

STRUCTURAL DESIGN OF COMPOSITE SUPER-COLUMNS FOR THE TAIPEI 101 TOWER

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SUMMARY

A total of 8 composite super-columns with the size up to 8'-0" X 10'-0" were used in the Taipei 101 tower which was scheduled to be open in late 2004 and will stand 1666' tall. The high strength and high performance steel plates of the box columns are described. The proportioning, creep and shrinkage effects of the 10,000 psi high performance concrete are also discussed. The structural design and construction of the CFT columns are presented for future reference.

INTRODUCTION

The Taipei 101 is located in the Hsinyi District of the city, the rapid-growing "Manhattan" of Taipei. This is the future center of financial power in Taiwan, and already home to a dynamic collection of retail and entertainment centers. The Taipei Municipal Government awarded development rights by tender for this Build-Operate-Transfer project, the first ever in Taiwan, to an unprecedented consortium of investors. The mission is to develop a state-of-the-art building that forms an integral part of the infrastructure for advancing Taipei towards becoming one of the Asia Pacific Financial Centers. This project symbolizes the outstanding achievements of Taiwan's economic development.

The 101-story Taipei 101 currently under construction in Taipei, Taiwan will top out at 508 m, a new world record for building heights. It's indeed more challenging to design and build a super-tall building in Taipei than any other location in the world because typhoon winds, large potential earthquakes and weak soil conditions all need to be overcome. The strength and the stiffness requirements of the structure to resist gravity and lateral loads were achieved by base structural members before the common problem of the occupant comfort encountered in tall buildings could be worked out in the structural design of the Taipei 101. A damping system was implemented to reduce the excessive lateral accelerations from wind. This article hereinafter covers the overview of the structural system of the Taipei 101 and analyses as well as the design of the damping system.

MEGASTRUCTURAL SYSTEM

Gravity Systems

The tower superstructure is a steel frame with 'H' shape steel beams acting composite with the floor slab through shear studs, and floor concrete acting composite with metal deck. A typical floor framing plan is shown in Fig. 1.

Gravity loads are carried vertically by a variety of columns. Within the core, sixteen columns are located at the crossing points of four lines of bracing in each direction. The columns are box sections constructed of steel plates, filled with concrete for added strength as well as stiffness at the 62nd floor and below. On the perimeter, up to the 26th floor, each of the four building faces has two 'super-columns,' two 'sub-super-columns,' and two corner columns.

Each face of the perimeter above the 26th floor has the two 'super-columns' continue upward. The

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'super-columns' and 'sub-super-columns' are steel box sections, filled with 10,000 psi high performance concrete on lower floors for strength and stiffness up to the 62nd floor. The balance of perimeter framing is a sloping Special Moment Resisting Frame (SMRF), a rigidly-connected grid of stiff beams and H shape columns which follows the tower's exterior wall slope down each 8 story module. At each setback level, gravity load is transferred to 'super-columns' through a story-high diagonalized truss in the plane of the SMRF.

The topmost section of the building above the 91st floor is much smaller in plan. Its loadings transfer to the core columns directly.

Lateral (Wind and Seismic) Systems

Lateral forces will be resisted through a combination of braced frames in the core, outriggers from core to perimeter, 'super-columns' and moment resisting frames in the perimeter and other selected locations. Fig. 2 shows the typical megaframe elevation.

By relative stiffness, the core bracing and outriggers carry most of the wind force and seismic force. Tower lateral systems are sized to limit tower story drift under the 50 year design wind load to an inter-story drift of $h/200$ at the 91st floor and below.

Within the core, bays between the core columns are stiffened by diagonal braces. On outer faces, middle bays have 'chevron' braces (inverted V's) through which the elevator lobby entrance passes. Side bays have single diagonals, eccentric only where required to clear minimum doorway requirements. On inner faces, middle bays are non-braced (special moment frames are provided) at office floors, to keep elevator lobbies open and spacious. Side bays have diagonal braces.

For additional core stiffness, the lowest floors from basement to the 8th floor have concrete shear walls cast between core columns in addition to diagonal braces. From core to perimeter, outrigger trusses occur at 11 locations in elevation. Outriggers at 6 locations are one story high, fitting in mechanical floors. The other 5 locations are double-height, working with architectural requirements. In plan, 16 outriggers occur on each such floor.

