DESIGN AND CONSTRUCTION OF THE MAHANAKHON TOWER IN BANGKOK

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Introduction

MahaNakhon Tower, at the end of the construction will be the tallest tower in Bangkok, Thailand standing at 314 meters. The vertical structure is made of a central RC core wall of 23x23 meters at the basement gradually reduced to 23x14m to the top of the tower providing structural stability to lateral loads such as wind and seismic. Gravitational load is mostly supported by 12 surrounding mega columns constructed of 60MPa concrete all the way up to the roof top. Lateral stiffness was strengthened by 3 RC outrigger walls linking the centre core walls to the mega columns at transfer floors on levels 19-20, 35-36 and 51-52. The slabs were mostly posttensioned band beams with RC flat slabs. Approximately 30% of the floor plates are in cantilever mode creating the 'pixelation' effect required by the architecture of the project.

MahaNakhon tower have a total gravitational load of 3,000MN which is the combination of superstructure self-weight of 1,600MN, superimposed dead load of 460MN, live load of 350MN and mat foundation self-weight of 590MN. The whole tower is supported by 8.75 meters thick mat foundation, with 129-1.2x3.0 meters barrette piles with the pile tip at -65.0 meters down, founded in Bangkok's second sand layer. The Tower structure is currently reaching completion, as shown in Fig.1.

This article is the summary of the main critical points for the design and the construction: it consists of the elements published in the Proceeding of the 2015 CTBUH Conference New-York. The direct link of this publication can be found at: global.ctbuh.org/paper/2403.

Fig.1. Construction of MahaNakhon tower (Source Bouygues-Thai)

Mat foundation

Fig.2 shows the footprint of the mat foundation accommodating 129 barrette piles with the safe working load of 29MN. Core walls are illustrated in blue colour and columns in red.

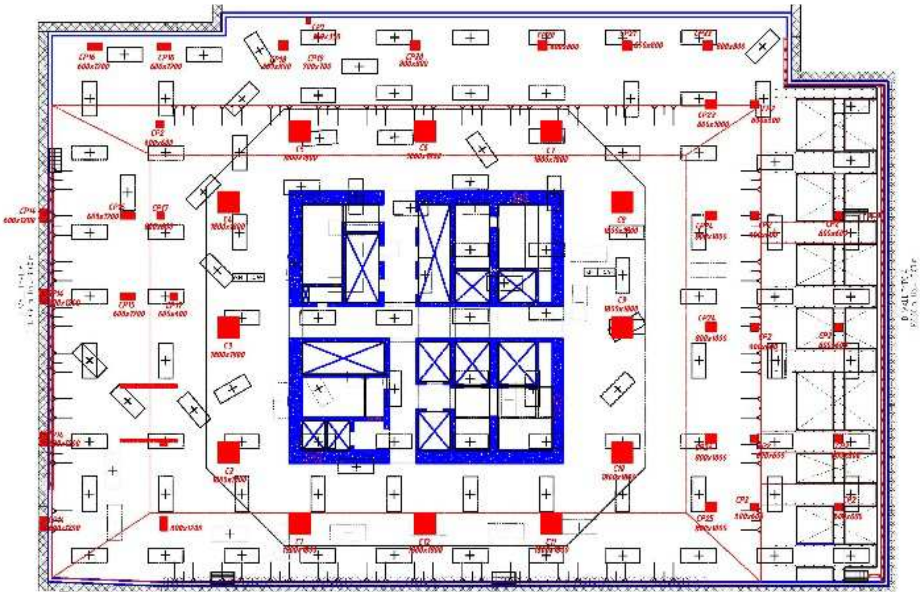

Fig.2. MahaNakhon mat foundations. (Source Bouygues – Thai)

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.

MahaNakhon mat foundation reinforcement was designed by the bending moments and shear forces from the thick shell elements in ETABS. Cross-checked by conventional equilibrium of free body diagrams of applied loads and barrette pile reactions and also the struts-and-ties were done to confirm the design globally and locally. Fig.3 shows the reinforcement work of MahaNakhon mat foundation with total quantity of 30MN.

