# Case Study: CCTV Building - Headquarters & Cultural Center

#### Authors

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Arup

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Figure 1. Architect's impression of the building

The new headquarters of China Central Television contains the entire television-making process within a single building. The 234m tall tower redefines the form of the skyscraper, with the primary system comprised of a continuous structural tube of columns, beams and braces around the entire skin of the building. In order to gain structural approval an Expert Panel process was necessary, for which a performance-based analysis was carried out to justify the design. This made extensive use of finite element analysis and advanced non-linear elastoplastic time history to evaluate the structural behaviour and ensure the building safety under different levels of seismic event. The leaning form and varied programme, including the need to accommodate large studio spaces, posed additional challenges for the gravity structure, and resulted in the introduction of a large number of transfer trusses throughout the tower. Erecting and connecting the two massive towers presented the structural engineers and contractors with further design and construction challenges.

#### Introduction

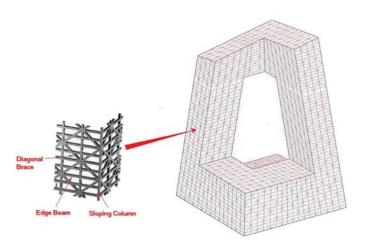
This article describes the structural design and construction of the CCTV Building in Beijing, including development of the structural concept, performance-based seismic design and Expert Panel Review process.

"Prior to connection, the two Towers would move independently of each other due to environmental conditions, in particular wind and thermal expansion and contraction. As soon as they were joined, therefore, the elements at the link would have to be able to resist the stresses caused by these movements."

#### Architectural Concept

China Central Television (CCTV), the country's state broadcaster, plans to expand from 18 to 200 channels and compete globally in the coming years. To accommodate this expansion, they organized an international design competition early in 2002 to design a new headquarters building. This was won by OMA (Office of Metropolitan Architecture) and Arup, which subsequently allied with the East China Design Institute (ECADI) to act as the essential local design institute (LDI) for both architecture and engineering.

The unusual brief, in television terms, was that all the functions for production, management, and administration would be contained on the chosen site in the new Beijing Central Business District (CBD), but not necessarily in one build-



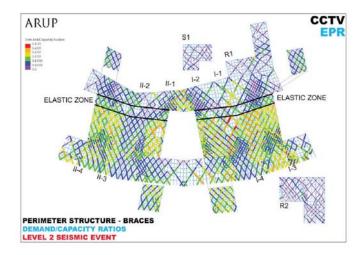


Figure 2. Uniform bracing pattern

Figure 3. Unfolded' view of final bracing pattern

ing. In their architectural response, however, OMA decided that by doing just this, it should be possible to break down the 'ghettoes' that tend to form in a complex and compartmentalized process like making TV programmes, and create a building whose layout in three dimensions would force all those involved to mix and produce a better end-product more efficiently.

The winning design for the 473,000m<sup>2</sup>, 234m tall, CCTV building (see Figure 1) thus combines administration and offices, news and broadcasting, programme production and services – the entire TV-making process – in a single loop of interconnected activities around the four elements of the building: the nine-storey 'Base', the two leaning Towers that slope at 6° in each direction, and the nine to 13-storey 'Overhang', suspended 36 storeys in the air.

The public facilities are in a second building, the Television Cultural Centre (TVCC), and both are serviced from a third Service Building that houses major plant as well as security. The whole development will provide 599,000m² gross floor area and covers 187,000m², including a landscaped media park with external features.

#### Development of the structural form

From the outset, it was determined that the only way to deliver the desired architectural form of the CCTV building was to engage the entire façade structure, creating in essence an external continuous tube system. This would give the structure the largest available dimensions to resist the huge bending forces generated by the cranked, leaning form – as well as loads from wind and extreme earthquakes.

The 'tube' is formed by fully bracing all sides of the façade. The planes of bracing are continuous through the building volume in order to reinforce and stiffen the corners. The system is ideally suited to deal with the nature and intensity of permanent and temporary loading on the building, and is a versatile, efficient structure which can bridge in bending and torsion between the Towers, provide enough strength and stiffness in the Towers to deliver loads to the ground, and stiffen up the Base to reinforce the lower Tower levels and deliver loads to the foundations in the most favourable possible distribution, given the geometry.

