SEISMIC DESIGN OF FLOOR DIAPHRAGMS IN PRECAST CONCRETE BUILDINGS. DESIGN EXAMPLE

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ABSTRACT

In typical seismic design procedures of precast concrete buildings, in the design of floor diaphragms is needed both a reliable evaluation of in-plane diaphragm forces and a design method including an appropriated load path. This paper shows examples of seismic design procedures for floor diaphragms for the case of a regular 20-story precast concrete building in a high seismic zone. In the evaluation of the in-plane diaphragm forces, a simplified procedure proposed by the authors is used. For the design of the floor diaphragms the stringer-and- panel method is used, which is not known among designers. This method is based in both equilibrium and compatibility and has the advantage of being design-oriented. Results found in this study are discussed and design recommendations are given in the paper

Keywords: Floors, Seismic Design, Buildings, Concrete, Precast, Diaphragms

INTRODUCTION

In typical seismic design procedures of buildings, in the design of floor diaphragms is needed both a reliable evaluation of in-plane diaphragm forces and a design method including an appropriated load path. Although these design procedures are needed in both buildings with cast-in-place floors and buildings with precast concrete floors, the latter are particularly vulnerable to the design of floor diaphragms because the jointed nature of precast concrete construction. This paper shows examples of seismic design procedures for floor diaphragms for the case of a regular 20-story precast concrete building in a high seismic zone. For the evaluation of in-plane diaphragm forces, several procedures are evaluated and discussed. These are the procedures specified by the Mexico City Building Code (MCBC, 2004), the ASCE 7-10 (ASCE 7-10, 2010), the First Mode Reduced Approach (Rodriguez et al., 2002), and a recent code proposal to BSSC for the seismic design of diaphragms.

For the design of the floor diaphragms of the building, the stringer-and- panel method (Blaaunwendraad and Hoogenboom, 1996) is used. This method is not well-known among designers. This method is based in equilibrium or also in equilibrium and compatibility with the former having the advantage of being design-oriented. Results found in this study are discussed and design recommendations are given in the paper. It must be mentioned that the Strut-and-Tie and the Finite Element methods are commonly used by designers when analyzing diaphragms in buildings subjected to seismic actions. The first method leads to a very conservative estimation of the steel reinforcement required in a diaphragm, and the second method is time consuming.

PRECAST CONCRET BUILDING CASE STUDY

This paper presents results of the analysis and design of a twenty-story office building showing in some detail procedures for the seismic design of floor diaphragms. The building is a concrete gravity frame structure whose lateral force resisting system is composed of reinforced concrete structural walls in the North-South direction along gridlines A, B, E, and F and in the East-West direction along gridlines 2 and 4 (see Fig. 1). As seen in Fig. 1, the building has several floor openings for elevators and stairs. The structure’s gravity system is composed of precast floor units spanning in the East West direction between reinforced concrete beams aligned along each gridline and spanning between reinforced concrete columns located at grid intersections (see Fig. 1). These precast floor units are formed by 8.5 m long prestressed tubular beams with 300 mm depth, at a spacing of 1000 mm, see Fig. 2. In addition, polystyrene vaults are seated on the tubular beams, and on top of both the tubular beams and the polystyrene vaults, a 60 mm concrete topping is cast in place, see Fig. 2.

The structure was designed following the MCBC (2004). Typical specified compressive and yield strengths for concrete and reinforcing steel were 40 MPa and 420 MPa, respectively.

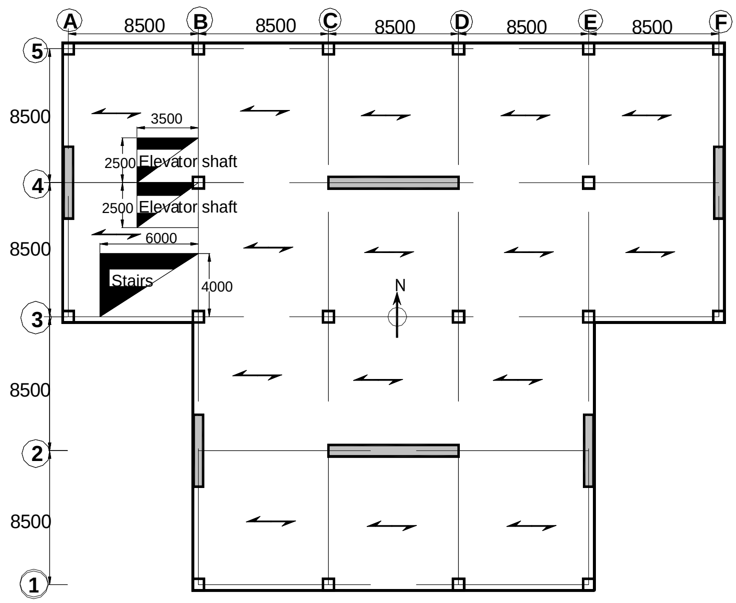

Figure 1 Plan of the building (dimensions in mm)

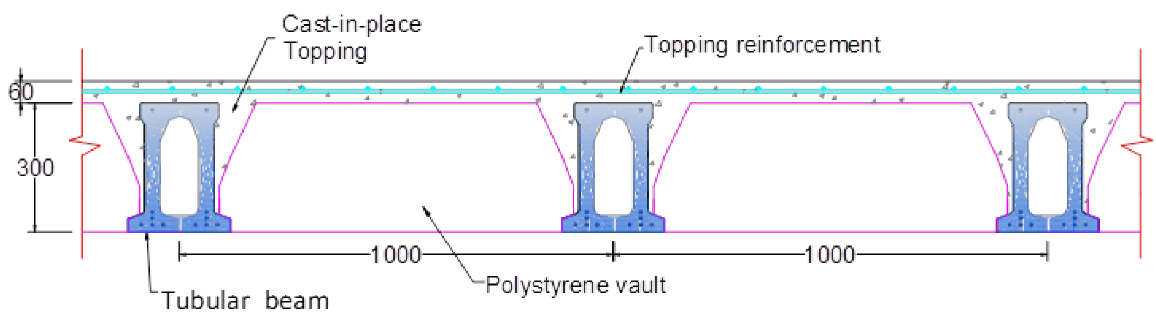

Figure 2. Typical details of precast floor (dimensions in

For typical floors design dead load was 5.8 kN/m^2, and specified live load for seismic design was 0.9 kN/m^2. The design reduced seismic coefficient was 0.108 which led to a design base shear equal to 28,200 kN. The calculated fundamental period of the structure in the North-South direction was 2.2 s. This direction is chosen here for the example of seismic design of floor diaphragms. The resulting transverse dimensions of RC columns were 1200mm x1200mm, and RC beams had a width and section height equal to 500 mm and 800 mm, respectively. RC walls in both directions were 400 mm thick.

