# STANDARD REINFORCEMENT FOR NARROW VERTICAL PARTS ALONG OPENINGS IN PRECAST CONCRETE WALLS

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ABSTRACT

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ABSTRACT

The purpose of this Bachelors' thesis was to describe the method of calculating the reinforcement in the narrow vertical parts along the openings in the precast concrete wall structures and to create standardized reinforcement tables and graphs.

Sometimes design procedure of precast concrete wall structures can be challenging and time consuming. This happens because often wall openings for doors or windows are designed to be close to the edge of the wall element. This can be an issue, which can slow down the work process significantly. The aim of this thesis was to standardize reinforcement for the narrow vertical members, in order to simplify the design process.

The method of calculation is based on Eurocodes and on the loads on the structure and the geometry of the structure.

As a result of the thesis, the standardized reinforcement tables with graphs were produced.

**Keywords** Precast concrete, wall openings, columns, reinforcement

**Pages** 24 pages including appendices 0 pages

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# 1 INTRODUCTION

## 1.1 Basis of research

Precast concrete structures are widely used nowadays. “The use of precast concrete in construction is regarded as an economical, durable, environmentally friendly, structurally sound and architecturally versatile form of construction. The precast concrete industry is continuously making efforts to keep in line with the demands of modern society: economy, efficiency, technical performance, safety, labor circumstances and environmental friendliness.”—fib Bulletin 74: Planning and design handbook on precast concrete structures.

Precast concrete wall structures are popular nowadays in Finland. In most of the cases, openings must be put to the wall, e.g. door openings, or window openings. Arranging the openings may slow down the process of the structural design, if there are too many openings. The worst scenario for this case is to have the opening closer to the wall’s end at the side. In this situation, the most problematic thing is to fit the reinforcement in such a way that the system works. Statistically, in such situations engineers spend a considerable amount of time designing and checking the “columns” next to the opening. For the solution of this problem, a proper design and a number of checks should be performed to each “column”. It is possible to make mistakes during design of such elements. Consequently, the process is slowed even more. Altogether, a great deal of time is spent for the design, which is repetitive.

The issue of repetitive design and design checks could be solved by having standardized reinforcement tables in hand, instead of performing the same calculation again and again. The process of creating those tables is based on the developed method, described in this thesis. The goal of this thesis is to optimize the work of engineers for the future design of precast wall structures.

## 1.2 Main objectives

One of objectives in the thesis is to describe the method of designing narrow vertical parts along the wall’s openings. In order to develop this approach, few aspects were taken in consideration: actions on the wall, geometry of the wall and geometry of the adjacent elements.

The final goal of the thesis is to standardize the reinforcement for the precast wall elements with the openings that are close to the side of the element. Only the most common cases in residential building’s projects are taken into consideration.

The sizes of elements and door or window openings are specified. Geometry of other elements that have an influence on calculation process is also specified.

As a result of the thesis, standardized reinforcement tables with graphs will be produced.

# 2 CALCULATION METHOD AND DETAILING

## 2.1 Walls' sizes

The idea of the research is to calculate the thin vertical parts along the openings separately as columns (Figure 1). The method is applicable to wall elements with a certain thickness of H. Those parts are assumed to be columns as long as the rule of cross-section: 4H<B is followed. (EN 1992-1-1 9.5.1) If the rule is not followed, those vertical parts should be calculated as walls.

