequence of Guangzhou Tower is envisioned as follows:

- Fabrication of Connections: The geometry of the proposed Guangzhou Tower dictates that the members have slightly varying configurations. The members in the structure are aligned at slightly varying orientation to each other, and the member lengths are also varying. The production of the connections will thus have to be computer-aided – REVIT CADD manufacturing – in which a computer is fed the geometry of the entire structure, and it would subsequently produce the desired geometry for a specified node. This connection geometry would then be exported to an adjustable casting system with a control mechanism, by which the desired geometry is configured, and hence the connection hub smelt. Other viable connection schemes could also be developed to comply with the architectural vision.

- Shop Welding of Members and Connections: The members connecting to a node are then welded to the connection hub. These members are half-length members extending to an elevation of 5m above and below the node. This is to allow construction to occur in 10m lifts.

- Shop Production of Lift Segment: A 10 m high lift segment consisting of the members and connections around the vertices of the triangle of each leg (see Figure 22) is formed. This lift segment could easily be stabilized by minor temporary bracing between the opposing members, to facilitate handling during transportation and erection.

- Erection: The segments are transported by barges down Pearl River to the job site and assembled on-site. These segments are erected by a crane and bolted to the members below. The bolt connections are now simple, conventional connections, as the members are being connected at mid-length, whose geometry is a simple straight line. A pair of cranes, working in tandem and crawling up the tower during erection are envisioned for construction of the towers.

7. CONCLUSIONS

When a building is important for the owner, community or public services, it should be designed as a Living or Smart Building.

REFERENCES

1. Lee, D. A., and Taylor, D. P., Viscous Damper Development and Future Trends, The Structural Design of Tall Buildings, Volume 10, p.p. 311-320, 2001
2. Symans, M. D., and Constantinou, C. C., Seismic Testing of a Building Structure with a Semi-Active Fluid Damper Control System, Earthquake Engineering and structural Dynamics, Vol. 26, p.p. 759-777, 1997
3. Corner, J. J., Introduction to Motion Control, Prentice Hall, 2003

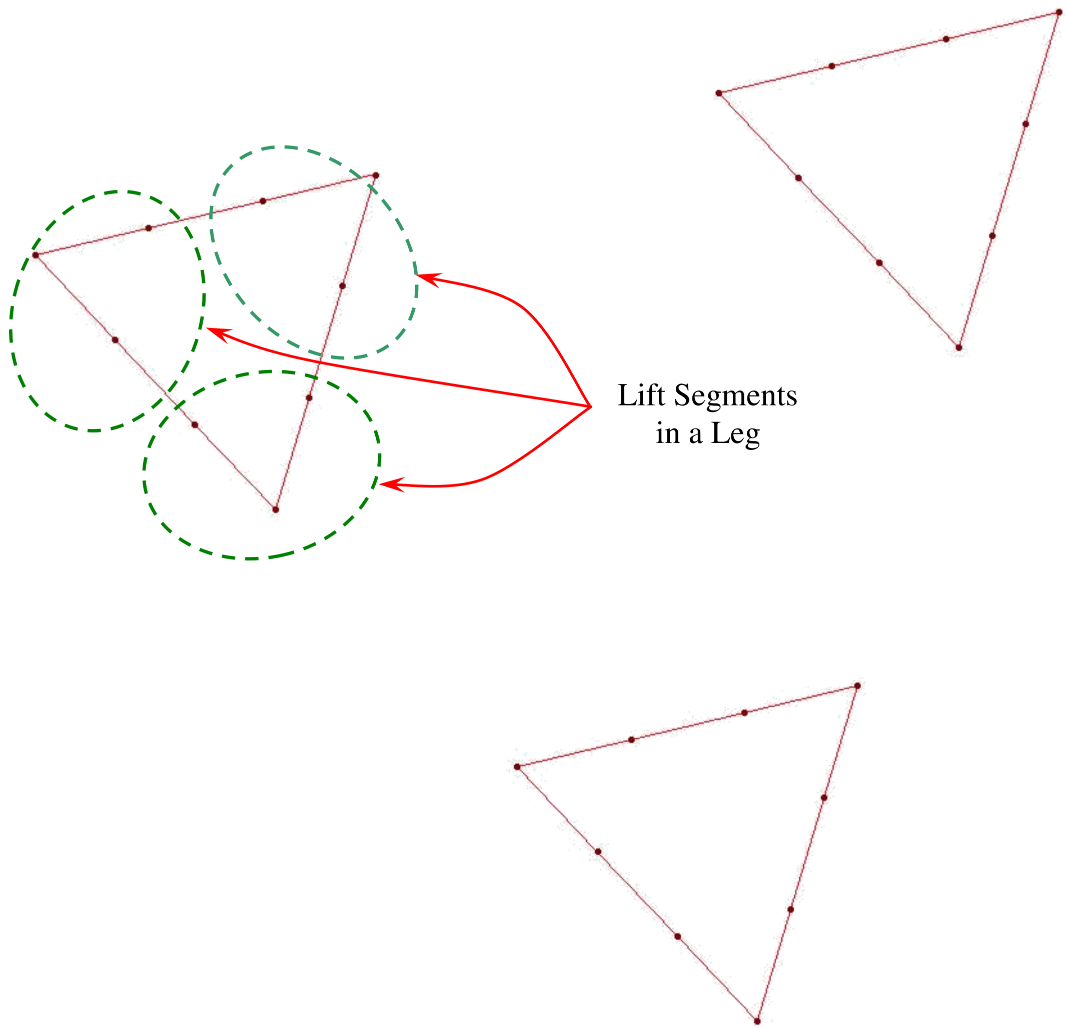


Figure 22 Plan View showing Lift Segments in a Leg of the Tower