EVALUATION OF HORIZONTAL FLOOR ACCELERATIONS FOR DETERMINING DIAPHGRAM DESIGN FORCES

ASCE 7-10 STANDARD (ASCE 7-10, 2010)

According to the ASCE 7-10 Standard, diaphragm seismic design forces, F_{px}, are calculated using the equivalent lateral forces, F_i, which are used for the design of the lateral force resisting system. Forces F_{px} are greater than forces F_i because the diaphragm design force F_{px} represents maximum values of diaphragm forces and they do not occur simultaneously. Forces F_{px} increase with floor level. Forces F_{px} are given by:

$$\frac{F_{px}}{w_{px}} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} \tag{1}$$

where n is the number of floors in the building. Force F_x is the design lateral force applied at level x. Weights w_i and w_x are the portions of the total seismic weight, W, corresponding to level i or x, respectively. Weight w_{px} is the floor weight at level x. The ratio F_{px}/w_{px} in Eq. (1) can be interpreted as the horizontal floor acceleration at level x divided by the acceleration of gravity, g. Forces F_{px} are limited to minimum and maximum values as shown by:

$$0.2 S_{DS} I_e < F_{px} / w_{px} < 0.4 S_{DS} I_e \tag{2}$$

where S_{DS} is the design, 5 percent damped, spectral response acceleration at short period as defined in Section 11.4.4 of ASCE 7-10, and I_e is the importance factor prescribed by ASCE 7-10. According to ASCE 7-10, Section 11.4.5, $S_{DS} = 2.5 PGA$, where PGA is the peak ground acceleration measured as a fraction of g, acceleration of gravity. Using this relationship and Eq (2) lead to:

$$0.5 I_e\, PGA < F_{px} / w_{px} < 1.0 I_e\, PGA \tag{3}$$

FIRST MODE REDUCED (FMR) APPROACH BY RODRIGUEZ, RESTREPO AND CARR

Rodriguez et al (2002) showed that the nonlinear response of the lateral-load-resisting system affects only the first mode of response, and they proposed an expression for evaluating the uppermost floor diaphragm force, F_n. Rodriguez et al (2007) simplified this expression and proposed an upper and lower bound for the roof floor acceleration. The upper bound for the roof floor acceleration was defined as:

$$\frac{F_n}{m_n\ g} = \sqrt{\left[\frac{\eta_1\ S_a(T_1,\xi)}{R_M\ g}\right]^2 + \eta_2\ \ln(n)\ C_{ho}^2} \tag{4}$$

where $S_a(T_1,\xi)$ is the spectral acceleration, parameter m_n represents the floor mass at the roof level. Parameters T_1 and ξ are the fundamental period of vibration and damping ratio, respectively, and R_M is the factor to reduce the design elastic forces to the inelastic design level. Parameters η_1 and η_2 are defined as $\eta_1 = 8/5$ and $\eta_2 = 1.75$. Parameter C_{ho} is the peak ground acceleration as a fraction of g.

BSSC TASK GROUP ON DIAPHRAGMS

A Task Group was formed in 2011 to propose to BSSC a diaphragm seismic design procedure in code language. According to the proposed procedure, diaphragms including chords and collectors should be designed to resist in-plane seismic design forces defined as:

$$F_{px} = \frac{C_{px}}{R_s} w_{px} \tag{5}$$

where C_{px} is the design acceleration coefficient at Level x calculated from C_{p0}, C_{pi}, and C_{pne} using linear interpolation as shown in Fig 3. Coefficients C_{p0}, C_{pi}, and C_{pne} are determined as shown later. Force reduction factor R_s is the diaphragm design force reduction factor. In the context of this paper we will assume reduction factor R_s = 2 to determine design forces for panels, sub-panels and for stringers that are deemed chords, whereas R_s = 1.5 for stringers that are deemed collectors.

The force determined by Eq (5) should not be less than

$$F_{px} = 0.2 S_{DS}\, I_e\, w_{px} \tag{6}$$

Design Acceleration Coefficients C_{p0}, C_{pi}, and C_{pne}

Design acceleration coefficients C_{p0}, C_{pi}, and C_{pne} should be calculated as given by Eqs (7), (8) and (9):

$$C_{p0} = 0.4\, S_{DS}\, I_e \tag{7}$$

$$C_{pi} = 0.9\,\Gamma_{m1}\,\Omega_0\, C_s \tag{8}$$

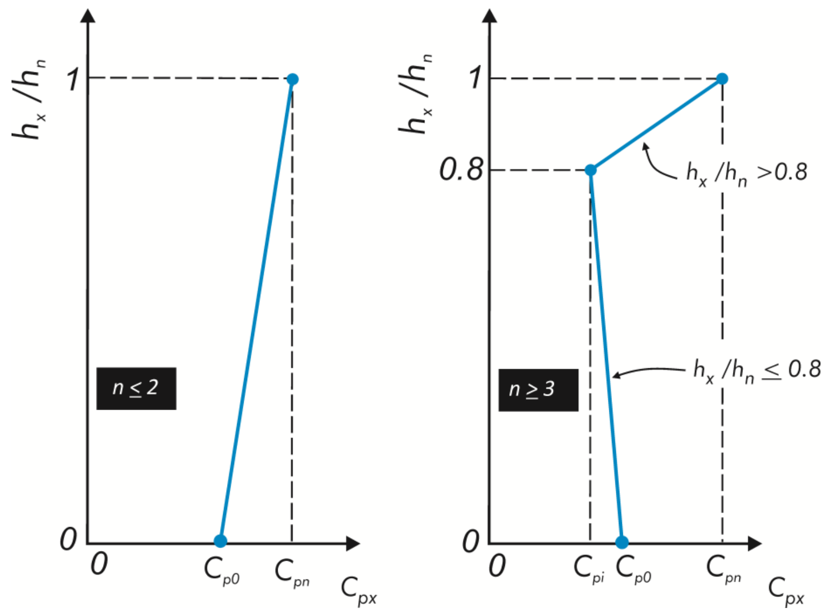

Figure 3. Assumed floor acceleration envelopes for calculating the design acceleration coefficient C_{px} in buildings with $n \leq 2$ and in buildings with $n \geq 3$

