Subject: Building Case Study
Structural Engineering

Paper Title: **Structural Design Innovation: Russia Tower**

Author(s): Halvorson, Robert
Warner, Carrie

Affiliation(s): Halvorson & Partners

Publication Date: 2007

Original Publication: CTBUH / Wiley Tall Journal, 2007

Paper Type:
1. Book chapter/Part chapter
2. Journal paper
3. Conference proceeding
4. Unpublished conference paper
5. Magazine article
6. Unpublished

THE STRUCTURAL DESIGN OF TALL AND SPECIAL BUILDINGS
Struct. Design Tall Spec. Build. **16**, 377–399 (2007)
Published online 26 October 2007 in Wiley Interscience (www.interscience.wiley.com). **DOI**: 10.1002/tal.423

STRUCTURAL DESIGN INNOVATION: RUSSIA TOWER

ROBERT HALVORSON AND CARRIE WARNER*
Halvorson & Partners, Chicago, Illinois, USA

SUMMARY

The Russia Tower will be Europe's tallest building and one of the most distinctive high rises in the world. The striking 600 m (1968 ft) tall form and its structure evolved through a collaborative process between the architect, Foster + Partners of London, and structural engineer Halvorson & Partners of Chicago. The innovative 'braced spine' structural system developed in this process is an efficient concept for super-tall structures and was a direct response to the design challenges and opportunities presented by this project. The design process between Foster and Halvorson that led to this unique tower design is described and several of its special structural engineering design considerations are discussed. Copyright © 2007 John Wiley & Sons, Ltd.

1. ONE-OF-A-KIND FORM

The STT Group, a prolific development company in Russia, approached Foster + Partners for a mixed-use project within Moscow City, a new development along the Moscow River 5·5 km (3·4 miles) west of the Kremlin. Similar to Canary Wharf, Moscow City is a major redevelopment of an entire section of the city and is home to many of Moscow's newest and largest buildings. The brief called for hotel, office and residential spaces to be arranged on the triangular site, with parking and retail at grade and below—for the 500 000 m^2 (5 400 000 ft^2) total project.

Foster first began exploring options for separate towers, but given the constraints of the site it was apparent that the proximity of individual towers would require a relationship between them. Halvorson and Partners was brought on board at this early stage for structural engineering services, in collaboration with Waterman International of London with offices in Moscow. In the design stages, Halvorson focused on the superstructure and Waterman on the substructure. Foster and Halvorson discussed the design opportunities for separate towers and some 'big picture ideas' of linking the individual structures in this project.

An optimal three-part radial arrangement was developed by Foster for the triangular site (see Figure 1) to maximize outward views and sunlight exposure. However, as separate towers, the resulting forms were very slender; the extreme being the 60-story hotel and residential tower with a 10 : 1 aspect ratio. Structural solutions were possible for this option of independent towers, but at these aspect ratios the solutions would be inefficient and have great impact on architectural flexibility and layout.

Halvorson suggested three possible structural concepts for linking the towers, and indeed any structural elements (see Figure 2):

* Correspondence to: Carrie L Warner, Halvorson & Partners, 600 West Chicago Avenue, Suite 650, Chicago, IL 60610, USA. E-mail: articles@halvorsonandpartners.com

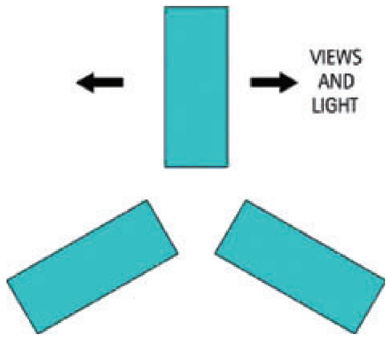

Figure 1. Radial arrangement of three towers

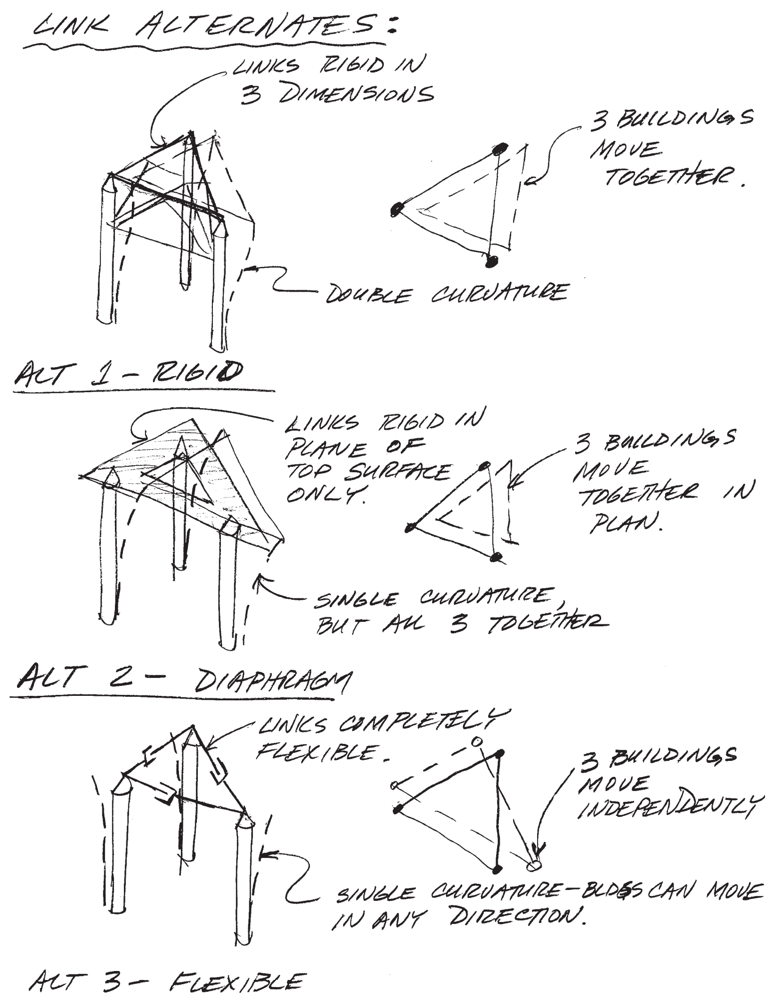

Figure 2. Structural concepts for linking towers

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(1) *Rigid*: the links transmit both horizontal and vertical shear forces, causing the elements to act as one and work together like a frame;
(2) *Diaphragm*: the links transmit horizontal shears only, so that the elements share lateral loads, but are not rigidly working together.
(3) *Flexible*: the elements remain structurally independent, and the links are provided with connections accommodating the differential movements.

Of these alternatives, the 'Rigid' concept had the greatest potential for increasing structural efficiencies since, when tied together, the strength and stiffness of the whole are substantially larger than the sum of the parts. Therefore, this concept was adopted for Russia Tower.

The design intent became, put simply, to 'lean' the three towers together. By continuously connecting their inner tips, the once separate forms became wings radiating from a central spine. The three parts were now rigidly linked, working together as a single structure—any one wing stabilized by the other two. Each wing was tapered in elevation to further express this leaning (see Figure 3) and

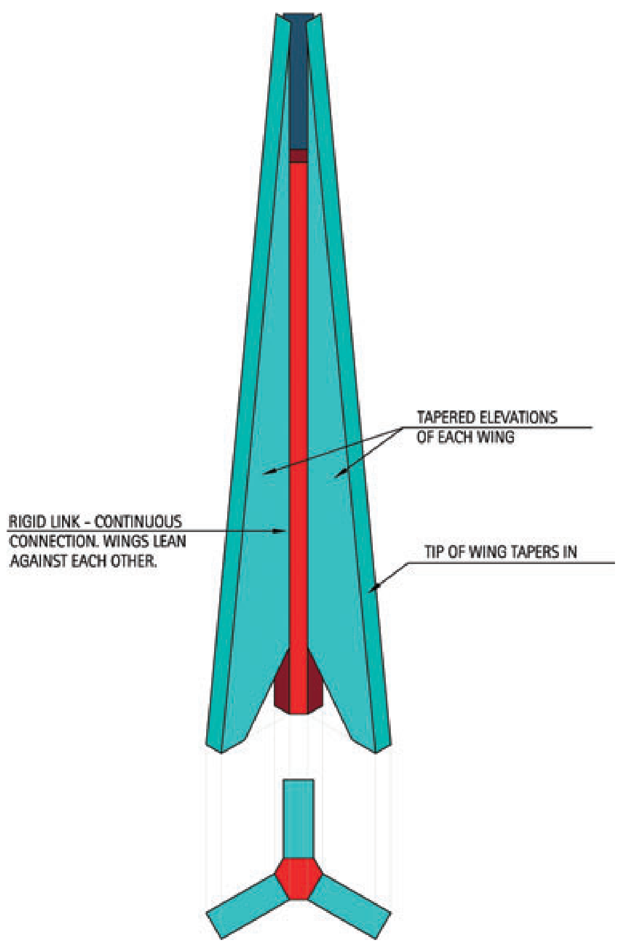

Figure 3. Three towers into one

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extended to 600 m (1968 ft) to maintain the floor area required in the brief. The tower actually appears to be quite slender due to this configuration, and maintains benefits for light and views—but in a far more structurally efficient form. The new aspect ratio is an astonishingly low 5 : 1.