For the dual seismic system, an independent Special Moment Resisting Frame (SMRF) is provided on each building face. From basement to the 26th floor, the SMRF consists of ductile steel beams framing between 'strong' columns – the exterior super-columns, exterior sub-super columns, and corner columns. Above the 26th floor, only two exterior super-columns continue to rise up to the 91st floor, so the SMRF consists of 600 mm deep steel wide flange beams and columns, with columns sized to be significantly stronger than beams for stability in the event of beam yielding. Each 7-story of SMRF is carried by a story-high truss to transfer gravity and outrigger forces to the super-columns, and to handle the greater story stiffness of the core at outrigger floors.

DESIGN OF SUPER-COLUMNS

Steel Plates

To minimize the member sizes and thickness used, high strength steel plates with 60 ksi yield strength was specified for the super-columns, even for steel plate up to 80 mm (3.15 inches) thick. Ductility in the steel plates and framing members was ensured by specifying the yield ratio, through-thickness characteristics and weldability as listed in Table 1. The specification was named SM570M since it was modified from CNS SM570, which tensile strength is 570 N/mm^2 .

The modifications focus on the ductility and weldability requirements for thick plates. As to ductility, the modifications cover the mechanical properties such as the range of yield strength, yield ratio, Charpy absorbed energy in longitudinal direction and reduction of area in through-thickness direction. The carbon equivalent and weld cracking sensitive composition as well as preheat temperature are required for adequate weldability. The ductility at heat affected zone is required to match that of the steel plate, except for the EGW or ESW welds at the continuity plates. The ductility of the steel plates therefore has to be improved in order to meet this welding requirement. Ultrasonic tests are specified to ensure the same quality throughout the steel plates as the tested specimens.

Table 1 SPECIFICATIONS OF SM570M STEEL PLATE

ITEM	SPECIFICATIONS	REMARKS
1. STRENGTH	1.1 YIELD STRENGTH (Fy) : $4200 \leq Fy \leq 5200$ Kg/cm ² 1.2 TENSILE STRENGTH (Fu) : $5800 \leq Fu \leq 7300$ Kg/cm ²	1.1 LIMIT STRENGTH RANGES
2. DUCTILITY	2.1 YIELD RATIO : BOX COLUMN, H COLUMN (Fy/ Fu \leq 0.85) GIRDER, BRACE (t \leq 40 mm ,Fy/ Fu \leq 0.85) GIRDER, BRACE (t > 40 mm ,Fy/ Fu \leq 0.8) 2.2 THROUGH-THICKNESS REDUCTION OF AREA : INDIVIDUAL SPECIMEN \geq 15%, AVERAGE OF 3 SPECIMENS \geq 25% 2.3 CHARPY ABSORBED ENERGY IN LONGITUDINAL DIRECTION : t \geq 12 mm, @1/4 PLATE t \geq 47J(-5°C) t \geq 50mm, ADDITIONAL CRITERIA @MID-DEPTH \geq 27J(-5°C)	2.1 SPECIFY UPPER LIMITS OF YIELD RATIO 2.2 SPECIFY REDUCTION OF AREA (CONFORM TO SN490C) 2.3 SPECIFY CMARPY ABSOR ENERGY @ MID-DEPTH FOR t \geq 50 mm
3. OTHERS	3.1 CARBON EQUIVALENT : t \leq 40 mm, Ceq \leq 0.44% ; t > 40 mm, Ceq \leq 0.46% 3.2 WELD CRACKING SENSITIVE COMPOSITION : t \leq 40 mm, Pcm \leq 0.26 ; t > 40 mm, Pcm \leq 0.29 3.3 ULTRASONIC TEST : t \geq 13 mm, CONFORM TO JIS G0901Y	3.1/3.2 SPECIFY UPPER LIMIT ON Ceq & Pcm (CONFORM TO SN490B) 3.3 SPECIFY REQUIREMENTS ON ULTRASONIC TEST

Filled-in Concrete

To provide sufficient structural strength and stiffness, the super-columns are filled with high strength concrete at level 62 and below. The columns above level 62 remain steel only to reduce the undesired gravity loads. Steel box column serves as the form of filled-in concrete while the overall column stiffness and strength are enhanced by the concrete. There has been a few building projects that use box columns with 8,000 psi filled-in concrete since it was first used in the 85 story T & C Tower in Kaohsiung, Taiwan. The design strength of high performance filled-in concrete of the Taipei 101 is 10,000 psi.