and

$$C_{pn} = \sqrt{\left(\Gamma_{m1}\,\Omega_0\, C_s\right)^2 + \left(\Gamma_{m2}\, C_{s2}\right)^2} \tag{9}$$

where Ω_0 is the overstrength factor given in table 12.2.1 de la ASCE 7-10 Standard. Coefficient C_s is determined in accordance with Section 12.8.1.1 de la ASCE 7-10 Standard. C_{s2} should be the smallest of values calculated from:

$$C_{s2} = \left(0.15\,n + 0.25\right) I_e\, S_{DS} \tag{10}$$

$$C_{s2} = I_e\, S_{DS} \tag{11}$$

$$C_{s2} = \frac{I_e\, S_{D1}}{0.03(n-1)} \tag{12}$$

The modal participation factors Γ_{m1} and Γ_{m2} in Eqs (8) y (9) are calculated from Eqs (13) and (14):

$$\Gamma_{m1} = 1 + \frac{z_s}{2}\left(1 - \frac{1}{n}\right) \quad (13)$$

and

$$\Gamma_{m2} = 0.9\, z_s \left(1 - \frac{1}{n}\right)^2 \quad (14)$$

where z_s is defined as:

z_S = 0.7 for buildings designed with Moment-Resisting Frame systems defined in Table 12.2-1,
or,
z_S = 0.85 for buildings designed with Dual Systems defined in Table 12.2-1 with Special or Intermediate Moment Frames capable of resisting at least 25% of the prescribed seismic forces
or,
z_S = 1 for buildings designed with all other seismic force-resisting systems.

DIAPHRAGM DESIGN FORCES FOR A 20-STORY PRECAST CONCRETE BUILDING

Diaphragm design forces using the proposal BSSC for the 20-story building above described are obtained in the following.

Using ASCE 7-10 definitions, factor I_e=1, and the seismic design forces specified by the MCBC (2004), we obtain C_{po}= 0.2, C_s=0.108, and S_{DS} = 0.8. Parameter C_{s2} needs to be the smallest of:

from Eq (10), C_{s2} = (0.15*20+0.25) S_{DS}= 3.25 S_{DS}
from Eq (11), C_{s2}= S_{DS}
from Eq (12), C_{s2}= S_{D1} / (0.03*(20-1))= 1.75 S_{D1}

From the above expressions C_{s2}= S_{DS}, from which C_{s2}= 0.8.

Factors Γ_{m1} and Γ_{m2} :

from Eq (13) and z_S = 0.85, Γ_{m1} = 1+ (0.85/2)*(1-1/20) = 1.4
from Eq (14) and z_S = 0.85, Γ_{m2} =0.9*0.85*(1-1/20)^2 = 0.73

Design Acceleration Coefficient at the Structural Height, C_{pn}:

Replacing values in Eq (9) and considering Ω_o=2

$$C_{pn} = \sqrt{(1.4*2*0.108)^2 + (0.73*0.8)^2} = 0.66$$

Design Acceleration Coefficient at Eighty Percent of the Structural Height, C_{pi}:

From Eq (8):

$$C_{pi} = 0.9*1.4*2*0.108 = 0.27$$

The distribution of diaphragm design forces along the building height can be obtained using Eq (5).

Figure 4 shows the distribution of diaphragm forces determined using different approaches. The thick solid line represents the envelope of diaphragm forces calculated using the proposal with the diaphragm design force reduction factor, R_s, equal to 1, and the thick dashed line represents same set of forces but reduced by a factor R_s equal to 2. Calculated diaphragm forces using the ASCE 7-10 Standard are shown with a thin solid line. Calculated diaphragm forces using the MCBC Standard are shown with a thin dashed line, where F_{px} is defined as:

$$\frac{F_{px}}{w_{px}} = c_o + \frac{F_x}{w_x} \qquad (15)$$

where c_0 is the design seismic coefficient at a period equal to 0.

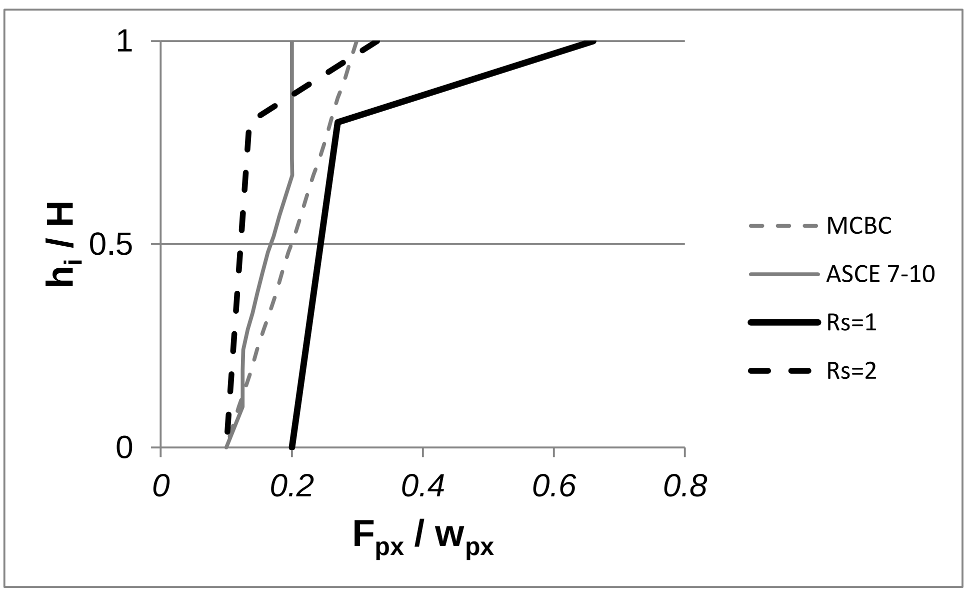

Figure 4. Design diaphragm forces calculated using different approaches.

As seen in Figure 4, results using the proposal and R_s=1 lead to the highest demands of diaphragm forces. Figure 4 also shows that diaphragm forces obtained using the proposal and R_s=2 are smaller than the forces obtained with the ASCE 7-10 Standard except at few floors at the upper levels. Also, these forces are smaller than those predicted by the MCBC.