The tube was originally envisaged as a regular pattern of perimeter steel or steel-reinforced concrete (SRC) columns, perimeter beams, and diagonal steel braces set out on a typically

two-storey module (see Figure 2). This was chosen to coincide with the location of several double-height studios within the Towers. A stiff floor plate diaphragm is therefore only guaranteed on alternate storeys, hence lateral loads from intermediate levels are transferred back to the principal diaphragm levels via the internal core and the columns.

However, results of the preliminary analysis showed that the forces in the braces varied considerably around the structure, with particular concentrations near the roof of the Overhang and at the connection to the Base. This led to an optimization process in which the brace pattern was modified by adding or removing diagonals (i.e. 'doubling' or 'halving' the pattern), depending on the strength and stiffness requirements of the design, based on a Level 1 earthquake analysis. This also enabled a degree of standardization of the brace element section sizes (see Figure 3).

This was an extremely iterative process due to the high indeterminacy of the structure, with each changing of the pattern altering the dynamic behaviour of the structure and hence the seismic forces that are attracted by each element. It was carried out in close \$\mathcal{D}\$

collaboration with the architect, since the pattern of visually expressed diagonals was a key aesthetic aspect of the cladding system.

The braced tube structure gives the leaning Towers ample stiffness during construction, allowing them to be built safely within tight tolerances before they are connected and propped off each other. The tube system also suits the construction of the Overhang, allowing its two halves to cantilever temporarily from the Towers.

The continuous tube has a high degree of inherent robustness and redundancy, and offers the potential for adopting alternative load paths in the unlikely event that key elements are removed.

Gravity loads are also carried by vertical columns around the building's central service cores, whilst a number of steel transfer trusses are introduced to support the floors in the Overhang, at high levels in the sloping towers, and over large studios in the Podium area.

Each tower sits on a piled raft foundation. The rafts vary in thickness up to 7metres, and extend beyond the footprint of the Towers to act as a 'toe', distributing forces more favourably into the ground. The foundation system is arranged so that the centre of the raft is close to the centre of load at the bottom of each tower, and no permanent tension is allowed in the 33m long piles. Limited tensions in some piles are only permitted in major seismic events.

#### Performance-based design approach

The legal framework in China governing building design practice is similar to those of Japan and some continental European countries where the design codes are legal documents published and enforced by the state government. Design engineers must comply with the codes when designing buildings and structures covered by their scope, but equally the codes provide legal protection to the

design engineers who are relieved of any legal responsibilities by virtue of compliance. The Chinese code for seismic design of buildings (GB50011 – 2001), sets out its own scope of applicability, limiting the height of various systems and the degree of plan and vertical irregularities. Design of buildings exceeding the code must go through a project-specific seismic design expert panel review (EPR) and approval process as set out by the Ministry of Construction.

Although the 234m height of the CCTV building is within the code's height limit of 260m for steel tubular structural systems (framedtube, tube-in-tube, truss-tube, etc) in Beijing, its geometry is noncompliant. The Seismic Administration Office of the Beijing Municipal Government appointed an expert panel of 12 eminent Chinese engineers and academics to closely examine the structural design, focusing on its seismic resistance, seismic structural damage control, and life safety aspects. In order to engage the expert panel early in the design process, three informal meetings were held to solicit feedback and gain trust before the final formal presentation and approval in January 2004.

As the seismic design lay outside the scope of the prescriptive Chinese codes of practice, Arup proposed a performance-based design approach from the outset, adopting first principles and state-of-the-art methods and guidelines to achieve set performance targets at different levels of seismic event. Explicit and quantitative design checks using appropriate linear and non-linear seismic analysis were made to verify the performance for all three levels of design earthquake.

The criteria for this performance-based design are beyond those usually applied to such buildings in China, and were set by the design team in consultation with the expert panel to reflect the importance of the building both to the client and to the Chinese Government. The basic qualitative performance objectives were as follows:

Seismic Fortifi- cation Level	Level 1	Level 2	Level 3
Description	Minor	Moderate	Severe
Peak ground acceleration	0.07g	0.20g	0.40g
Average Return Period	1 in 50 years	1 in 475 years	1 in 2475 years
Probability of exceedance	63% in 50 years	10% in 50 years	2% in 50 years
Fortification Criteria	No damage (remain elastic)	Repair- able damage	No col- lapse

Table1. Seismic performance objectives

#### Elastic superstructure design

A full set of linear elastic verification analyses were performed, covering all loading combinations including Level 1 seismic loading, for which modal response spectrum analyses were used. All individual elements were extensively checked and the building's global performance verified. Selected elements were also assessed under a Level 2 earthquake by elastic analysis, thus ensuring key elements such as columns remained elastic.