The specifications of high performance concrete were fully discussed by the design engineers and concrete experts from the industry and universities. In addition, a lot of tests were performed to confirm the concrete mixture proportion is able to meet the design requirements on the strength, shrinkage and other performance.

For concreting of 10,000 psi high performance filled-in concrete, it is pumped in to the bottom of the column, so the flowability is crucial to ensure there is no air trapped underneath the continuity steel plates. A high slump flow of 60 \pm 10 cm is specified to ensure good workability. Bleeding and segregation are also not permitted. Design age is 90 days to keep autogeneous shrinkage to as low as 300x10⁻⁶ m/m and increase durability through low water and low cement usage. Two mock-up tests were performed prior to construction to confirm the quality and workability of the concrete.

The actual concrete proportion is listed in Table 2 and the actual slump flow is 65~70 cm. Concrete strength at the design age is about 12,000 psi and still increasing.

Table 2 10,000 PSI CONCRETE MIXTURE PROPORTION

Design Strength (90 Days)	Slump (cm)	Slump Flow (cm)	Max. Diameter of Coarse Aggregate (in)	Unit Quantity (Kg/m ³)								
				Water	Cementitious Materials				Aggregate			Admixture
					Water Cementitious Ratio	Cement*	Slag	Silica Fume	Coarse Aggregate	Fine Aggregate	Limestone	
				W	W/(C+F)	C	F1	F2	G	S	LS	HP
10,000psi (HPC)	25 \pm 2	60 \pm 10	1/2"	160	0.31	340	150	30	760	957	--	6.24
	25+2	60 \pm 10	1/2"	155	0.31	340	130	30	772	920	80	7.25

Note: * Belite-rich cement.

Cross-section of Super-columns

Super-column is the primary vertical members of the megastructural system. Ease of steel erection and concreting need to be considered when a complicated cross-section is used to achieve the strength and stiffness

when designing the columns. Fig. 3 illustrates the typical column cross-section with the maximum size of 2.4 m x 3.0 m and 2 or 3 vertical stiffener plates are provided at each side of the column to: 1. reduce width to thickness ratio of the plate, 2. increase strength, 3. prevent the column plates from deforming by welding rebars to the vertical stiffeners and enhance confinement to the concrete, and 4. stiffeners are spliced with bolts to prevent the splice joint to be located at the same section.

A round manhole at the center of each continuity plate is provided as the access for welding, bolting, rebar splicing and concreting. The continuity of concrete and composite action between concrete and steel plates might be affected by shrinkage and creep after the structure loaded, so shear studs are provided at the face of vertical column plates as well as the continuity plates. Steel reinforcements are provided to increase axial strength and decrease the effects of shrinkage. Rebar cages were fabricated in the shop and lifted to place through the manholes after the vertical stiffeners spliced.

Analyses and Design of Super-columns

Analyses of the super-columns were based on the stiffness calculated from the transformed composite section of structural steel and reinforced concrete. Stress ratios were calculated using AISC LRFD Code. Taking the super-columns at level 1 as an example, the design maximum factored axial force was 38,000 tons while the maximum factored moment was 4,800 ton-meters.

The time dependent differential deformations from shrinkage and creep in the concrete are going to induce additional stresses in all the frame members, so these two effects can't be neglected in reinforced concrete and steel reinforced concrete tall buildings since the deformations of shrinkage and creep are normally close to elastic deformations. For buildings with design not governed by lateral forces, the steel ratios in the concrete columns are relatively lower and the long term deformations from shrinkage and creep are larger. The effects accumulate as the building gets taller and shall be reflected in the design.

The concrete quantity in the super-columns used in the Taipei 101 was far more than regular CFT columns and the concrete reaches a height of 270 m. Therefore, the force redistribution resulted from the effects of shrinkage and creep shall be accounted for in the analyses by calculating equivalent stiffness of the composite columns.

Creep Strain

Creep strain is a function of stresses and time, and is the product of elastic strain and a factor.

