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An Innovative Braced Megaframe for Torre BBVA Bancomer in Mexico City


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This paper discusses the design and delivery of an Eccentrically Braced Megaframe (EBMF) for a 52-story building currently under construction in Mexico City. The EBMF for this project has been constructed external to the building's envelope and at an unprecedented scale. The large spacing of the external composite columns assisted in creating an inherently stiff structure and enabled the number and size of columns on the façade line to be minimized. A performance-based approach has been adopted to justify the design of this unusual building in the highly seismic zone of Mexico City.

Introduction

The focus of the paper is the deployment of advanced analytical techniques in the design of tall buildings. Using nonlinear response history analysis, the design team was able to demonstrate the likely performance of the building during representative earthquakes. Using the same analytical tools, the team undertook detailed analysis at a component level to understand the complex low-cycle fatigue behavior of the tower and satisfy a third-party peer reviewer of the design methodology.

The basic capacity design procedures of existing codes would have made this building uneconomic and even unfeasible to build. Through the use of nonlinear response history analysis, the design team has delivered a highly efficient and flexible building with a robust seismic resisting system.

Overview

When it opens in 2015, Torre BBVA Bancomer will be the Latin American headquarters of the BBVA Bancomer banking group. Designed by an Anglo-Mexican team of architects and engineers, the final level of the 52-story tower was erected in late 2013 (see Figure 11).

Standing in the heart of Mexico City's rapidly developing business district, the tower's location required the design team to contend with notoriously challenging ground conditions. Structural engineers from Arup worked alongside the building's architect LegoRogers, a joint venture between Rogers Stirk Harbour + Partners (London) and Legorreta+Legorreta (Mexico City) to develop a structural system that provides excellent seismic performance and architectural freedom in space planning.

Central to the design strategy is an Eccentrically Braced Megaframe (EBMF), which provides stiffness, strength, and ductility. The EBMF provides the tower's lateral stability, resisting design wind, and moderate earthquakes elastically. Energy from larger earthquakes is dissipated through nonlinear yielding of "seismic links" (see Figure 2). The nonlinear response of the Tower has been designed using performance-based approaches, including global response history analysis and detailed low-cycle fatigue modeling.

Mexico City has a subtropical highland climate; the temperature rarely goes outside the range of 3 °C to 30 °C. This benign climate enables the EBMF to be positioned outside of the building's thermal envelope, maximizing its effectiveness in resisting lateral loads and removing the need for a structural core. This solution helps reduce the seismic weight of the tower and the associated foundation loads. It also provides an interior that is largely free of structure. The absence of a concrete core enabled the architect to terminate the primary elevator core at Level 11. Below this level, the floor plates are open, maximizing the net usable area of the tower.

The design team's integrated approach to architecture and structure has produced a unique system that maximizes the developable area of a prestigious location. Through the application of a clear design strategy and sophisticated analysis, the project provides the client with an iconic, yet efficient, building.

Structural System

The EBMF system provides the complete lateral resistance for the tower; the seismic link elements provide ductility.

There were a number of particular drivers on this project that influenced the structural design:

- The site location is characterized by deep strata of soft soils, where foundation capacity comes at high cost.
- Mexico City is highly seismic. The soft soils on this site give rise to the classic Mexico City seismic hazard, where distant subduction-zone earthquakes are modulated and amplified to create long-duration, long-period ground motions.
- In order to comply with local parking requirements, extensive parking, and circulation was required in the tower footprint, not only in the basement, but also in the lower section of the superstructure. Consequently the office accommodation starts at Level 12.

The high seismicity and poor ground conditions created a clear rationale for a low-weight structural solution. The unusual location of the main elevator lobby at Level 12 meant that the primary vertical transportation only started at this level, such that core areas below this could be reduced. The combination of the low-weight driver and the desired flexibility of the height of the tower challenged the design team to consider solutions beyond a traditional concrete core, which would have been highly restrictive for vehicle circulation in the lower portions of the tower.