Now, the challenge became finding an elegant and efficient structural solution for this one-of-a-kind tower.

2. STRUCTURING THE FORM: THE BRACED SPINE STRUCTURAL CONCEPT

The design team discussed and took to heart the key aspect of designing an efficient tall building: resisting lateral loads. A system is most efficient if it resists wind loads and gravity loads with the same structural elements. Also, overturning forces due to wind should be resisted by elements as far from the building center as possible and on members carrying sufficient gravity load to avoid uplift.

There were limited locations to place structure in this configuration: along the wing faces and tips and around the central zone were the obvious places. Halvorson suggested two initial structural concepts in plan (see Figure 4):

(1) a perimeter closed tube around each wing, with an interior triangle tube at their link—a ‘bundled tube’ organization; and
(2) a central closed hexagonal tube providing torsional resistance at the center of the plan, with stiff planes along the sides of each wing to brace the central tube laterally.

Foster was excited by the opportunities in the second suggestion and saw design potential for expressing the structural system at the exterior. Numerous structural concepts were discussed—including a core with outriggers, a diagrid exoskeleton, stepped core bracing, and more (see Figure 5).

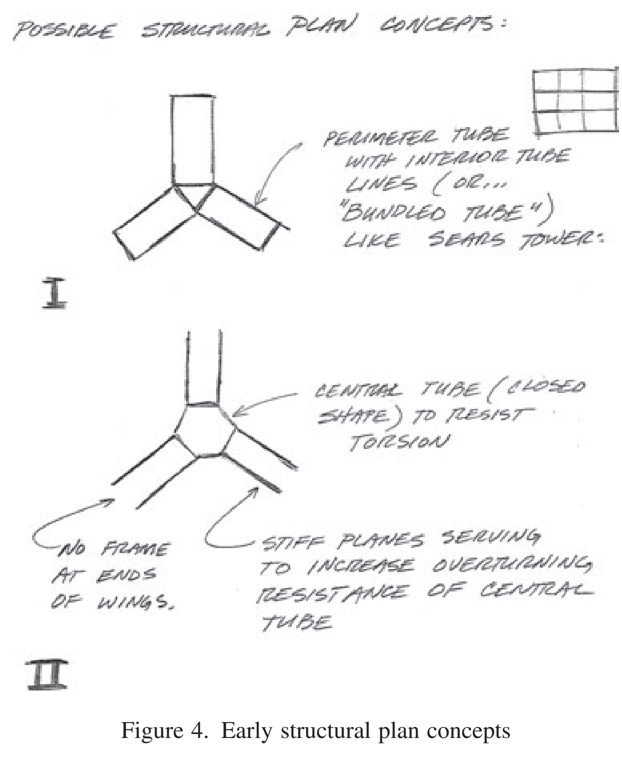

Figure 4. Early structural plan concepts

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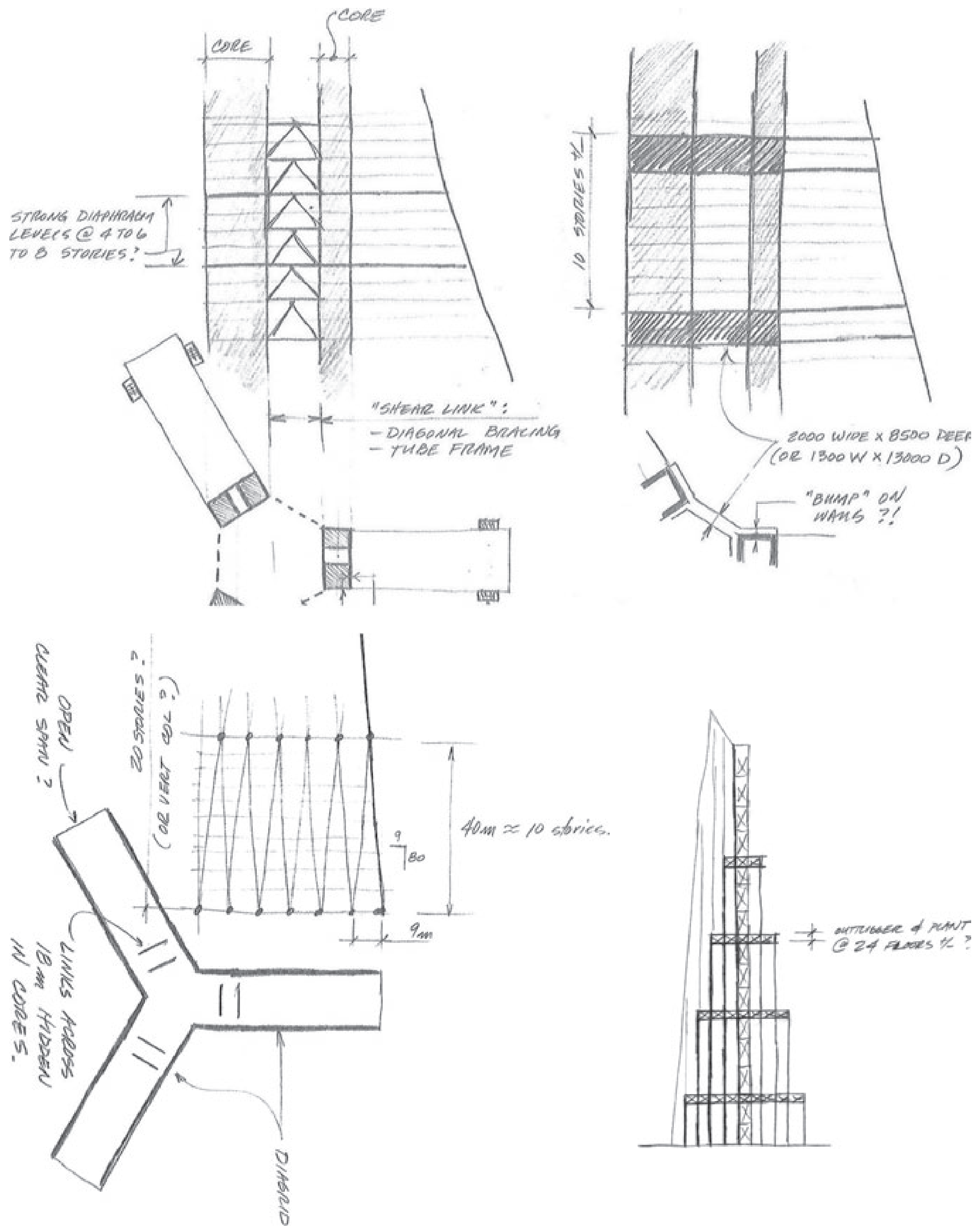

Figure 5. Early structural system concept sketches

One structural concept that related to the form of the building was particularly intriguing to the design team. It suggested a series of sloping, parallel columns at a regular spacing to brace a central core laterally and to carry gravity loads (see Figure 6a). This arrangement maintained consistent bays that offered efficiency in horizontal framing. Foster appreciated this concept, but questioned whether these sloping columns might instead originate at one point at the base of each wing (see Figure 6b). This arrangement allowed all overturning forces to be resolved at the furthest point from the building center. And visually, it was stunning.