EXAMPLE OF THE SEISMIC DESIGN OF A DIAPHRAGM OF THE 20 STORY PRECAST CONCRETE BUILDING

In the following an example is given to describe the seismic design procedure for the diaphragm at the roof level of the 20 story precast concrete building. According to the previous calculations, it has been shown that C_{pn}=0.66, then according to Eq (5) and considering w_{pn}=10,690 kN and R_s= 2, the diaphragm seismic design force at the roof level is obtained as:

$$F_{pn} = \frac{C_{pn}}{R_s} w_{pn} = \frac{0.66}{2} * 10,690\, kN = 3,528\, kN$$

The stringer-and-panel method (Blaaunwendraad and Hoogenboom, 1996) is described in the following to design the diaphragm for resisting the force of 3,528 kN at the roof level. Examples of the suggested seismic design of diaphragms for both the region of diaphragm without openings and the region of diaphragm with openings are given in this paper.

INRTRODUCTION TO THE STRINGER-AND-PANEL METHOD

In the following the typical beam span length of 8.5 m is termed *a*. It is assumed that the diaphragm is divided in equal panels with in-plane dimensions equal to $a*a$. Fig 5 shows that the diaphragm has been divided in 16 panels. It is also assumed that the in-plane force in each panel, F_j, is defined as a portion of the design diaphragm force $F=F_{pn}$:

$$F_j = \frac{F}{16} = \frac{3528\ kN}{16} = 221\, kN$$

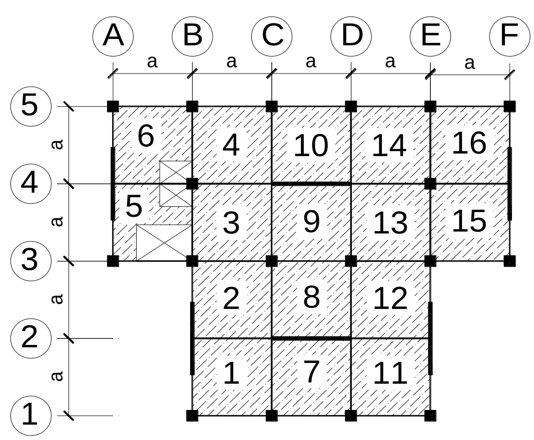

Figure 5. Panels in the diaphragms of the building

It must be noted that conservatively it is assumed here that the masses in the panels with openings, panels 5 and 6 (see Fig 5), are equal to those of the panels without openings. The seismic design procedure for panels 5 and 6 is described later.

The next step is the evaluation of nodal forces of each panel in the direction of the analysis, and to do that the force *F*/16 in a panel is distributed in four equal forces at each node of the panel, as seen in Fig 6. When grouping the nodal forces acting at each panel considering the full diaphragm, we obtain the distribution of forces shown in Fig 7. These forces are in equilibrium with the wall shear forces, which at each wall is assumed equal to *F*/4, see Fig 7.

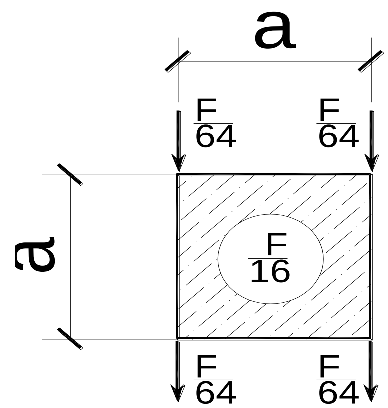

Figure 6. Nodal forces in a panel

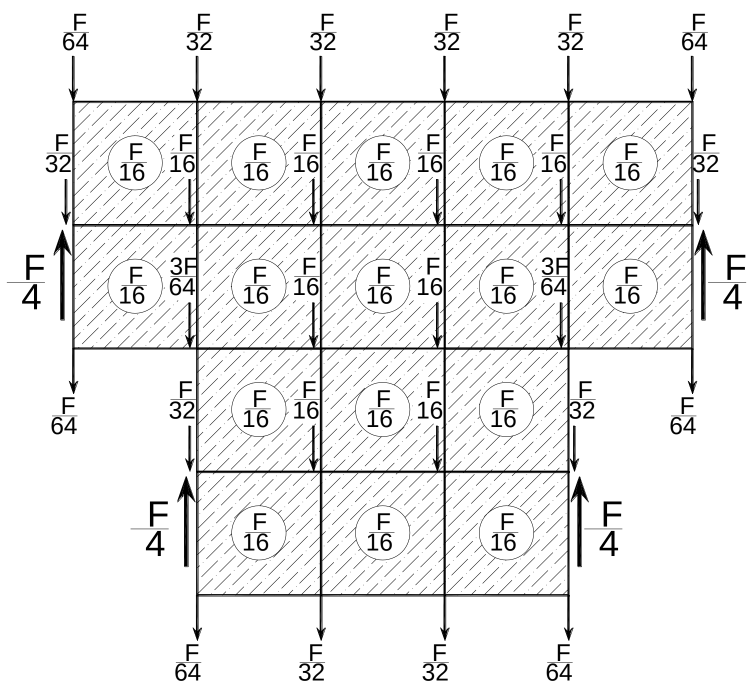

Figure 7. Distribution of nodal forces in the panels of the diaphragm

Equilibrium of Forces in a Stringer and Panel

In the stringer-and-panel method it is assumed that in each panel acts a shear flow equal to q. This shear flow also acts at the interface between the shear panel and the stringers bordering the panels. As a consequence, the axial force in a stringer increases or decreases linearly. Fig 8 shows the forces acting on the stringers and panel. These forces can be obtained using equilibrium considerations. For example, in the stringer at the lower edge, see Fig 8, the axial force N_2^x is obtained from an equation of equilibrium as:

$$N_2^x = N_1^x - q\,a \qquad (16)$$

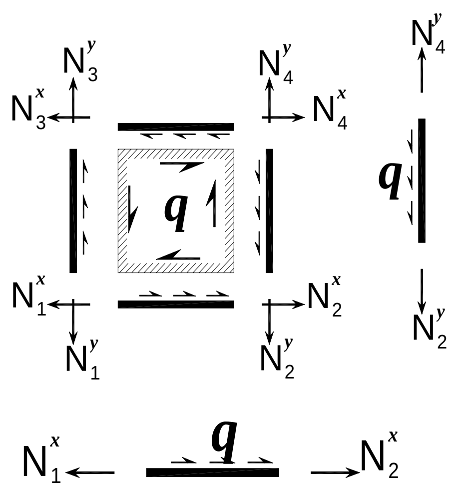

Figure 8. Shear flow and normal forces in the stringers and panel

Shear in the Diaphragm Panels

Fig 9 shows the proposed distribution of shear flows, q, in the diaphragm panels without openings of the building. It must be noted that the symmetry in both the diaphragm geometry and nodal loads leads to the condition that the shear flow, q_i, at one side of the symmetry axis needs to be equal to the shear flow q_i at the other side of the symmetry axis but with opposite sign, see Fig 9. In addition, due to symmetry, the axial loads in the chords along axis C and D need to be equal in magnitude and sign, and to reach this condition the following needs to be satisfied:

$$q_3 = q_6 = q_8 = q_{10} = 0 \qquad (17)$$

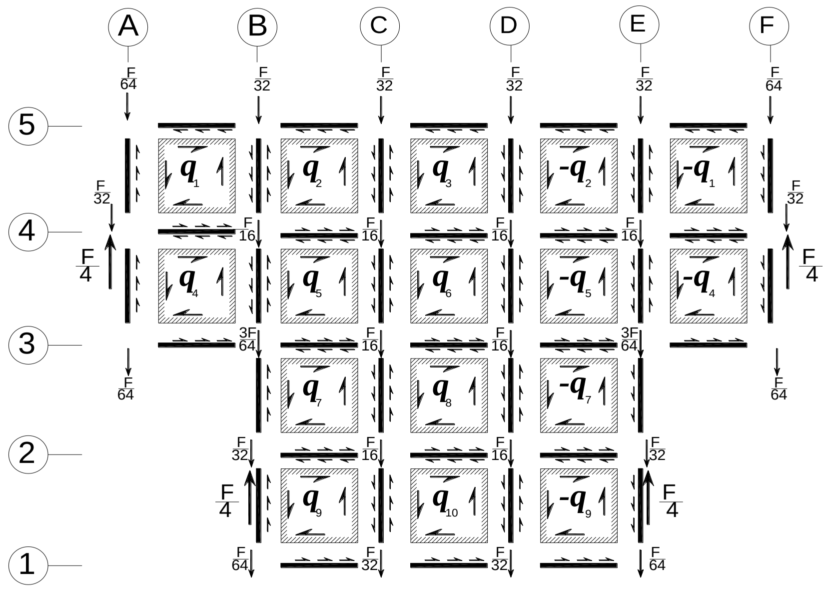

Figure 9. Proposed distribution of shear flows in the panels.

The remaining shear flows, q_i, in the diaphragm, can be obtained from equilibrium equations in axis A and B, as shown in the following.

As seen in Fig 10, equilibrium in axis A leads to:

$$(q_1 + q_4)\,a = -\tfrac{3}{16}\,F \tag{18}$$

and from equilibrium in axis B, see Fig 11, we obtain:

$$(q_2 + q_5 + q_7 + q_9)\;a - (q_1 + q_4)\;a + \frac{1}{16}F = 0 \tag{19}$$

Eqs (18) and (19) are indeterminate equations and they can be solved <u>as long as equilibrium is satisfied</u>. For example, we can assume the shear flow carried in panels 1 and 4 are equal and also the shear flow carried by panels 2, 5, 7 and 9 are also equal. This results in the following expressions for the shear flows,

$$q_1 = q_4 \tag{20}$$

and

$$q_2 = q_5 = q_7 = q_9 \tag{21}$$

From Eqs (18) and (20) we obtain:

$$q_1 = q_4 = -\tfrac{3}{32}\,F\,/\,a \tag{22}$$

and from Eqs (19) and (21) we obtain:

$$q_2 = q_5 = q_7 = q_9 = -\tfrac{1}{16} F / a \tag{23}$$

With the values for q_i given by Eqs (17), (22) and (23), and considering the proposed shear flow distribution shown in Fig 9, we obtain the distribution of shear flows in the panels of the diaphragm shown in Fig 12.

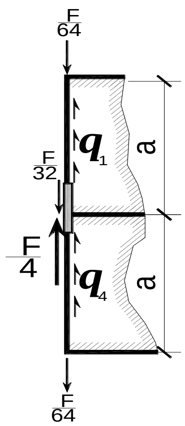

Figure 10. Forces and shear flows in axis A.

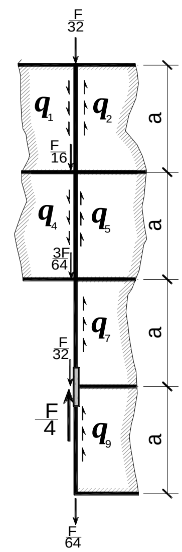

Figure 11. Forces and shear flows in axis B

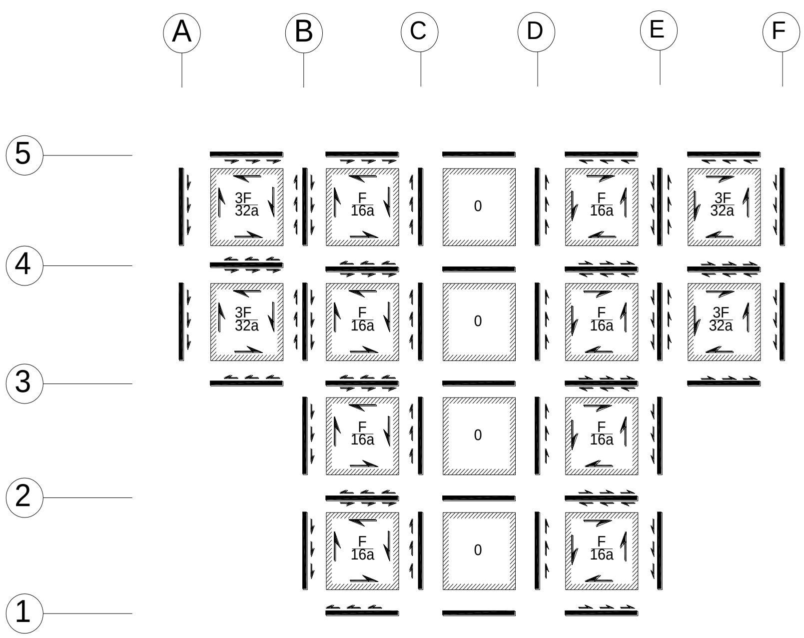

Figure 12. Computed shear flows in the panels without openings.

STRINGER-AND-PANEL METHOD FOR THE DIAPHRAGM WITH OPENINGS

The seismic design procedure for the panels 5 and 6, which have openings, is shown in the following. These panels are further divided in nine smaller sub-panels shown in Fig 13. As seen there, in each of these nine sub-panels acts the shear flow q_{ij}, where i corresponds to the region where acts shears q_1 or q_4 (see Fig 9), and j is the number of the panel at region 1 or 4. It must be mentioned that the shear flow $F/16a$ acting in axis B, see Fig 13, was obtained from the distribution of shear flow shown in Fig 12.