In order to achieve large column-free floor plates at a low structural weight, composite steel framing was selected for the floors. The framing layout requires only a single internal column in each of the two 33.5-by-33.5-meter triangular spaces. Three pairs of columns flank the vertical transportation and technical zone in the central diagonal band. This layout also enables provision of large external sky gardens, without the need for transfer structures (see Figure 3).

Figure 1. Torre BBVA Bancomer, Mexico City. © LegoRogers

A clear lateral structural system was developed at the competition stage of the design, and this was maintained through to construction. The system comprises an external megaframe with six perimeter columns, continuous eccentric bracing on the four orthogonal sides of the building, and intermittent eccentric bracing on the two shorter sides of the building. The structural system is described as an EBMF and is the first of its kind to be constructed (see Figure 2).

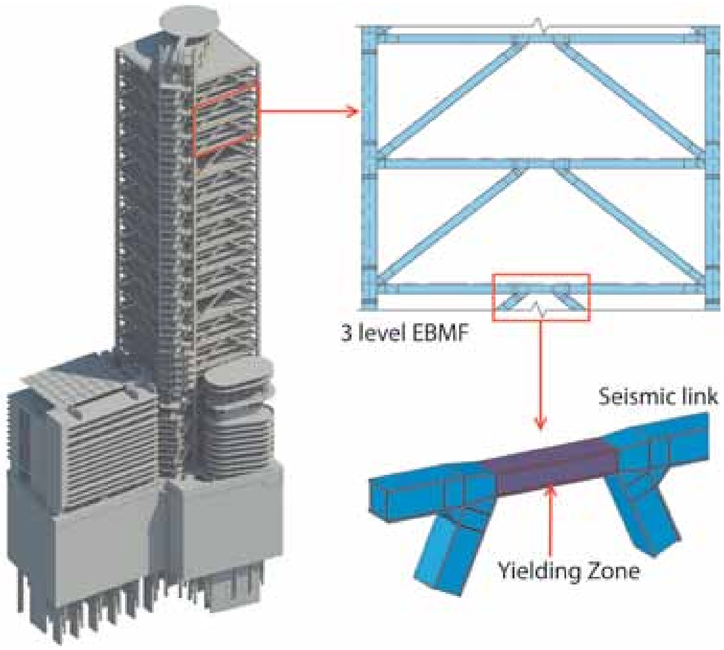

Figure 2. Overview of Eccentrically Braced Megaframe (EBMF).

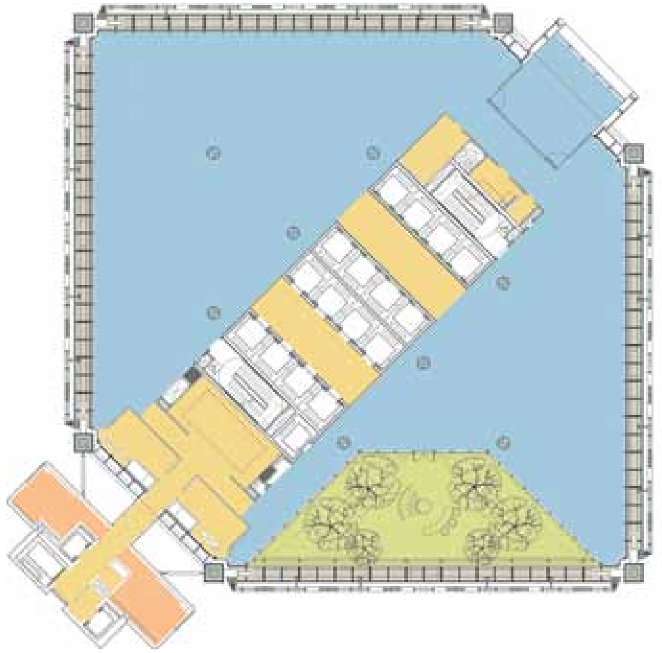

Figure 3. Typical sky lobby floor plan. © LegoRogers

> “The Eccentrically Braced Megaframe is the first of its kind to be constructed. The system comprises an external megaframe with six perimeter columns, and continuous eccentric bracing on the four orthogonal sides of the building, and intermittent eccentric bracing on the two shorter sides of the building.”

