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# FULL PRECAST STRUCTURE WITH UNBONDED POSTTENSION PRESTRESSED HYBRID FRAME STRUCTURES AT THE TAMANSARI HIVE OFFICE PARK BUILDING, JAKARTA, INDONESIA

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## ABSTRACT

*The need for high rise buildings in big cities like Jakarta is very urgent right now. Requirements regarding the quality of concrete, speed and ease of implementation have become demands. The Tamansari Hive Office Park is designed to meet these terms and conditions. This building consists of 3 basement floors and upper structure of 12 stories. The basement and shear wall structures are constructed from cast in place conventional concrete. While the top structure uses precast components for floor plates, beams and columns. This paper will describe the shape of beam, column and floor modeling in precast system structures. Indonesia is one of areas affected by earthquake events. Thus, earthquake load is a problem to be considered. Design of earthquake resistant buildings follows the provisions in Building Requirements for Structural Concrete (ACI 318-11), Indonesian Earthquake Resistance Design Procedures for Building and Non Building Structures (SNI 1726 – 2002) and some related regulations, particularly design regulations concerning precast buildings. The earthquake-resistant concept of this building does not use the concept of strong columns weak beam as earthquake absorbers, but uses the concept of self centering as described in the PRE cast Seismic Structural System (PRESSS). This concept is implemented with Unbonded Post-*

*Tensioned Precast Concrete for Special Moment Frames that connect beams and columns which meets the requirements of the ACI T1.2-03 Special Hybrid Moment Frames Composed of Discretely Jointed Precast and Post-Tensioned Concrete Members and ACI 550.3M-13 Design Specification for Unbonded Post-Tensioned Precast Concrete Special Moment Frames Satisfying ACI 374.1. In addition, the beam is also connected to a column with normal reinforcement as a weak connection. Meanwhile the column to column connection uses grouted splice sleeve. The diaphragm has important uses in the distribution of seismic forces, so that elements such as beams and columns accept the forces according to their rigidity. Hollow core slabs are one-way slab elements that are sheet-shaped and have a certain size. The elements must be arranged into unity, in order to function as a rigid diaphragm. This paper explain the using a method of construction without scaffolding for this project. This method will speed up the installation of precast components and avoid work disruption at under the floor. While the column to column connection using grouted splice sleeve.*

**Keyword:** Precast, Self Centering, PRESSS, Unbonded Posttension Prestressed, Hollow Core Slab, Unshoring, Diaphragm.

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## 1. INTRODUCTION

The Tamansari Hive Office Park is an office building located in Jakarta, Indonesia. This building consists of 3 basement floors and 12 floors for the upper structures. The structures of basement and shear walls are constructed from local casting conventional concrete. While its upper structures use precast components for floor plates, beams and columns. The purpose of using the pre-cast system is to speed up the implementation, and maintain quality. The narrow location also becomes a consideration for the use of precast system. To accelerate the implementation of this building, strategic steps are taken namely a) using Hollow Core Slab with spans of ± 8 meters, b) not using scaffold on beams to get free space under floor. Beams are seated on corbel which is expected to be strong when they gets vertical loads during construction, c) columns to columns using large-tolerated grouted splice sleeve, making it easier to mount columns, d) connection between columns is not positioned on the beam-column connection to keep the quality of concrete in the area and avoid complexity during installing due to tight reinforcement.

This earthquake-resisting structure system uses PRE-cast Seismic Structural System (PRESSS) by applying Unbonded Post-Tensioned Precast Concrete Special Moment as required on ACI 374.1-05 [4] and ACI 550.3M-13 {7]. This system uses the principle of self-centering in which lateral deformation will be restored by unbonded post-tensioned prestressed.

To distribute lateral loads to the structures of portal and shear walls, diaphragm is required on the floor. In the conventional structure, the diaphragm is a rigid massive floor. While the structure with the HCS floor requires a special solution by combining the sheets of HCS into a single unit.

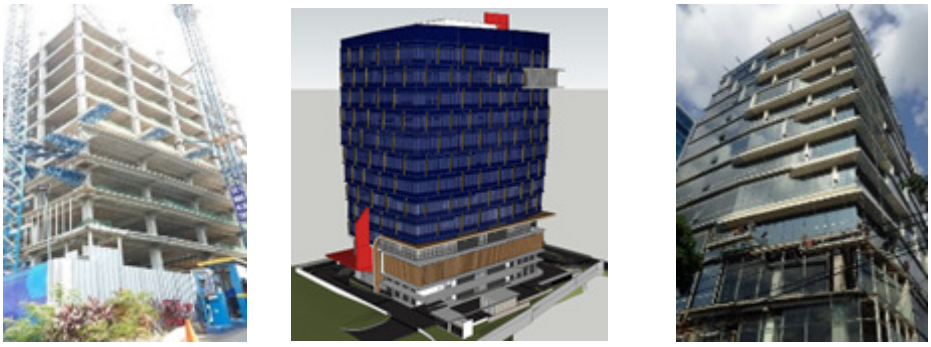


**Figure 1** Tamansari Hive Office Park Building, Jakarta [17]

## 2. PRECAST STRUCTURE

### 2.1. Floor

The original floor design of this building is a local cast plate reinforced with secondary beams. The plates are designed as two-way slabs. The shortages of this system are that it requires the formwork and scaffolding to be installed continuously at least 3 floors and requires a long time for concrete maintenance.

**Figure 2** Conventional building plan [17]

Figure 2 shows the existence of secondary beams that can cause complexity in creating formwork, especially when there are differences in the dimensions of crossing secondary beams. To accelerate and facilitate the implementation, it needs another alternative. One of the alternatives that can be applied is the use of Hollow Core Slab (HCS). HCS is installed at the one same direction, so it is called one-way slab. By being installed at one direction, then workers can focus more on two sides than two-way slabs that should focus on 4 sides, making it easier to install HCS.

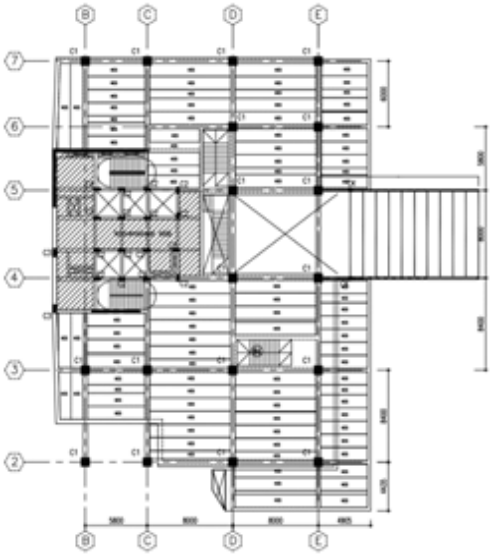


**Figure 3** Building Plan with Hollow Core Slab [17]

HCS installation in one direction as shown in Figure 3. The mounting direction does not always have to follow the pattern as the image in question. As for example, installed in perpendicular direction to the previous pattern. Benefits obtained when using HCS namely it does not require secondary beam to stiffen floor, does not require formwork and scaffolding, does not require long time for concrete maintenance, can use long span so as to reduce secondary beams, can function as Rigid-Diaphragm through interaction between wet joints and HCS and beams; and long spans will increase productivity in HCS installation, thereby shortening the construction implementation.