Creep Coefficient ν_t : ratio of creep strain to initial strain. For a loading age 7 days, ν_t can be calculated as follows according to ACI Committee 209.

$$\nu_t = \frac{t^{0.60}}{10 + t^{0.60}} \nu_u$$

t : time in days after loading

ν_u : the ultimate creep coefficient defined as ratio of creep strain to initial strain

Creep is usually affected by many factors to be determined through tests using actual proportion. ACI Committee 209 provides a formula to calculate the ultimate creep coefficient using estimated parameters when testing data are not available.

$$\nu_u = 235 \gamma_b \cdot \gamma_\lambda \cdot \gamma_h(\gamma_w) \gamma_t \cdot \gamma_s \gamma_\phi \gamma_\alpha$$

where γ_b : correction factor for loading age

γ_λ : correction factor for ambient humidity

$\gamma_h(\gamma_w)$: correction factor for average thickness (or volume-surface ratio)

γ_t : correction factor for temperature

γ_s : correction factor for slump

γ_ϕ : correction factor for fine aggregate percentage

γ_{α} : correction factor for air content

$\nu_u=0.728$ for the filled-in concrete used in the super-column based on the actual concrete properties.

Creep Effects on the Frame Members

The steel ratios in the CFT super-columns are much higher than regular reinforced concrete columns. For instance, the steel ratio is 12.3% for a 2400×3000×70×70 super-column with the area of vertical stiffeners of 0.15 m², and contribution from the area of rebars neglected. The axial stiffness ratio of the concrete portion is 54.3% of the composite section. For an 1200×1200×50×50 interior column, the steel ratio is 15.9% while the axial stiffness of concrete portion is 46.8%. These ratios indicate that SRC columns are less sensitive to creep effects, and differential deformations are more significant in structural analyses. The super-columns above level 62 are steel columns and can be considered as not affected by creep effects. ACI Committee 209 suggests that a rough effective modulus E_e can be used to calculate the creep strain.

$$E_e = \frac{E_{ci}}{1 + \nu_t}$$

where E_e : effective concrete modulus
 E_{ci} : actual concrete modulus at the time of initial loading
 ν_t : creep coefficient

The effective modulus $E_e=E_{ci}/(1+0.728)$ was used to replace E_{ci} in analyzing the structure. As a result, the axial deformation of dead load at level 91 was increased from 101 mm to 121 mm, while that of the corner column at the service core was increased from 106 mm to 126 mm. However, the force redistribution was mainly from differential deformation and Fig. 4 illustrates the axial force at different elevations of a super-column. The stress ratio of the super-columns subjected to gravity loads only were around 0.4 since lateral forces are significant. So, the creep effects are not very obvious to the super-columns as compared to other loadings.

Shrinkage

Shrinkage strain after age 7 days can be predicted as a function of ultimate shrinkage strain according to the recommendations of ACI Committee 209,

$$(\epsilon_{sh})_t = \frac{t}{35+t} (\epsilon_{sh})_u$$

In the absence of specific shrinkage data for local aggregates and conditions, the average suggested ultimate strain is as follows,

$$(\epsilon_{sh})_u = 780 \times 10^{-6} \cdot \gamma_{\lambda} \cdot \gamma_h \cdot (\gamma_{vs}) \cdot \gamma_s \cdot \gamma_{\phi} \cdot \gamma_c \cdot \gamma_{\alpha}$$

where γ_{λ} : correction factor for ambient humidity
 $\gamma_h(\gamma_{vs})$: correction factor for average thickness (or volume-surface ratio)
 γ_s : correction factor for slump
 γ_{ϕ} : correction factor for fine aggregate percentage
 γ_c : correction factor for cement content
 γ_{α} : correction factor for air content

The maximum shrinkage strain can be calculated using the actual concrete properties,

$$\text{Interior columns} : (\epsilon_{sh})_u = 75.9 \times 10^{-6}$$

$$\text{Super-columns} : (\epsilon_{sh})_u = 21.36 \times 10^{-6}$$

The maximum deformation of the interior column at the elevation of 284 m, the topmost point of filled-in concrete, is 2.1 cm and is much smaller than that of a regular reinforced concrete column because of higher steel ratios. The final shrinkage deformations are 0.33 and 1.00 cm respectively based on the stiffness ratios of 54.3% and 46.8% as mentioned in the previous paragraph. Although there is a differential deformation of 0.67 cm, the shrinkage deformations occur in a short period. The final deformation will be compensated at each floor as the construction proceeds, and won't be accumulated. For a story with the height of 4.2 m, the differential shrinkage is only 0.01 cm and can be neglected when compared to other loads.