The EBMF provides the complete lateral (wind, seismic) resistance for the tower. The intention of eccentrically braced frames (EBF) is that ductility takes place in the seismic link element, with the remainder of the frame remaining essentially elastic (see Figure 2). This creates a clear and explicit location for yielding under severe seismic response and protects the rest of the structure from damage.

The megaframe module spans three levels of office and four levels of car park. Small posts provide intermediate support to the floor plate perimeter within the megaframe modules, distributing loads through the braces to the corner columns. The accumulation of both gravity and overturning loads in the corners of the building means that column tension is largely avoided, and structural material is located where it provides maximum stiffness.

The floor plates are architecturally independent of the external megaframe. Lateral stability within each module is ensured by the six perimeter columns spanning across the three or four stories, transferring lateral forces from intermediate floors to the megaframe levels (see Figure 4). The 1.6-square-meter perimeter columns act as small cores, effectively stabilizing the floors and interior columns within each module. The floor plates within the megaframe module attach to the perimeter columns though rigid connections of the primary floor beams. Floor plates at megaframe levels additionally contain on-plan shear connections to the perimeter megaframe beams. Restraints for the seismic link articulate in order to allow link beam movement independent of the floor plates.

Moment connections are provided within the primary floor framing for additional redundancy. The perimeter megaframe is inherently able to redistribute column loads if integrity of a column is lost during an extreme event. The structural system is therefore resilient, even though the number of principal elements is smaller than for typical buildings of these dimensions.