## 2.2. Beam

Beam is designed as half beams with dimension of 400 × 550 mm (precast) as shown in Figure 4. This concept aims in order to meet the concrete monolithic property. The shape of beam is formed like an invert T to give splace for HCS holder. After casting, overtopping becomes 150 mm, the theoretical size of the beam becomes 700 mm. At the time of construction, beam was seated on corbels, so it does not require scaffolding.

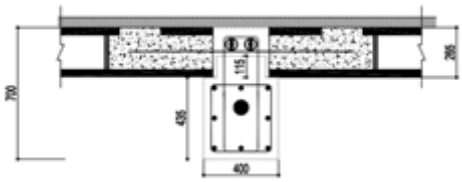


**Figure 4** Connection between Hollow Core Slab to precast beam

## 2.3. Column

The dimensions of the columns are 700 × 700 mm and 800 × 800 mm. Column is equipped with corbel as a place for beam. The meeting of columns is planned not as local beam-column joint. It is intended that beam-column joint to be intact, as they are made in pre-cast way. Inter-column joint is not positioned along with beam-column joint, but it is placed at a certain distance from the floor elevation. That distance can be taken between 20 - 50 cm. With this method, joining columns becomes easier.

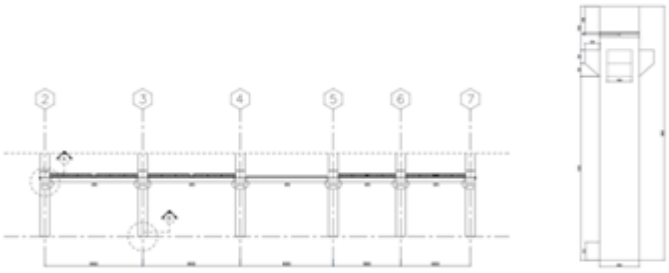


**Figure 5** Precast column with corbel. [17]

# 3. EARTHQUAKE RESISTANT BUILDING SYSTEM

## 3.1. Seismic Loads

The original design of this building refers to SNI 03-1726-2002, Earthquake Resistance Planning Procedures for Buildings, because this design has been completed, SNI 1726 - 2012 on Earthquake Resistance Planning Procedure for Building Structure and Non-Building. The location of buildings in Jakarta falls into zone III area based on the Map of Indonesia Seismic Zone refers to SNI 1726 - 2002.

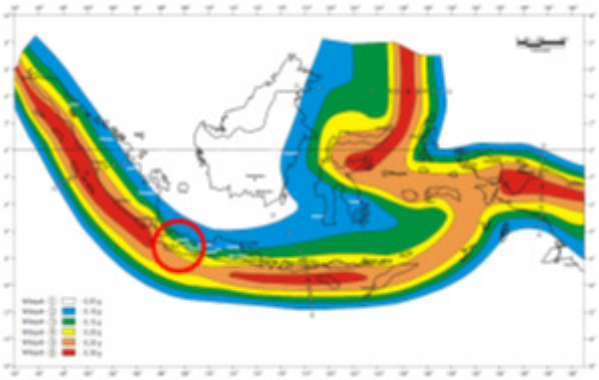


**Figure 6** Seismic Zone with Peak Ground Acceleration (PGA) at 500 years return period for Tamansari Hive Office Park Building Indonesia. [9]

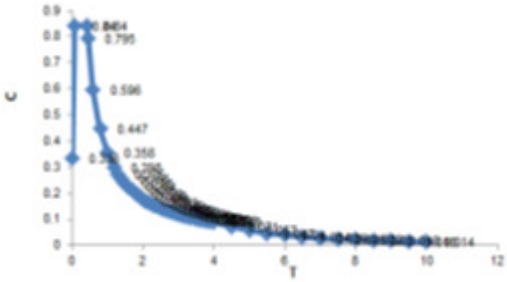



**Figure 7** Design of Site Response Spectrum for Tamansari Hive Office Park Building Indonesia.[17]

This building uses dual system consisting of 1) space truss that resists the entire gravity loads, 2) the lateral load resisting in the form of shear wall with moment-resisting frame. The moment-resisting frame should be planned separately to be able to bear at least 25% of all lateral loads; 3) both systems should be planned to bear jointly all lateral loads by taking into account multiple interactions/systems. Reinforced concrete is designed as SMRFS (Special Moment Resisting Frame System) with Reduction Factor (R) = 8.5. Priority factor (I) minus 1.0, because it serves as an office building.

To anticipate the eccentricity between the center of mass and the center of rotation as well as the additional eccentricity, a) for 0 < e < 0.3 b, ed = 1.5 e + 0.05 b or ed = e - 0.05 b and selected between the two of which has the most decisive influence, b) e > 0.3 b, ed = 1.33 e + 0.1 b or ed = 1.17e – 0.1 b or and is selected between the two which has the most decisive influence. b is the largest horizontal measure of building structure plan on that level floor, measured perpendicular to the direction of earthquake loading.