Fig.3. Reinforcement work of MahaNakhon mat foundation. (Source image courtesy of PACE Development PLC)

Concrete QA/QC

Due to the thickness of the piled raft of the tower, the special concrete mixes have been used to prevent all the problems of early-age thermal effects. To reduce the heat of hydration, "fly ash" was used to replace some portions of cement content. Fly ash added other advantages to the concrete mix with better workability and less segregation due to its smaller 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 amount of ice was 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 25mm thick polystyrene foam to insulate the concrete. The maximum allowable differential temperature was limited to 20°C. Thermocouple poles were installed inside each layer to monitor the concrete temperatures. It was automatically recorded in the data logger at every hour for at least 5 days or until the mat foundation temperatures were stabled as shown in Fig.4 below.

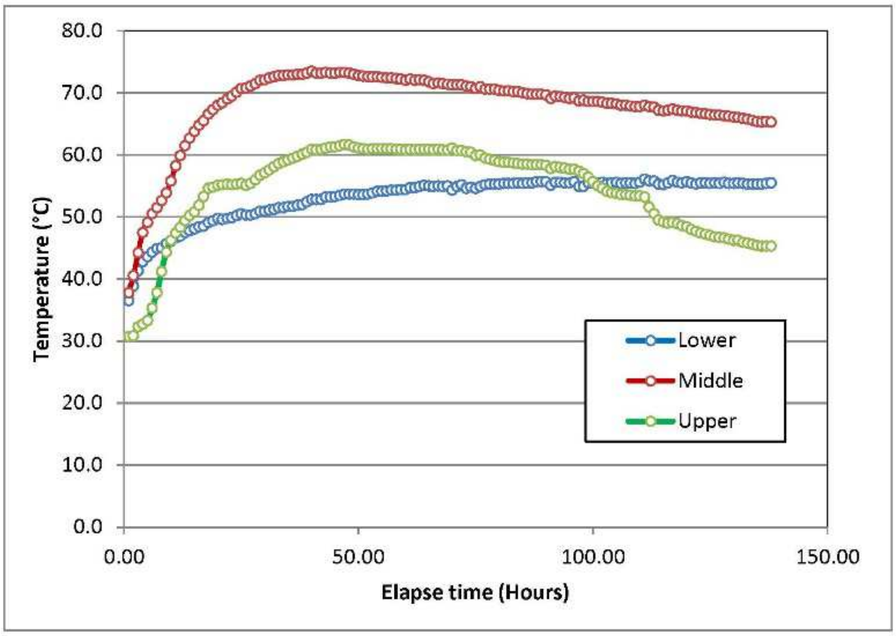

Fig.4. Concrete temperature monitoring. (Source Bouygues – Thai)

Casting sequences

The total concrete quantity is approximately 22,000 m^3. 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^3 per day (see Fig.6 to get a sense of a raft casting day). The combined constraint of heat control and concrete availability led the construction sequence to be done in 12 horizontal pouring layers. Each layer being about 1m thick as shown in Figure 5 with adequate shear transfer rebars. Twelve (12) continuous working hours is required for each pour.