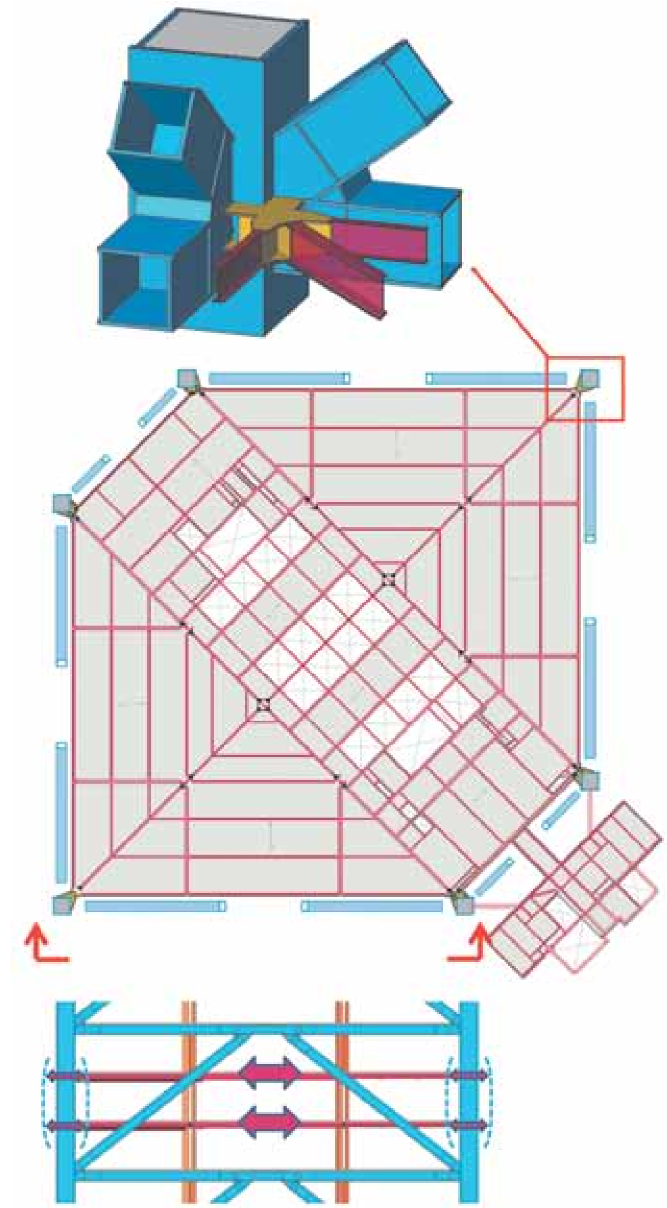

Figure 4. Typical structural floor plate showing relationship with megaframe.

The tower, the associated 12-story annex building, and the 14-story ramp/auditorium building, are located on a common seven-story basement. The ground on the site comprises lake clay strata down to around 26 meters depth. These strata are generally very soft, with variable plasticity. Below this, down to 33 meters depth, there are sandy clays of medium to hard consistency. Between 33 and 77 meters, there are interbedded layers of gravel, sand, and clay of a reasonably hard consistency. Below this there are variable strata of sandy silt, with gravel of hard consistency and high compactness. The design range for the water table is set between –4 and –25 meters. The foundations comprise of a perimeter slurry wall to 50 meters typical depth, accompanied by internal 1.4-meter diameter piles and 6.2-meter barrette piles to the same depth. The tower is founded partly on the perimeter

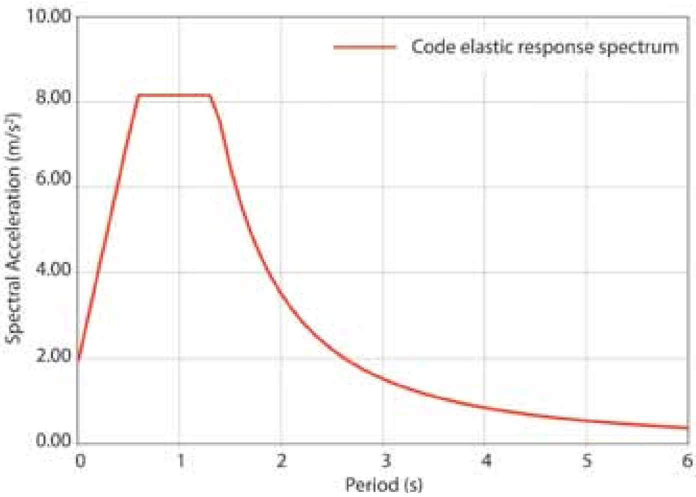

Figure 5. Elastic response spectrum for code assessment.

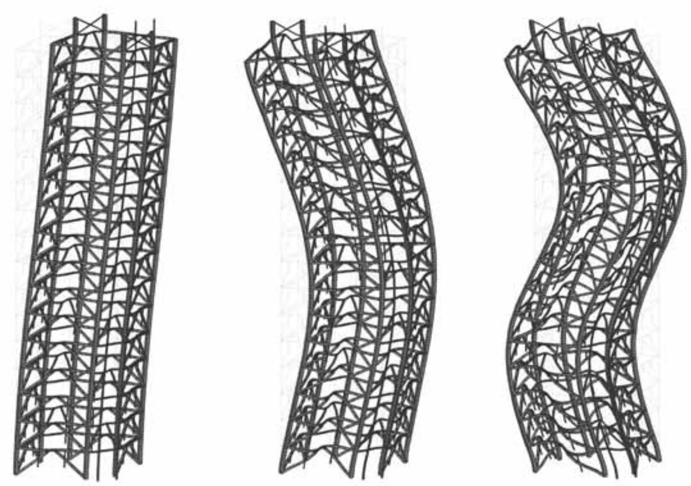

Figure 6. Three sway modes of Torre BBVA Bancomer. Higher modes play a significant role in the lateral response of a tall building (periods of vibration: 5.2s, 1.65s & 0.95s).

slurry wall and partly on interior piles and barrettes. The mat slab is typically 2 meters thick.