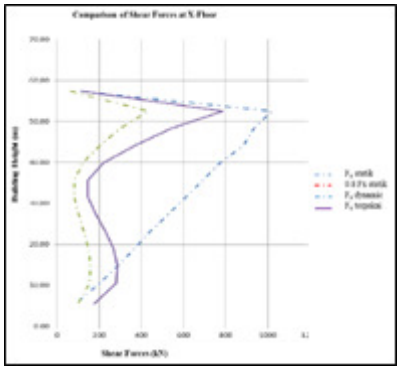


**Figure 8a** Comparison of shear forces at X-dir.[17]

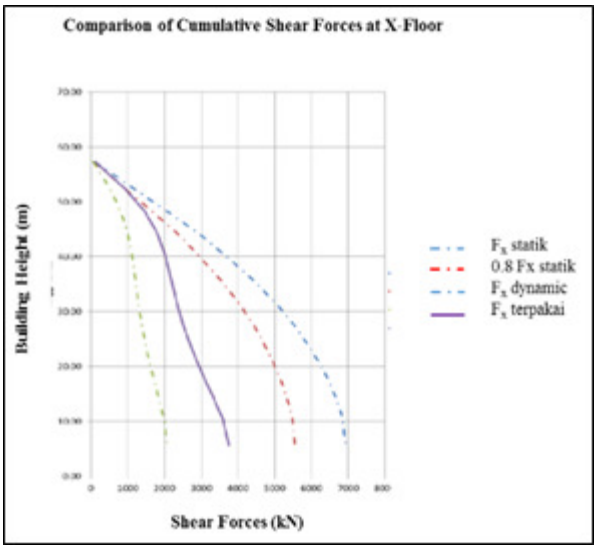



**Figure 8b.** Comparison of Cumulative shear forces at X-dir.[17]

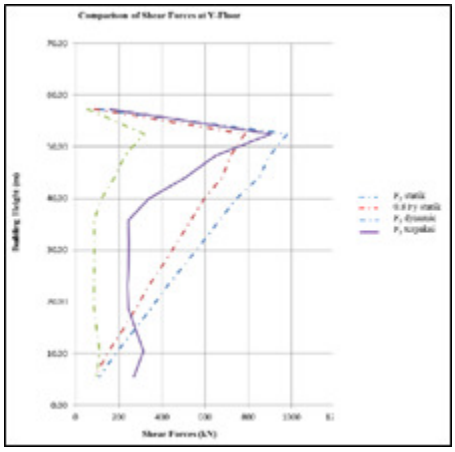


**Figure 8c** Comparison of shear forces at Y-dir.[17]

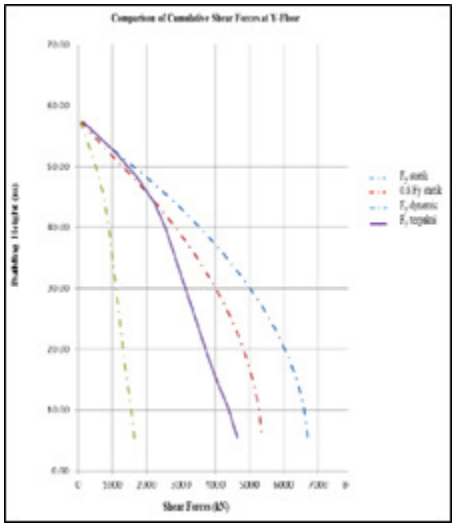


**Figure 8d** Comparison of Cumulative shear forces at Y-dir.[17]

The dual system requires at the resisting frame for bearing at least 25% against the seismic load of the observed direction.

**Table 1** Contribution of the shear force between Shear Wall and Frame [17]

| | Total Shear Forces(kN) | | Shear Wall (kN) | | Frame (kN) | |
|---|---|---|---|---|---|---|
| X | 2392 | | 1927 | | 465 | |
| Y | | 1887 | | 1096 | | 791 |
| % | 100% | | 81% | | 19 | |
| | | 100% | | 58% | | 42% |

Based on table 1 above, in the X direction the frame contribution to the seismic load is 19% <25% (multiple system requirements). For that, the static shear force of X direction multiplied by 25/19 = 1.316. This does not apply to Y direction, because the contribution of this portal is 42% which is already more than 25%. This provision will affect the design of beam and column reinforcement. To avoid flexible structures, the value of natural vibration time of fundamental building structure is limited. $T_1 < \zeta.n$ where the coefficient of $\zeta$ is set according to Table 2 (Table of SNI 03-1726-2002).

**Table 2** $\zeta$ Coefficient that limits the natural vibration time of the fundamental building structure [9]

| Seismic Zone | $\zeta$ |
|---|---|
| 1 | 0.20 |
| 2 | 0.19 |
| 3 | 0.18 |
| 4 | 0.17 |
| 5 | 0.16 |
| 6 | 0.15 |

$\zeta.n = 0.18 \times 12 = 2.16$ seconds

T1 direction X = 2.166 seconds $\approx \zeta.n$

T2 direction Y = 1.751 seconds $< \zeta n$

### 3.1. Service Performance Limits of X Direction

#### 3.1.1. Service Performance Limits

The performance of service limit is determined based on the inter-level deviation. The maximum limit of inter-level deviations is 0.03 H/8.5 or 30 mm, taken the smallest value. H is the height between floors.[9]

#### 3.1.2. Ultimate Performance Limits

The performance of ultimate limits is determined based on the inter-level deviation calculated from the deviation of the building structure due to the nominal of seismic loading, multiplied by a multiplier factor as follows: 1) for a regular building structure: $\xi = 0{,}7\ R$, 2) for irregular building structures: $\xi = \frac{0.7\ R}{Scale\ Factor}$, where R is the seismic reduction factor of building structure and the Scale Factor is $\frac{0.8V_1}{V_t} \geq 1$, where $V_1$ is the nominal shear force as the first dynamic response only and $V_t$ is nominal of shear force obtained from the analysis of various spectrum responses that have been performed. For the state of this building, with R = 8.5, then $\xi = 0{,}7 \times 8.5 = 5.95$.[9]

## 4. PRESSS SYSTEM

This building is designed with PRESSS concept. PRESSS is an earthquake resisting structure system based on the principle of self-centering, a structure when gets lateral force will be deformed to a certain distance and then will return to its original position. With self-centering, seismic energy in the structure will be muffled. This self-centering can be achieved by implementing unbonded post-tensioned mounted centrically on cross-section of beam and weak reinforcing installed on the fiber as energy dissipater.

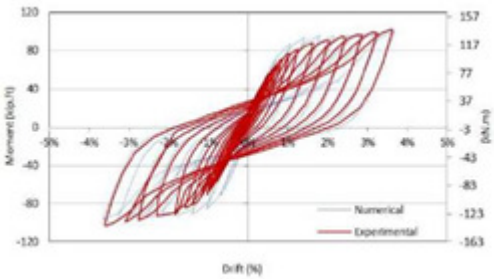



**Figure 9a.** Hysteresis Loop Rigid Frame

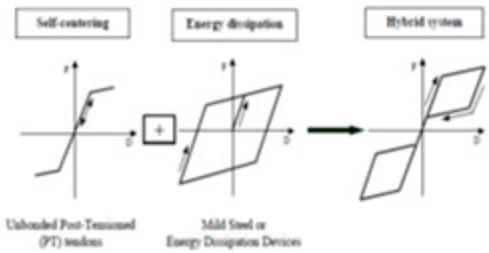


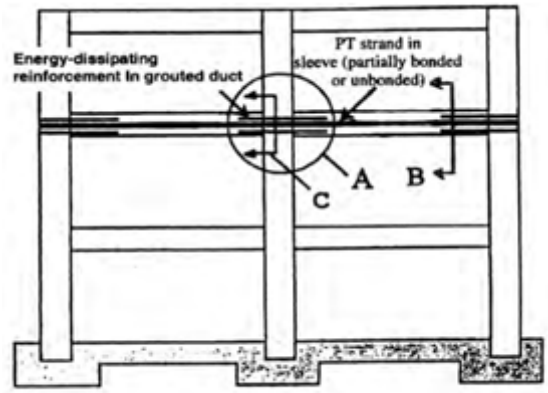

**Figure 9b.** Hysteresis loop Unbonded Post-Tensioned

Hysteresis loop at Unbonded Post-Tension are different to hysteresis loop with conventional structure (rigid frame). Conventional moment resisting frames are designed with so-called strong-column weak-beam criteria to dissipate seismic energy by forming plastic hinges at the beam ends and at the base of the first story columns to form a global beam-sway mechanism

(Figure 9a.). Although the structure is expected to behave in a ductile manner and dissipate a significant quantity of seismic energy, there may be significant inelastic residual deformation.

While on the Unbonded Post-Tensioned, structure is capable of deforming to form graphs such as butterflies (Figure 9b). At 0% drift, frame does not lose its ability. The greater the force at 0% drift, the structure has the ability to absorb seismic energy. Self-centering concrete moment frames consist of concrete beams and columns horizontally post-tensioned together so that a gap can open at the beam-column interface when subjected to a specific applied moment. Energy dissipation is supplied at the beam-to-column joint through a variety of mechanisms such as unbonded mild reinforcing steel, friction damping elements and other devices.

