

# Direct Analysis Method Case Study - Addressing Stability for the Russia Tower

Authors:

Robert A. Halvorson, SE, PE, FStructE, FASCE, Principal  
Halvorson and Partners, 600 West Chicago Avenue, Suite 650, Chicago, Illinois 60654  
312-274-2402 [rhalvorson@halvorsonandpartners.com](mailto:rhalvorson@halvorsonandpartners.com)

Carrie Warner, SE, PE, LEED AP, Associate Principal  
Halvorson and Partners, 600 West Chicago Avenue, Suite 650, Chicago, Illinois 60654  
312-274-2413 [cwarner@halvorsonandpartners.com](mailto:cwarner@halvorsonandpartners.com)

Alex Lang, Staff Engineer  
Halvorson and Partners, 600 West Chicago Avenue, Suite 650, Chicago, Illinois 60654  
312-274-2400 [alang@halvorsonandpartners.com](mailto:alang@halvorsonandpartners.com)

## Introduction

The 600 m (1968 ft) tall Russia Tower has an extremely efficient and intuitively clear structural concept. But its unique form raised a number of key structural challenges to be addressed – all related to stability. The following paper outlines Halvorson and Partners’ approach to these: applying stability concepts from 2005 AISC Specification for Structural Steel Buildings (AISC, 2005), including the newly introduced Direct Analysis Method (DM), as well as material secondary effects for concrete from ACI to assess, analyze, and design for stability in this complex, composite structure – the Russia Tower.

## Tower Structure Overview

The Russia Tower will be Europe’s tallest building and one of the most distinctive high rises in the world. Its Y-shaped plan extrudes to 600 m, tapering in elevation, to create a visually slender form of three 23 m (75 ft) wide wings extending from a central spine. The expressed “braced spine” structure developed by Halvorson and Partners, in collaboration with architect Foster + Partners, is an extremely efficient concept for super-tall structures. A complete description of the tower’s structural concept, components and collaborative design process can be found ASCE 2009 paper by Robert Halvorson, entitled “Russia Tower: Design Challenges”.

