Standing Tall In Madrid

A visually light Vierendeel frame wraps Spain's tallest building.

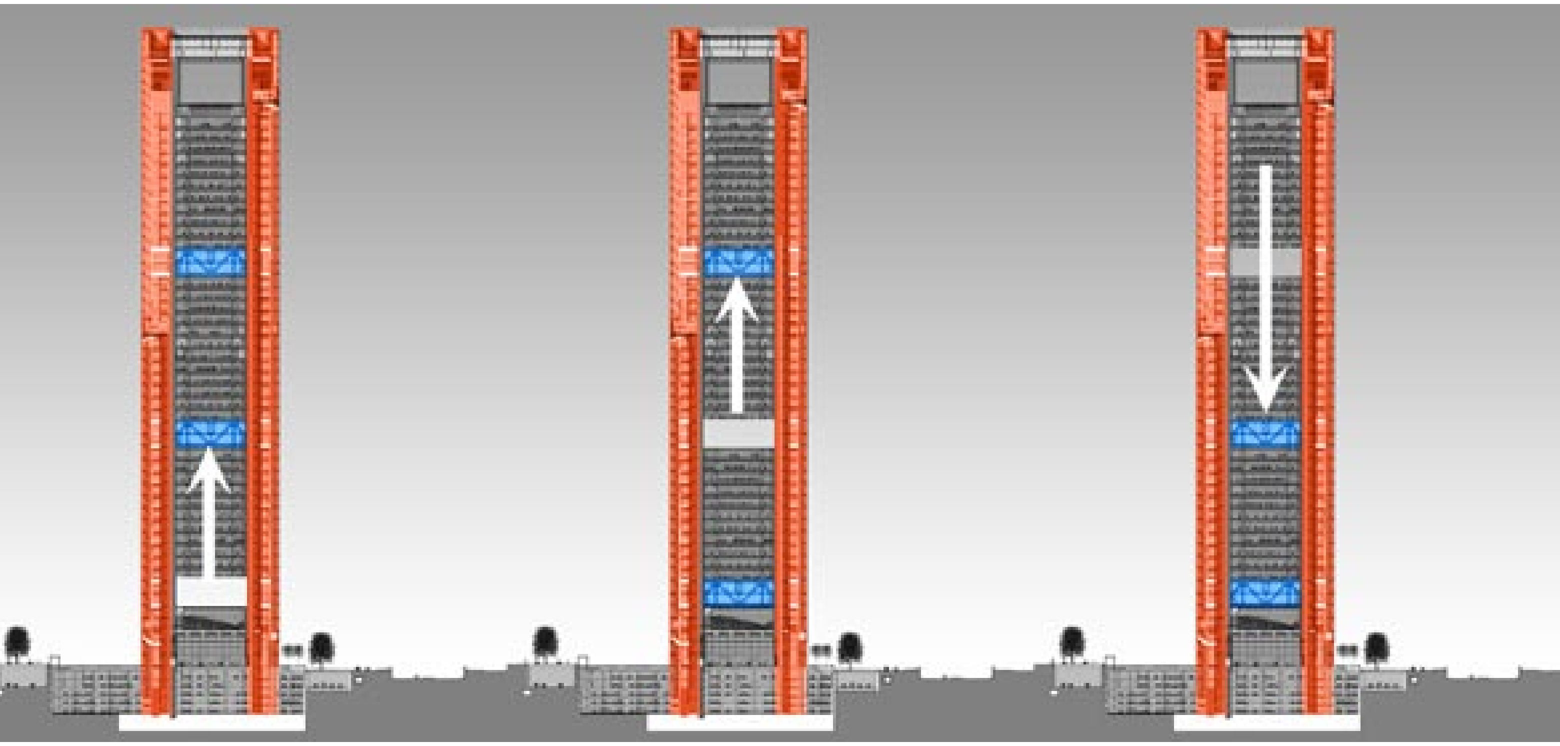
BY GREG LAKOTA, P.E., S.E.

MAJOR CHANGES CAN, AND USUALLY DO, occur during a building's construction process, but project ownership isn't typically one of them. The new corporate headquarters for Caja Madrid, one of Spain's largest banks, was originally designed to be the corporate headquarters for Repsol YPF, Spain's largest oil company. However, during construction the increase in value of the building was so great that Repsol YPF decided to build their headquarters elsewhere and sell their new building to Caja Madrid.

