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# The Structural Design and Construction of the MahaNakhon Tower

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*Bouygues-Thai was appointed by PACE Development Corporation PLC as the structural contractor for the design-build contract of MahaNakhon Tower, soon to be the tallest tower in Bangkok, Thailand standing at 314 meters. The “behind-the-scenes” story for the design and construction of the main structural components – including the mat foundation, core walls, mega columns, outriggers and floor plates – is truly unique. It is particularly notable that the set-back architectural layout on the higher floors creates an undesired lateral displacement of the tower due to its unbalanced gravity loads. A pre-setting strategy was adopted to address this problem.*

The vertical structure of the MahaNakhon Tower is made of a central RC core wall of 23 x 23 m at the basement and is gradually reduced to 23 x 14 m to the top of the tower, providing structural stability to lateral loads such as wind and seismic action. The gravitational load is mostly supported by 12 megacolumns (constructed with 60 MPa concrete) surrounding the core along its height. Lateral stiffness was strengthened by 3 RC outrigger walls linking the center core walls to the megacolumns at transfer floors on levels 19–20, 35–36, and 51–52. The first 3 dynamic modes have natural periods of 7.05, 6.80, and 2.17 seconds. The slabs consist of post-tensioned band beams with RC flat slabs. Approximately 30% of the floor plates are cantilevered, creating the “pixelation” effect required by the architecture of the project.

MahaNakhon tower has a total gravitational load of 3,000 MN, which is the combination of the superstructure self-weight of 1,600 MN, the superimposed dead load of 460 MN, the live load of 350 MN, and the mat foundation self-weight of 590 MN. The whole tower is supported by an 8.75-meter-thick mat foundation, with 129 barrette piles measuring 1.2 x 3.0 m with the pile tip reaching 65 m below grade, founded in Bangkok’s second sand layer.

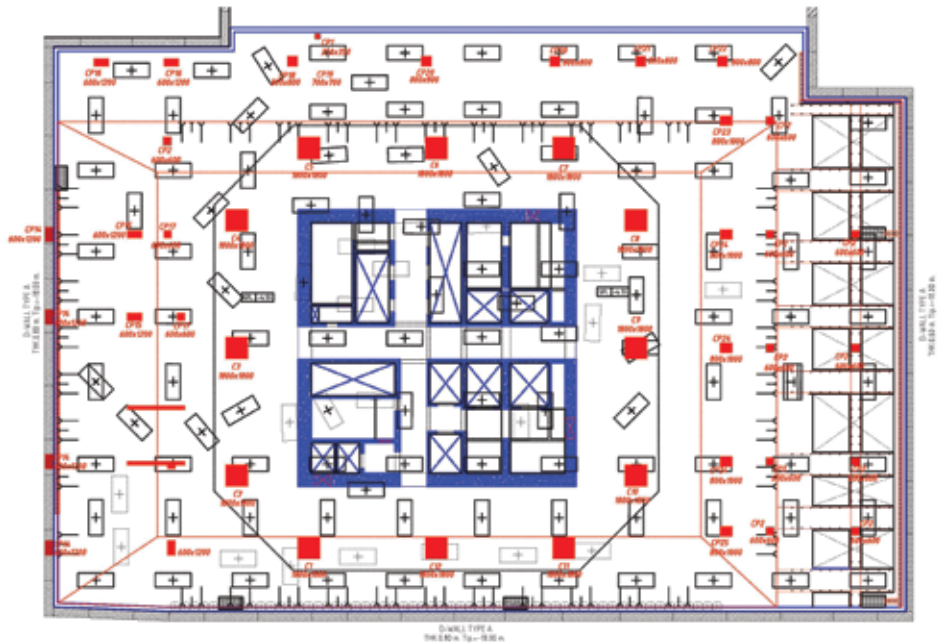
## Mat Foundation

The footprint of the mat foundation accommodates 129 barrette piles with a safe working load of 29 MN. Twelve concrete megacolumns surround the core walls of the tower.



Right: MahaNakhon mat foundation – core walls are illustrated in blue and columns in red. Source: Bouygues Thai Ltd

Bottom: Construction of MahaNakhon as of December 2014. Source: Bouygues Thai Ltd



### Soil-Structure Interaction

The design of the pile-raft took into account the soil-structure interaction by estimating the most appropriate set of stiffness of the barrettes with PLAXIS. Each individual barrette pile behaves like a spring to support the mat foundation. Spring stiffness varies from pile to pile due to the “group-effect” involved by the stress interference from the surrounding piles. An iteration process was undertaken with PLAXIS and ETABS to converge to an appropriate set of stiffness/ load distribution on barrettes.