The elastic analysis and design was principally performed using SAP2000, a computer-based nonlinear structural analysis program, and a custom-written Chinese steelwork code postprocessor in Excel. This automatically took the individual load cases applied to the building and combined them for the limit state design. Capacity ratios were then visually displayed, allowing detailed inspection of the critical cases for each member. Due to the vast number of elements in the model (10,060 primary elements) and the multitude of load cases, four post-processors were run in parallel for each of the four types of element in the external tube (steel columns, SRC columns, steel braces, and steel edge beams respectively).

The post-processor provided a revised element list which was imported back into SAP2000, and the analysis and post-processing repeated until all the design criteria were met. As the structure is highly indeterminate and the load

### ...structure CCTV

66 Maybe we could best describe it as a tube folded in space... All the outer surfaces are covered in a diagonal steel mesh and this mesh is folded and allows the weight to flow around the building until it finds the ideal path to the ground. 99

Ole Scheeren, Partner at the Office for Metropolitan Architecture, and architect of the CCTV Building discusses the building's unique structural system. From 'CCTV, the new state television headquarters, will broadcast China's rise', The Times, August 9<sup>th</sup>, 2008.

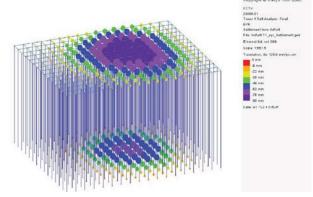


Figure 4. Foundation settlement analysis

paths are heavily influenced by stiffness, each small change in element property moves load around locally. Optimizing the elements only for capacity would result in the entire load gradually being attracted to the inside corner columns, making them prohibitively large, so careful control had to be made of when an element's section size could be reduced and when there was a minimum size required to maintain the stiffness of the tube at the back face

To further validate the multi-directional modal response spectrum analyses, Level 1 time-history checks were also made using real and artificially-generated seismic records.

#### Non-linear superstructure seismic design

For the performance-based design, a set of project-specific 'design rules' were proposed by the design team and reviewed and approved by the expert panel, for example allowable post-yield strains in each type of element. Appropriate linear and non-linear seismic response simulation methods were selected to verify the performance of the building under all three levels of design earthquake. Seismic force and deformation demands were compared with the acceptance limits established earlier to rigorously demonstrate that all three qualitative performance objectives were achieved.

Inelastic deformation acceptance limits for the key structural brace members in the continuous tube were determined by non-linear numerical simulation of the post-buckling behaviour. LS-DYNA, commonly used to simulate car crash behaviour, was used for this work. The braces are critical to both the lateral as well as the gravity systems of the building and are also the primary sources of ductility and seismic energy dissipation. Non-linear numerical simulation of the braces was needed to establish the post-buckling axial force/axial deformation degradation relationship to be used in the global 3-D non-linear simulation model. It was also used to determine the inelastic deformation (axial shortening) acceptance limit in relation to the stated performance criteria. Post-buckling inelastic degradation relationship curves illustrate the strength degradation as the axial shortening increases under cyclic axial displacement time history loading. The acceptable inelastic deformation was then determined from the strength degradation 'backbone' curve to ensure that there was sufficient residual strength to support the gravity loads after a severe earthquake event.

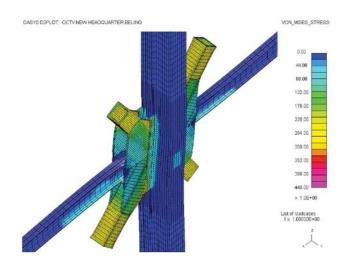
Having established the inelastic global structure and local member deformation acceptance limits, the next step was to carry out non-linear numerical seismic response simulation of the entire 3-D building subjected to Level 2 and Level 3 design earthquakes. Both the non-linear static pushover analysis method and the non-linear dynamic time his-

tory analysis method were used to determine the seismic deformation demands in terms of the maximum inelastic inter-storey drifts and the maximum inelastic member deformation. These deformation demands were compared against the structure's deformation capacities storey-by-storey and member-by-member to verify the seismic performance of the entire building. All global and local seismic deformation demands were shown to be within their respective acceptance limits.