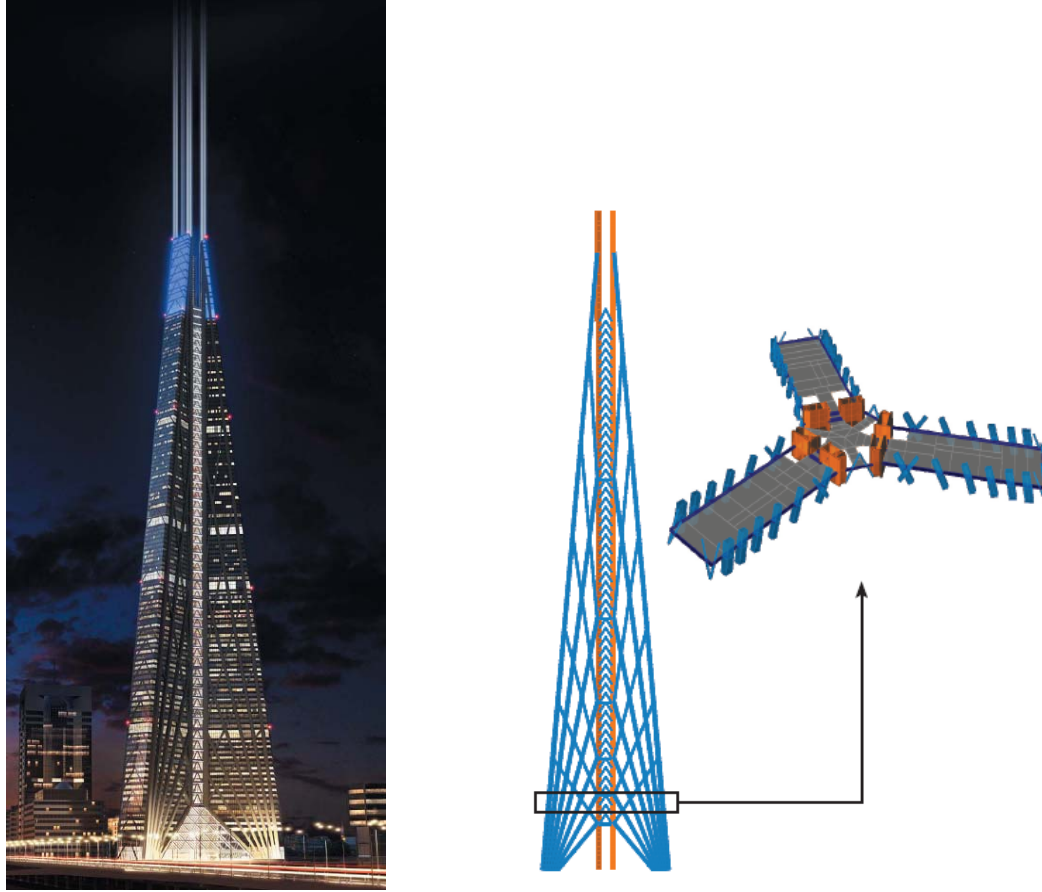


FIGURE 1. Russia Tower Rendering (Foster + Partners, 2007) Structural Diagram ( Halvorson and Partners, 2008)

The central spine of the structure consists of reinforced concrete walls hidden in core elements – which are linked together across the open space in the central spine by two-story steel chevron bracing. 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 (see figure 1). These reinforced concrete columns, which vary in cross-sectional area from 1.0 to 6.25 m<sup>2</sup> (10 to 67 ft<sup>2</sup>), brace the central spine for lateral loads and carry gravity loads. Where the fan columns intersect the spine, substantial horizontal reinforced concrete beams are provided to resist the thrusts of the fan columns. 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. Steel floor framing interfaces with the steel bracing directly, and connects light steel erection columns within the concrete walls and sloped columns.

The structural system is extremely efficient. Lateral loads are resisted as axial forces in the sloped columns and the ‘spine’ must, in effect, only span like a continuous beam between the points where the fan columns brace the spine. Gravity loads are also resisted as axial forces in the sloped columns and walls. Given this and its low aspect ratio, gravity stresses exceed any wind tensions in all but a few stories of select columns, with no concerns for uplift at the base. And lateral displacements and accelerations are well below standard limits.

## Stability Challenges

With the unique configuration of this structural system (cruciform plan, the bracing with nodes at variable intervals, and so forth), there were a number of key structural challenges to be addressed– 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 insure adequate strength and stiffness to maintain the plan configuration of the tower, and, to laterally brace the columns.

Given its composite configuration, Halvorson called upon guidance from both AISC and ACI. The 2005 AISC specification has greatly expanded and refined its treatment of stability in steel structures. First, Chapter C “Frames and Other Structures” provides clearer requirements for general stability, 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. This method allows all columns to 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.

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.

Halvorson also retained Dr. Jerry Hajjar of the University of Illinois to assist in developing an 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.

## Global Stability Analysis

Stability Analysis Concepts:

Russia Tower’s form provides a very stable base for resisting lateral loads, but it was necessary to evaluate global buckling, especially for torsion, of this “Y” shaped form. This 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 non-linearity, 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. For the buckling analysis and investigation, the p-Delta plus large displacement method offered in SAP2000 was adopted. This non-linear (non additive) method was used in conjunction with a linear buckling analysis as later described. Small p-delta could be accounted by dividing members into two pieces; but this was not necessary during assessment of global buckling.

Horizontal notional loads can be applied to the structure to address Item 3. In line with new AISC recommendations, these loads can be orientated to most de-stabilize 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 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. This is an area where ‘engineering judgment’ comes into play. 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 non-linear 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 Eigen value ratio.

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.

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 non-linear 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 non-linearity, modifiers were applied to material and member properties. For steel, the Modulus of Elasticity was taken as  $0.80 \times E_{\text{steel}}$ , in line with AISC Appendix 7. For concrete buckling assessment, ACI recommends a  $1 / (1 + b_d)$  modifier, which Halvorson elected to apply this ratio to  $E_{\text{concrete}}$  ( $0.625 \times E_{\text{concrete}}$ ). The columns' I modifier was established to insure  $EI = 0.20 E I_g$ , further in line with ACI recommendations for buckling. Other concrete members 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 (In-plane stiffness modifier = 0.10 along wing, to insure the gravity arching action was carried in the perimeter beams and not the diaphragm; =0.50 perpendicular to the wing; and = 0.20 in both directions within the centre where the wings meet). Where steel framing and diaphragm bracing provided additional diaphragm stiffness, this was also included in the full model buckling analysis.