After obtaining the spring stiffness, the structural analysis was performed with ETABS to obtain the barrette pile reactions. These reactions were then checked and confirmed to be lower than the allowable working capacity of 29 MN. In cases where barrette piles have slightly higher reactions than geotechnical working capacity (with reaction forces still less than the ultimate structural capacity), a redistribution of barrette pile reactions will occur to the adjacent piles, with a slight increase in settlement.

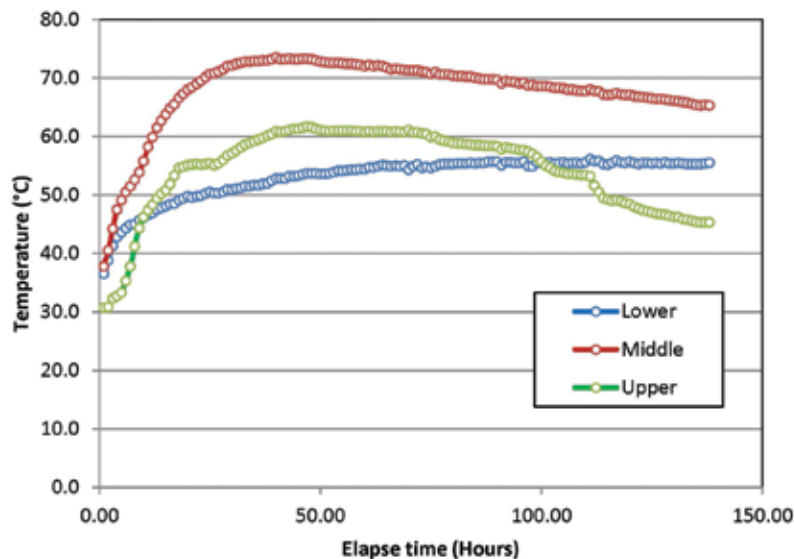
The reinforcement design of MahaNakhon's mat foundation was informed by the

bending moments and shear forces from the thick shell elements in ETABS. The design was cross-checked by referencing the conventional equilibrium of free body diagrams for applied loads and barrette pile reactions, and the struts-and-ties were designed to global and local standards. The reinforcement work of MahaNakhon's mat foundation totaled 30 MN.

### Concrete QA/QC

Due to the thickness of the piled raft of the tower, special concrete mixes have been used to prevent early-age thermal effects. To reduce the heat of hydration, “fly ash” was used to replace some portions of cement content. Fly ash also provided better workability and less segregation due to its smaller particle size and lighter weight than cement. Other raw materials such as coarse aggregates and sand were stocked in a shaded area with automatic water sprinklers to control their temperature. Reasonable amounts of ice were added into the water to lower the temperature of the fresh concrete.

Immediately after the concrete curing process, a plastic sheet was placed on top of the concrete then overlaid by 25 mm of



Graph 1: Concrete temperature monitoring. Source: Bouygues Thai Ltd

Bottom: Casting sequence of the mat foundation.  
Source: Bouygues Thai Ltd

polystyrene foam for insulation. The purpose of this foam is to keep the temperature of the top concrete layer as close as possible to that of the middle concrete layer. The maximum allowable differential temperature was limited to 20°C. Thermocouple poles were installed inside each layer to monitor concrete temperatures. Temperatures were automatically recorded in the data logger every hour for at least 5 days or until the mat foundation temperatures were stabilized, as seen in Graph 1.

### Casting Sequences

The total concrete quantity is approximately 22,000 m<sup>3</sup>. Due to the fact that the project is located at the heart of business district in downtown Bangkok, the maximum delivery rate of concrete was limited to 4,000 m<sup>3</sup> per day.

The combined constraints of heat control and concrete availability led the sequential pouring of 12 horizontal layers. Each layer was about 1 m thick, with adequate shear transfer rebar. Twelve (12) continuous working hours were required for each pour.