#### Foundation design

The design of the foundations required that the applied superstructure loads be redistributed across the raft so as to engage enough piles to provide adequate strength and stiffness. To validate the load spread to the pile group, an iterative analysis process was used adopting a non-linear soil model coupled with a discrete model of the piled raft system (see Figure 4). Several hundred directional load case combinations were automated in a spreadsheet controlling the GSRaft soil-structure interaction solver.

The analysis iteratively modelled the redistribution of load between piles when their safe working load was reached. The analysis was repeated for each load case until the results converged and all piles were within the allowable capacities. Finally, the envelope of these analyses was then used to design the raft reinforcement.  $\Rightarrow$ 



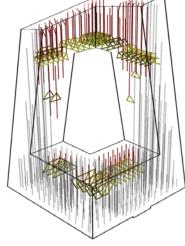


Figure 6. Transfer trusses

Figure 5. Connection analysis

#### Connection Design

The force from the braces and edge-beams must be transferred through and into the column sections with minimal disruption to the stresses already present in the column. The connection is formed by replacing the flanges of the steel column with large 'butterfly' plates, which pass through the face of the column and then connect with the braces and the edge-beams. No connection is made to the web of the column to simplify the detailing and construction.

The joints are required to behave with the braces, beams, and columns as 'strong joint/ weak component'. The connections must resist the maximum probable load delivered to them from the braces with minimal yielding and a relatively low degree of stress concentration. High stress concentrations could lead to brittle fracture at the welds under cyclic seismic loading, a common cause of failure in connections observed after the 1994 Northridge earthquake in Los Angeles. Two connections, representing the typical and the largest cases, were modelled using powerful finite element analysis software such as MSC/NASTRAN (see Figure 5).

The models were analyzed, subjected to the full range of forces that can be developed

before the braces buckle or yield - assuming the maximum probable material properties - to evaluate the stress magnitude and degree of stress concentration in the joints. The shape of the butterfly plate was then adjusted by smoothing out corners and notches until potential regions of yielding were minimized and the degree of stress concentration reduced to levels typically permitted in civil and mechanical engineering practice. CAD files of the resulting geometry of the joints were exported from the finite element models and used for further drawing production.

#### **Gravity Structure and Transfer Trusses**

Whilst the external tube structure slopes to give the unique geometry, the internal steel columns and cores are kept straight for functional layout and to house lift and services shafts. This resulted in a different configuration for every floor - the spans from core to façade, and internal column to façade, change on each level.

Sloping cores were considered, to allow consistency of floor plate layout, but ruled out due to constraints on the procurement of the lift systems. Therefore, additional columns are needed on upper storeys where the floor spans increase significantly on one side of the core. Transfer trusses support these additional

columns, spanning between the internal core and the external tube structure. They are typically two storeys deep and located in plant floors so as to be hidden from view and to minimize the impact on floor planning. The sizes of the transfer trusses mean that they could potentially act as outriggers linking the external tube to the internal steel cores - undesirable as this would introduce seismic forces into the relatively slender internal cores. The transfer trusses are thus connected to the internal cores and the external columns at singular 'pin-joint' locations only.

Further transfer trusses are introduced to support internal columns within the Overhang, and to support floors above the large studios in the Base (see Figure 6). As with the 'butterfly' plates, forces in the truss diagonals are carried only by the flanges at connections, with the webs stopping short of the chords to simplify construction.

#### Physical Testing

As part of the expert panel approval process, there was a requirement for three physical tests to be carried out, in order to verify the analytical calculations:

1. Joint Test ('butterfly plate'): Beijing's Tsinghua University tested a 1:5 scale model of the column-brace joint to confirm

its performance under cyclical loading, in particular the requirement that failure takes place by yielding of the element rather than at the connection.

- 2. Composite column: Tongji University in Shanghai tested 1:5 scale models of the project's non-standard steel reinforced columns. These tests resulted from concerns that the high structural steel ratio might lead to reduced ductility.
- 3. Shaking table test: A 7m tall 1:35 scale model of the entire building was constructed to test the structural performance under several seismic events including a severe Level 3 earthquake. The tests were undertaken by the China Academy of Building Research (CABR) in Beijing, using the largest shaking table outside America or Japan (see Figure 7).