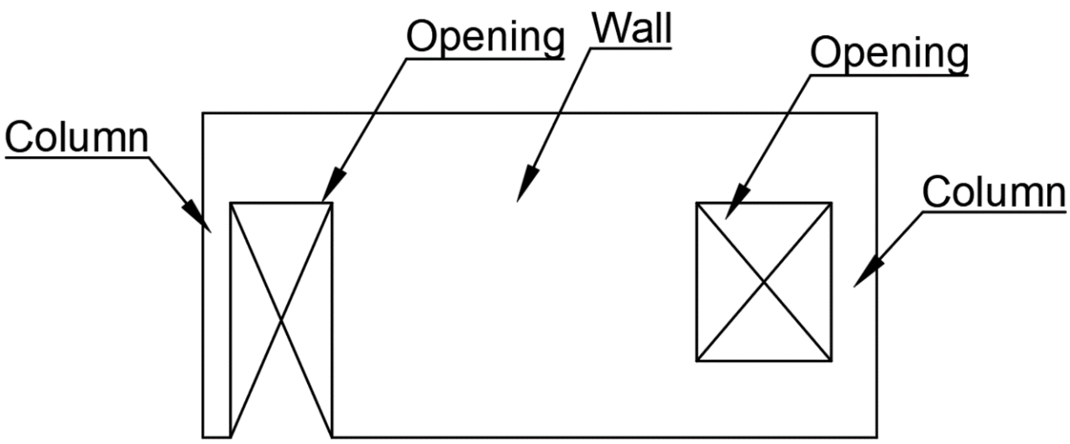


**Figure 1.** Precast concrete wall with openings.

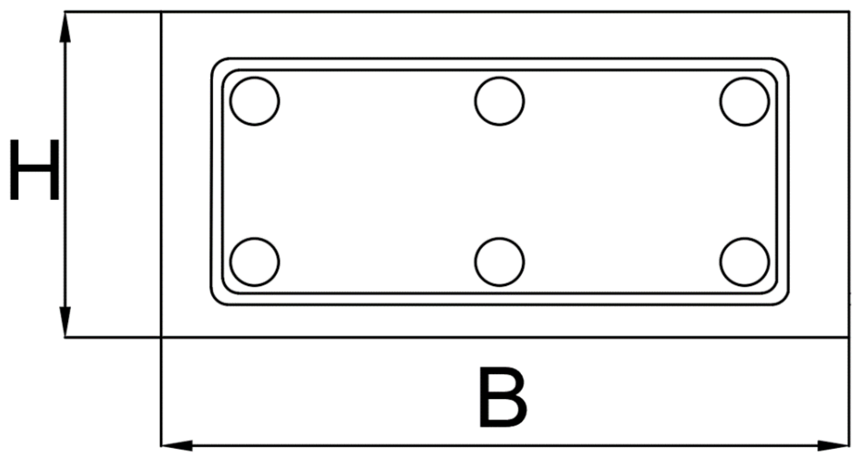


**Figure 2.** Cross-section of the column.

## 2.2 **Actions on the element**

For the thin wall part along the opening, it was considered loading coming from three different parts i.e. normal forces from the member on top of the wall, moment from the opening beam and moment and normal force from the slab right on top of the element.

The load from slabs forms moment $M_{Ed}$ on the beam (Figure 4). The opening beam transfers the Moment to the column, consequently, the moment should be taken into account for the calculation. The support moment of the beam $M_{Ed}$ is assumed to be totally transferred to the column. The moment $M_{Ed}$ is further considered as moment $M_{yy}$ at the top of the column (Figure 3, Figure 4).