CONCLUSIONS

Super-column is one of the major elements that make up the megastructural system. The high stress concentration and size effects shall be seriously considered in detailing, although the humongous members can provide more flexible spaces for architectural design purpose. The design of super-columns needs to implement the highest specifications to achieve the strength requirements. In addition, the stress distribution and transfer between different materials of the composite member due to different strengths and stiffness tend to complicate the construction. It requires the cooperation among the structural engineers, the supervising engineers and the contractor to perform fabrication and erection of the super-columns and filled-in concrete. The new materials and new construction methods worked out by the engineers, steel makers, steel fabricators, concrete provider and the researchers can assure the stable mechanical behaviors of the super-columns. We would like to express our appreciation to all people who make the construction of the super-column successful during the design and the construction stages.

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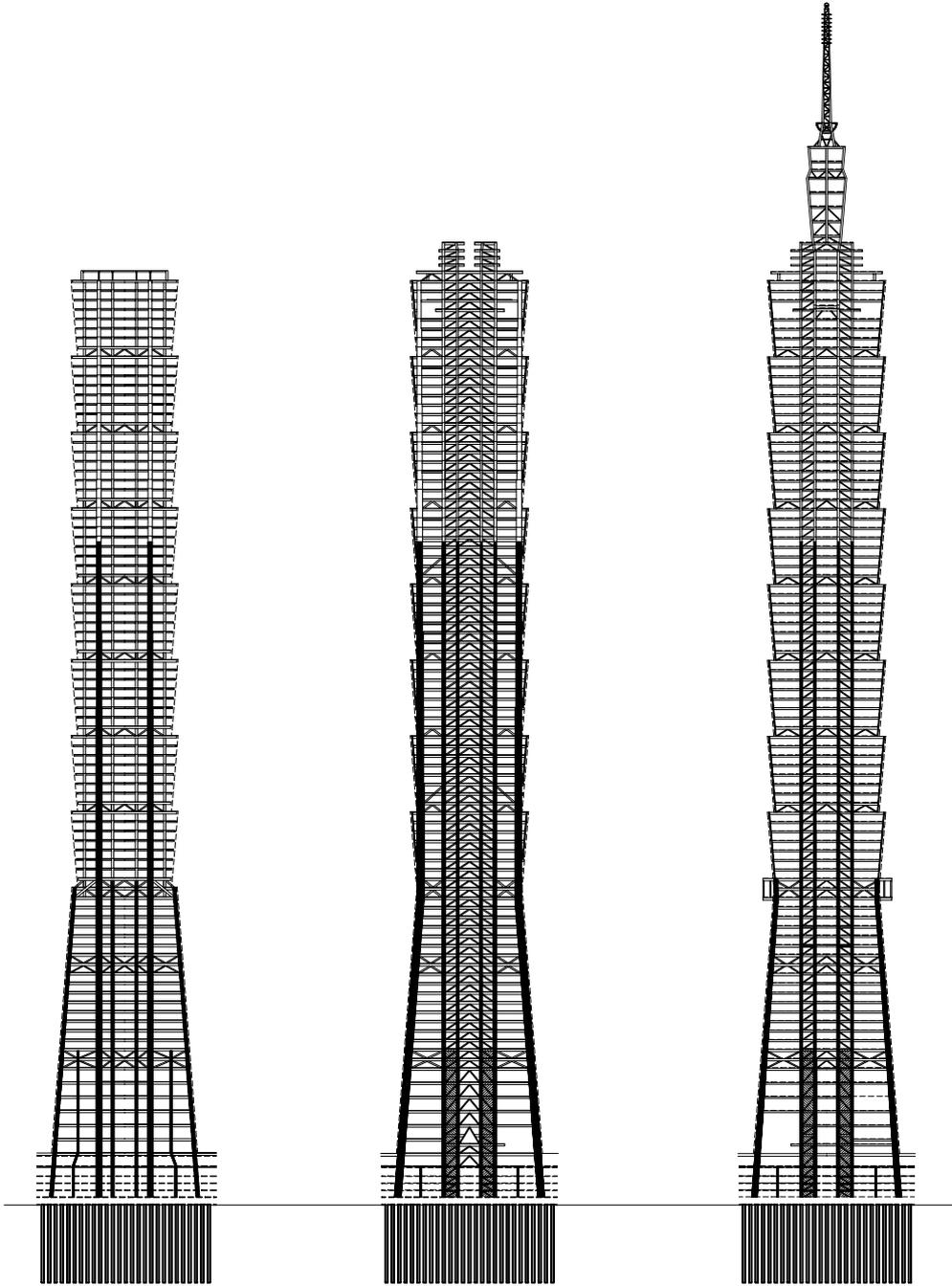