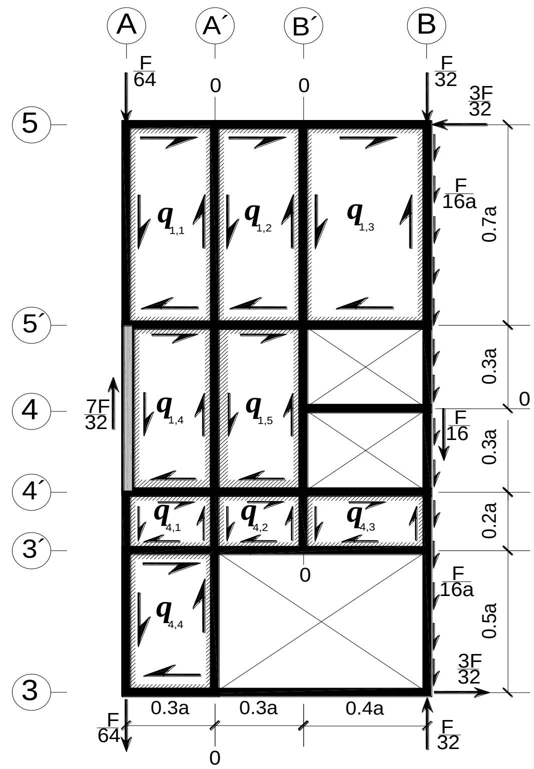

Fig 13. Distribution of shear flows in the panels with openings

Shear flow in the Panels with Openings

The distribution and values of shear flow q_{ij} in the panels with openings can be obtained by equilibrium considerations in axis 3, 5, A, B and B'. For example, from equilibrium in axis 3, see Fig 14, we obtain:

$$q_{4,4} = -\frac{5F}{16a} \tag{24}$$

and from equilibrium in axis 5, see Fig 15, we obtain:

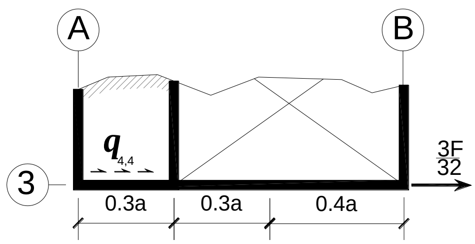

Figure 14 Forces in axis 3

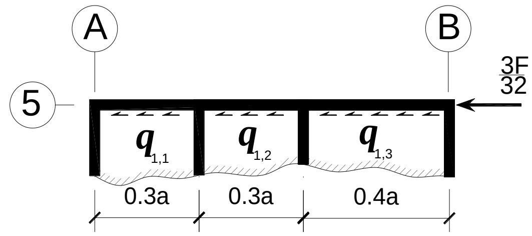

Figure 15. Forces in axis 5

$$q_{1,1}(0.3a)+q_{1,2}(0.3a)+q_{1,3}(0.4a)+\frac{3F}{32}=0 \qquad (25)$$

It can be shown that three additional equilibrium equations can be obtained from axes A, B and B', and they are:

$$q_{1,1}(0.7a)+q_{1,4}(0.6a)+q_{4,1}(0.2a)+q_{4,4}(0.5a)+\frac{3F}{16}=0 \qquad (26)$$

$$q_{1,3}(0.7a)+q_{4,3}(0.2a)+\frac{3F}{16}=0 \qquad (27)$$

$$(q_{1,2}-q_{1,3})(0.7a)+q_{1,5}(0.6a)+(q_{4,2}-q_{4,3})(0.2a)=0 \qquad (28)$$

We have nine unknowns and five equilibrium equations (Eqs (24) through (28)), therefore, we need four additional equations. Like an in Strut-and-Tie method, a statically admissible solution (equilibrium solution) is acceptable as long as redistribution of internal forces can take place at the ultimate load. In this example we assume:

$$v_{1,1}=v_{1,4} \qquad (29)$$

$$v_{1,1}=v_{4,1} \qquad (30)$$

$$v_{1,3}=v_{4,3} \qquad (31)$$

$$v_{1,5}=v_{4,2} \qquad (32)$$

The solution of Eqs (24) through (32) leads to the distribution of shear flows in the panels shown in Fig 16.

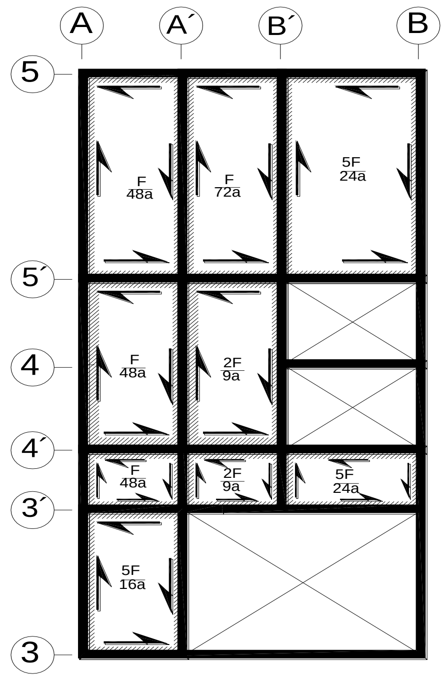

Figure 16. Computed shear flows in the panels with openings

Design for Shear in the Diaphragm with Openings Bordering by Axes 3, 5, A and B

Panel between axes 3 and 3':

The diaphragm panels between axes 3 and 3' have a shear flow, q, equal to (see Fig 16):

$$q = \frac{5F}{16\,a} = \frac{5}{16}\,\frac{3{,}528\,kN}{8.5\,m} = 129.7\ \frac{kN}{m}$$

According to ACI 318-11 Section 21.11.9, the nominal shear strength in diaphragms, ϕV_n, is given by:

$$\phi\,V_n = \phi\,(V_c + V_s)$$

where the design strength ϕV_c in a unit of width (1 m) is given by:

$$\phi\,V_c = \phi A_{cv}\,(0.17\sqrt{f'_c}) = 0.75*1000mm*60mm*0.17\sqrt{40MPa} = 48.4\,kN$$

and the required shear strength provided by reinforcement in a unit of width is given by:

$$\phi V_s = (V_u - \phi\,V_c) = 129.7kN - 48.4kN = 81.3\,kN$$

which results in ϕ 9.5 mm bars @ 275 mm in both directions. Note that this reinforcement should not be in addition to the reinforcement placed to resist flexure in the slab or in addition to the shrinkage and temperature reinforcement. However, in the context of the example presented in this paper, the reinforcement so found will be placed in addition to the reinforcement determined to resist gravity load.