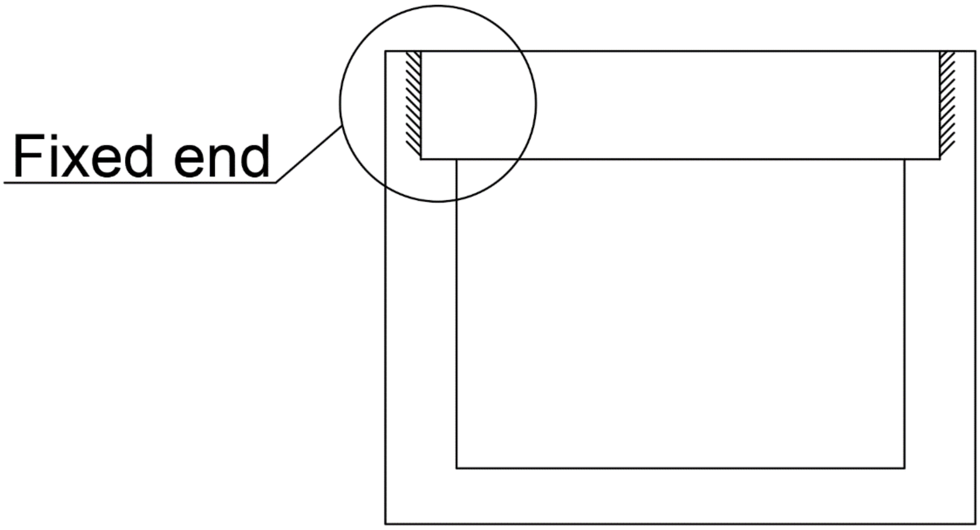


**Figure 3.** Beam's fixed support.

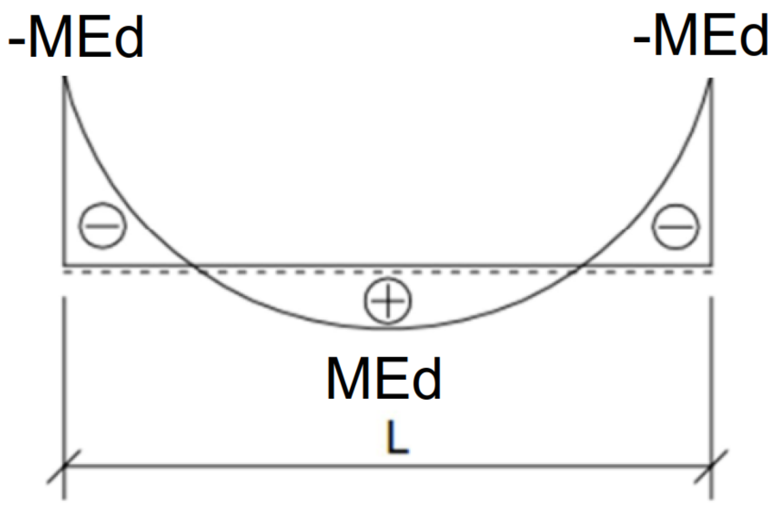


**Figure 4.** Moment on the opening beam.

The loading is calculated as:

The Self-weight of the slab $Q_{slab}$ ($kN/m^2$) multiplied by half of the slab span $L_1/2$ gives us the dead load on top of the element. The density of reinforced concrete is $25kN/m^3$. Additionally to the slab's weight, a load for floor finishing $n_1$ ($kN/m^2$) and a load for hanging equipment $n_2$ ($kN/m^2$) are added and consequently also multiplied by half of the slab span $L_1/2$ to give us the load on the element

$$G_k = L_1/2 * Q_{slab} + L_1/2 * n_1 \, kN/m^2 + L_1/2 * n_2 \, kN/m^2 \quad (1)$$

Live load is considered according to the Tables 6.1 and 6.2 of EN (1991-1-1). The building is in category A "Domestic or residential activities". The recommended live load value for the floors of category A buildings is $2kN/m^2$. Moreover, based on the most common cases, $n_3$ $kN/m^2$ is added as the weight of the lightweight concrete walls.

$$Q_k = L_1/2 * 2kN/m^2 + L_1/2 * n_3 \, kN/m^2 \quad (2)$$

Then the load is combined according to EN(1990).

$$Q_d = 1{,}35 K_{FI} G_{kj,\,sup} \quad (3)$$

$$Q_d = 1{,}15 K_{FI} G_k + 1{,}5 K_{FI} Q_{k,1} + 1{,}5 K_{FI} \sum_{i>1} \psi_{0,i} Q_{k,i} \quad (4)$$

where:

$K_{FI}$ is the factor depending on reliability class.

$G_k$ is unfavourable permanent action.

$Q_{k,1}$ is leading variable action.

$Q_{k,i}$ is variable action.