### Megacolumns and Core Walls

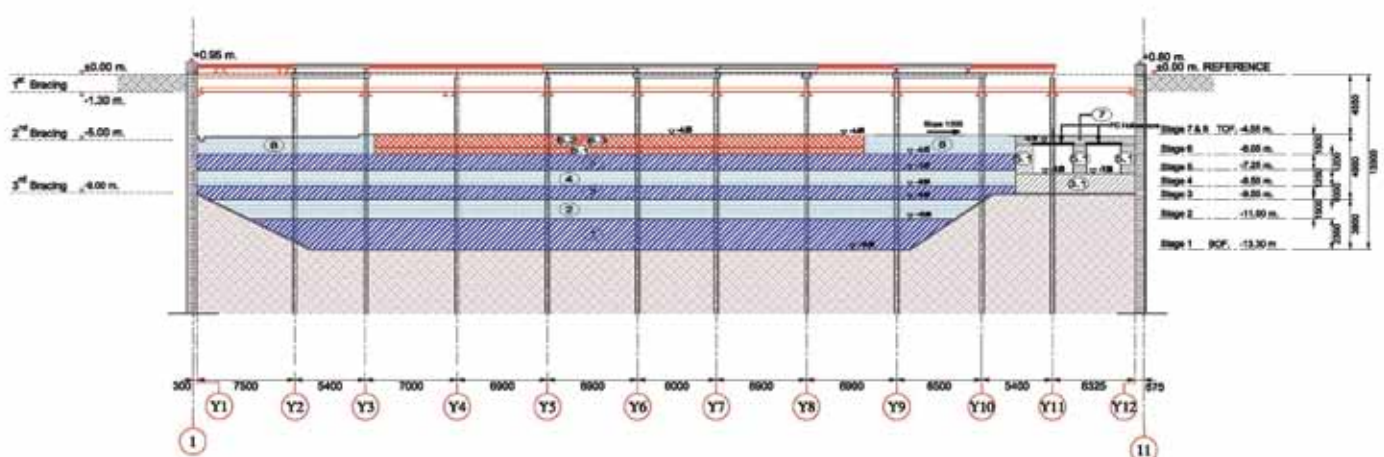
#### Gravity Models & Lateral Models

Based on sensitivity studies, it was necessary to cast outriggers in the early stages to reduce the differential axial shortening between core walls and columns. However, since floor slab dead loads were always directly supported by megacolumns before the casting of upper outrigger walls, it was concluded that the construction sequence would have a significant influence on the internal column load paths. Therefore,

making a construction sequence finite-element gravity model was necessary. This gravity model was analyzed separately from the traditional, complete and instantaneously built model, also known as a “wished-in-place” model. The latter is used to comprehensively analyze short term loads such as wind and earthquakes.

Support conditions were also considered for both flexible and rigid cases to account for all possible load paths.

All Finite Element (FE) models developed for the design of the MahaNakhon Tower are listed in Table 1.





	Flexible foundation		Rigid foundation	
Construction sequence FE gravity system models	"Model naming ""CS"" (C-construction sequence, S-spring support) Long term spring supports Stage 1: Raft foundation only Stage 2: Raft to L19 Stage 3: Raft to L35 Stage 4: Raft to L51 Stage 5: Raft to Roof		"Model naming ""CF"" (C-construction sequence, F-Fixed support) Fixed supports Stage 1: Fixed support to L19 Stage 2: Fixed support to L35 Stage 3: Fixed support to L51 Stage 4: Fixed support to Roof	
Wish-in-Place FE lateral system models	"Model naming ""US 475"" (U-ultimate lateral forces, S-spring support, short term)  475 year RP seismic with 5% damping ratio with R=4.0 to design + Wind loads 50 years return period with 1.5% damping ratio to design	"Model naming ""US 2475"" (U-ultimate lateral forces, S-spring support, short term)  2475 years return period seismic with 2% damping ratio without any response modification factor (R=1.0) to check intermediate ductility detail requirements	"Model naming ""UF 475"" (U-ultimate lateral forces, F-fixed support)  475 year RP seismic with 5% damping ratio with R=4.0 to design + Wind loads 50 years return period with 1.5% damping ratio to design	"Model naming ""UF 2475"" (U-ultimate lateral forces, F-fixed support)  2475 years return period seismic with 2% damping ratio without any response modification factor (R=1.0) to check intermediate ductility detail requirements

Table 1: Model details and naming system. Source: Bouygues Thai Ltd

Concrete strength (MPa)	ACI 8.5.1 Elastic modulus (MPa)	Laboratory Elastic modulus (MPa)	Applied stresses (40% of compressive strength, MPa)	Elastic strains based on ACI elastic modulus (x10-6)	Creep & Shrinkage strains from test results (x10-6)	Actual long term creep coefficients (based on ACI elastic modulus)
35	29,910	40,245	14	468	895	1.91
40	31,975	40,632	16	500	840	1.68
50	35,750	44,612	20	559	843	1.51
60	39,162	47,104	24	613	668	1.09

Table 2: Elastic modulus and creep & shrinkage test results. Source: Bouygues Thai Ltd

### Performance Based Check and Seismic Design

Referring to Table 2, there are two main cases for the lateral loads of each support condition.

First case:

- Seismic design was done according to Thai local codes with a response spectrum analysis based on a 475-year return period, with a 5% damping ratio and response modification factor of R=4.
- A 50-year return period for wind loads with a 1.5% damping ratio.

Second case:

- CTBUH Recommendations For Seismic Design of High-Rise Buildings (2008), Appendix B for low seismic hazard regions, were implemented for the performance check, with an amplified return period up to 2,475 years and a damping ratio of 2.0% without a response modification factor (R=1).