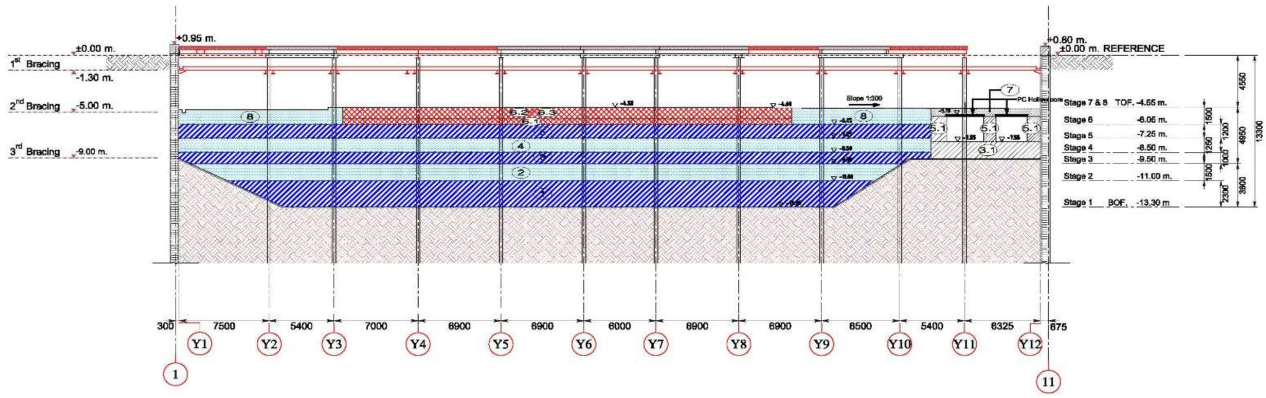

Fig.5. Casting sequence of mat foundation. Source Bouygues – Thai)

Fig.6. Concrete trucks on the mat foundation casting date. (Source Bouygues-Thai)

Mega-columns and core walls

Gravity models and 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 mega columns before upper outrigger walls casting, it was concluded that the construction sequence 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 analysed separately to the traditional, complete and instantaneously built model, also known as "Wished-in-Place" models.

The latter is used to analyse in one go for short term loads such as wind and seismic; Support conditions were also considered for both flexible and rigid cases to envelop all possible load paths. All Finite Element (FE) models developed for the design of the MahaNakhon Tower are listed in Tab.1.

<table>
<tr><th></th><th colspan="2">Flexible foundation</th><th colspan="2">Rigid foundation</th></tr>
<tr><td>Construction sequence FE gravity system models</td><td colspan="2">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</td><td colspan="2">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</td></tr>
<tr><td>Wish-in-Place FE lateral system models</td><td>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</td><td>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</td><td>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</td><td>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</td></tr>
</table>

Tab.1. Model details and naming system.

Performance based check in addition to local design for seismic

Referring to Tab.1, there are two main cases for the lateral loads for each support condition.

First case:

- *Seismic design was done by the Thai local codes with a response spectrum analysis based on a 475 years return period seismic spectrum, with 5% damping ratio and response modification factor R equal to 4.*
- *50 years return period wind loads with 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 2475 years and a damping ratio at 2.0% without any response modification factor (R equals to 1).*

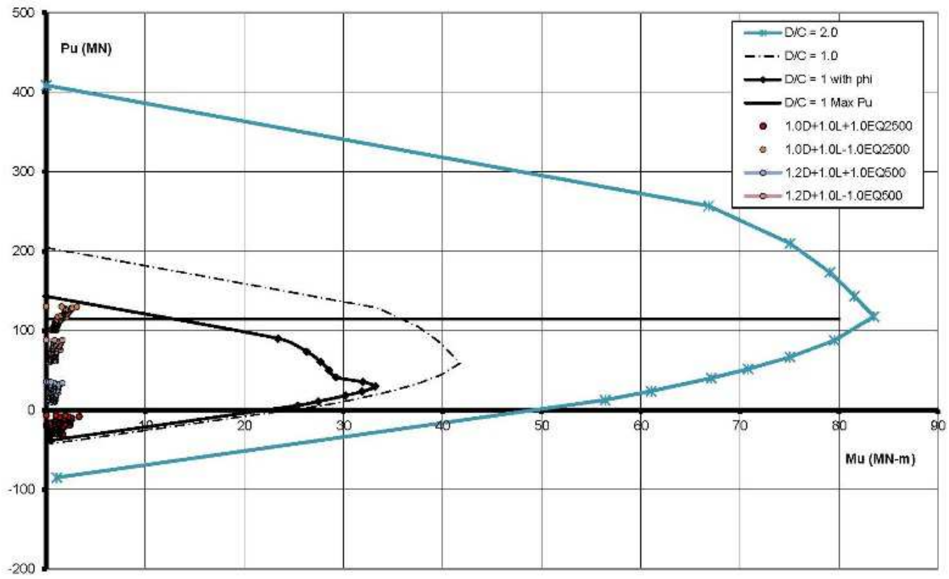

Fig.7. Interaction diagram with CTBUH Appendix B (Source Bouygues – Thai)

Result of the analysis shows that the demand to capacity ratio was always less than 1.0 for both 475 years seismic return period and CTBUH recommended seismic 2475 years return period.