$\psi_{0,I}$ is coefficient from (EN1990, Annex A1, Table A1.1)

The load on the beam is $Q_d$. The length of the opening (the beam) is $L_2$ Consequently, the $M_{ed}$ (Figure 4) is calculated:

$$M_{Ed} = Q_d * L_2^2/12 \qquad (5)$$

Then, $M_{Ed}$ is assumed as a moment around stronger axis ($M_{yy}$) for the column. (Figure 5)

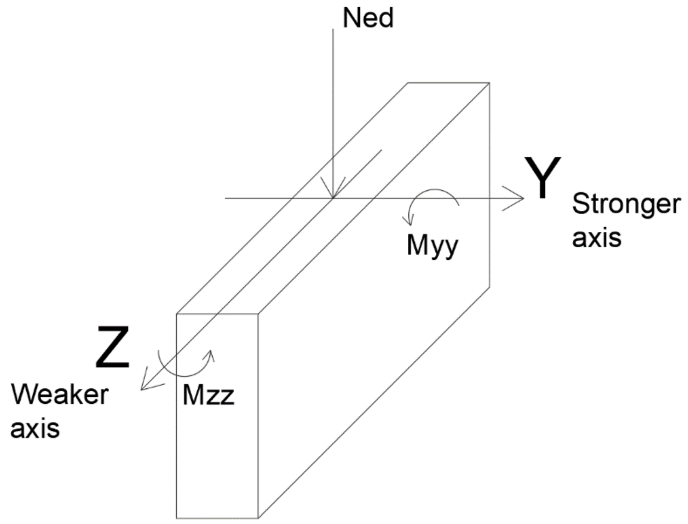


**Figure 5.** Moments on the column caused by the load of the floor.

Eccentricity around weaker axis. When the load comes from slabs to the wall, small eccentricity appears at the support. Eccentricity is calculated as:

$$e_{0E} = (h_{wall} - l_{support})/2 \qquad (6)$$

So that the moment around weaker axis occurs. $M_{zz}$ is then calculated as:

$$M_{zz} = Q_d * B_{column} * e_{0E} \qquad (7)$$

## 2.3 **Reinforcement detailing**

### 2.3.1 Nominal concrete cover

In order to find the minimum cover, the exposure class of the element should be chosen according to the table EN 1992-1-2 4.1N. Based on the exposure class of the building the minimum concrete cover can be chosen. (Table 1.)

Table 1. Concrete cover according to exposure class. (adapted from: RIL 202-2011/by 61 table 4.3N)

| Ympäristöolosuhteista johtuva betonipeitteen vähimmäisarvovaatimus $c_{min,dur}$ (mm) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Kriteeri | Rasitusluokka taulukon 4.1 mukaan | | | | | | | |
| | X0 | XC1 | XC2<br>XC3 | XC4 | XD1 | XS1 | XD2 | XD3<br>XS2,3 |
| Betoniteräs | 10 | 10 | 20 | 25 | 30 | 30 | 35 | 40 |
| Jänneteräs | 10 | 20 | 30 | 35 | 40 | 40 | 45 | 50 |
| 100 vuoden suunniteltu käyt-töikä [1)] | +0 | +0 | +5 | +5 | +5 | +5 | +5 | +5 |
| Lujuusluokka ≥ | C20/25<br>-5 | C30/37<br>-5 | C35/45<br>-5 | C35/45<br>-5 | C35/45<br>-5 | C40/50<br>-5 | C35/45<br>-5 | C45/55<br>-5 |

The minimum cover also depends on the fire exposure class. (Table 2.)

Table 2. The minimum distance for fire protection. (from: RIL 202-2011/by 61 table L4.1)

**Taulukko L4.1.** Suorakaidepilarien ja pyöreiden pilarien paloluokkia.

| *b* (mm) = | 180 | 280 | 380 | | 480 | | 580 | | 680 | |
|---|---|---|---|---|---|---|---|---|---|---|
| *a* (mm) = | 40 | 40 | 40 | 50 | 40 | 50 | 50 | 60 | 50 | 60 |
| Paloluokka, kun hyväksikäyttöaste $\mu_{fi} = 0{,}7$ | R30 | R60 | R60 | R90 | R90 | R120 | R120 | R180 | R120 | R180 |

According to (EN 1992-1-1), a nominal concrete cover $C_{nom}$ is defined as a minimum cover $C_{min}$ plus an allowance in design for deviation $\Delta C_{dev}$.