This large-scale shaking table test was of particular interest. In China it is the norm for buildings that fall outside the code to be thus studied, and the CCTV model was the largest and most complex tested to date. The nature of the testing required the primary structural elements to be made from copper (to replicate as much as possible in a scale sense the ductility of steel). The model also included concrete floors (approximately 8mm thick) to represent

the 150mm thick composite floor slabs. In all cases, the physical tests correlated closely with the analysis.

#### Handover and Tender

In August 2004, after receiving approval for the structural design from the Chinese Ministry of Construction, Arup handed over the extended preliminary design (EPD) documents to ECADI, which then began to produce the Construction Documents (CDs). Arup, however, maintained an extensive involvement on completion of the EPD design phase, including production of tender documentation for the main structure and interaction with the tenderers for the works, as well as being part of the tender review process. Together with the architects OMA, Arup also had a continuous site presence during construction, working with the contractor in implementing the design.

#### Particular Technical Specification

One of the key tender documents was the Particular Technical Specification (PTS), which placed several requirements on the contractor that were specific to the design of CCTV.

The PTS outlined specific measures to address key issues in the construction of the building including:

- 1. Construction sequencing and its effect on the final stress in the structural elements
- 2. Ensuring the building and elements are constructed to the designed setting out and positions, within allowed construction tolerance
- 3. Construction and linking of the overhang

Further requirements were contained in separate Construction Stages and Movement reports, complementary to the PTS.

Some of the detailed issues identified in the PTS included:

- 1. Weight audits placing the onus on the contractor to convey the weight added to the building at stages during the construction. The contractor would then use this information in the prediction of deformation and movements, which would then enable calibration and presetting of the building during construction.
- 2. Specific monitoring of the tower deformation.
- 3. Specific monitoring of deformations of the foundations.
- 4. Presetting of the structure.
- 5. Monitoring of daily variation in the difference between the position of connection points as the Overhang construction advanced prior to linking.
- 6. The requirement to connect when the relative movement between the connection points of the Overhang would be manageable.
- 7. A means of showing that the extent of connection was commensurate with the daily movement measurement, so as to prevent the connection ripping apart once it had been firmly made.



Figure 7. Shake table test model

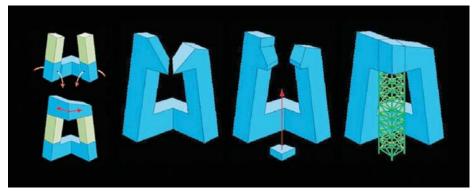


Figure 8. Alternative methods of constructing the Overhang

#### Construction sequencing

The final stresses in the building are linked to its construction sequence. In addition to regular gravity and lateral forces acting on the structure, there are significant additional construction stage forces due to the fact that the building comprises two separate leaning Towers with cantilever up until the point at which they are joined to become one structure. The additional bending and overturning stresses that get "locked" into the Towers and foundations prior to joining depend on the amount of structure and façade completed at the time of connection.

In essence, the greater the construction load applied to the building prior to connecting the two Towers, the more this would manifest itself as increased locked-in base moments in the Towers. After the connection was made, any added weight would result in a thrust between the two Towers via the Overhang.

As part of the Particular Specification, the Construction Sequence report defined an upper and lower bound range of permissible locked-in stress, allowing the contractor some flexibility in choosing his final construction sequence.

A number of construction methods were proposed for the Overhang. These included constructing of a temporary tower the full 162m height to the underside of the Overhang, providing a working platform to build the Overhang connection in situ; constructing

the lower part of the Overhang at ground level and strand jack the assembly into position; and constructing incremental cantilevers from each Tower until the two met and connected at the centre of the Overhang (see Figure 8). The latter approach was as described in Arup's documentation, though any construction approach was deemed acceptable provided it could satisfy the locked-in stress limits defined in the Particular Specification.

China State Construction Engineering Corporation (CSCEC) was awarded the main contract in April 2005. CSCEC tendered on this third approach.

#### Construction team

CSCEC, a state-owned enterprise under the administration of the central government, was established in 1982 and is China's largest construction and engineering group. CSCEC now enjoys an international reputation, having completed an increasing number of projects abroad including the Middle East, South America and Africa. The steelwork fabricators were Grand Tower, part of the Bao Steel group based in Shanghai (China's largest steel manufacturer), and Jiangsu Huning Steel, based in Jixing, Jiangsu Province.