For example in Fig.7, the mega column C1 interaction diagram is shown. It was apparent that all internal forces from all load cases were located inside the demand capacity ratio of 1.0 curve shown in dotted line. This was due to the fact that 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 more detailed analysis about the seismic design, refer to MahaNakhon Tower and the Use of CTBUH Seismic Guidelines, article published during CTBUH Convention 2014 in Shanghai (Chanvaivit 2014).

Outriggers

In a tall building with high slenderness ratio, the human comfort needs to be checked carefully. The presence of 3 levels of outriggers on level 19-20, 35-36 and 51-52 was necessary to improve the stiffness of the tower by linking the centre core walls with the surrounding mega columns to create a push-pull mechanism with a tension and compression forces in the outer columns and apply a couple to the core which 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.

The first 3 dynamic modes have natural periods of 7.05, 6.80 and 2.17 seconds.

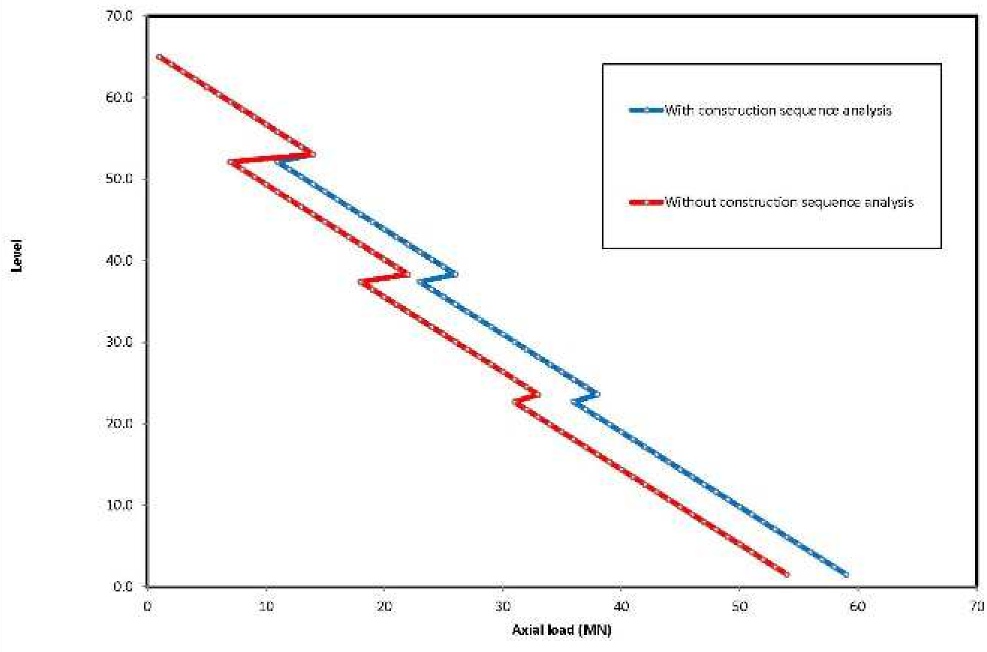

Fig.8. Effect of a construction sequence in column loads. (Source Bouygues-Thai)

Staged analysis for system with outriggers

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 one part of the forces. It was found that the underestimation was approximately 10% as illustrated in Fig.8.