$$C_{nom} = C_{min} + \Delta C_{dev} \tag{8}$$

A column is a part of the wall element. Consequently, mesh reinforcement should be considered. In this thesis, the capacity of the mesh is not taken into account, as we cannot know exactly the positioning of the mesh in the element. Moreover, stirrups should be put inside the mesh, in order to simplify the manufacturing process, so extra distance is added for nominal concrete cover of main reinforcement for this calculation. The final nominal concrete cover can be defined as:

$$C_{nom} = C_{min} + \Delta C_{dev} + \varphi_{mesh} \tag{9}$$

### 2.3.2 Transverse reinforcement

In accordance with (EN1992-1-1), the diameter of the stirrups should be bigger than 6 millimeters or one quarter of the maximum diameter of the longitudinal bars whichever is greater, as shown in the equation below (10).

$$\varphi_{stirrup} > \max\,(6mm \text{ or } 0.25 * \varphi_{main\ bar}) \tag{10}$$

According to (EN 1992-1-1) the spacing of the transverse reinforcement along the column should not exceed $S_{cl.tmax.}$

The lowest of the following three values applies:

- 15 times the minimum diameter of the longitudinal bars used
- the lesser dimension of the column
- 400 mm

### 2.3.3 Anchorage

Minimum anchorage length is calculated according to EN (1992-1-1).

The Basic required Anchorage length $l_{b,rqd}$ can be calculated as:

$$l_{b,rqd} = (\varphi_{main\ bar}/4)*(\sigma_{sd}/f_{bd}) \tag{11}$$

where:

$\sigma_{sd}$ is the stress of the bar. In this calculation $\sigma_{sd}$ is assumed to be the maximum value. This means that $\sigma_{sd} = f_{yd}$.

$f_{bd}$ is the ultimate bond stress.

The designed value of the ultimate bond stress $f_{bd}$ can be calculated as:

$$f_{bd} = 2.25 * \eta_1 * \eta_2 * f_{ctd} \tag{12}$$

where:

$f_{ctd}$ is the design value of concrete tensile strength according to (EN 1992-1-1 3.1.6)

$\eta_1$ is the coefficient related to quality of the bond condition and position of the bar during concreting. Check (EN 1992-1-1 8.4.2)

$\eta_2$ is the coefficient related to the bar diameter. Check (EN 1992-1-1 8.4.2)

The design anchorage length is calculated according to EN (1992-1-1) as:

$$l_{bd} = \alpha_1 * \alpha_2 * \alpha_3 * \alpha_4 * \alpha_5 * l_{b,rqd} \tag{13}$$

where:
$\alpha_1$, $\alpha_2$, $\alpha_3$, $\alpha_4$ and $\alpha_5$ are the coefficients from the (EN 1992-1-1 table 8.2)

### 2.3.4 Spacing of bars

Spacing of main bars is calculated according to EN (1992-1-1) and NATIONAL BUILDING CODE OF FINLAND.
The minimum spacing of the main reinforcement is chosen as the maximum of:
- Bar diameter
- The maximum size of aggregate + 3mm
- 20mm

### 2.3.5 Minimum area of reinforcement

The minimum area of reinforcement is calculated according to EN (1992-1-1).
The total amount of longitudinal reinforcement should not be less than $A_{s,min.}$ The minimum area of reinforcement is calculated as:

$$A_{s,min} = \max\,(0.10 * N_{ed} / f_{yd} \text{ or } 0.002 * A_c) \tag{14}$$

where:
$f_{yd}$ is the design yield strength of the reinforcement.
$A_c$ is the area of column's cross-section.

### 2.3.6 Maximum area of reinforcement

The maximum area of reinforcement is calculated according to EN (1992-1-1) and NATIONAL BUILDING CODE OF FINLAND.
The area of longitudinal reinforcement should not exceed $A_{s,max.}$ The value of the maximum area of longitudinal reinforcement at laps is $A_{s,max}$ = $0.12A_c$ and outside of laps it is $A_{s,max} = 0.06A_c$.