Located in Madrid on the former training grounds of the Real Madrid football (soccer) team, the tower is part of a new business park, Cuatro Torres, which includes three other new office buildings. At 820 ft tall and nearly completed, Caja Madrid Tower is now the tallest building in Spain, and the new owner will be moving in within the year.

Structural engineer Halvorson and Partners of Chicago collaborated with architect Foster and Partners of London to design an "iconic building," as directed by Repsol YPF, to consolidate the oil company's many smaller offices into one central location. The tower consists of five below-grade parking levels and 34 office floors that are divided into three distinct office blocks of 11, 12, and 11 floors. Below each office block is a two-story space dedicated to mechanical equipment that services the floors above. Within these spaces are two pairs of perpendicular two-story steel trusses, one pair spanning between the two end cores and supporting a transverse pair that align with the steel columns above. This truss system transfers all gravity loads to the reinforced concrete cores at the building ends, which are the only vertical load-carrying elements that extend to the foundation. The result: sufficient gravity load on the cores to counter tensions due to wind overturning as well as a dramatic, column-free lobby.

The typical office floor plate, about 14,530 sq. ft, is framed in structural steel and is unique in that the four corners of the floor cantilever roughly 27½ ft from the core and 23 ft from the nearest exterior column through the use of an aesthetically light



A progressive collapse resistance diagram of the building. Halvorson and Partners steel Vierendeel frame that wraps the perimeter of the building. The building uses approximately 9,000 tons of structural steel in all.

Floor Framing System

All the steel floor framing in the tower is S355 K2G3/G4 steel (approximately equivalent to ASTM A992). The typical office floor slabs have 3-in. deck plus 3 in. of lightweight concrete. The floor slabs at levels 1, 12 and 24, which correspond to the top chords of the two-story truss system, and the mechanical level slabs, which correspond to the bottom chords of the truss system, are 3-in. deck plus 6 in. of normal-weight concrete; the thicker slab is primarily required to minimize sound transmission from the mechanical equipment. The floor framing at each level is supported by the two reinforced concrete cores and by four interior and four exterior HD400 (W14) steel columns.

Long-Span Transfer Trusses

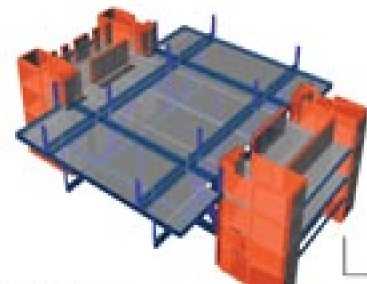
The north-south lateral loads are resisted by pure cantilever action of the two cores, and with gravity load for the entire building supported only by the cores, there is no uplift or tension in the core walls, even with an aspect ratio of 11 to 1. For east-west lateral loads the cores are too narrow to provide adequate strength and stiffness as pure cantilevers, and therefore are linked together at the three truss levels such that the core and truss system behaves like a large moment frame to resist lateral forces.

The three truss levels not only serve as a primary component of the lateral system, they also transfer the eight gravity load columns (four interior and four exterior) from each office block to the two cores. At each of the three truss levels the system of trusses consists of the following: two “primary” trusses that span 105 ft between the cores and two “secondary“ trusses that cantilever 33 ft to the north and south past the primary trusses and transfer the four exterior columns back to the primary trusses.

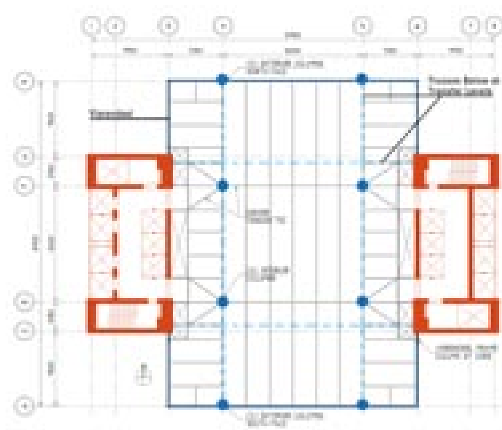
Ideally, the primary trusses would have been designed as simple span between the cores. However, since the primary trusses also interact with the cores to resist lateral loads, the top chord of the truss had to be connected to the core. With the truss top chord connected to the core walls, negative bending moments due to gravity loads would develop in the truss, resulting in large top chord tensions at the connection to the core. In order to minimize the gravity load negative moments in the truss, the top chord connection of the primary trusses to the core has been detailed to allow horizontal movement, and this connection was not pretensioned until the full structural dead load of the office block it supported had been applied to the truss. So, in the permanent condition, top chord tensions in the truss only result from floor live loads and east-west lateral loads.