Results of the analysis show that the demand-to-capacity ratio was always less

than 1.0 for both the 475-year seismic return period and the CTBUH-recommended 2,475-year return period.

For example, from the megacolumn C1 interaction diagram seen in Graph 2, it was apparent that the internal forces from all load cases were located inside the demand capacity ratio of the 1.0 curve (illustrated by a dotted line). This was due to the fact that the sizing of the structural elements did not depend only on the strength requirements, but also on the serviceability requirements (wind displacement and wind acceleration for human comfort criteria).

For a more detailed analysis about the seismic design of the building, refer to MahaNakhon Tower and the Use of CTBUH Seismic Guidelines, an article published for the CTBUH 2014 Conference in Shanghai (Chanvaivit 2014).

### Outriggers

In a tall building with a high slenderness ratio, human comfort must be carefully addressed. The presence of 3 levels of outriggers on levels 19–20, 35–36, and 51–52 was necessary to improve the stiffness

of the tower by linking the center core walls with the surrounding megacolumns to create a push-pull mechanism with tension and compression forces in the outer columns, applying a couple to the core that acts against the cantilever bending under wind loads. The belt truss surrounding the building brings all external columns into action. This mechanism minimizes the fundamental period of the tower, the dynamic part of the wind loads, the lateral drifts and accelerations, lowering the risk of human discomfort.

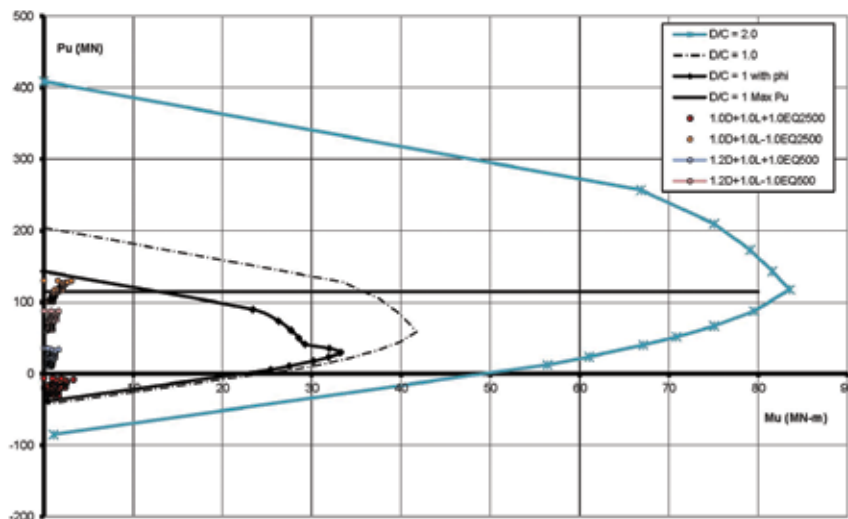
### Staged Analysis for the Outrigger System

For such a structure, the gravitational dead load distribution resulting from a "wished-in-place" model (i.e., a model that does not account for the construction sequence) will underestimate the gravity loads in the columns as the stiffness of the outriggers will hang the columns and attract some of the forces. It was found that the underestimation was approximately 10%, indicated in Graph 3.

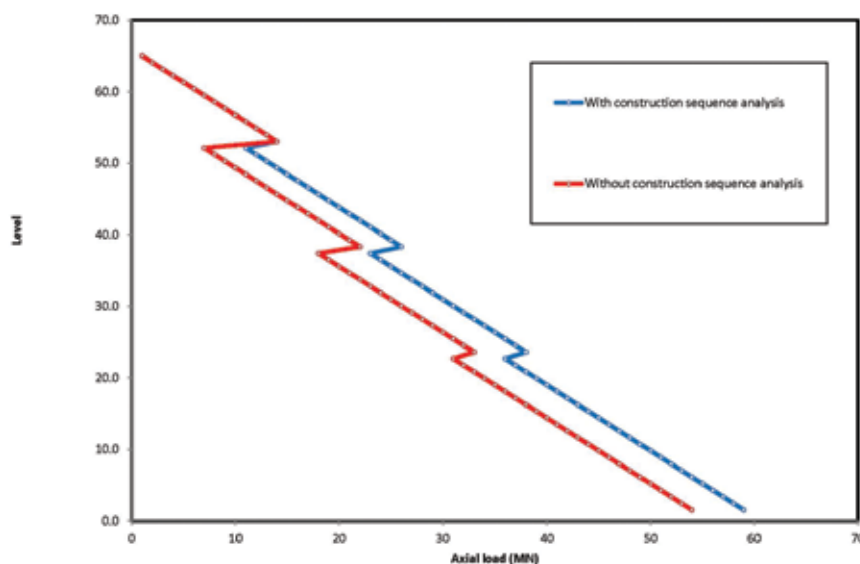
### Delay of the Outrigger Connections

Different sensitivity studies were performed during the design process with ETABS to estimate the impact of delaying the connection of the columns to the outriggers.