## 2.4 Column calculation procedure

Column calculation procedure is calculated according to (EN 1992-1-1)

### 2.4.1 The designed moment

The designed moment $M_{Ed}$ is calculated as:

$$M_{Ed} = M_{0Ed} + M_2 \tag{15}$$

where:
$M_{0Ed}$ is the 1st order moment, including the effect of imperfections.
$M_2$ is the nominal 2nd order moment.

The 1st order moment $M_{0Ed}$ can be calculated as:

$$M_{0Ed} = \max\,(0.6M_{02} + 0.4M_{01} \text{ or } 0.4M_{02}) + M_i \tag{16}$$

where:
$M_{01}$ and $M_{02}$ are the first order end moments. $M_{01}$ and $M_{02}$ should have the same sign if they give tension on the same side, otherwise opposite signs. Furthermore, $|M_{02}| \geq |M_{01}|$.
$M_i$ is the moment appeared because of the geometrical imperfections.
$M_2$ is the nominal 2nd order moment.

The moment appeared because of imperfections $M_i$ is calculated as:

$$M_i = N_{Ed} * e_i \tag{17}$$

where:
$N_{Ed}$ is the force from the member on top of the column.
$e_i$ is the eccentricity given by the imperfections (section 2.4.3 of this work).

The nominal 2nd order moment $M_2$ is calculated as:

$$M_2 = N_{Ed} * e_2 \tag{18}$$

where:
$N_{Ed}$ is the force from the member on top of the column.
$e_2$ is the eccentricity deflection.

The eccentricity due to deflection $e_2$ is calculated as:

$$e_2 = (1/r) * l_0^2/c \tag{19}$$

where:
$1/r$ is the curvature (check EN 1992-1-1 5.8.8.3)
$l_0$ is the effective length (Figure 6).
c is a factor depending on the curvature distribution (check EN 1992-1-1 5.8.8.2)

### 2.4.2 Imperfection

Imperfection $\theta_i$ is calculated as:

$$\theta_i = \theta_0 * \alpha_h * \alpha_m \tag{20}$$

where:
$\theta_0 = 1/200$
$\alpha_h$ is the reduction factor for length or height (check EN 1992-1-1 5.2)
$\alpha_m$ is the reduction factor for number of members (check EN 1992-1-1 5.2)

### 2.4.3 Imperfection eccentricity

Eccentricity because of imperfection $e_i$ can be calculated as:

$$e_i = \theta_i * l_0/2 \tag{21}$$

where:
$l_0$ is the effective length of the column (Figure 6)
$\theta_I$ is the imperfection of the column. (Chapter 2.4.2 of this thesis)

### 2.4.4 Slenderness

The slenderness of column λ is calculated as:

$$\lambda = l_0/i \tag{22}$$

where:
i is radius of gyration of uncracked concrete section (check EN 1992-1-1 5.8.3.2)
$l_0$ is the effective length of the column (Figure 6)

The slenderness check should perform $\lambda < \lambda_{lim}$ (check EN 1992-1-1 5.8.3.1)