Other members of the team were Turner Construction (USA), providing support to CSCEC on construction logistics, China Academy of Building Research (CABR), one of the major design institutes in Beijing, and Tsinghua University, which carried out the presetting analy-

sis and is one of China's foremost universities. The independent site supervisor was Yuanda International.

#### Excavation and foundations

The ground-breaking ceremony took place on 22 September 2004, and the excavation of 870 000m³ of earth began the following month under an advance contract. Strict construction regulations in Beijing meant that spoil could only be removed at night: nonetheless, up to 12 000m³ of soil was removed each day, the entire excavation taking 190 days. Dewatering wells were also installed, since the groundwater level was above the maximum excavation depth of 27.4m below existing ground level.

The two Towers are supported on separate piled raft foundations with up to 370 reinforced concrete bored piles beneath each, typically 33m long and up to 1.2m in diameter. In total, 1242 piles were installed during the spring and summer of 2005.

The Tower rafts were constructed over Christmas 2005. The 7m thick reinforced concrete slabs each contain up to 39 000m³ of concrete and 5000 tonnes of reinforcement. Each raft was constructed in a single continuous pour lasting up to 54 hours. At one stage, 720m³ of concrete was being delivered every hour, using a relay of 160 concrete trucks from three suppliers. Chilled water pipes were embedded inside the pour and temperatures were monitored for more than two weeks to ensure



Figure 10. Typical baseplate

that the concrete did not experience too high a temperature gradient during curing. The two rafts, poured within days of each other, were the largest single continuous concrete pours ever undertaken by China's building industry. In total, 133 343m<sup>3</sup> of concrete went into the foundations of the Towers and podium.

The seismic analysis indicated that some columns and their foundation piles could experience tension during a severe design earthquake. Some of the perimeter columns and their baseplates were therefore embedded 6m into the rafts to enhance their anchorage (see Figure 9). Certain piles were also designed for tension.

#### Steelwork construction

The first column element was placed on 13 February 2006. In total, 41 882 steel elements with a combined weight of 125 000 tonnes, including connections, were erected over the next 26 months, at a peak rate of 8000 tonnes per month.

During the design it was thought that some high-grade steel elements would need to be imported, but in the end all the steel came from China, reflecting the rapid advances of the country's steelwork industry. Steel sections were fabricated at the yards of Grand Tower in Shanghai and Huning in Jiangsu, and then delivered to site by road (see Figure 10), with a size limit of either the tower crane capacity (80 tonnes) or the maximum physical dimensions that could be transported (18m length).

The elements were lifted into place by two tower cranes working inside each Tower, including M1280D cranes imported from Australia – the largest ever used in China's building industry.

Each crane not only had to be raised up to 14 times during construction, but also skewed sideways up to four times when it reached the upper levels, to maintain position relative to the edges of the progressively shifting floorplate.

Due to the 6° slope of the Towers, the perimeter elements needed to be adjusted to approximately the correct installation angle after

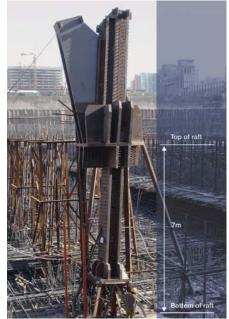


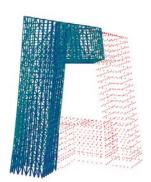
Figure 9. Column embedded in raft

being lifted a short distance off the ground, using a chain block. This simplified the erection process at height.

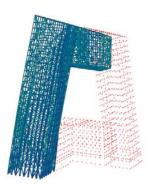
The vertical core structure was generally erected three storeys ahead of the perimeter frame. This meant that the perimeter columns could be initially bolted in place and braced to the core columns with temporary stays, then released from the tower crane before final surveying and positioning. The welders could then start the full-penetration butt welds required at every connection: a time-consuming task requiring shift work to achieve a continuous 24-hour process.

The maximum plate thickness of the columns is 110mm and the volume of weld sometimes reaches as much as 15% of the total connection weight. At the extreme case, a few connection plates near the base of the Tower required a 15m long site splice of 100mm thick plate, each taking a week to complete. The plate thickness of some elements exceeded the maximum assumed in design, which had been determined by likely steel availability. Onerous material specifications were laid out for thick sections to ensure satisfactory performance.