Opposite: Preliminary casting of the core walls and megacolumns. Source: Bouygues Thai Ltd



Graph 2: Interaction diagram with Appendix B of the CTBUH Seismic Guidelines in blue. Source: Bouygues Thai Ltd



Graph 3: Effect of the construction sequence on column loads. Source: Bouygues Thai Ltd

The studies shows that the forces that went to the outriggers were reduced by only 2.6 MN out of 38.6 MN in cases where all outriggers were delayed until the construction reached the rooftop. As a consequence, the decision was made not to delay the connection of the outriggers to the core walls because delaying connections would negatively impact the construction cycle. As curtain wall installation was scheduled shortly after the construction of the structure, the connection would have to be done in a closed space, which is a very difficult operation. Secondly, delaying would have a very important impact on the differential axial shortening between columns and the core walls.

### *Differential Axial Shortening Between Columns and Core Walls*

For MahaNakhon, the gravitational stresses in the megacolumns were significantly greater than in the core walls due to the floor layout. This differential stress caused the megacolumns to shorten faster relative to the core walls. In the case of the connection of the outrigger being delayed, dead loads would go directly through the columns until the connection was fully braced.

Studies showed that column axial load caused differential axial shortening between the core walls and the columns of approximately 100 mm and made the floor plate tilt. However, if there was no delay of the outrigger connection time, the maximum differential axial shortening would become only 5 mm.

### *Outrigger Design Approach*

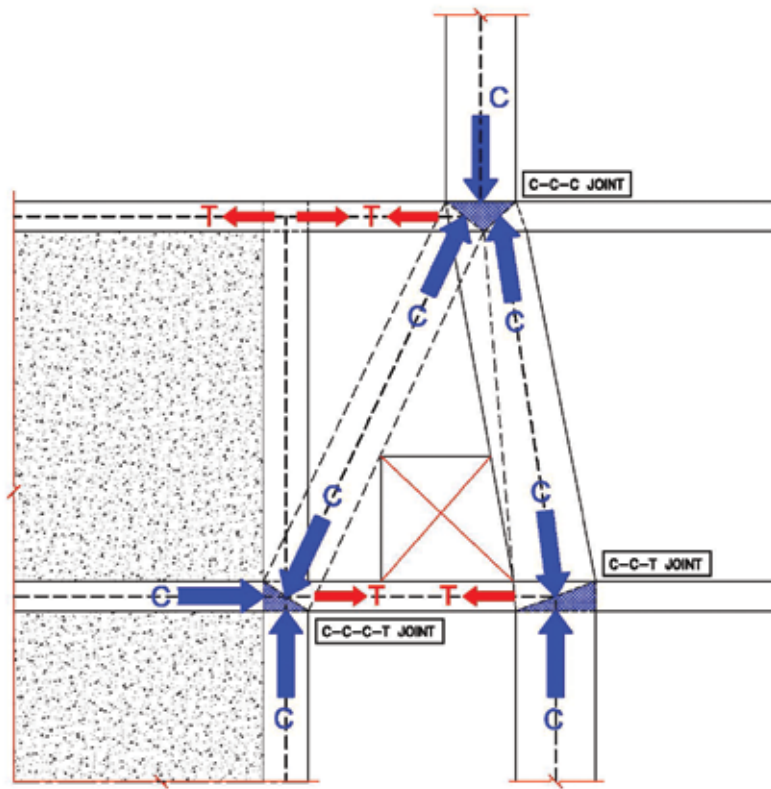
Outriggers can be considered “deep beams” due to their span-to-depth ratios. The strut-and-tie method from ACI318-11, Appendix A was adopted to design outrigger walls. Incoming and out-going column forces acting on an outrigger wall were obtained from the superstructure finite element model to develop a strut-and-tie truss model. After equilibrium was achieved, reinforcement was designed accordingly.



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### Constraints of the Outriggers

The outrigger elements and the belt trusses have a huge impact on the construction cycle. One outrigger wall level has a self-weight of 1.5 MN with heavily congested rebar and 1 MN of concrete formwork. This required 8 to 14 floors of back-propping. A total of 48,000 couplers were required for all outrigger levels.

### Floor Plates

#### Cantilever PT Slabs

MahaNakhon Tower has a special feature called "pixelation," creating an iconic form in which a three-dimensional ribbon wraps around the building's full height. This pixelation is made from stacked surfaces of the long cantilever terraces.