Since the top chord connections of the primary trusses were not pretensioned, they would act as simple-span trusses during erection of each office block, developing large tension forces in the bottom chords, which would be resolved as a horizontal thrust against

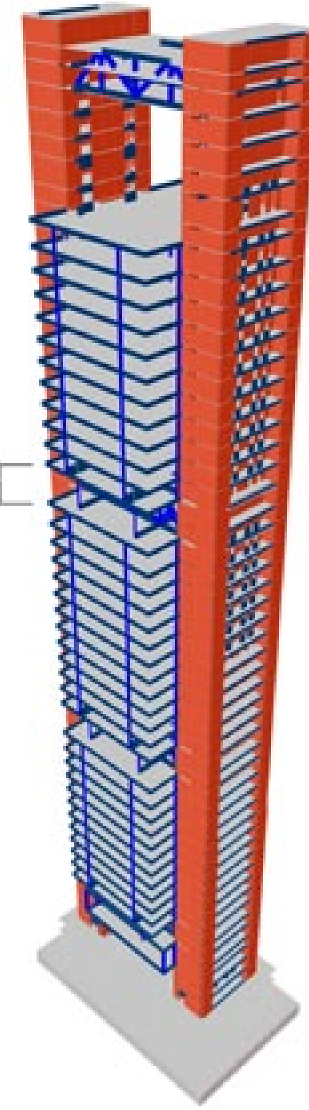
Typical and transfer levels of Caja Madrid.



Transfer Level



Typical Level



Images: Halvorson and Partners

The typical floor area in the building is approximately 14,530 sq. ft.

Foster + Partners

Caja Madrid Tower, at 820 ft, is the tallest building in Spain.

the cores. As the cores try to resist the thrust and move outward, away from the floor plate, tension forces are introduced into the floor framing members on multiple floors above and below the truss bottom chord levels. In essence, the floor framing above and below the truss levels is trying to hold the cores together as the thrust from the trusses tries to push the cores apart, which was an undesirable design solution. To eliminate the trusses' thrust against the cores, post-tensioning tendons are provided along the bottom chord of the primary truss and anchored to an embedded column in the cores. In addition to minimizing the axial thrust on the core, the post-tensioning provides a level of redundancy for the connection of the bottom chord truss to the core.

At each level within the core, where the truss top and bottom chords attach, a 6.2-ft-thick slab is provided. The thick slabs, along with a two-way system of multi-strand post-tensioning tendons and reinforcement, provide the required strength and a means of engaging the full cross-section of the core to resist the truss chord forces. The connection of the bottom and top chords of the primary trusses to the core are critical, since these 12 connections transfer all gravity loads and east-west lateral loads to the cores, and ultimately both the top and bottom chords of the truss will transmit tension and compression forces to the core. Once the bolts of the top chord connection to the core are tightened, live loads on each office block will induce negative bending in the truss and thus tensions in the top chord; east-west wind loads will induce load reversals in the truss chords. So, robust positive connections of the truss to core are provided by embedding two built-up steel columns within each core. The column is in turn anchored with post-tensioning rods into the 6.2-ft slab, providing adequate strength to transmit truss forces into the core cross-section.

Given the critical nature of the trusses and their connections to the core, redundancies were built into the entire system. Each set of primary trusses and connections to the core have sufficient strength to resist service loads from two office blocks, should a catastrophic failure of a single truss level occur. For instance, if a failure in the lowest level truss occurred, the first 11-story office block could hang from the truss level above. One might think that designing each truss level with enough capacity to support two office blocks is an inefficient; however, the premium to build in this redundancy was marginal. The truss member sizes were not controlled by strength to resist gravity and lateral loads forces; they were sized to provide adequate stiffness for lateral drift in the east-west direction. With the trusses sized for stiffness, an increase in the material strength from S355 to S460 (50 ksi to 65 ksi) was sufficient to provide the necessary strength to support service loads from two office blocks on one truss level.

Although the truss system was designed with robustness and redundancy for a catastrophic failure, the core walls themselves did not have this additional capacity to resist the negative bending forces that would develop, and the building would still be susceptible to a progressive collapse. The solution to protect the core walls was to design a "structural fuse" for the top chord-to-core wall connection, which essentially proportioned the splice plates such that they would yield prior to any overstressing of the core walls.