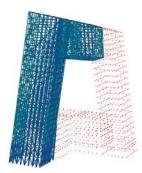
The geometrical complexity made construction slower than for other steel-framed buildings. Although the rate of erection increased as the contractor became more familiar with the process, CCTV has no "typical floors". Nevertheless, up to six storeys per month was achieved for the relatively uniform \$\mathcal{D}\$



(a) Tower deflects under its own weight.



(b) Preset upwards and backwards.



(c) Resultant: no deflection under self-weight.

Figure 11. Basic concept of presetting

levels at Tower mid-height. Concreting the composite columns and floor slabs took place several storeys behind steel erection, off the critical path.

#### Movements and presets

Arup's calculations included a "construction time history" analysis to take account of the effects of the predicted construction method and sequence on the completed building's deflections and built-in forces. This indicated that the corner of the Overhang would move downwards by approximately 300mm under the building's dead weight. For there to be no overall downward deflection under this load case, the whole structure needed to be preset upwards and backwards to compensate (see Figure 11). The contractor continuously monitored construction to ensure that the actual movements corresponded to analysis assumptions and predictions.

- (a) Tower deflects under its own weight
- (b) Preset upward and backward
- (c) Resultant: no deflection under self-weight

The presetting process was further complicated by the fact that when completed, almost all the columns have different stresses, depending on the ratio of gravity to seismic loads, unlike in a conventional building where all perimeter elements will be similarly stressed. As a result, different presets were required on different sides of the Towers, the exact values also depending on the final construction sequence. In practical terms, this meant fabricating the columns longer on one side of each Tower, so that they would eventually shorten to the correct geometry under load.

Presetting was in two stages: at the fabrication yard, based on the results of the analytical modelling, and then at installation, if required, to suit the actual building deformation as monitored during the course of construction. Progress of floor plate concreting was also controlled to suit the assumptions made in the presetting estimation.

The contractor commissioned CABR to carry out the movement monitoring, while Tsinghua University performed the building movement prediction and presetting analysis as required by the Arup specification. This required a more detailed time history analysis of the final construction sequence, dividing the process into 53 assumed stages based on estimated progress for the perimeter tube, core, slab concreting, façade, services, and interior fit-out. This was compared with the results of the movement monitoring, and checks and adjustments were made as necessary.

The studies found that the movements during Overhang construction would be far more significant than those at the earlier stages caused by the Towers' lean only. Due to the large number of variables needed for the presetting calculation (variable axial stiffness, final construction sequence, foundation settlement, thermal movements, etc), the main focus of the analysis was on the critical Overhang construction stage. By the time

## ...safety vs. cost

**66** It does not take a NIST report or a rocket scientist to figure out that requiring additional exit stairs will improve overall occupant evacuation times... The bigger question that needs to be answered is at what economic cost to society?

David Frable, a General Services Administration fire safety engineer, asks the International Code Council to repeal stronger safety requirements for new skyscrapers that were added to the country's most widely used building code last year, arguing that they would be too expensive to meet. From 'Agency Fights Building Code Born of 9/11, The New York Times, September 7th, 2008.



Figure 12. The Overhang before connection

Overhang erection commenced, there was already much movement data from the Tower construction that could be used to calibrate the analysis.

#### Overhang construction

Construction of the Overhang began after the steelwork for the two Towers was completed to roof level. Tower 2 Overhang began first, in August 2007, and the structure was cantilevered out piece-by-piece from each Tower over the course of the next five months (see Figure 12). This was the most critical construction stage, not only in terms of temporary stability but also because its presence and the way it was built would change the behaviour of those parts of the Tower already constructed. The forces from the two halves of the partly constructed Overhang would be concentrated in the Towers until such time as the two halves were sufficiently linked and the building became a single continuous form, when the loads would start being shared between all of the permanent structure.

The bottom two levels of the Overhang contain 15 transfer trusses that support the internal columns and transfer their loads into the external tube. In the corner of the Overhang, these trusses are two-way, resulting in some complex 3-D nodes with up to 13 connecting elements, weighing approximately 33 tonnes each.

Fabrication accuracy was therefore crucial for this part of the structure, with erection being carried out piece-by-piece 160m above ground level. Trial assembly of these trusses at the fabrication yard prior to delivery was essential to ensure that minimal adjustment would be needed at height.