Opposite Top: The strut-and-tie truss model for the level 19–20  
outrigger. Source: Bouygues Thai Ltd

Opposite Bottom: Outrigger walls on level 19–20.  
Source: PACE Development PLC

Bottom: Post-tensioning layout. Source: Bouygues Thai Ltd

Thirty percent of the total slab area of the MahaNakhon project is in cantilever span due to this pixelation. Typical cantilever span at residential floors is approximately 8.0 m. In particular locations, the cantilever span went up to approximately 10.0 m. Pre-stressed concrete was the key behind the success of this design. This bonded post-tensioned system was designed and installed by VSL, the specialist company from Bouygues Construction Group. Due to the limited available space for the structural floor system, the shapes of post-tensioned beams were relatively wide with respect to their thickness.

#### Deflection Control for the Façade

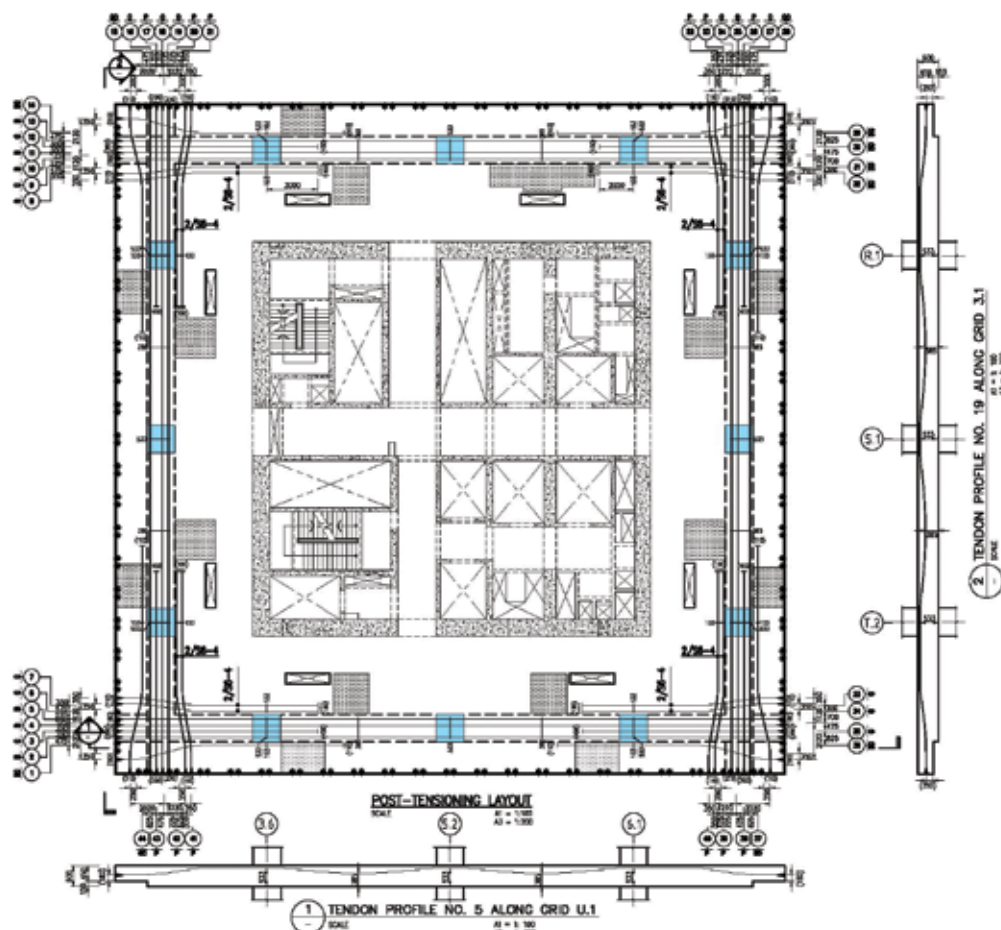
The curtain wall façade was designed to be hung from the edge of each floor slab with special horizontal joints between panels

called “stack joints.” This stack joint demanded the most stringent criteria for the slab design. Basically, this stack joint allowed a relative vertical movement of 25 mm between panels. However, 6 mm was reserved for the temperature axial shortening/expansion of the glass, and the creep and shrinkage of the core walls and columns from floor to floor. Hence, the remaining allowable relative vertical movement between façade panels was only 19 mm. Both short-term and long-term relative slab deflections should be compatible with this acceptable movement of 19 mm, otherwise the two adjacent panels may clash and cause damage to each other.