Seismic Design Strategy

Code Design

As stipulated by the client, the seismic design of Torre BBVA Bancomer is in accordance with the Mexico City building code (FDBC 2004). Given this requirement, the authors additionally adopted a performance-based seismic *verification* (rather than *design*) approach for the Tower. This two-pronged design and verification strategy was intended to provide a locally appropriate seismic design that was also consistent with international best practice for the seismic design of tall buildings.

This Mexico City building code specifies seismic design parameters, based on the site soil conditions and the seismic load-resisting system. Figure 5 shows the elastic response spectrum applicable to the site. The code also includes a minimum base shear strength requirement. In practice, this limits the ductility factor that may be applied to the design of the seismic system. Together with bidirectional interaction, accidental torsion, and load factors, this sets the code minimum strength requirements for the building.

The Mexico City code does not include specific requirements or guidance for EBFs. As an alternative source, the *American Institute of Steel Construction (AISC 341-05)* was adopted for the design of the EBF system. AISC 341-05 provides detailing rules for EBFs, but requires column forces to be calculated based on all EBF links yielding in the same direction coincidently. This approach is reasonable in low-rise buildings where seismic response in the first mode only is a reasonable approximation of the overall behavior. However, it would be highly conservative for the 52-story tower, which, typical for a building of its height, exhibits significantly higher-mode seismic response (see Figure 6). Higher-mode response that contributes to building shear does not contribute equally to overturning, and thus not to the column forces. The practical consequence is that a higher-mode response reduces the overall column forces.

The design team presented an approach where the column forces are based on nonlinear analysis, which gives a representative interaction between higher modes. This approach was accepted by the peer reviewers and enabled the authors to reduce column design forces by over 30%. The latest version of the code (AISC 341-10) now explicitly allows this approach.

Performance-based Seismic Verification

The overall objective of the performance-based seismic verification was to establish a detailed understanding of the seismic hierarchy and behavior of key elements, such that holistically sound seismic performance could be demonstrated for the most economical and flexible structural solution.

At a global level, the purpose of the performance-based seismic verification was threefold:

1. Demonstrate acceptable performance during service-level earthquakes. The minimum performance target for this level was essentially elastic response of the structure (including EBF links) and compliance with appropriate drift limits.

2. Verify column forces at code level earthquakes.

3. Demonstrate acceptable performance at Maximum Considered Earthquake (MCE). MCE evaluation considered ductility demand in yielding link elements, drifts, and forces in non-yielding frame elements (columns, braces).

In addition to global assessment, the exceptional scale of the EBMF warranted detailed performance-based evaluation of the plastic fatigue behavior of the ductile link elements.

The framework for the performance-based seismic verification was the *Pacific Earthquake Engineering Research (PEER) Report 2010/05* and *ASCE 7-10: Nonlinear Response History Analysis (NLRHA)* was completed at each design level, using earthquake records spectrally matched to the site hazard, using the approach set out in Grant et al. (2008)

Analytical Results

Conventional response spectrum analysis does not predict the cumulative plastic strain demand in yielding elements. This is a particular concern for the long-duration ground motions that are characteristic of the Mexico City seismic hazard. Furthermore, the representation of the modal interaction and distribution of yielding in such analysis is basic and not necessarily appropriate as a basis for design of tall buildings. The design team therefore applied global Nonlinear Response History Analysis (NLRHA) to assess these effects directly.

NLRHA comprises a structural representation of the building, with nonlinear properties for the elements that are intended to go beyond elastic behavior. This structural representation is then subjected to ground motion records applied at the base and solved in the time-domain to give the transient dynamic response of the structure.