Delay of the outrigger connections

Different sensitivity studies were performed during the design process with Etabs to estimate the impact of the delay of the connection of the columns to the outriggers. The studies shows that the forces that went to the outriggers were reduced by only 2.6 MN out of 38.6 MN in the case where all outriggers were delayed until the construction reached the roof top. As a consequence, the decision was made not to delay the connections of the outriggers to the core walls because delaying connections would impact the construction cycle. As curtain wall installation was scheduled shortly after the structure construction, 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 project, the gravitational stresses in the mega columns were significantly greater than in the core walls due to the floor layout. This differential stress caused the mega columns to shorten faster relatively to the core walls. In the case of the connection of the outrigger being delayed, dead loads would go through columns directly until the connection was fully braced.

Studies showed that column axial load caused a differential axial shortening between the core walls and the columns of approximately 100mm 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 5mm.

Outrigger design approach

Outriggers can be considered as "deep beams" due to their span to depth ratios. Strut-and-tie method from ACI318-11, Appendix A was adopted to design outrigger walls. In-coming 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 (Fig.9). After equilibrium was achieved, reinforcement was designed accordingly.

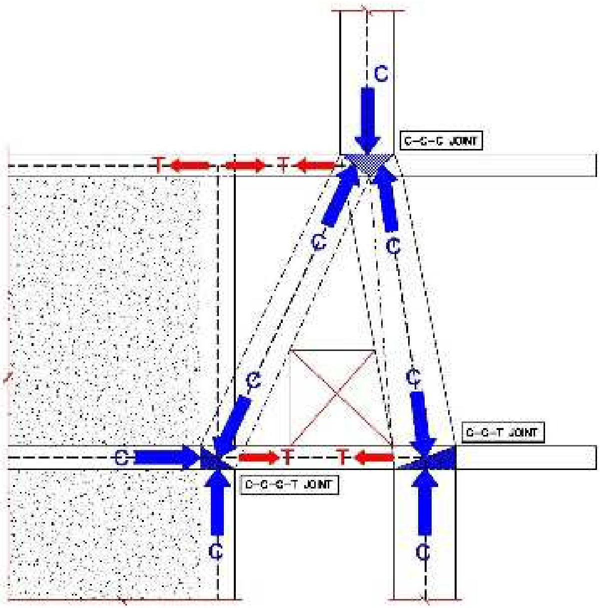

Fig.9. Strut-and-tie truss model for the level 19-20 outrigger.(Source Bouygues-Thai)

Constraints of the outriggers

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

Floor plates

Cantilever PT slabs

MahaNakhon Tower has a special feature called 'pixelation' which creates an iconic form which a three-dimensional ribbon wrapping around the building's full height. This pixelation is made from stacked surfaces of the long cantilever terraces as shown in Fig.10.

Fig.10. Pixelation of MahaNakhon tower. (Source image courtesy of PACE Development PLC)

30% 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 8.0m. In particular locations, the cantilever span went up to approximately 10.0m. Prestressed concrete was the key behind the success. 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 structural floor system, the shapes of post-tensioned beams were relatively wide with respect to their thickness, so-called "band beams" as shown in the post-tensioning layout in Fig.11.

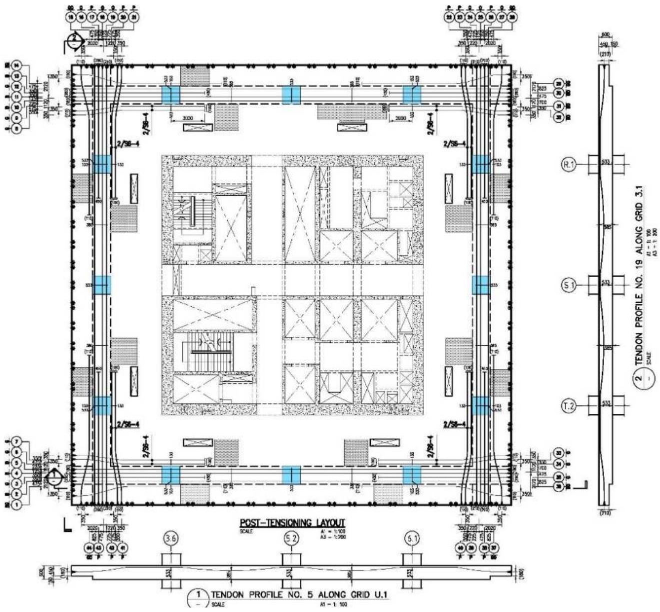

Fig.11. Post-tensioning layout (Source Bouygues-Thai)