At this point, the design essentially had arrived at a familiar structural form: that of a cable stayed mast. Instead of tension cables, however, the tower has sloped columns acting in compression (which became known as 'fan columns') to prop the center core, or 'spine', against wind loads and also carry gravity loads. A cable-stayed system is common for very tall, light structures. The cables are pulled away from the mast at ground to provide a low aspect ratio for the system, creating a stiff system that carries overturning forces as far from the center as possible for efficiency. The system also efficiently relies on the axial stiffness of the bracing cables, rather than bending stiffness of any bracing members. The braced spine structure for Russia Tower has these same benefits for its lateral system. However, unlike the conventional cable stayed mast system, this structure must support significant gravity loads. The fan columns therefore carry gravity loads as compression forces along their sloped axis and are stabilized by leaning into each other—essentially acting like three-dimensional arches.

A parametric study illustrates the structural efficiency for carrying lateral loads with this system, as compared to a conventional core-and-outrigger system used in tall towers (see Figure 7). For the braced spine system, lateral loads are resisted as axial forces in the fan columns and the 'spine' must, in effect,

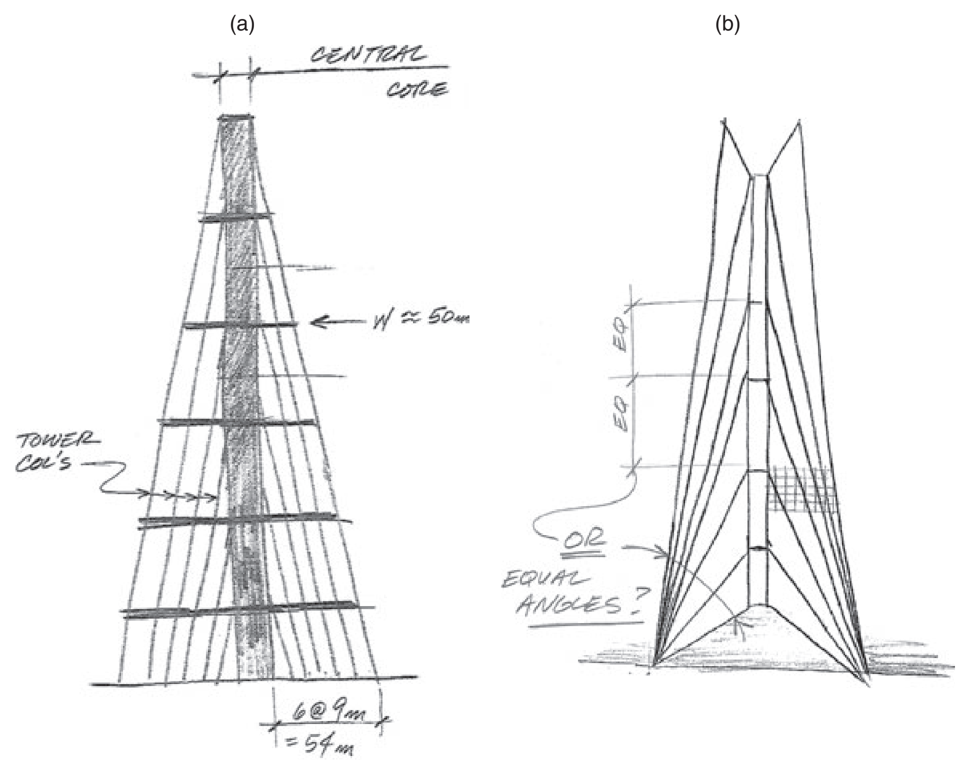

Figure 6. Reaching current structural form. (a) Parallel sloped columns. (b) Sloped columns to one point

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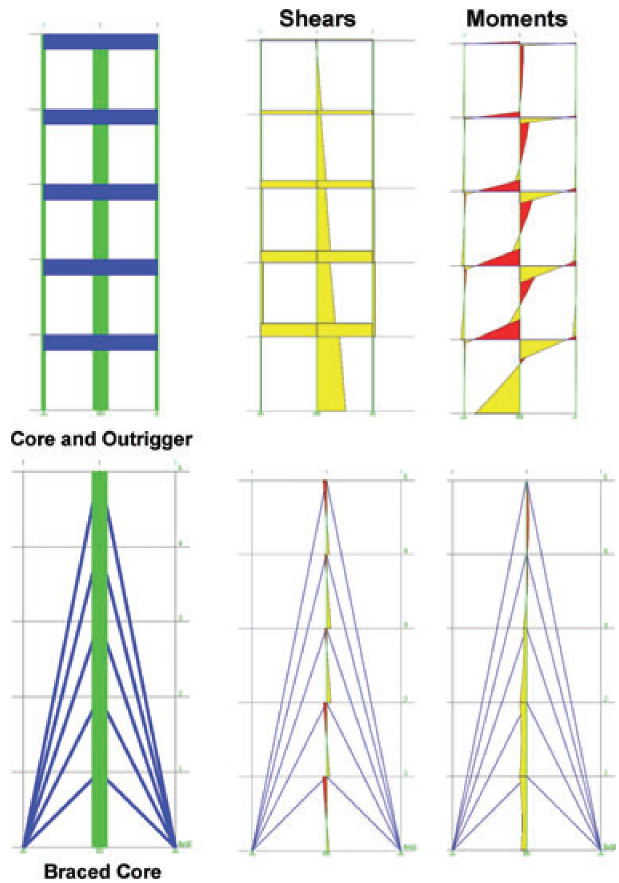

Figure 7. Lateral load path diagrams

only span like a continuous beam between the points where the fan columns brace the spine. In contrast, for a core and outrigger system, all the lateral shear loads on the building accumulate in the core, leading to large bending moments in the core and in the outriggers; core moments in this system are more than five times those found in an equivalent braced core system. Therefore. the stiffness and strength required in the core of a braced spine structure are much less than would be required in the core of a core and outrigger system.

Russia Tower's low aspect ratio (5 : 1) combined with the system's reliance on the axial stiffness, rather than the bending stiffness of its members, results in a very stiff tower. Lateral accelerations, which often govern the design of tall structures, are well within recommended limits without the use of any supplemental damping. Furthermore, this stout aspect ratio imparts a stable base and satisfies one of the important structural principles mentioned earlier—carrying the lateral loads out far from the building center to minimize axial forces due to overturning.

In order to carry vertical gravity loads on the sloping columns, horizontal stabilizing forces are induced and balanced between the three wings. Where each fan column intersects the central spine these major horizontal thrusts are resolved by links across the spine (see Figure 8). Horizontal forces are also induced in the floor structure and carried in the spandrel beams, as this arching action leaks out from the primary fan column and link load path.

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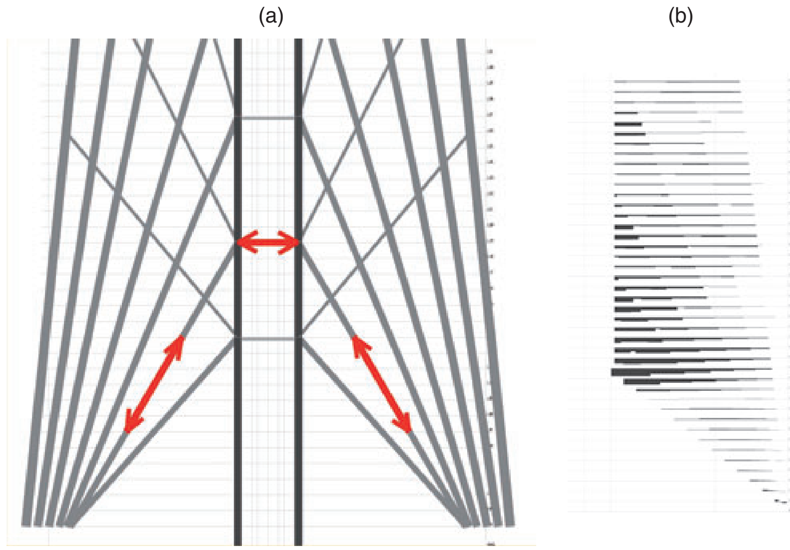

Figure 8. Gravity load path diagrams: (a) in fan column and links; (b) in spandrel beams

For the Russia Tower, the efficiencies of the braced spine system, the low aspect ratio, and the significant gravity loads carried in the primary lateral brace members result in the ultimate efficiency for a tall buildings—wind loads are almost carried ‘for free’! Fan column sizes were established by strength design under gravity load combinations; when checked for wind load combinations, only nominal column size and/or reinforcing increases were required. In addition, the wind loads never exceed the gravity loads to cause tension in the columns (except at a few columns near the top of the tower).