The relative deflections floor plates were analyzed based on the actual construction program. Incremental long term deflections

were calculated after the façade was installed including (1) creep deflections from the slab self-weight, (2) creep deformations from the post-tensioning, (3) short-term and long-term deflection due to the finishes, services, ceilings, partitions and façade self-weight and (4) live load. The elastic deformations due to slab self-weight and post-tensioning at the jacking stage were not considered for the incremental long-term deflections since these deformations occurred prior to the façade installation.

MahaNakhon's floor plates vary from floor to floor throughout the tower, therefore providing a limited number of typical floors. Hence, all floors were analyzed to represent the vertical and rotational deflections from a floor plate to adjacent floor plates.



Finally, adjacent floor plate deflections are compared with the live load on the higher floors, while the lower floors have no live load. No single differential deflection was greater than 19 mm.

### Actual Slab Deflections

The slab on level 11 was selected to compare the actual in-situ deflection with the finite element results. The comparison was made at the slab corner where the maximum deflection occurred. Reading the analysis results, it was shown that the self-weight deflection was 9.7 mm downward while the deformation from post-tensioning was 1.3 mm upward. After treating the effects of shrinkage and creep, the in-situ long-term deflection was estimated at only 22 mm, comparing to the total long term deflection of 28 mm that was derived from theoretical calculations. This lower deflection is due to the higher actual concrete elastic modulus as well as a lower tested creep coefficient.

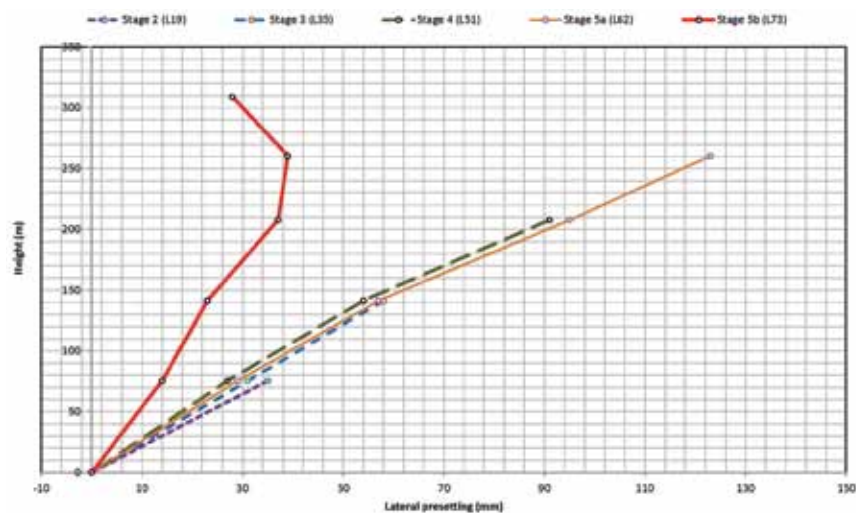
### Tower Pre-Setting

#### Unbalanced Tower Loads

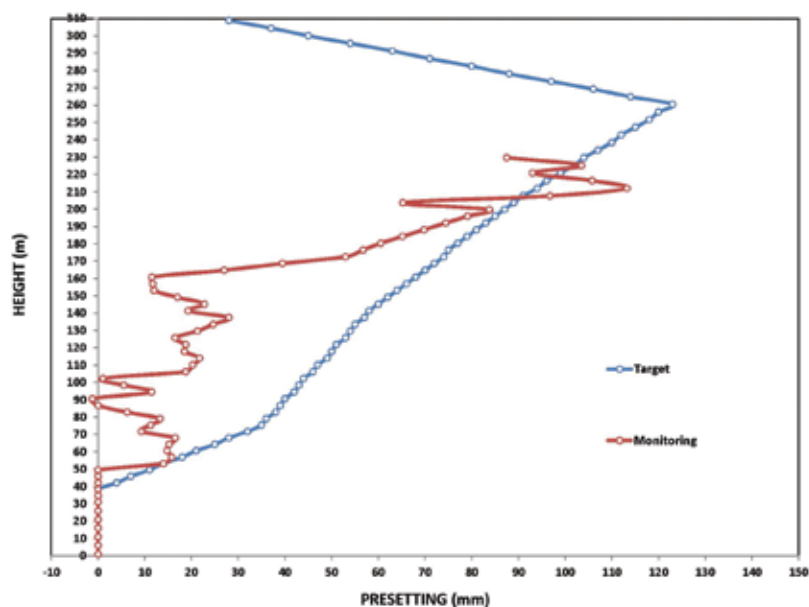
The architectural design of the top of the tower affects the center of the gravity of the upper floors to shift westward. This is called an “unbalanced tower load.” From the structural analysis, it was indicated that the tower has a long term horizontal displacement of approximately 440 mm westward due to gravity loads only. When combined with a horizontal displacement of 340 mm by wind loads, total displacement reaches 780 mm. This value is higher than the limitation for elevator operation (H/500), so a pre-setting construction method was used to correct the lateral displacement. Initial design evaluation and laboratory and field tests (creep, monitoring) were adapted to the pre-setting process to affect the real behavior of the building during construction.