Deflection control for façade

Curtain wall façade was designed to be hung from the edge of slab full-height from floor to floor with the special horizontal joint between panels called "stack joint". This stack joint demanded the most stringent criteria to the slab design. Basically, this stack joint allowed the relative vertical movement between panels for 25mm. However, 6mm was reserved for the temperature axial shortening/expansion of the glass and the creep & shrinkage of the core walls and columns from floor to floor. Hence, the remaining allowable relative vertical movement between façade panels was only 19mm. Both short term and long term slab relative deflections calculated are in accordance with this limit of 19mm.

Tower pre-setting

Unbalanced tower loads

This architectural design of the top of the tower affects the centre of the gravity of the upper floor to shift westward. This is called "unbalanced tower loads". From the structural analysis, it was indicated that the tower has a long term horizontal displacement of approximately 440mm westward due to gravity loads only. Superposition with the horizontal displacement by wind loads of 340mm, reaching 780mm total displacement. This value being higher than the limitation for elevator operation (H/500), a pre-setting construction method was then used to correct the lateral displacement. Initial design evaluation and laboratory and field tests (creep, monitoring) were adapted to the pre-setting to achieve the real behaviour of the building during construction.

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	*5 750*	*44 612*	*20*	*559*	*843*	*1.51*
60	*39 162*	*47 104*	*24*	*613*	*668*	*1.09*

Tab.2. Elastic modulus and creep & shrinkage test results

Creep test

This pre-setting calculation was originally based on the ACI creep assumption without any specific data available in Thailand. Bouygues - Thai worked with King Mongkut University of Technology Thonburi to develop a creep testing machines and a creep testing room according to ASTM C512 standard. The temperature was controlled at 23°C ±1°C with the controlled relative humidity of 50% ±4%. The creep test results are summarized in Tab.2. It can be found that the elastic modulus of all concrete specimens were higher than the code models approximately by 15% to 30% while the creep strains were relatively lower than the code recommendations. The higher concrete strength had the lower creep strain.

Pre-setting method

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 counter balance the westward long term horizontal displacement, the tower was needed to be pre-set horizontally eastward. There was no pre-setting for the north-south direction. A procedure was set-up to monitor various points and their eastward shifting. Fig.12 shows the necessary pre-setting for various stages of the construction cycle.