With an efficient system established, the fundamental remaining challenge was torsion. Because the core alone did not provide adequate torsional stiffness, the entire perimeter of the tower needed to become a ‘closed section’. After much unsuccessful sketching about how to incorporate diagonals sloped oppositely to the fan columns, Halvorson suggested that where the fan columns hit the central core a ‘reverse fan’ column be created at the same relative angle to the core, and extend upward and outward; this would triangulate the wing facades (see Figure 9). Foster and the team liked it! The rigid wing faces were then linked across the wing tips by four-story steel chevron bracing to ultimately provide the desired ‘closed section’. Through these modifications, the fundamental torsional period of the tower was reduced from 12 s to 5 s, which allowed the tower to meet recommended torsional acceleration and velocity limits based on a preliminary wind assessment prepared by Dr Nick Isyumov. The basics of the system were set.

3. DEVELOPING THE SCHEME: DEFINING THE STRUCTURAL COMPONENTS

3.1 Structural materials

Concrete is the structural material of choice in Moscow, and for carrying pure compression forces it is very economical. In the Russia Tower, the fan columns, reverse fan columns, and the core structure (or ‘spine’) carry primarily compression forces, with low shear and bending forces and no tensions. Reinforced concrete seemed a logical choice for these elements.

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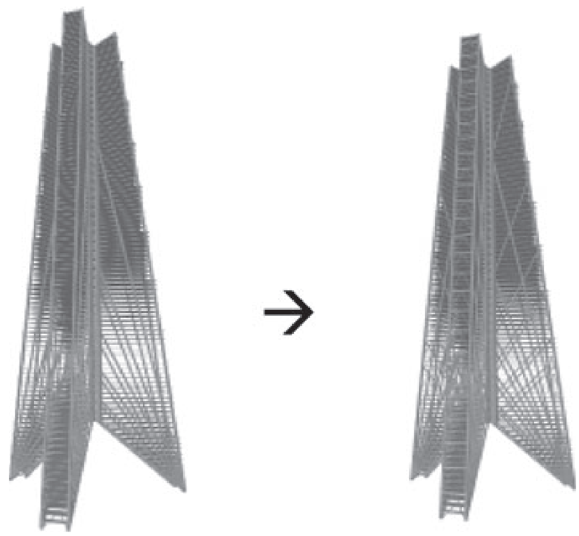

Figure 9. Solving torsion issue—add reverse fan columns on façade

Steel was appropriate for much of the remainder of the structure. Perimeter spandrel beam spans vary due to the fan column layouts, reaching as much as 18 m (60 ft). The developer also wanted column-free office floors, requiring a 21 m (69 ft) clear span across the wings. Post-tensioned concrete framing was not a viable option in Moscow, without which these long spans would require prohibitively deep reinforced concrete beams. Thus, long-span composite steel trusses became the choice for office floor framing members.

For the hotel and residential levels, a flat plate reinforced concrete system would be the conventional solution, since the necessary interior columns could be accommodated in the floor layouts and a flat plate offers advantages in floor-to-floor heights. However, the design team felt that having one structural concept, i.e., not mixing composite and concrete systems, from bottom to top of the tower was important for avoiding large interruptions in the construction sequence when systems changed. Further, transferring interior concrete columns would require story-deep transfer walls within and along the edges of each wing, which could not be accommodated. The large-perimeter spandrel beam spans would also have presented difficulties for reinforced concrete. For these reasons, the design team decided to utilize steel floor framing with composite slabs for the full height of the tower.

Since a steel system is lighter than a reinforced concrete system, this also offered significant savings for the gravity load-carrying elements and foundations. Steel bracing was also the most appropriate solution for linking across the tips of the wings and linking across the center of the tower, both of which wanted as much transparency as possible.

3.2 Constructability

The team addressed some of the fundamental concerns for constructability when developing the structural system, primarily focusing on the issues of composite construction. Integrating steel and concrete systems is always tricky. The interface must reconcile tolerances between the two systems. It must also transfer forces via relatively low-capacity elements, such as shear studs or reinforcing bars, embedded in the concrete and welded to steel plates for field connections.

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As described above, the concrete fan columns and core walls primarily support gravity and wind loads, and the steel floor framing and secondary bracing must transfer their significant loads into these concrete members. This typical connection of the floor framing to the fan columns also requires capacity to resist progressive collapse tie forces and brace the heavily loaded columns. Additionally, the typical column connection at Russia Tower has over 1 m (3·3 ft) of eccentricity, as the floor framing perimeter beams are offset 825 mm (32·5 in) from the inside face of the fan columns.

In response to these conditions, Halvorson recommended that the entire structure be constructed as a steel building, using steel erection columns for the fan columns and core walls. Concrete encasement of the erection columns would follow a few floors behind. This would allow all framing and bracing connections to be steel-to-steel, with forces then transferring into the concrete along the erection column's length (see Figure 10). Additionally, all erection tolerances and the construction scheduling could be established by standards of steel construction, reducing the challenges of reconciling tolerances for concrete and steel construction that might occur if a stepped-form concrete forming system was instead used.

3.3 Summary of the tower's structural system

At this point, having made the fundamental decisions for the structure, Fosters and Halvorson continued ongoing collaboration to further define, refine, and optimize the elements. A summary of the structural system's primary components, including some of their design issues, is provided. (see also figure 11 for summary)