Self-centering seismic lateral force resisting systems are capable of fully re-centering when the lateral forces are removed (herein referred to as full self-centering), eliminating residual drift. The flag-shaped hysteresis demonstrates self-centering ability since the displacement returns to negligible values when the lateral forces are removed; however, the system should be proportioned such that this self-centering can occur. (Figure 9b).

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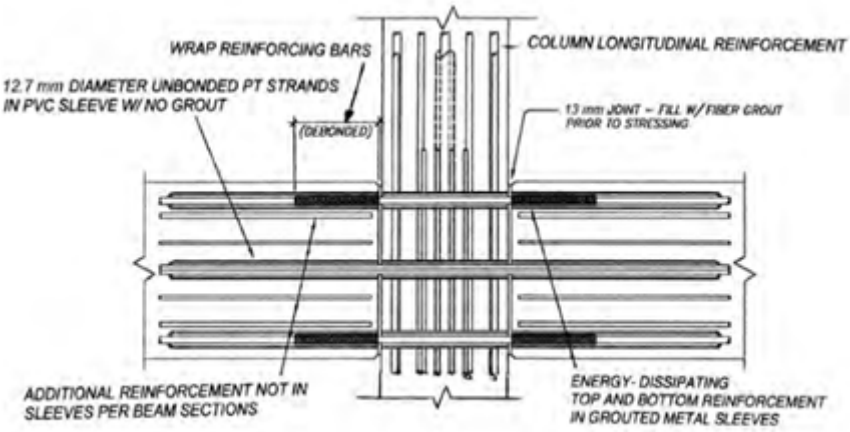


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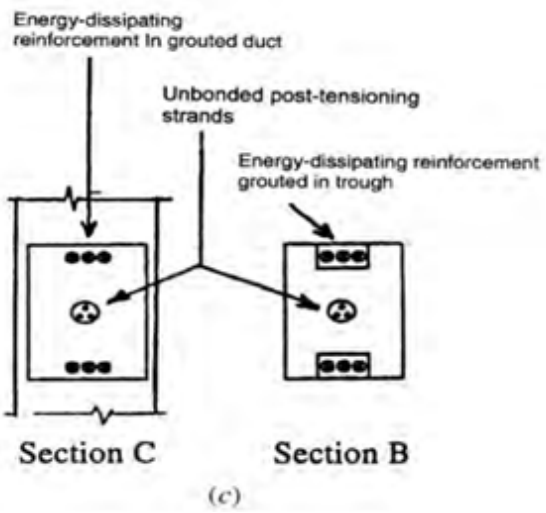


**Figure 10.** Typical moment frame composed from discretely jointed precast concrete members: (a) elevation of typical interior moment frame; (b) detail of connection—A; and (c) typical sections—B and C.[5, 8]

## 4.1. Calculation of unbonded post-tensioned and dissipater reinforcement.

Minimum Prestressed Force $A_{ps}.f_{se}$ should be taken [7]

$$A_{ps} = \frac{(1.2V_D + 1.6V_L)}{\emptyset\mu} \quad (1)$$

where $V_D$ is shear force due to dead load, $V_L$ is shear force due to live load, $\phi$ is strength reduction factor and $\mu$ is the friction coefficient (0.6).

Under earthquake action, the shear demand at the beam-column interface is a function of both the gravity loads acting on the precast beam and the earthquake moments induced in it.

$$V_u = 0.75(1.2V_D + 1.6V_L) + \frac{(M_{pr1} + M_{pr2})}{L_{clear}} \quad (2)$$

where $M_{pr1}$ and $M_{pr2}$ are $M_{pr}$ for opposite ends of the deforming precast beam, $L_{clear}$ is the face-to-face distance between columns.

### *4.1.1. Energy-dissipating reinforcement*

The yield force in the energy-dissipating reinforcement: [7]

$$A_s f_y \geq \frac{V_D + V_L}{\emptyset} \quad (3)$$

where $\phi$ is the strength reduction factor for shear and equals 0.85 in accordance with 9.3.4(c) of ACI 318M-11.

Where the energy-dissipating reinforcement is properly anchored in both the column and the precast beam, and the width of the joint at the interface is relatively small, the shearing yield strength of a reinforcing bar is approximately half its tensile yield strength. The performance requirement of Section 5.4.5(b) can be met by satisfying the condition

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$$\frac{(A_s f_y + A'_s f_y)}{2} = A_s f_y = \frac{V_D + V_L}{\emptyset} \quad (4)$$

The probable flexural strength $M_{pr}$ is the sum of the contributions from the energy-dissipating reinforcement $M_s$ and the post-tensioned reinforcement $M_{prs}$.

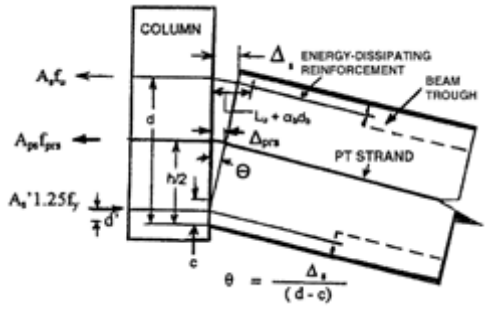
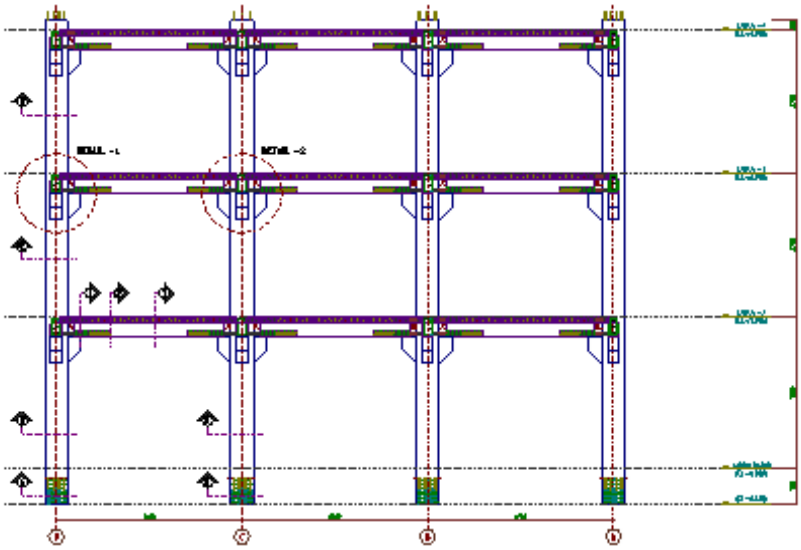