Fig 17 shows the reinforcement needed for shear in the region of the diaphragm with openings.

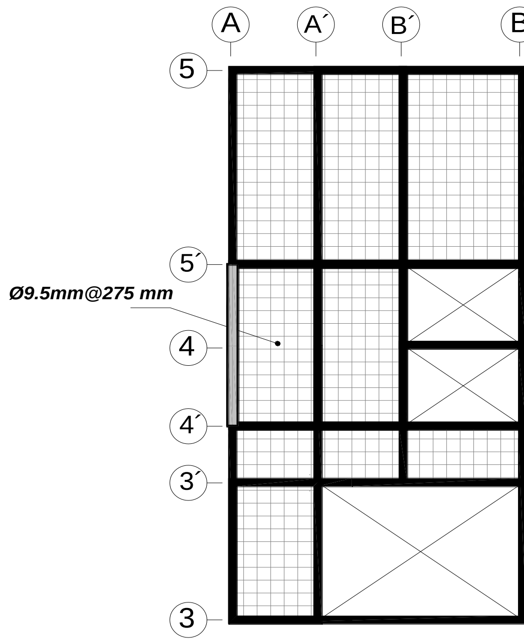

Figure 17. Required shear reinforcement in the region of the diaphragm with openings

FLEXURE-AXIAL FORCE BEAM DESIGN CONSIDERING NORMAL FORCES IN STRINGERS

In the following the flexural reinforcement provided for a beam in the building to resist the design seismic forces obtained from a conventional seismic analysis of the building is revised to check if this reinforcement could also resist the axial forces introduced in the beam when applying the stringer-and-panel method. The beam design shown in the below example correspond to the beam in axis 3 between axes C and D. As shown later, this beam is the most demanded beam in tension. Fig 18 shows the flexural reinforcement at the beam in axis 5 at the roof level and between axes C and D, which is required to resist the design seismic forces obtained from a conventional seismic analysis of the building.

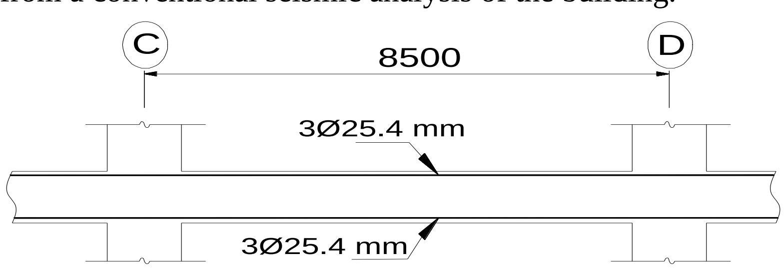

Figure 18. Longitudinal reinforcement in beam of axis 5

As mentioned before, axial forces in stringers can be obtained from equilibrium considering the shear flow q acting at the interface between the shear panel and the stringers bordering the panels. Fig 19 shows the distribution of axial forces in the stringers obtained following this approach using the values of q shown in Fig 12. As seen in Fig 19, the maximum tensile force in the stringers is equal to 5F/32 and is located at the beam in axis 5 between axis C and D. At the roof level, this force is equal to 5*3,528/32= 551 kN.

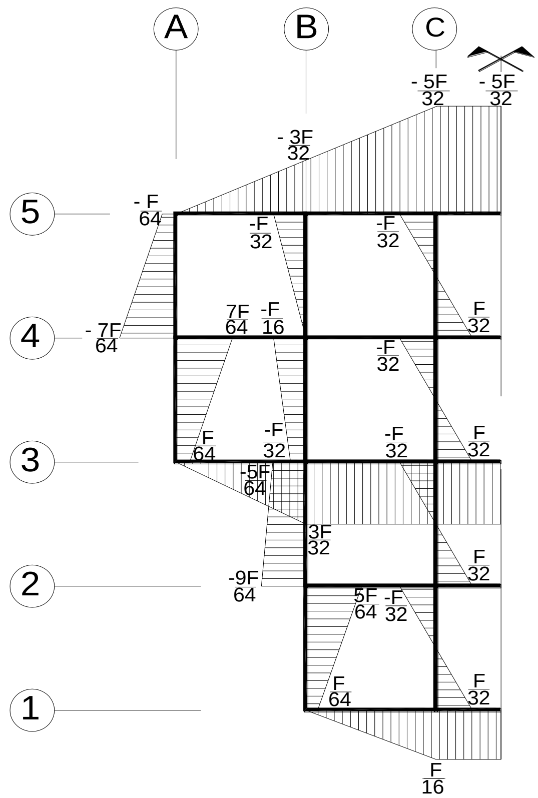

Figure 19. Computed axial forces in stringers

Figure 20 shows the calculated demands of flexural moments (from conventional seismic analysis of the building) and axial load (obtained from using the stringer-and-panel method) for this beam at the roof level. It is necessary to revise if the longitudinal reinforcement shown in Fig 18 could resist the forces shown in Fig 20. Fig 21 shows the interaction moment-axial load diagram for this beam. As can be seen in Fig 21, the flexural moment-tensile force design for the beam with the proposed longitudinal reinforcement can be considered satisfactory for the design values (M_u, P_u) or (265kN-m, 551kN). Further revisions of other critical sections in the beam subjected to flexure and a tensile axial load equal to 551 kN shown that the provided longitudinal reinforcement is acceptable.

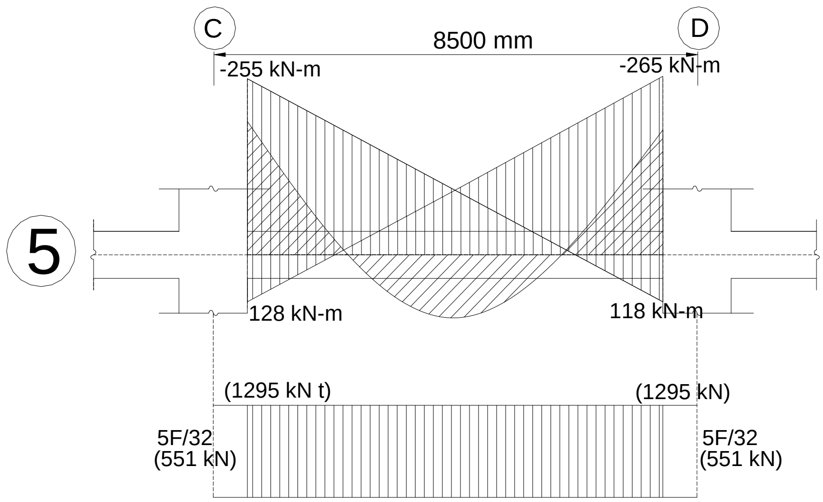

Figure 20. Flexural moment and axial force demands in the beam at axis 5 at the roof level.