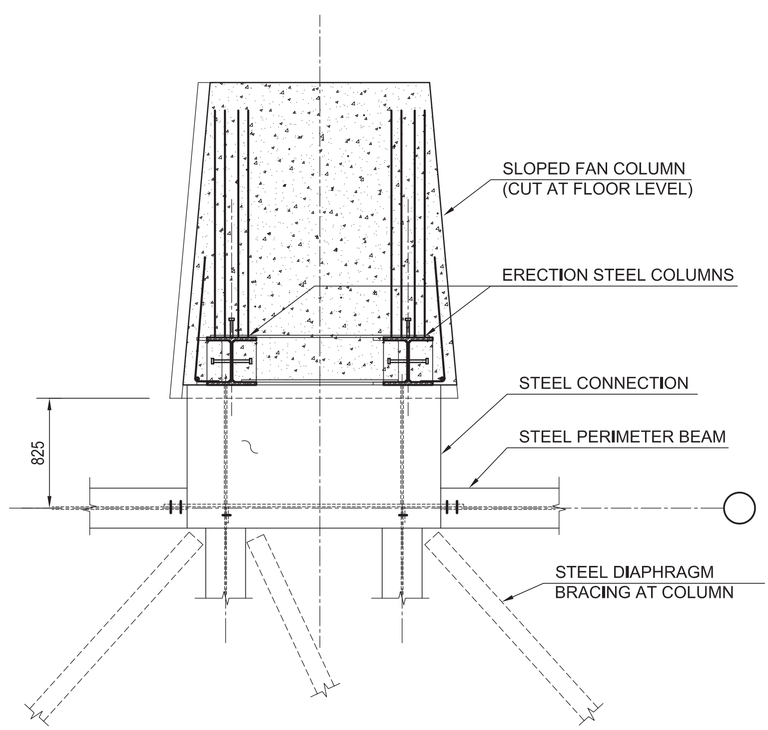

Figure 10. Connection of fan column to floor framing

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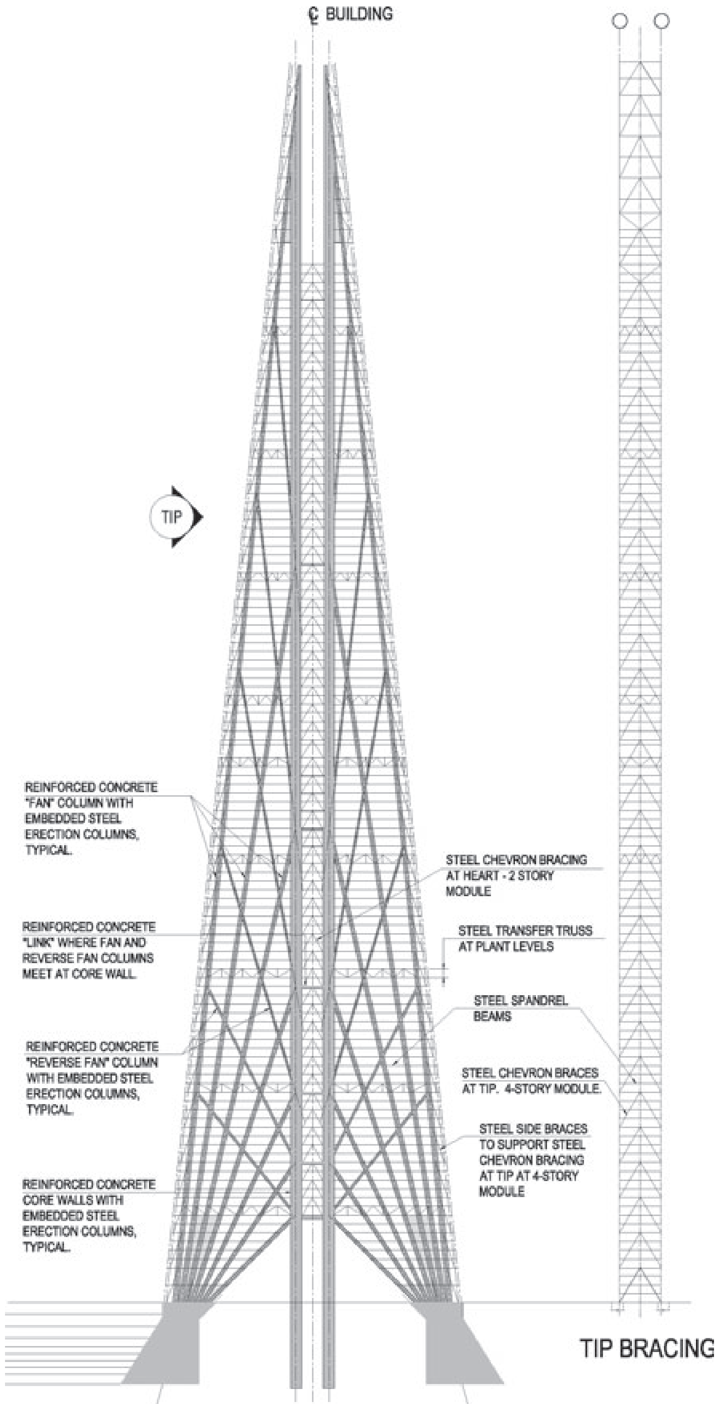

Figure 11. Structural system summary elevation

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Fan and reverse fan columns

On each wing face, seven fan columns radiate from a base abutment toward the central spine, where they then 'bounce off' the spine to create the reverse fan columns. The fan columns are set outside the floor spandrel, but still within the building envelope. The columns (which vary in cross-sectional area from 1·0 to 6·25 m^2, 10 to 67 ft^2) have a trapezoidal profile that allows them to appear more slender from the exterior and reduces interference on views looking out from tower. Relatively light, steel erection columns designed only for temporary construction loads are first erected at each fan column location along with the floor framing as for a conventional steel building. These erection columns are later encased by reinforced concrete, which provides the final fan column strength for permanent loads. Floor framing loads are delivered to the steel erection columns, which then shed their loads into the concrete column via shear studs.

The spine: core walls and spine bracing

The central spine is formed by reinforced concrete walls hidden in core elements in each wing, which are linked together across the open space in the central 'spine' by two-story steel chevron bracing. Where the fan columns intersect the spine, substantial horizontal reinforced concrete beams are provided to resist the thrusts of the fan columns. Like the fan columns, the core walls utilize steel erection columns sized for temporary loads, allowing the steel construction to proceed first, followed by placement of the permanent concrete structure.

Tip bracing

Four-story steel chevron bracing at the wing tips serve to link the rigid faces formed by the fan and reverse fan columns, creating 'closed sections' in each wing for torsional stiffness. The bracing also props up the framing of the cantilevered wing tips, which extend 4·1 m (13·5 ft) beyond the outermost fan columns. The support offered by the bracing allows shallower spandrel beams to be used than would have been possible with a cantilevered floor structure. The bracing also allows the corners to be column free to further increase views.

The base

The fan columns on each face meet at a massive abutment, with its centroid aligned with the resultant thrust of the fan columns (see Figure 12). A large, 11-story atrium is located in the space created below the sloping columns. Between the base and Level 11, each of the wings is an independent element since it is not connected to or braced by the other wings. Stability for each of the wings is created by locating bracing on the underside of the wings and by interconnecting several of the fan columns along each face (see Figure 13).

The vertical elements of the spine carry down to the foundations, providing a direct load path for gravity loads; but all beams and bracing linking these vertical core elements together are removed to provide transparency in the atrium. Consequently, the independent structures of each wing also serve to stabilize the building as a whole at its base.

Also at the base, a steel and glass roof encloses the column-free 11-story atrium (see Figure 14). For its 75 m (246 ft) column-free spans, this 1·5 m (4·9 ft) diagonalized steel structure acts effectively as a stiffened bent plate, with the 10 m (32·8 ft) cantilevered horizontal canopy plane stiffening for wind loads and the sloping plane stiffening for gravity loads.

Office levels

In the lower third of the tower, office floor framing typically spans the width of the wing (21 m or 69 ft) to provide a column-free floor area. This framing consists of 950 mm (37 in.) deep built-up steel trusses composite with the 80 mm (3 in.) metal deck plus 80 mm lightweight concrete topping.

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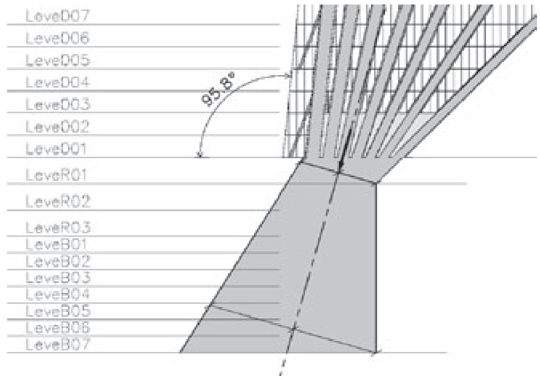

Figure 12. Fan column abutment

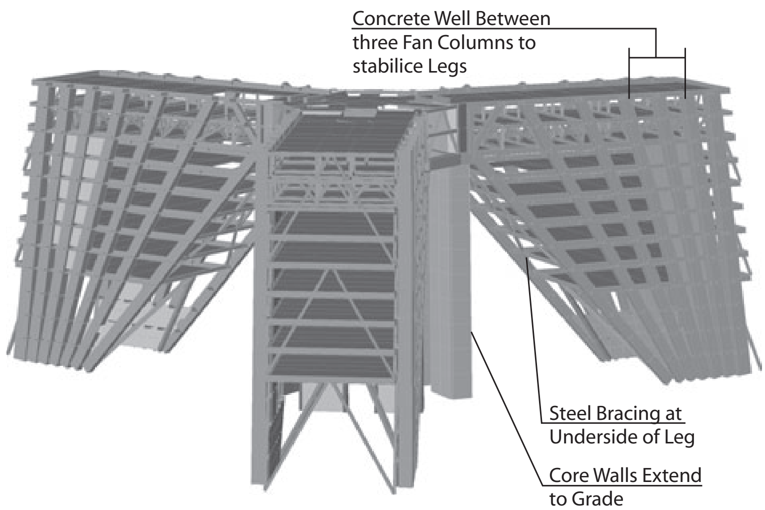

Figure 13. Base of tower structure

Building services are coordinated through voids in the trusses. At the perimeter, 900 mm (maximum, 35·4 in.) deep steel spandrel beams support the interior floor framing and resist axial loads induced by sloping columns and floor diaphragm forces. In the core areas where the wings intersect at the spine of the tower, there are many significant floor openings. Here, the typical slab thickness is increased, and in-plane floor diaphragm bracing is added to channel diaphragm forces around these openings.

Hotel and residential levels

In the hotel and residential levels for the remaining stories above the office, a pair of internal column lines along the centre of the wings reduces spans to allow shallow steel floor framing (300–450 mm deep, 12–18 in.) and maximize ceiling heights. The composite floor slab matches the office levels. At the perimeter, steel spandrel beams were limited to 600 mm (24 in.) to achieve desired ceiling heights.