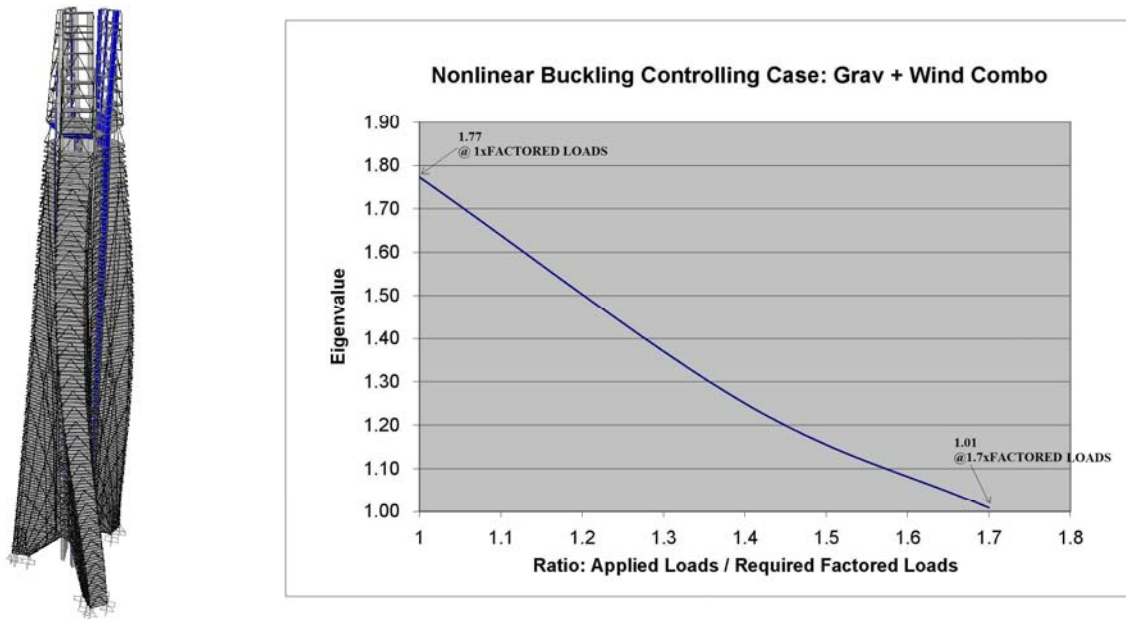


FIGURE 2. Torsional buckling of tower at ratio of 1.7 x factored loads (including wind), or 2.1 x service loads.

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., non-linear 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 2). 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. Therefore, it was concluded that the tower was stable in a global sense.

## 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 insuring 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-story bracing at the wing tips, and sloping columns triangulating over varied story heights, see Figure 3), it was challenging to determine the unbraced column lengths for use in a conventional design approach.

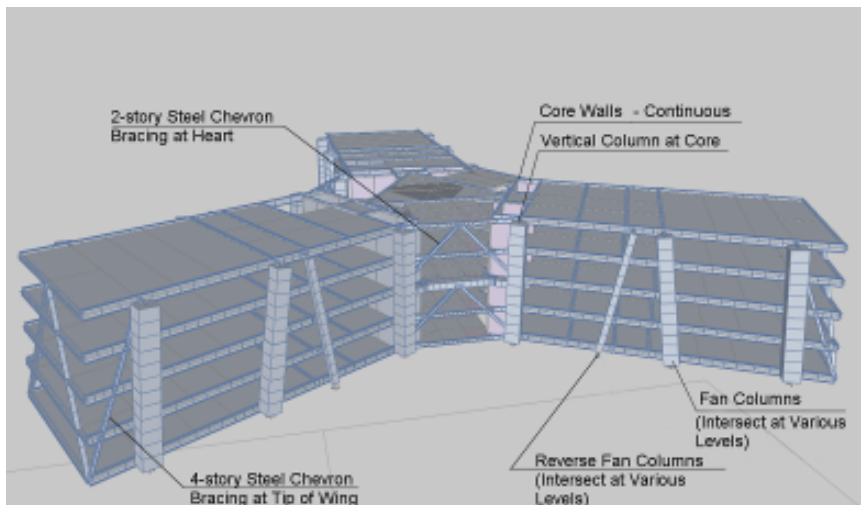


FIGURE 3. Four-story portion of tower showing varied modules of bracing

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 designs of the columns were taken as the more conservative of the two.