Vierendeel Frame

The architectural design intent was to minimize the number of exterior columns on the typical office floors and eliminate corner columns. This was achieved by providing only two columns on the north and south faces of the building; the columns are spaced 59 ft apart, with a 23-ft cantilever to the east and west of each column. To eliminate the columns from the corners, spandrel beams on the east and west side of the building would cantilever from the cores out to the 23 ft cantilevers on the north and south faces of the building.

The two exterior columns on the north and south sides are supported directly on the secondary trusses at the three truss levels. To minimize the depth of the 23-ft cantilevers, the spandrel beams on the east and west are moment connected to the core. A moment connection of the steel spandrel beam to the concrete core wall would have been difficult to erect, so a steel column was placed 6 in. from the core wall to provide moment fixity for the spandrel beam at the core. The column adjacent to the core is connected with a simple shear connection, a simpler detail to construct. The perimeter spandrel beams and exterior columns form the Vierendeel frame, minimizing the depth and weight of the frame and helping to control deflections.

With a steel column located just 6 in. from the core wall, the effects of creep and shrinkage of the concrete core had to be addressed. Since the steel column would not creep or shrink with the concrete core, the core would be transferring axial load to the column over time and overstressing the column and the connection between the column and the core. Since the adjacent steel column is only required to provide bending stiffness for the cantilevered spandrel beam, the axial loads could be released, allowing the core to creep and shrink as it wants to without overstressing the columns. A vertical slip detail was provided at the mid-depth of the column, at approximately the inflection point. The slip detail still allowed for a shear transfer such that the column could provide bending stiffness for the cantilevered spandrel beam of the perimeter Vierendeel. MSC

Greg Lakota, AISC Professional Member, is a principal with Halvorson and Partners and was the lead project engineer for the Caja Madrid Tower.

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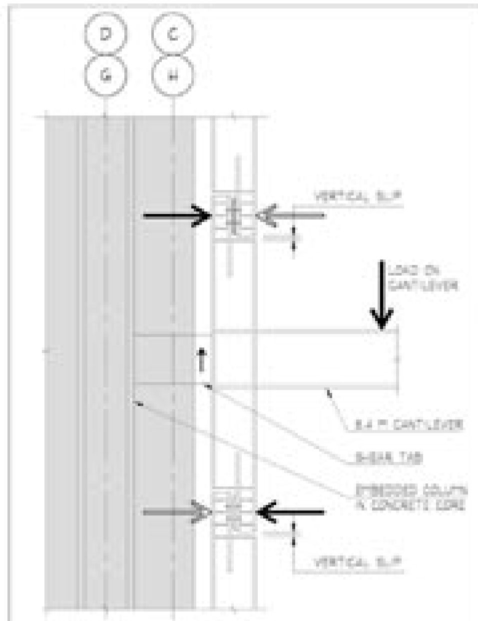

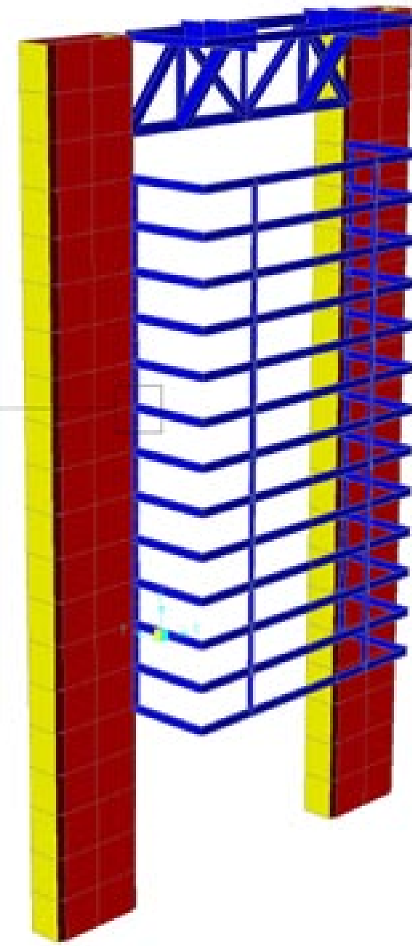

A detail of the connection between the Vierendeel exterior framing and a concrete core.

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