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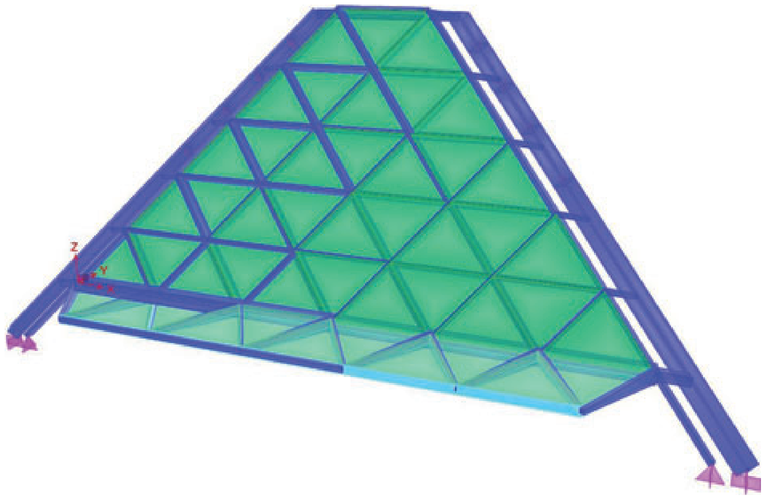

Figure 14. Structural model of atrium structural steel and glass roof enclosure

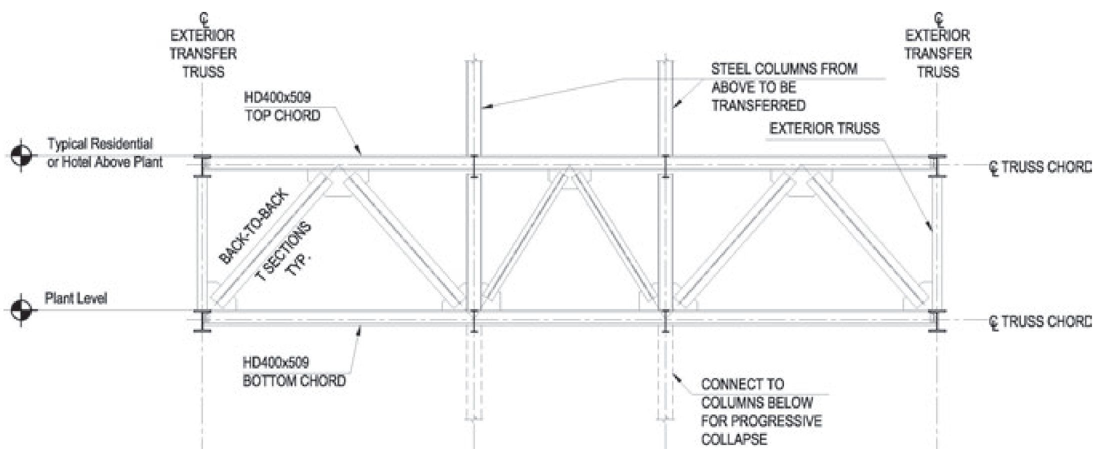

Figure 15. Full-depth transfer truss at plant levels

The loads from the interior columns in the hotel and residential floors transfer to the perimeter structure at 'plant' levels approximately every 12 or so floors by means of story-high steel trusses (see Figure 15). This transfer is necessary to avoid differential shortening issues between the steel interior columns and the concrete fan columns. Other benefits of the column transfers are that they direct gravity loads to the fan columns, which are concrete and therefore more cost-effective in carrying the compression loads; and, by getting sufficient gravity loads into the fan column net tensions under wind load conditions are effectively eliminated.

Plant levels

At the plant levels, interior story-deep trusses span 21 m (69 ft) across the wing to perimeter story-deep trusses, which transfer loads to the fan columns. These trusses utilize steel column shapes for their top chords and back-to-back T-sections for their diagonal web members. The floor structure within the plant levels span between the story-deep steel trusses. The typical maximum girder depth is 600–700 mm (24–28 in.), with shallower framing between for MEP clearance needs. The floor structure consists of 80 mm (3 in.) composite deck plus 145 mm (5·75 in.) normal weight concrete topping with two layers of reinforcing, as the composite deck serves only as formwork.

The masts

For the top 100 m (328 ft) of the tower, three independent masts are formed where the wings extend above the observation roof level to house mechanical services. They are structured by the extension of the outermost fan columns in each wing and portions of the core wall around the spine. A diagonalized steel and glass roof structure again creates a column-free space, this time covering the observation lobby, and serves as the highest diaphragm linking the three masts. Typical composite framing is used at enclosed plant levels and heavy-duty open metal grating is used at cooling tower levels.

4. UNIQUE STRUCTURAL DESIGN CHALLENGES

Fazlur Khan was quoted as saying ‘Mathematics is supple and the friend of intuition.’ By that he meant that if a structure is well conceived and if it ‘looks right’, then the mathematics of the structural design effort will usually justify the designer’s intuition. Intuition led the design team to the solution for the Russia Tower, but next came a lot of mathematics. There were a number of key structural challenges to be addressed for this unique form—all related to stability:

(1) global stability, particularly in torsion considering the ‘Y’ shape of the tower;
(2) stability of the highly stressed concrete fan columns resulting from uncertainty on how to interpret their unbraced lengths according to conventional design methods; and
(3) design of the floor diaphragms to ensure adequate strength and stiffness to maintain the plan configuration of the tower, and to laterally brace the columns.

Design codes such as AISC and ACI offer guidance on these issues, but it was felt that none addresses the complexities inherent in the scheme for the Russia Tower: the cruciform plan, the bracing with nodes at variable intervals, and so forth.

The 2005 AISC specification has greatly expanded and refined its treatment of stability in steel structures. First, Chapter C, ‘Frames and Other Structures’, provided clearer requirements for general stability requirements, giving the engineer more guidance and at the same time allowing more flexibility in addressing this issue. Second, the new Direct Analysis Method in Appendix 7 presents a clear and encompassing methodology that will likely soon become the standard for design. Following this methodology, all columns can simply be designed using an unbraced length equal to one story height. Third, and perhaps most notably, ‘notional’ loads are defined to address geometric imperfection; previously, this was handled less transparently. (A forthcoming AISC design guide for stability will offer further help in understanding and applying these new recommendations.)

ACI’s criteria for concrete structures have not changed in some time and they similarly recommend general requirements for safety and stability. Some more specific guidance is offered in section ACI 318-05, Sections 10·10 and 10·11, for considering secondary material affects such as creep and shrinkage by modifying member and material properties.

Owing to the limitations of existing codes, Halvorson retained Dr Jerry Hajjar of the University of Illinois to consult on developing a more sophisticated analysis approach appropriate to the unique characteristics of the Russia Tower. The resulting approach incorporated three separate, but closely related, analyses and design checks to address each of the three issues.

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4.1 Global stability analysis

Stability analysis concepts

As described earlier, the Russia Tower's form provides a very stable base for resisting lateral loads. However, it was necessary to evaluate torsional buckling for this 'Y'-shaped plan, just like, for example, a cruciform-shaped column should be checked for torsional buckling. One complexity in the analysis was the flexibility of the floor diaphragms: with the many openings in the center of the floor plate, these needed to be modeled accurately for a valid buckling analysis. For assessing global stability, the buckling analysis needed to accurately reflect the flexibilities in the system and consider all secondary effects. A 'rigorous' second-order analysis was developed to include:

(1) large *P*-Delta (global) and small *p*-delta (local member) effects;
(2) flexure, axial, and shear deformation;
(3) geometric imperfections, such as out-of-plumb (yet still within tolerance) construction;
(4) material nonlinearity, resulting in reduced stiffness of members due to residual stresses in steel and creep and shrinkage in concrete; and
(5) flexible floor diaphragms.

Items 1 and 2 can be handled by most of the high-end analysis programs available today. The design team concluded that for the design of the Russia Tower the iterative approximated p-Delta method in ETABs was sufficient for capturing deformations and large P-Delta forces. Small p-delta forces could also be included, although members may require division into two pieces. For reasonably small deformations, the iterative method is sufficiently accurate and this method also conveniently allows load cases to be added, making it excellent for design. However, for the buckling analysis and investigation, the more rigorous p-Delta plus large displacement method offered in SAP2000 was adopted. This more truly nonlinear (non additive) method was used in conjunction with a linear buckling analysis as later described.

Horizontal notional loads can be applied to the structure to address item 3. In line with new AISC recommendations, these loads can be oriented to most destabilize the structure. For the Russia Tower, notional loads causing the tower to twist were most critical; however, those inducing sidesway were also separately considered.

Item 4 can most easily be considered by applying modifiers to members and materials; both ACI and AISC offer some guidance for this, although there are still some areas of ambiguity. Elements can be assessed for extent of cracking and its affect on behavior to arrive at approximate factors as well. This is an area where 'engineering judgment' comes into play.

Finally, item 5 can be captured by accurately modeling floors, and not assuming typical 'rigid diaphragms'.