Fig. 2 Frame Elevations

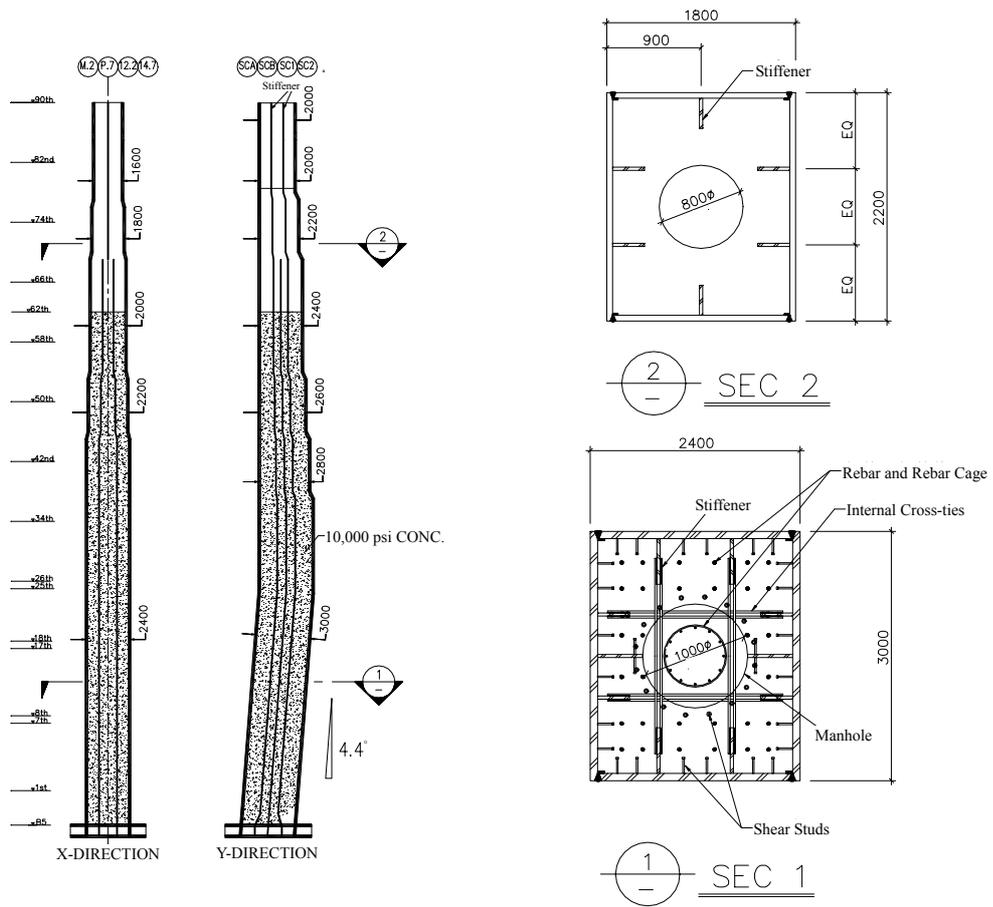


Fig. 3 Section through the Super-Columns

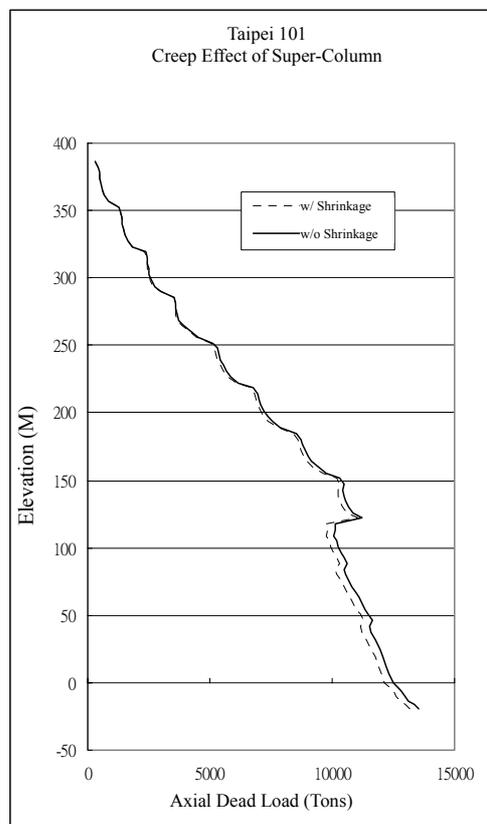


Fig. 4 Column Loads with and without Creep