### Direct Analysis Method:

The concepts and modeling assumptions applied for the Direct Analysis Method are nearly identical to those outlined previously for the Global Buckling Analysis, with the following exceptions outlined by previous numbered items:

- 1) For capturing Large P-Delta (global) effects, the team concluded that for the design of the Russia Tower, the iterative approximated p-Delta method in ETABs was sufficient for capturing deformations and secondary forces; for reasonably small

deformation this approximated iterative method was warranted sufficiently accurate and this method also conveniently allows load cases to be added, making it excellent for design. Also, a comparison of 2<sup>nd</sup> and 1<sup>st</sup> order analyses found  $\Delta_{2nd}/\Delta_{1st} < 1.5$  (usually  $< 1.1$ ). Small p-delta forces could also be included, although members may require division into two pieces or including a moment multiplier for these.

- 2) Flexure, Axial and Shear Deformation could be accounted for by ETABS analysis.
- 3) Notional loads of 0.3% gravity loads were instead used, since columns were highly stressed and in lieu of calculating  $\tau_b$ . But, since  $\Delta_{2nd}/\Delta_{1st} < 1.5$ , were still not required to be included in the factored combination with wind loads for design.
- 4) Material and members modifiers were identical to those previously outlined for Global Stability. Except concrete Fan and Reverse Fan Column Moment of Inertias were assumed  $I_{effective} = 0.70 I_{gross}$ . This was an increase from buckling checks, and in line with ACI standards.
- 5) Flexible floor diaphragm assumptions aligned with those earlier described.

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 small 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 buckling 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.

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 non-linear 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.

## 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, 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.

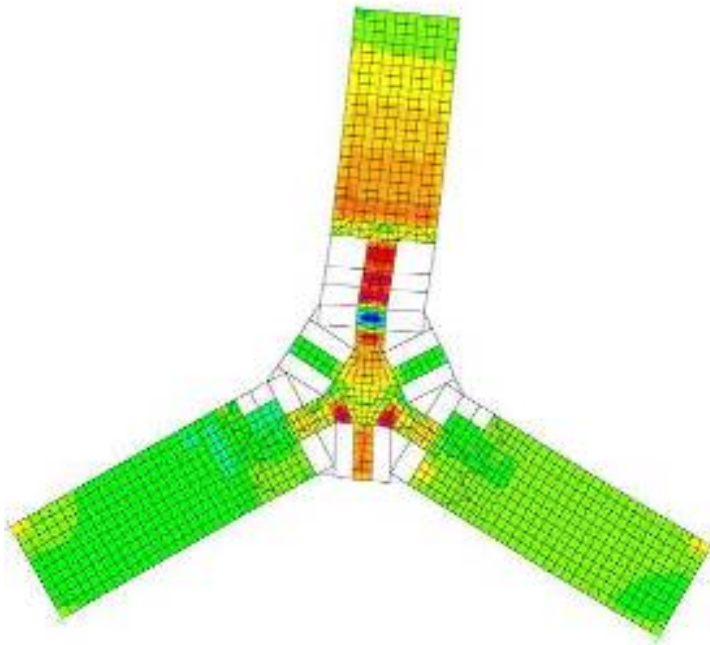


FIGURE 4. Diaphragm stresses due to horizontal loads

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
- 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.

## **Summary**

Stability was the key design consideration for the unique configuration of the Russia Tower. The recently updated stability sections of the AISC specification, including the introduction of the Direct Analysis Method, were a timely development for our effort – as these offer engineers a practical, fundamental-based methodology for addressing such issues. By integrating ACI requirements as explained, Halvorson and Partners arrived a reasonably conservative approach to feel confident in our assessment, analysis and design for the stability of Russia Tower.

## References

[American Institute of Steel Construction, Inc. (AISC) 2005], "Load and Resistance Factor Design Specification for Structural Steel Buildings", Chicago, IL.

[American Institute of Steel Construction, Inc. (AISC) 2005] "Code of Standard Practice for Steel Buildings and Bridges", Chicago, IL.

ACI 318-05 (2005), "Building Code Requirements for Structural Concrete and Commentary", American Concrete Institute, Detroit, MI.

[Abdelrazaq, A., Baker, W., Hajjar, J. F., and Sinn, W. 1993] "Column Buckling Considerations in High-Rise Buildings with Mega-Bracing," Is Your Structure Suitably Braced?, Proceedings of the Structural Stability Research Council Annual Technical Session and Meeting, Milwaukee, Wisconsin, April 6-7, 1993, SSRC, Bethlehem, Pennsylvania, pp. 155-169.