#### Creep Test

This pre-setting calculation was originally based on the ACI creep assumption without any specific data available in Thailand.



Graph 4: Lateral pre-setting. Source: Bouygues Thai Ltd



Graph 5: Target and monitoring pre-setting. Source: Bouygues Thai Ltd

Bouygues-Thai worked with King Mongkut University of Technology Thonburi to develop a creep testing machine and a creep testing room according to the ASTM C512 standard. The temperature was controlled at  $23^{\circ}\text{C} \pm 1^{\circ}\text{C}$  with the controlled relative humidity of  $50\% \pm 4\%$ . The creep test results are summarized in Table 2. It can be found that the elastic modulus of all concrete specimens were higher than the code models by approximately 15% to 30%, while the creep strains were relatively lower than code recommendations. The higher concrete strength had the lower creep strain.

#### Pre-Setting Method

The verticality of the tower was controlled by laser plummets from the survey system. Monitoring and adjustment of the slipform

was done by slipform operators, technicians, and Bouygues-Thai surveyors. In order to counterbalance the westward long-term horizontal displacement, the tower needed to be pre-set horizontally eastward. There was no pre-setting for the north-south direction. A procedure was setup to monitor various points and their eastward shifting. The necessary pre-setting for various stages of the construction cycle are shown in the opposite top graphic.

#### Actual Pre-Setting Data

In-situ pre-setting data was collected from the production team. After level 12, the pre-setting strategy was starting to be implemented on site. From level 12 up to level 35, the average achieved pre-setting was approximately 50% of the expected

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value. After level 51, the target average pre-setting had been successfully reached, as seen in Graphs 4 and 5.

Table 3 presents the expected pre-setting and movements at each level of the tower when the structures were being cast on the floors above. The numbers in blue in diagonal cells represent the in-situ initial pre-setting when each floor was cast. The numbers in black represent the expected tower position on that particular floor when upper floors were cast at each stage. The numbers in red were the actual tower position on that particular floor when upper floors were cast at each stage. For example on level 19, when it was cast, the pre-setting was at 13 mm eastward. When the slab on level 31 was built, the slab on level 19 is supposed to move from 13 mm to 10 mm, while the in-situ data is at 11.9 mm, which is slightly slower

than expected. It was the same case when the level 35 slab was casted, the slab at level 19 was supposed to move to 9 mm while the actual recorded data was only 10.2 mm, which is slightly slower than expected.

However, the pre-setting data after level 35 was cast is slightly faster than expected. This was not due to the change in concrete properties, but the fact that the core walls were cast 10 floors above the casting slab, which was different from the analysis models that assumed the core walls and the slabs were cast close to each other.

As confirmed by the creep test results with better creep properties, the lateral displacements of the tower reduced in magnitude significantly. From this recorded data, the lateral pre-setting strategy was adapted. It was recommended that

from level 66 onward the core could be constructed vertically.

**Final Thoughts**

The design-build process involves the structural designers to work in full conjunction with the construction teams. The design was adapted to the methods used on-site and vice versa. This way, the design assumptions embrace reality and the design results can be more efficient. This article shows the important link between the construction site and design development, which is important to realize a successful and efficient project. The construction of MahaNakhon’s structure provided an interesting opportunity to compare the design projections and the field data and monitoring results. Our findings show that the current in-situ results tend to follow the expected values, with a better result.

	Cast L19	Cast L27	"Cast L31 Monitoring 1 (Aug 21, 2014)"	"Cast L35 Monitoring 2 (Sep 12, 2014)"	"Cast L43 Monitoring 3 (Nov 10, 2014)"	"Cast L50 Monitoring 4 (Dec 10, 2014)"	"Cast L51 Monitoring 5 (Dec 17, 2014)"	Cast L62	Cast L73
L73									N/A
L62								N/A	
L51							97		
L50						89			
L43					57				
L35				21			17/14.1		
L31			21						
L27		19			14.5/17.1	13.3/9.5	14.5/6.3		
L19	13		10/11.9	9/10.2	7/3.8		5/-		

Table 3: Actual pre-setting monitoring. Source: Bouygues Thai Ltd

**References:**

Chanvaivit, Kanokpat. (2014) **MahaNakhon Tower and the Use of CTBUH Seismic Guidelines**. Future Cities Towards Sustainable Vertical Urbanism, Proceedings of the CTBUH 2014 Shanghai Conference. p. 587-593