For Torre BBVA Bancomer, the design team applied the approach outlined in ASCE 7-10 (with seven bidirectional ground motions) using LS-DYNA software for the analysis. Only the seismic links in the EBMF are intended to respond inelastically; the element formulation for these was therefore given properties that

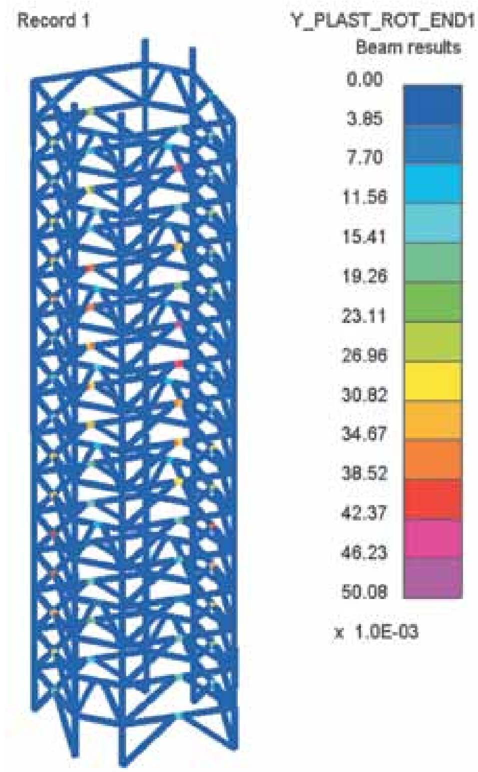

Figure 7. Example response history analysis plot, showing ductility occurring in seismic links while the remaining structure remains elastic.

realistically represent the nonlinear hysteretic behavior of the links (yielding, strain-hardening, and unloading). In the global NLRHA, a basic lumped plasticity representation of this was introduced by incorporating nonlinear hinges at the ends of EBF links, based on parameters from ASCE 41-06.

Figure 7 shows a graphical representation of the ductility demand in the structure due to one of the MCE earthquake records, where the plot contours show the peak plastic rotation angle in the hinges in radians. Yielding is widespread, but confined to the seismic link elements as intended. The cumulative ductility demand in each of the links is recorded, with an example of the strain history during the event shown in Figure 8. The recorded strain history is unique for each link and each earthquake record, and gives detailed insight into the ductility demands on the elements.

Acceptable performance of the seismic load-resisting system at MCE level relies on the ductility demands in the EBF links being within their cyclic strain capacity. There is reasonable test data available on the cyclic strain capacity of EBF links, e.g., Okazaki & Engelhardt (2007), as well as codified maximum absolute rotation level (AISC 341-05). However, the megaframe scale of the tower's EBF installation warranted special investigation of the cyclic strain demands.

Highly refined submodels were used to evaluate the plastic fatigue performance of the inelastic elements under the cumulative strain demands due to the MCE earthquake records. As with the global studies, LS-DYNA was used for response-history analysis of the assembly. The material formulation for the steel in the model included a continuum damage model as well as strain hardening. This model captured low-cycle fatigue degradation

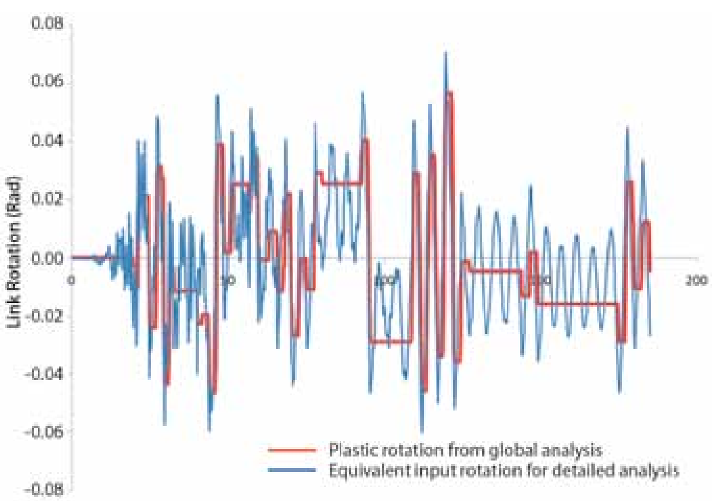

Figure 8. Example plastic strain demand for seismic link, extracted from global NLRH analysis and used for low-cycle fatigue assessment.