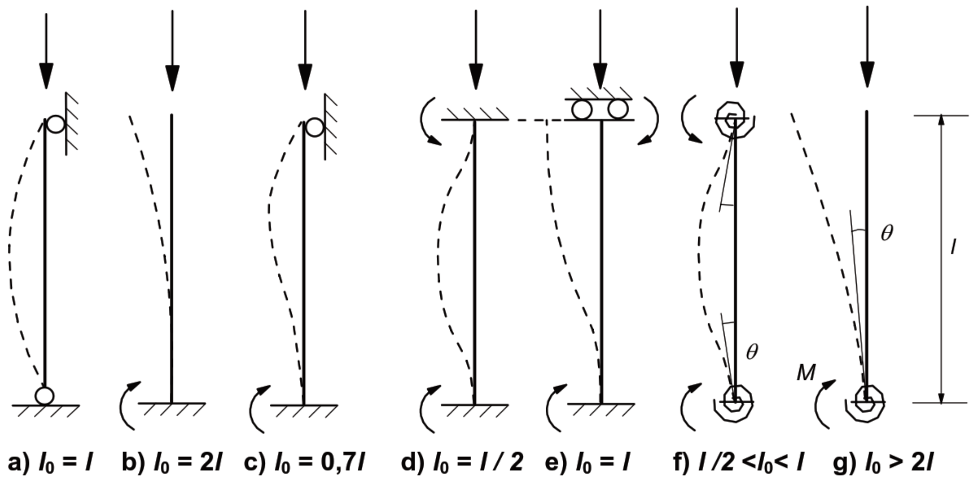


**Figure 6.** Effective lengths of the columns.

### 2.4.5 Interaction table

The resistance of reinforced concrete column is usually expressed with the use of interaction diagrams in order to relate the design axial load ØPn to the design bending moment ØMn (Figure 7). Each control point on the column interaction curve represents one combination of design axial load ØPn and design bending moment ØMn corresponding to a neutral-axis location. — Al-Ansari MS, Afzal MS;2019

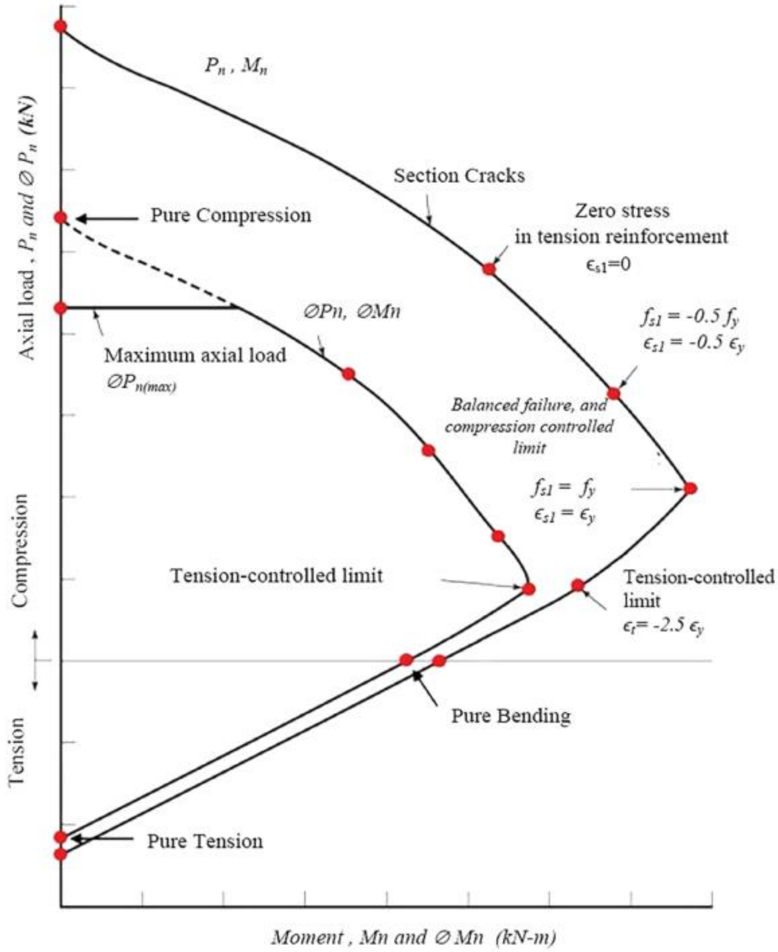


**Figure 7.** An interaction curve diagram.

Another calculation procedure is based on standard interaction diagrams, which are based on standardized curves from the interaction points mentioned above. The reinforcement in this case comes from the standard curves and graphs and from non-dimensional factors based on actions and the column geometry. (Figure 8).