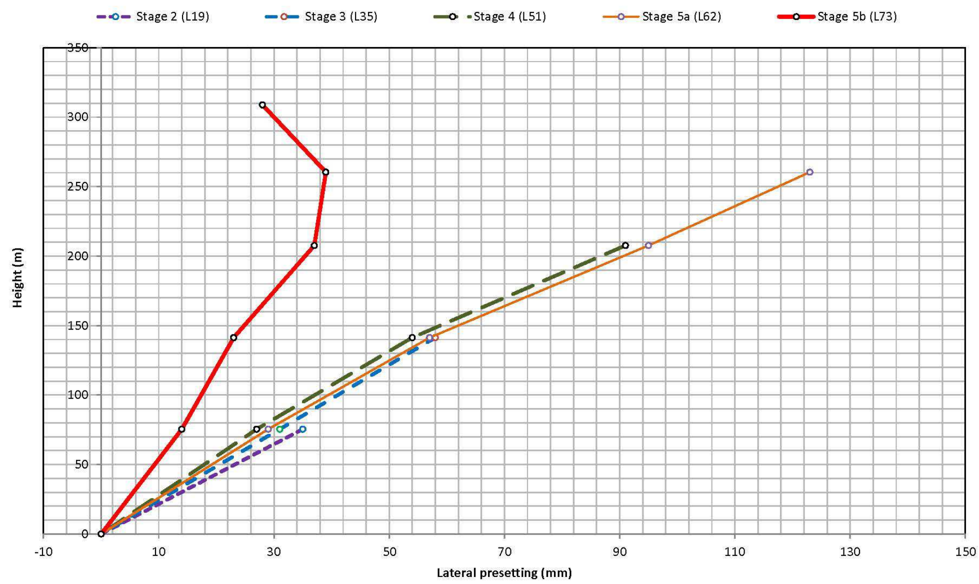

Fig.12. Lateral presetting. (Source Bouygues-Thai)

Actual pre-setting data

In-situ pre-setting data was collected from the production team. After level 12, the pre-setting strategy was started to be implemented on site. From level 12 up to level 35, the average achieved pre-setting was approximately 50% of the expected value as shown in Fig.13. After L51, the target average pre-setting has been successfully reached.

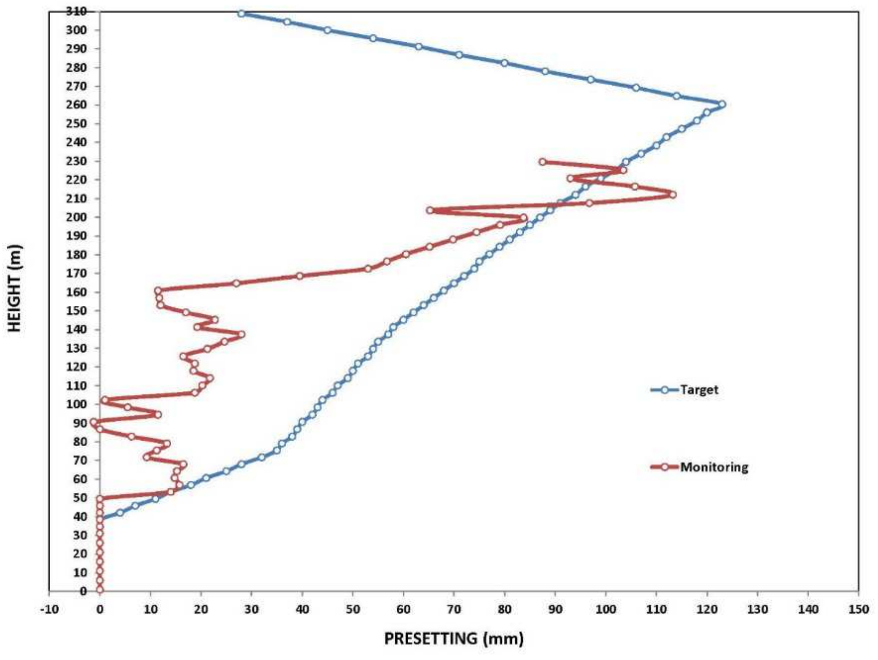

Fig.13. Target and monitoring pre-setting (Source Bouygues-Thai)

Tab.3 presents the expected tower pre-setting and movements at each level when the structures were being up on the floors above. The numbers in blue color in diagonal cells represented the in-situ initial pre-setting when each floor was *cast. The numbers in black color represented the expected tower position on that particular floor when upper floors were cast at each stage. The numbers in red color were the actual tower position on that particular floor when upper floor were cast at each stage. For example on level 19, when it was cast, the pre-setting was at 13mm eastward. When the slab L31 was built, the slab level 19 is supposed to move from 13mm to 10mm while the in-situ data is at 11.9mm which is slightly slower than expected. It was the same case when the slab level 35 was casted, the slab at level 19 was supposed to move to 9mm while the actual recorded data was only 10.2mm which is slightly slower than expected.*

However, the pre-setting data after the 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 Level 66 onward the core could be constructed vertically.

	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/-		

Tab.3. Actual pre-setting monitoring

Conclusions

The Design and 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 the reality and the design results can be more efficient. This article shows the important link between the construction site and the design development in order to lead to a successful and efficient project.