**Figure 11** Rotation at beam-column interface



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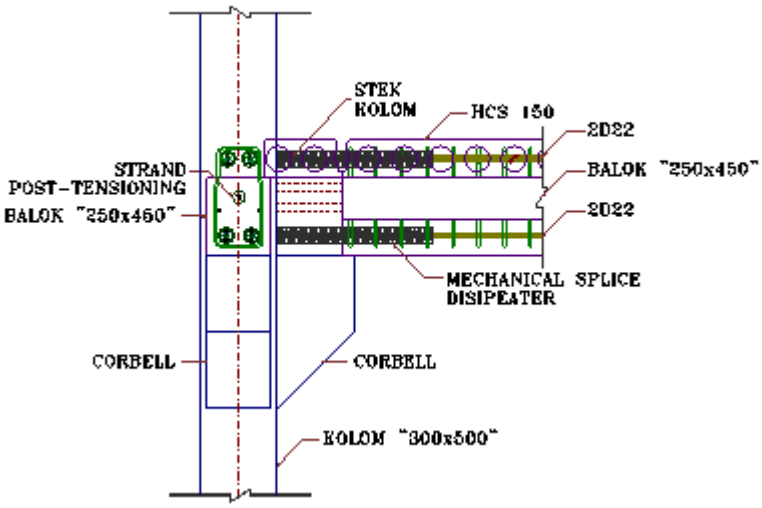


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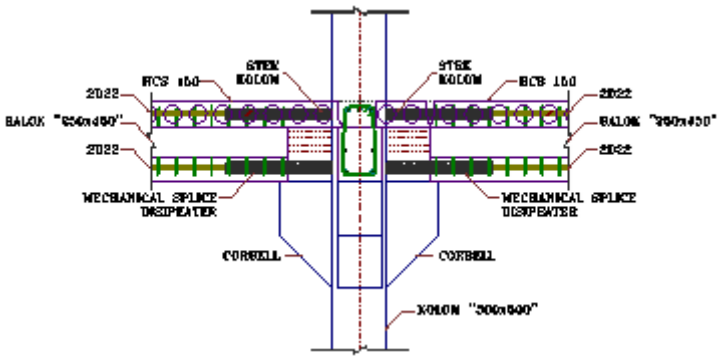



(c)

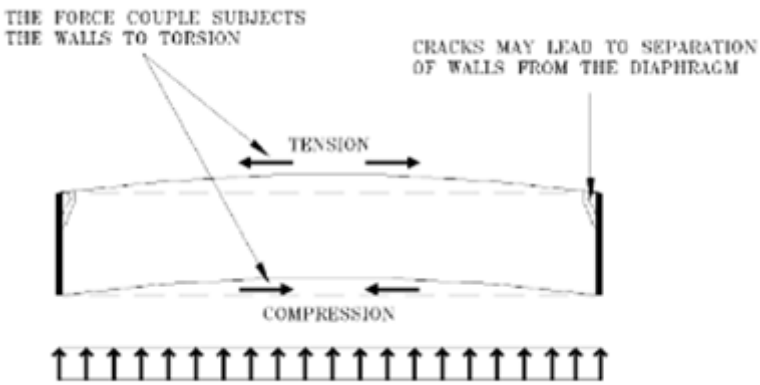
**Figure 12** Detail of beam-column connection for Tamansari Hive Office Park Building [17]

## 4.2. Diaphragm

The primary function of floor and roof systems is to support gravity loads and to transfer these load to other structural members such as columns and walls. Furthermore, they play a central role in the distribution of wind and seismic forces to the vertical elements of the lateral load resisting system.

In the earthquake resistant design of building structures, the building is designed and detailed to act as a single unit under the action of seismic forces. Design of a building as a single unit helps to increase redundancy and intregrity of the building. The horizontal generated by earthquake excitations are transferred to the ground by the vertical systems of the building which are designed for lateral load resistance. (e.g. frames, bracing, and walls). These vertical systems are generally tied together as a unit by means of the building floors and roofs. In the sense, the floor/roof structural systems, used primarily to create enclosures and resist gravity (or out of plane) load are also designed are horizontal diaphragms to resist and transfer horizontal (on in plane) load to appropriate vertical elements.

The analysis and design of the floor of roof deck under the influence of horizontal load is performed assuming that the floor or roof deck behaves as a horizontal continues beam supported by vertical lateral load resisting elements (hereafter referred to as VLLR elements). The floor deck is assumed to act as the web of the continuous beam and the beams at the floor periphery are assumed to act as the flange of the continuous beam (Figure 13).

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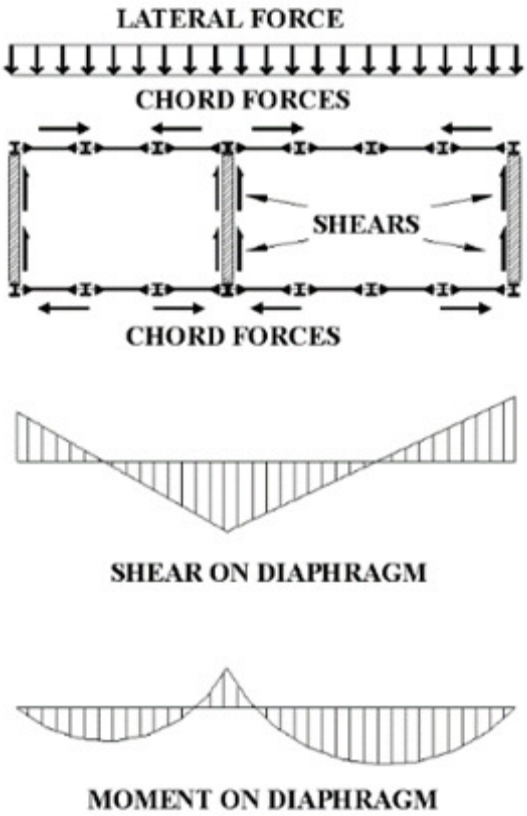


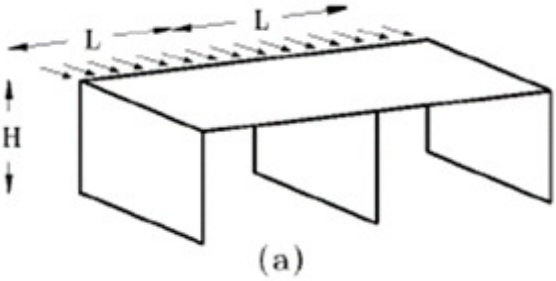

(b)

**Figure 13** Bow action subjects the end walls to torsion.[13]

The floor of the diaphragm can be modeled as a rigid or semi-rigid or flexible floors. The rigid diaphragm floor has infinite rigidity. In this model, every point at a given floor level will move together when the structure gets lateral load (Figure 14b). The distribution of lateral forces based on the rigidity of vertical elements such as columns and shear walls.

On the diaphragm floor, the semi-rigid of diaphragm is taken into account in structural analysis, so that the lateral load distribution is based on the rigidity of diaphragm floor (Figure 14b). The software structure can be used to help the calculations.

On flexible diaphragm floor, the diaphragm span is considered as the simple span. Deformation of the points due to floor flexibility will lead to inner forces on the perimeter beam (Figure 14c).