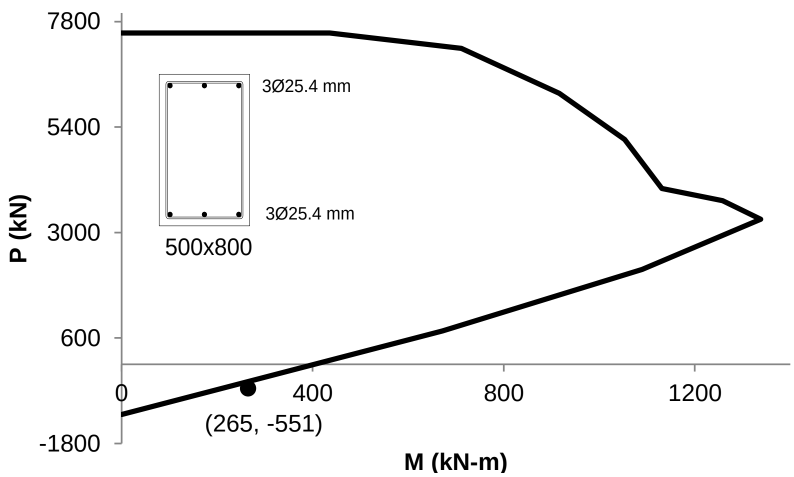

Figure 21. Moment-axial load interaction diagram for the beam at axis 5 at the roof level.

Fig 22 shows the distribution and values of axial forces obtained from the stringer-and-panel method for the region of diaphragm with openings. The design of chords bordering the openings was performed using results shown in Fig 22, which led to the longitudinal reinforcement shown in Fig 23.

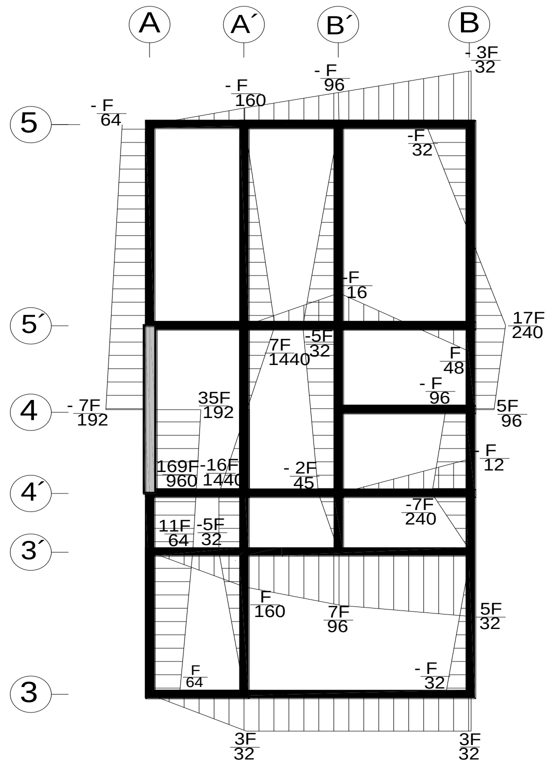

Figure 22. Computed axial forces in stringers of the region of diaphragm with openings

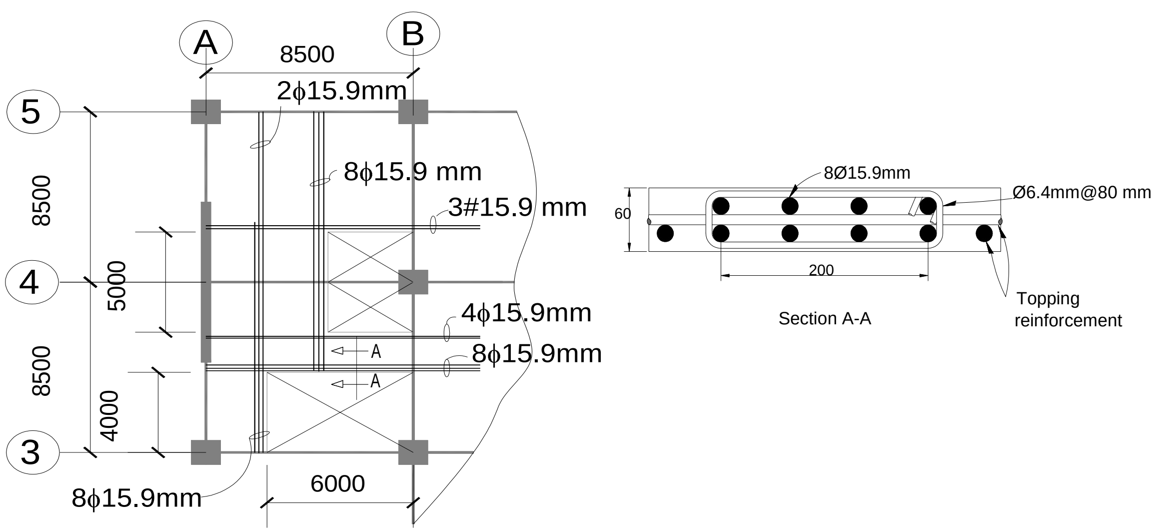

Figure 23. Longitudinal reinforcement in secondary beams of the region of diaphragm with openings (dimensions in mm).

CONCLUSIONS

1. Design diaphragm forces were calculated for a precast concrete 20-story building using different approaches. A comparison of results from these approaches shows that diaphragm forces obtained using the proposal and a R_s value equal to 2 lead to comparable forces as those obtained using the ASCE 7-10 Standard, and they were significantly smaller than the diaphragm forces obtained using a R_s value equal to 1.

2. The stringer-and-panel method was presented in this paper as a tool for the seismic design of diaphragms in buildings. This method is based on equilibrium, it is little known among designers, and it is much simpler than the Finite Element method commonly used by designers when analyzing diaphragms in buildings.

3. It was shown that the stringer-and-panel method allows performing a rational design of diaphragms with openings when subjected to diaphragm forces. This method is more effective than typical procedures followed by practicing engineers when designing diaphragm with openings to resist diaphragm forces.

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