Stability analysis methodology

In carrying out the study to assess the tower stability, a nonlinear buckling analysis was carried out under factored loads, sequentially increasing these loads until a lowest eigenvalue of 1·0 was obtained—representing incipient buckling. The full tower model analyzed in SAP2000 accurately represented all primary framing and wall elements for the lateral system, including perimeter beams. Floors were modeled with slab elements, with openings conservatively represented such that flexibility of the floor diaphragms was modeled. Analysis was done in an iterative process. A second-order analysis was first run under factored loads and its results were used as input for the linear buckling analysis. This two-step process was carried out multiple times, each time increasing the factored loads until an eigenvalue of 1·0 was reached. The ratio of the final loads to the required factored loads represents the true non-linear buckling eigenvalue ratio.

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Gravity loads, notional loads, and wind loads were all considered in factored load combinations per ASCE 7-02. Three directions of notional loads were considered: two in lateral directions and the third in a torsional manner. The value for the notional load suggested in AISC, 0·2% of the gravity loads applied to each column at every floor acting horizontally, represents a condition wherein all columns and walls are constructed out of plumb (or in this case off of the specified slope!) by height/500 to match typical AISC tolerance limits. This assumption is conservative for tall structures since there is an upper limit on the horizontal offset of columns per AISC. Further, the statistical likelihood that all columns in the tower would be out of alignment in the same direction is small.

Buckling cases with wind were conservatively considered, but wind loads were not taken simultaneously with notional loads. Wind loads are not usually considered in buckling analysis because this load is transient and would not induce material nonlinear effects, such as creep. However, since the gravity and wind loads are both carried as axial loads in the same members, this was conservatively done.

To capture material nonlinearity, modifiers were applied to material and member properties. For steel, the modulus of elasticity was taken as $0{\cdot}80 \times E_{\text{steel}}$, in line with AISC Appendix 7, to capture residual stress effects. For concrete, ACI recommends a $1/(1 + b_d)$ modifier on $E^*\ I_g$ for buckling checks (where I_g is member's gross moment of inertia); Halvorson elected to apply this ratio to E_{concrete}, using $0{\cdot}625 \times E_{\text{concrete}}$. The modifier for the columns' I was established to provide $EI = 0{\cdot}20\ EI_g$, in line with ACI recommendations for buckling. Other concrete members were modified based on the effects of cracking, such as a 0·50 modifier on I_g for link beams, which were provided with sufficient reinforcing.

The floor diaphragm stiffness needed to represent contributions from steel diaphragm bracing, floor framing, and concrete slabs. For the slab itself, only the concrete above the flutes was considered for in-plane stiffness, with reductions applied for cracking. Along the axis of each wing, the in-plane stiffness was reduced to 0·10 of actual to ensure the gravity arching action was carried in the perimeter beams and not the diaphragm. The in-plane stiffness was also reduced to 0·50 perpendicular to the wing and 0·20 in both directions within the center where the wings meet. Where steel framing and diaphragm bracing provided additional diaphragm stiffness, this was also included in the full model buckling analysis.

Based on this analysis, the lowest, and therefore governing, buckling mode was found for the case considering gravity loads plus 100% of torsional wind loads acting together with a portion of lateral wind loads. An eigenvalue of 1·0 was achieved (i.e., nonlinear buckling occurred) when this load combination was increased to approximately 1·7 times the required factored design loads, or roughly 2·1 times actual loads (see Figure 16). Similarly, considering gravity loads acting alone, an eigenvalue of 1·0 would not be reached until the factored load combination was increased by a factor 2·4, or roughly a factor of 3 times actual loads. For comparison, a conventional linear buckling analysis was carried out, still considering all conservative member and material modifiers noted above, and achieved an eigenvalue of 4·1 under factored loads considering wind (almost 5 times service loads) and 4·6 under factored gravity loads alone (nearly 6 times service loads). Therefore, it was concluded that the tower was stable in a global sense.

4.2 Column stability and design

The sloping fan columns are the key to Russia Tower. Their visual expression is a (if not 'the') defining architectural feature. They are the primary lateral bracing and, along with the core walls, gravity support for the tower. Every effort was made to minimize their size while ensuring the strength and stability of these highly loaded members. Since the fan columns were apparently braced at varied modules (core walls continuous at all levels, two-story bracing at the spine between the cores, four-

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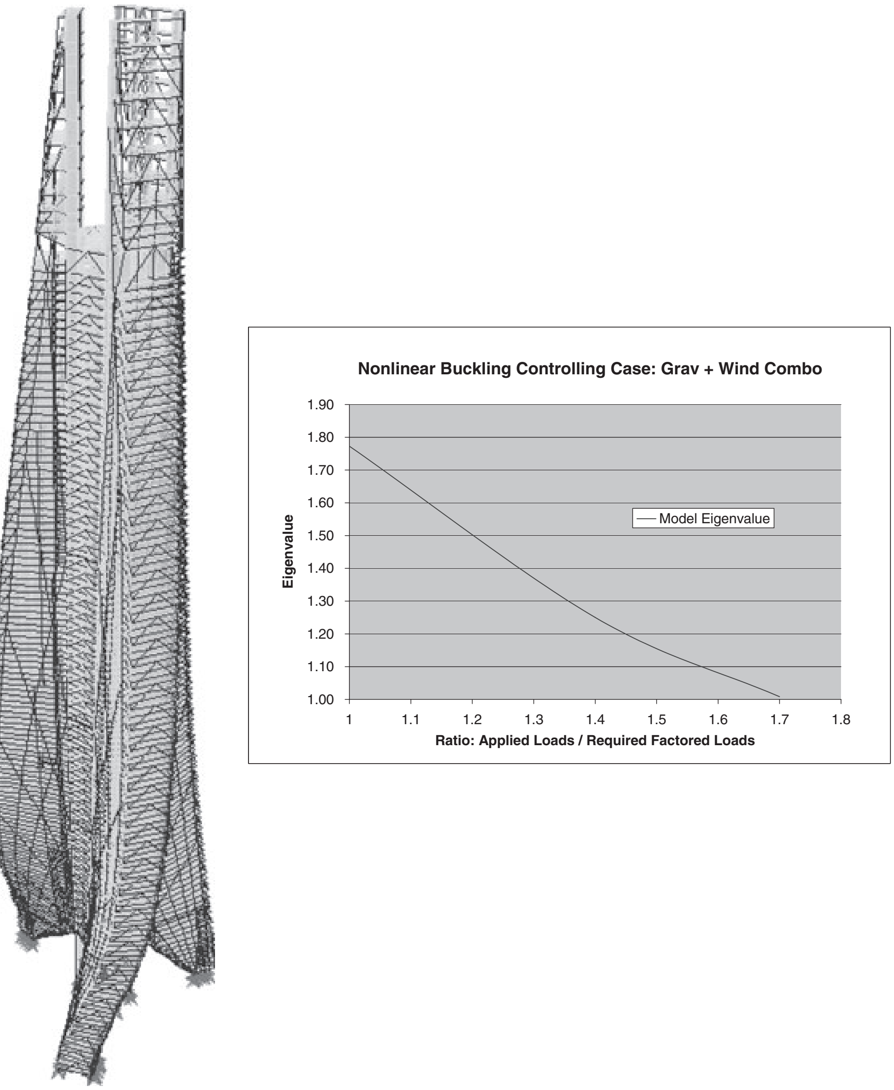

Figure 16. Torsional buckling of tower at ratio of 1·7× factored loads (including wind) or 2·1× service loads

story bracing at the wing tips, and sloping columns triangulating over varied story heights, see Figure 17), it was challenging to determine the unbraced column lengths for use in a conventional design approach.

AISC's direct analysis method for steel structures offered a method for handling this issue, and served as a model for the method developed for the Russia Tower's composite structure. By carrying out an analysis that considers all secondary effects, the direct analysis method allows all columns be designed with $k = 1$, meaning considering a one-story column height, subject to the guidelines given in AISC's Appendix 7. In the final design of the tower, the strength analysis and design was actually carried out in two versions: a direct analysis method and a conventional method. The final design of the fan columns was taken as the more conservative of the two.