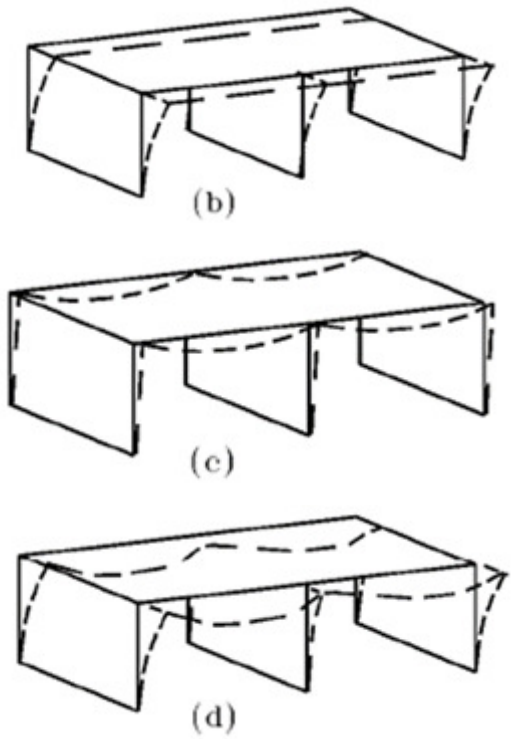



**Figure 14.** Lateral deformation of floor diaphragm [13]

The semi and flexible rigid models are applied to a) irregular structure , b) structure with set back, c) torque exceeding the provisions, d) structures having ratio of height to width building (H/B ratio) >3, and e) structures using precast flooring.

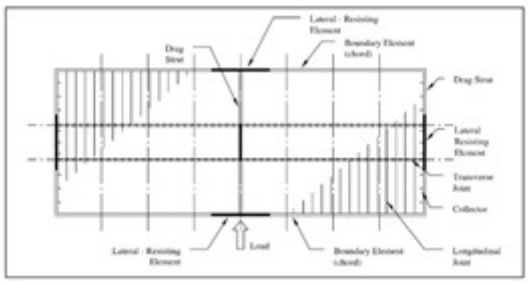
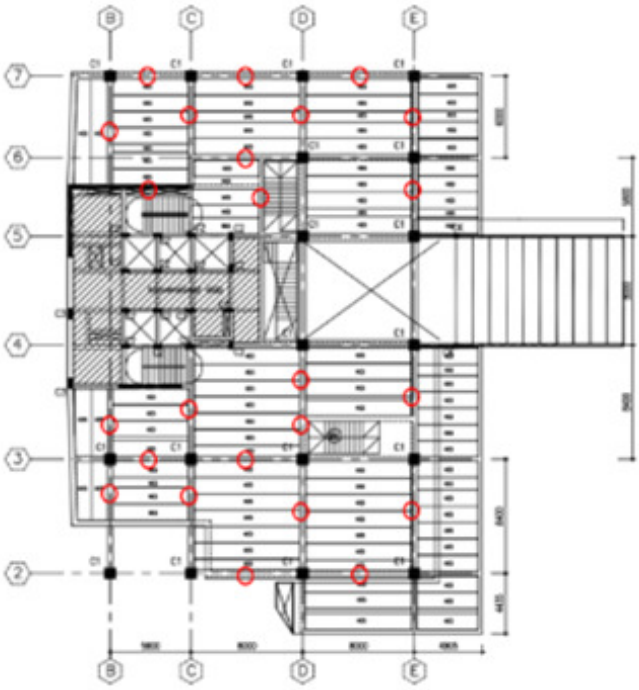
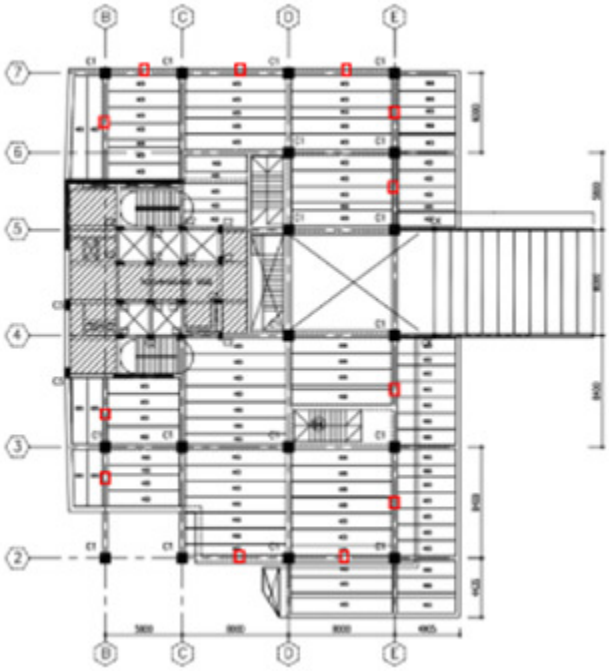



**Figure 15** Elements of floor diaphragm as earthquake resistance component[13]

1) **Boundary Element** is edge element around the diaphragm or around the openings on diaphragm that bind the diaphragms together. The boundary element can serve as a chord or drag strut. 2). **Chord** is a wing element of diaphragm that is experience tensile or compressive to achieve integrity when the diaphragm is flexible. 3) **Drag Strut** is an element to drag the lateral load onto lateral load resisting element and to distribute shear that exceeds the length of diaphragm strut. 4) **Collector** is an element that moves shear force from the diaphragm to lateral load resisting element. 5) **Longitudinal Joint** is the parallel joint of span to slabs. 5) **Transverse Joint** is a perpendicular joint of span to slab.



Boundary Element



Chord

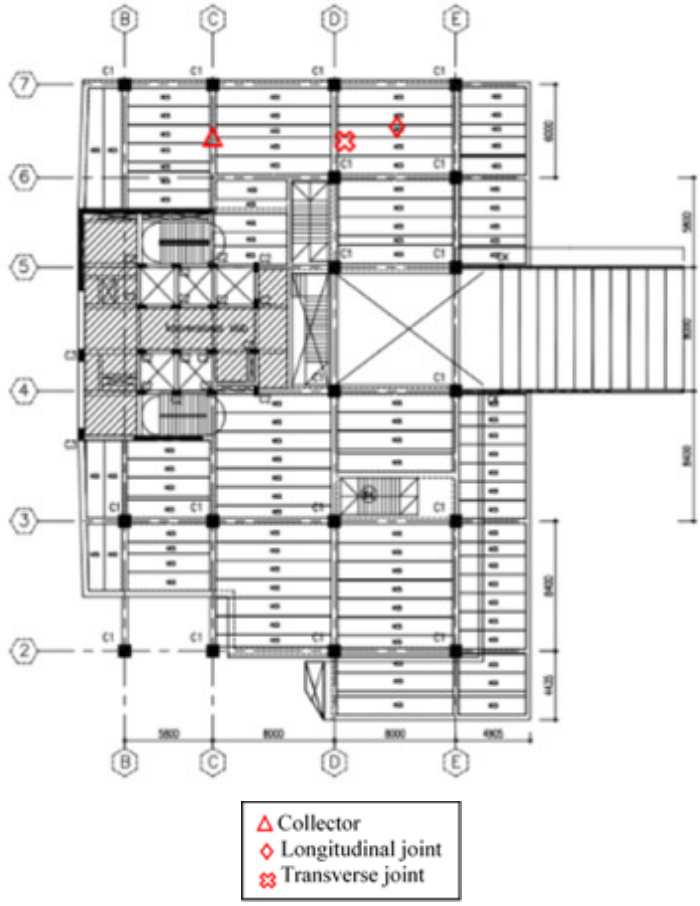



**Figure 15** The parts of floor diaphragm [17]

### *4.2.1. Diaphragm Design Force [9, 10, 12]*

The floor and roof diaphragms should be designed to bear the seismic force designed from structural analysis based on the equation below:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_{px}} w_{px} \quad (6a)$$

where $F_{px}$ is a diaphragm design style, $F_i$ is design force applied at $i$ level, $w_i$ is a tributary of weights up to $i$ level and $w_{px}$ is the tributary weight up to diaphragm at $x$ level.