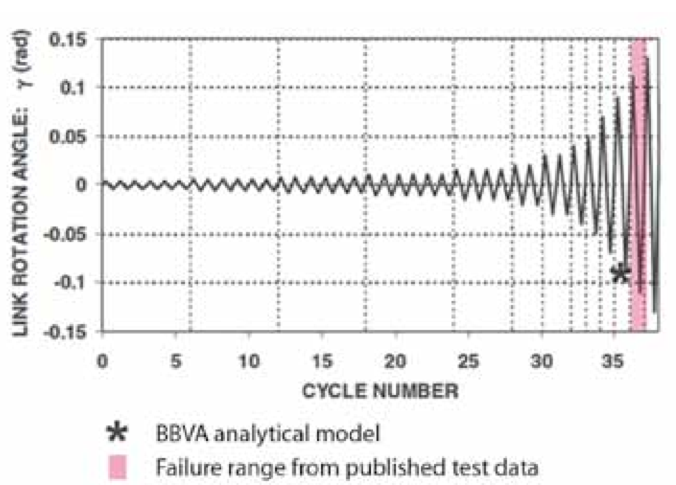

Figure 9. Comparison of analytical and published test data for low-cycle fatigue performance of seismic links.

benchmarked against the Manson-Coffin strain-range-vs.-cycle-number relationship (Coffin 1954). In effect, the material model accumulates fatigue damage during cyclic plastic straining (Huang et al. 2010).

The analytical framework with the continuum damage model was validated against physical test results for link members tested under standard AISC cyclic-loading protocols. Figure 9 shows the as-analyzed breakage point compared with as-tested breakage points for links of similar moment/shear ratios. The comparison indicates the analytical assessment is close to, but conservative compared with the tested range, indicating some conservatism in the minimum versus typical material properties. Figure 10 shows one of the analysis models employed for the study.

The detailing of the seismic links on Torre BBVA Bancomer is such that inelastic behavior is governed by shear strain, rather than bending strain. This distributes yielding over the full length of webs, whereas bending-governed links concentrate yielding at the end of the flanges, providing less energy dissipation capacity. The distribution of strain accumulation in Figure 10 clearly demonstrates the shear-yield behavior intended in the design.

Design Realization

Torre BBVA Bancomer was under construction, and topped out in early 2014. Figure 11 shows the state of construction in January 2014, with the upper megaframe module being completed. ■

Unless otherwise noted, all photography credits in this paper are to Arup.

Figure 11. Construction photo of Torre BBVA Bancomer, January 2014.

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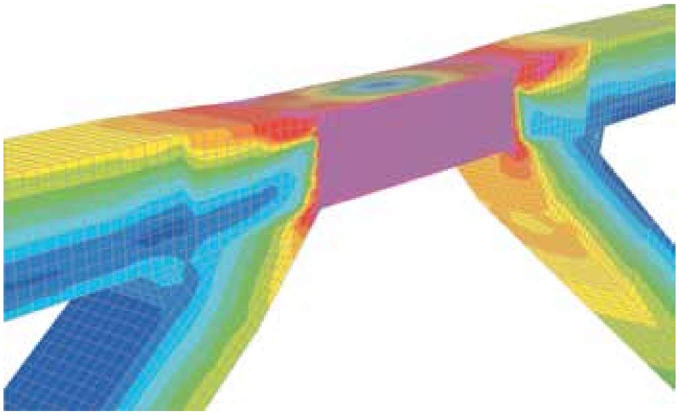

Figure 10. Detailed analysis model showing shear yielding of seismic links over full length of the webs (Pink color represents yielding, blue to red elastic).

"The analytical framework with the continuum damage model was validated against physical test results for link members tested under standard AISC cyclic-loading protocols."