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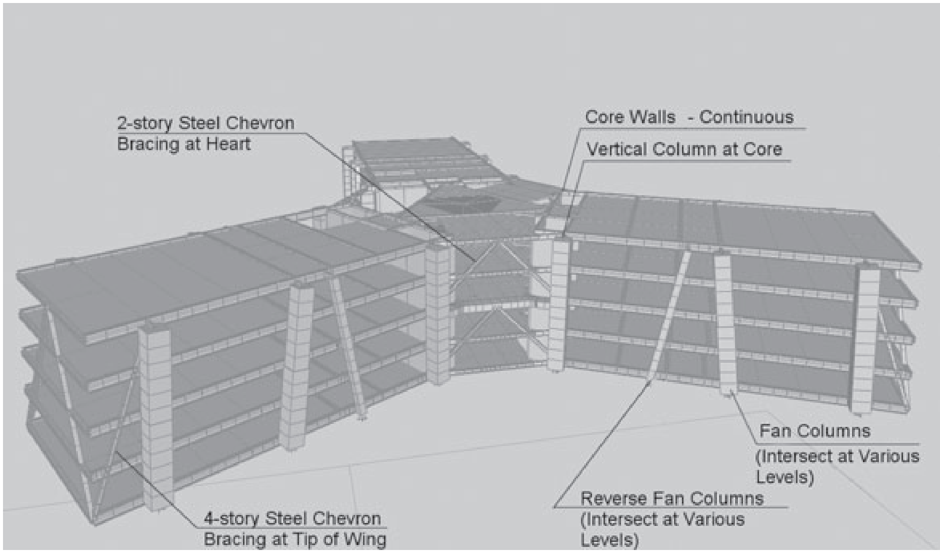

Figure 17. Four-story portion of tower showing varied modules of bracing

Direct analysis method

This method was carried out using factored loads in ETABS, using its iterative *P*-Delta method. Modeling assumptions for loads and materials were identical to those for the global buckling analysis described above, with the following exceptions:

- Notional loads of 0·2% gravity loads were used but, unlike the buckling analysis, were included in the combination with wind loads.
- Concrete fan and reverse fan column moments of inertia $I_{effective} = 0{\cdot}70 I_{gross}$. (this was an increase from buckling checks, and in line with ACI standards).

Using this method, all columns connected at a floor could be designed for a single-story height (k = 1). However, given the presence of openings and potential for future tenant openings already established by the design team, the columns were conservatively considered with $k = 2$ for all cases. As a final conservatism, additional moments were included in the column design for *p*-delta effects in lieu of dividing all columns into two finite elements in the analysis.

Conventional method

For comparison, a 'conventional' design was also carried out in accordance with ACI standards. This design and analysis was still done in ETABS, capturing secondary effects due to *p*-Delta (global) and deformations. However, notional loads were no longer included. Member modifiers remained similar to the direct analysis method as these were in line with typical ACI standards, but E was no longer reduced. Additional moments were again included in column design for local member *p*-delta effects.

Independent bucking studies were carried out on four-story modules of the building (established by the height of the four-story tip-bracing module) to establish reasonable estimates for unbraced column lengths. The resulting minimum k values became $k = 4$ for fan columns perpendicular to the wing and $k = 2$ for all other members in all other directions.

DOI: 10.1002/tal

This conventional model was analyzed for gravity and wind loads, in versions with and without springs at the base to bracket potential differential settlement design effects. A version was analyzed in ETABS to consider the potentially different distribution of gravity loads due to construction sequence, and a second sequential model was analyzed in SAP2000 considering nonlinear creep and shrinkage effects on concrete, where E varies with time.

The final column designs for the Russia Tower represent the most conservative design for each member from all model versions given.

4.3 Floor diaphragm design

In the previous sections on global stability and column design, the need to provide adequate stiffness and strength in the floor diaphragms to maintain the tower plan form and to laterally brace the columns was raised. The earlier analyses evaluated the tower structure considering realistic stiffness of the floor diaphragms, with successful results. The remaining issue was to provide adequate strength in the floor diaphragms. This final section explains how these slabs were analyzed and designed for the required in-plane diaphragm forces (see figure 18), both for stabilizing the columns to one another and transferring lateral forces to the core walls and fan columns. Around major openings, steel floor framing and added bracing members work together as horizontal trusses transferring in-plane diaphragm forces.

Three primary horizontal forces are acting within the typical floor diaphragm:

(1) horizontal component to stabilize the sloping columns, carried through axial force in spandrel beams;
(2) horizontal wind loads, transferring forces into the primary lateral framing members: walls and sloped columns;

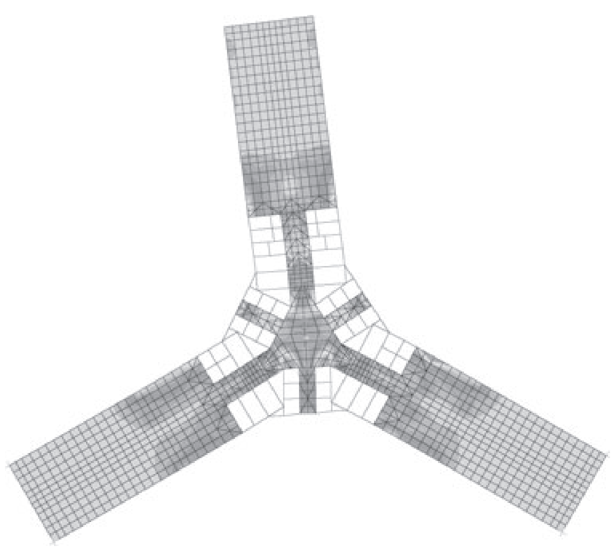

Figure 18. Diaphragm stresses due to horizontal loads

DOI: 10.1002/tal

(3) horizontal forces to brace columns to one another; the most conservative case would be all columns being out of alignment in parallel and all kinking at the same floor level.

The first type of diaphragm force created by stabilizing the sloped columns is resisted by a load path provided in the perimeter spandrel beams (for this analysis, the concrete slab properties were significantly reduced so as to eliminate load shedding from the spandrel beams into the concrete slab). Of the other two types of diaphragm forces, the forces created by wind loads are of significantly lower magnitudes than the forces required to brace the columns. Therefore, the third load case was considered for the diaphragm design. For Russia Tower, a tolerance limit of height/800 was set for the columns. If a column slopes at height/800 above a level, 'kinks' at that level, and then slopes at height/800 below in the opposite direction, the required horizontal force acting at that level to brace the column is equivalent to 0·25% of the axial load in the column (e.g., 1/800 + 1/800 = 0·0025).

This bracing force of 0·0025 times the total axial load in the columns of one wing was applied horizontally to one floor level of the tower at a time in various orientations—pushing a cantilevered wing sideways, pulling a cantilevered wing away from the tower, etc. The diaphragm forces in the steel framing and in the concrete slabs were bracketed for two extremes: one with the slab stiffness 'turned off' and only the steel diaphragm bracing considered, and another with the steel diaphragm bracing omitted and only the concrete floor slab considered. The analysis revealed that only a portion of the bracing forces had to be resisted by the floor diaphragm at the level where the loads were applied—a significant portion of the bracing forces transferred vertically through the stiff columns and walls to levels above and below.

Reasonable amounts of reinforcing were sufficient in the concrete slabs to handle the in-plane diaphragm forces. Steel framing and diaphragm bracing and their connections were designed to resist the axial loads due to these in-plane forces.

5. CONCLUSION

The architectural form and the structural engineering concepts of the Russia Tower developed from a unique set of circumstances: its program, its site, its developer and the intuition of its design team working in collaboration. The structural concept responds very directly to the imperatives of a tall building and to the loads acting upon it. The structural concept is also intimately linked to the architectural form of the tower—it is hard now to imagine one without the other (see figure 19). While the structural concept is elegant in its simplicity, the mathematics of its behavior and design were necessarily complex, and some of the unique structural engineering approaches adopted for its design have been presented.

RUSSIA TOWER DESIGN TEAM

Client:	STT Group
Architect:	Foster + Partners
Structural engineers:	Halvorson and Partners
	Waterman International
Services engineer:	Waterman International
Wind engineering consultant:	Dr. Nicholas Isyumov
Wind tunnel testing and analysis:	RWDI Anemos
Vertical transportation and façade access:	Lerch Bates Ltd

DOI: 10.1002/tal

Figure 19. Russia Tower rendering

ACKNOWLEDGEMENTS

The authors wish to express their gratitude to the Developer, STT Group, and the design team Foster + Partners, with local consulting from Waterman International, for the collaborative approach to the design that led to this exciting building concept.

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DOI: 10.1002/tal