The forces in (6a) equation cannot be less than:

$$F_{px} = 0.2 S_{DS} I_e W_{px} \quad (6b)$$

and cannot be exceeded

$$F_{px} = 0.4 S_{DS} I_e W_{px} \quad (6c)$$

### *Longitudinal Joints [13]*

Grounded keyways between slabs that have capacity to divert longitudinal shear forces from one slab to another. Taking into account the shear stress of 80 psi (0.55 MPa), then the strength of the elongated shear threshold is:

$$\phi V_n = \phi(0.08) h_n . l \text{ (kips)} \quad (7a)$$

$$\phi V_n = \phi(0.55) h_n . l \text{ (N)} \quad (7b)$$

where $h_n$ is the net height of grout key (in inch or mm), $l$ is the lenght of grout key joint (in inch or mm) and $\phi$ is taken by 0.85.

If grout strength is exceeded, or ductility is required, longitudinal joint reinforcement is installed (Figures 16a and 16b). Longitudinal reinforcement is determined by the formula:

$$A_{vf} = \frac{V_u}{\emptyset f_y \mu} \quad (8)$$

where $V_u$ = *factored applied shear*

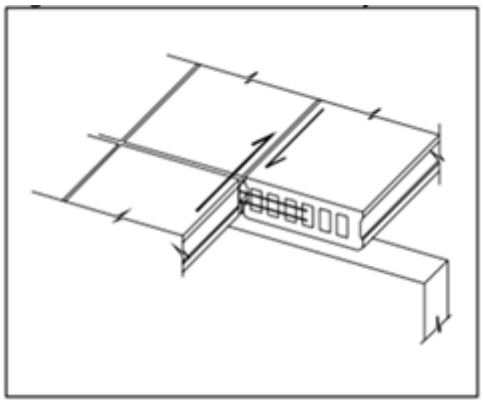


**Figure 16a.** Shear friction steel in butt joint [13]

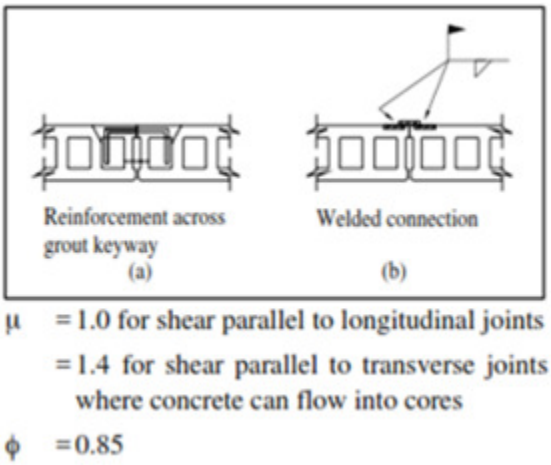



**Figure 16b.** Alternate longitudinal shear connections [13]

### *4.2.3. Transverse Joints*

The functions of transverse joints are a) Shear reinforcement for longitudinal joints, b) working forward the axial tensile or compressive forces that carry loads on the diaphragm to the lateral load resisting element, c) as a chord rod where tensile strength due to flexure is retained, d) separating the horizontal beam body where horizontal shear must be transferred to maintain the height of the composite diaphragm.

The tensile force on the chord is borne by reinforcement which gives bending strength to the diaphragm.

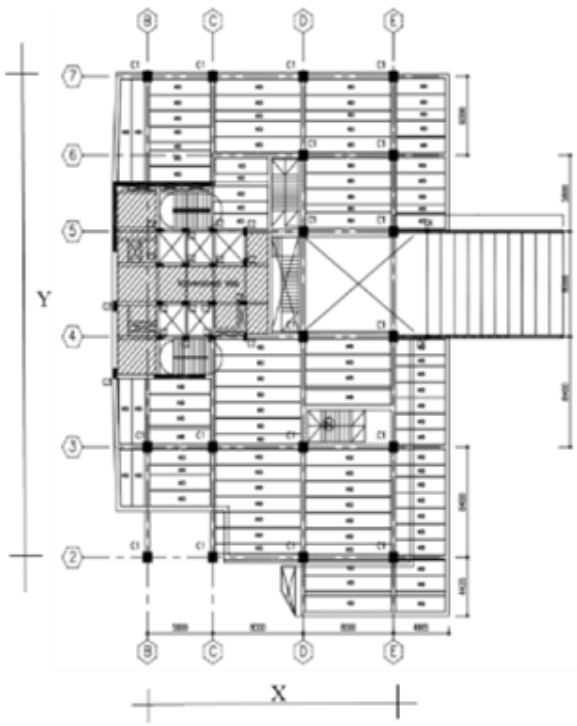



**Figure 17** Length of arm moment for building plan

$$A_{sx} = \frac{M_{ux}}{\emptyset 0.8 x f_y} \quad (9a)$$

$$A_{sy} = \frac{M_{uy}}{\emptyset 0.8 y f_y} \quad (9b)$$

where $M_{ux}$ is the moment of x direction on the diaphragm, $M_{uy}$ is the moment of y direction on the diaphragm, x and y are the respective width of building on the reviewed direction and $f_y$ is the melt tension of the reinforcing steel.

The shear reinforcement of the boundary element can be calculated by the following formula:

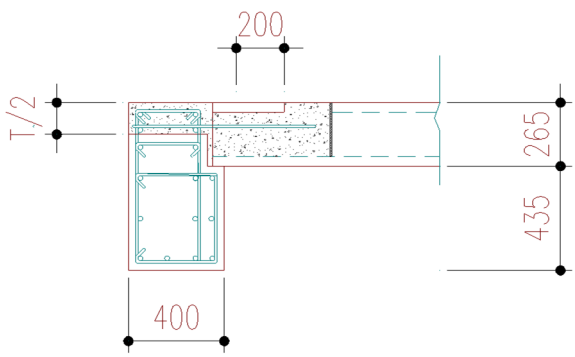
$$V_h = \frac{V_u Q}{I} \quad (10a)$$

where Q is the first moment of area.