Prior to connection, the two Towers would move independently of each other due to environmental conditions, in particular wind and thermal expansion and contraction. As soon as they were joined, therefore, the elements at the link would have to be able to resist the stresses caused by these movements. As a result, the connection strategy required a delay joint that could allow a sufficient number of elements to be loosely connected between the Towers, then locked off quickly to allow them all to carry these forces safely before any relative movement took place.

Arup specified that this should take place early in the morning on a windless day, when the two Towers would be at a uniform temperature and the movements at a minimum.

In the lead-up to connection, Arup's specification required one week of monitoring of global and relative movements so that the correct dimensions of the linking elements could be predicted. The relative movements of the Towers during the day were found to be around ±10mm. The contractor made the final measurements of the gap exactly 24 hours beforehand (i.e. at identical ambient conditions)

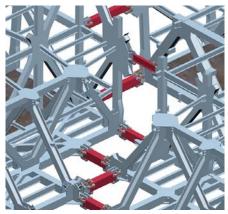


Figure 13. The seven initial connection elements

so that final adjustments could be made to the length of the linking elements while they were still on the ground prior to installation.

The contractor chose to connect seven link elements at the inside corner of the Overhang during this initial connection phase (see Figure 13). These were lifted into place – to less than 10mm tolerance – and temporarily fixed with pins in the space of a few minutes at 9.00am on 8 December 2007, before the Towers started to move relative to each other (see Figure 14). The pins allowed them to carry the thermal loads while the joints were fully welded over the following 48 hours.

The specification originally called for the connection to take place while ambient temperatures were between 12-28°C (i.e. close to the standard room temperature assumed in analysis). Since the connection took place during winter, the temperature at the time was around 0°C, so further analysis of the structure was carried out by the design team to check the impact of the increased design thermal range.

Once the initial connection was made, the remainder of the Overhang steelwork was progressively installed. With the building now acting as one entity, the Overhang was propping and stabilising the two Towers, and continued to attract locked-in stresses as further weight was applied. In addition to the primary steelwork elements, a continuous steel £



Figure 14. Installation of first connection element

plate deck up to 20mm thick was laid down on the lowest floors of the Overhang to resist the high in-plane forces that were part of this propping action. The concrete floor slabs were only added once the entire primary structure had been completed, so as to reduce the loads during the partially-constructed stage. Again, the construction stage analysis needed to take account of this sequencing.

A topping-out ceremony on 27 March 2008, on a specially-constructed platform at the corner of the Overhang, marked the completion of the steelwork installation.

#### Post-installation of key elements

Arup's early analysis showed that the corner columns on the inside faces of the Towers would attract a huge amount of dead load from the Overhang, and thus have little spare capacity for resisting seismic loads. Increasing the column sizes was rejected since they would become stiffer and hence attract even higher loads. Instead, the corner column and brace elements directly below the Overhang were left out until the end of construction, forcing the dead loads to travel via the diagonals down adjacent columns and enabling the full capacity of the corner elements to be available for wind and seismic loads in the as-built condition.

Key elements at the intersection of the Towers and podium were also post-fixed for similar reasons. In addition, this process enabled the architectural size of the elements to be controlled, while giving the contractor additional flexibility to deal with construction movements.

Delay joints were introduced between the Towers and the Base to allow for differential settlement between the two structures' foundations. It should be noted that over half the predicted settlements were expected to take place after the Towers were constructed to their full height, due to the disproportionate effect of the Overhang on the forces in certain columns. These were fully closed after completion of the main structure. Further late-cast strips were also provided at several locations around the basement to control shrinkage.

#### CONCLUSIONS

The project demonstrated that a building with many complex technical challenges could be delivered successfully within a tight programme. An international team was mobilized to make best use of the firm's experience and knowledge, which required seamless co-ordination between a number of locations and cultures.



Figure 15. The completed tower

The performance-based design approach pioneered on CCTV has since been used successfully for many other projects in China. The structure of the CCTV building was completed in May 2008, with the façade finished by the start of the Beijing Olympic Games.

That the contractor could construct such a vast and complex building with few delays was a credit to the design team and to CSCEC, in particular the attention paid to devising a feasible construction sequence from an early stage, and the careful thought about the buildability of the primary structural elements and connections.

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#### Credits

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#### Illustrations

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