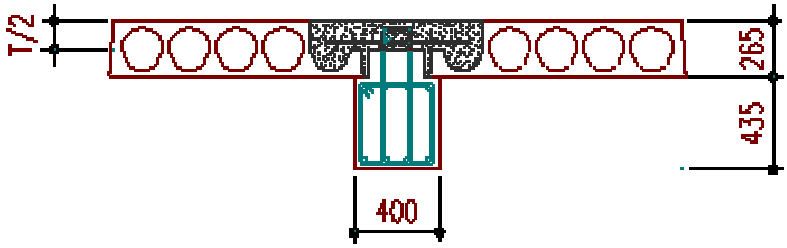
or

$$V_h = \frac{M_u}{0.8h} \quad (10b)$$

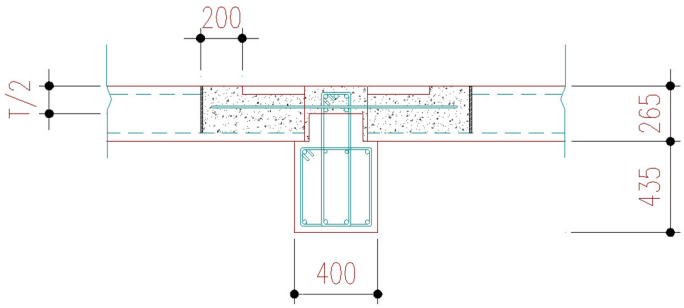
where $h$ is $x$ or $y$, depends on the reviewed direction.




(a)




(b)



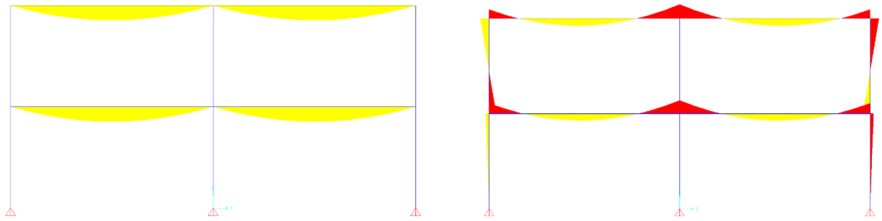

(c)

**Figures 18** Detail of chord, longitudinal and transverse reinforcement

## 5. CONSTRUCTION METHOD

The precast construction process of this building consists of 1) columns installation, 2) beams installation and 3) Hollow Core Slab (HCS) installation.

1. The column installation includes placement of column in proper axis, elevation and vertical control and grouting work. The placement of columns in proper axis is done by giving the marking on the floor and the column itself. The elevation control is performed by the installation of the shim plate, while the vertical control is obtained by measurement using theodolite. Grouting work on the column is very important in the installation of the column, because this process confirm the column position. Grouting also gives strength to column joints. There are 2 commonly selected methods for grouting work: a) the column sits on shim and mortar plate, and grouting is inserted into the splice sleeve holes, b) the column sits on shim plate, then the gap between mounted columns and column underneath it is mounted with formwork around it. Grouting injected into splice sleeve will fill the gap and the splice sleeve itself.
2. Installation of precast beam (half-beam) including beam elevation control, unbonded prestressed steel cables installation, dissipater reinforcement installation, beam-column joints casting, and stressing work. The work continued with overtopping on precast beam. Precast beam sits on top of corbel, so it does not require scaffolding during installation. Corbel is designed to bear its own dead loads of precast beams ($DL_{Beam}$), its own dead load of HCS ($DL_{HCS}$) and live load of construction ($LL_{construction}$). At this stage, the beam pedestal is the joint pedestal (Figure 18a), so that the loads are already loaded on the precast beam. After overtopping, the beams become full beams (Figure 18b). At this stage, the column-beam joint becomes a fixed joint. The working loads are dead load plus service live load and seismic/wind loads. With this principle, the reinforcement on beam pedestal can be saved.

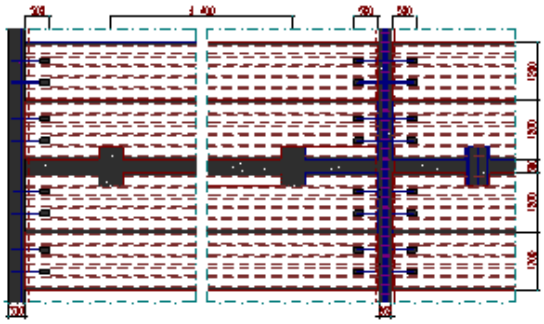


(a) (b)

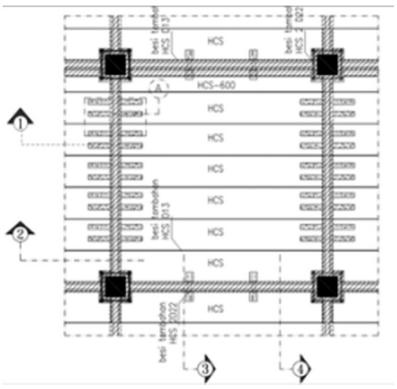
(c)

**Figures 19** Construction stage at precast installation [17]

3. Installation of HCS includes placement on precast beam, then installation of chord, longitudinal, transverse joints and negative reinforcements on HCS. HCS width is in uniform size by 1200 m, but not necessarily can cover the width of beam distance. Normally there will be gap that must be closed using local cast concrete as the closure. The last part of HCS installation is overtopping over the precast beam.



(a)

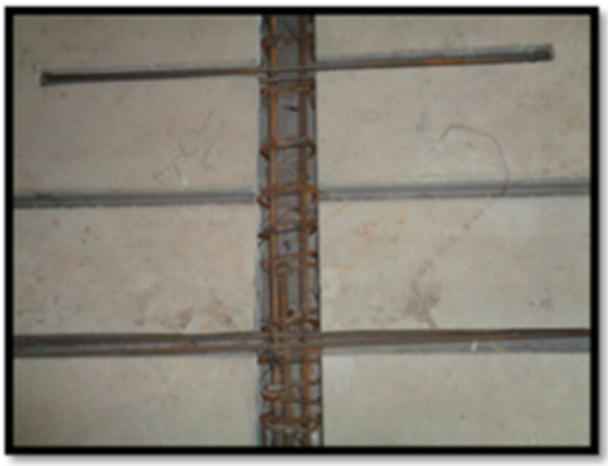


(b)

**Figures 20** Steel arrangements at floor diaphragm [17]

(a)

(b)



(c)

**Figures 21** Detail of steel reinforcments at floor diaphragm [17]

## 6. SUMMARY

1. Precast system can be applied to multi-storey building in seismic areas. Almost all building components such as floors, beams and columns can be made with precast system.
2. Tamansari Hive Office Park building uses PRESSS system to anticipate seismic load. This system is expected to reduce the damage to the building, because it uses the principle of self-centering obtained from the use of unbonded posttension prestressed.
3. Hollow core slabs should be designed as diaphragms that can channel the lateral forces to vertical elements (columns and shear walls). The main structure with HCS is taken into account as a semi rigid structure. This applies as a structure that uses precast flooring. HCS is integrated by installing reinforcements on gaps between HCS (longitudinal and transverse joints) and reinforcements around floor areas (tension and compression chords).
4. Hollow core slab is a one-way slab that simplifies installation, as there are only 2 installation limits, compared to two-way slabs that have 4 installation limits. With its long span, then high installation productivity will be obtained.
5. Installation of pre-casted beams without scaffolding can accelerate the construction and makes the floor seems roomy and clean.

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