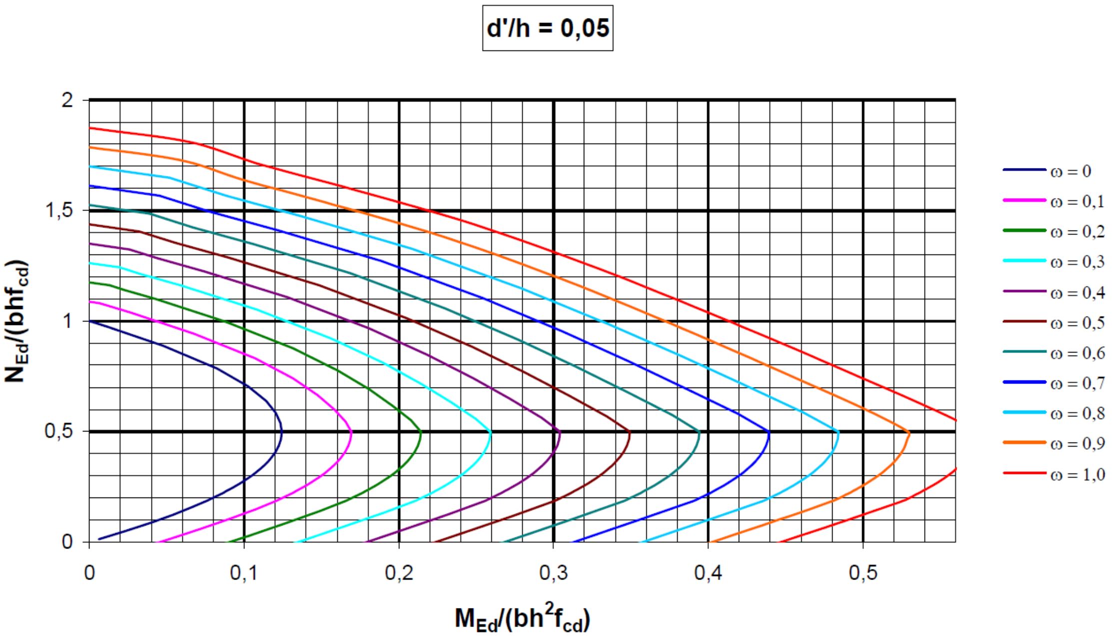


**Figure 8.** Standard interaction table.

The vertical non-dimensional factor $\upsilon$ is calculated as:

$$\upsilon = N_{Ed} \,/\, (A_c * f_{cd}) \tag{23}$$

where:
$N_{Ed}$ is the force from the member on top of the column.
$A_c$ is the cross-sectional area of the column.
$f_{cd}$ is the design compressive strength of concrete.

The horizontal non-dimensional factor μ is calculated as:

$$\mu = M_{Ed} \,/\, (A_c * H * f_{cd}) \tag{24}$$

where:
$M_{Ed}$ is the designed moment. (Chapter 2.4.1 of this thesis)
$A_s$ is the cross-sectional area of the steel.
H is the height of cross-section. (Figure 2)
$f_{cd}$ is the design compressive strength of concrete.

Based on the interaction diagram, the value of $\omega$ can be defined by interacting of the factors, mentioned above.

$$\omega = (A_s * fyd) / (A_c * f\mathrm{cd}) \tag{25}$$

After the transformation of the equation above (), we get:

$$A_s = (\omega * A_c * f\mathrm{cd}) / fyd \tag{26}$$

where:
$f_{yd}$ is the design value of steel yield strength.

After determination of the cross-sectional area the proper reinforcement can be chosen, based on the amount of steel and geometrical properties of the column's cross-section.

### 2.4.6 Biaxial bending

Biaxial bending check is performed as:

$$(M_{zz} / M_{Rdz})^a + (M_{yy} / M_{Rdy})^a \leq 1.0 \tag{27}$$

where:
$M_{zz/yy}$ is the design moment around the respective axis, including a 2nd order moment.
$M_{Rdz/y}$ is the moment resistance in the respective direction, obtained from the column interaction diagram.
a is the exponent (see EN 1992-1-1 5.8.9).

# 3 METHODOLOGY

## 3.1 Chosen material and geometry parameters

The chosen thicknesses of walls are 150mm and 200mm. Therefore, calculation is applied to the columns with the certain widths. (Figure 9)

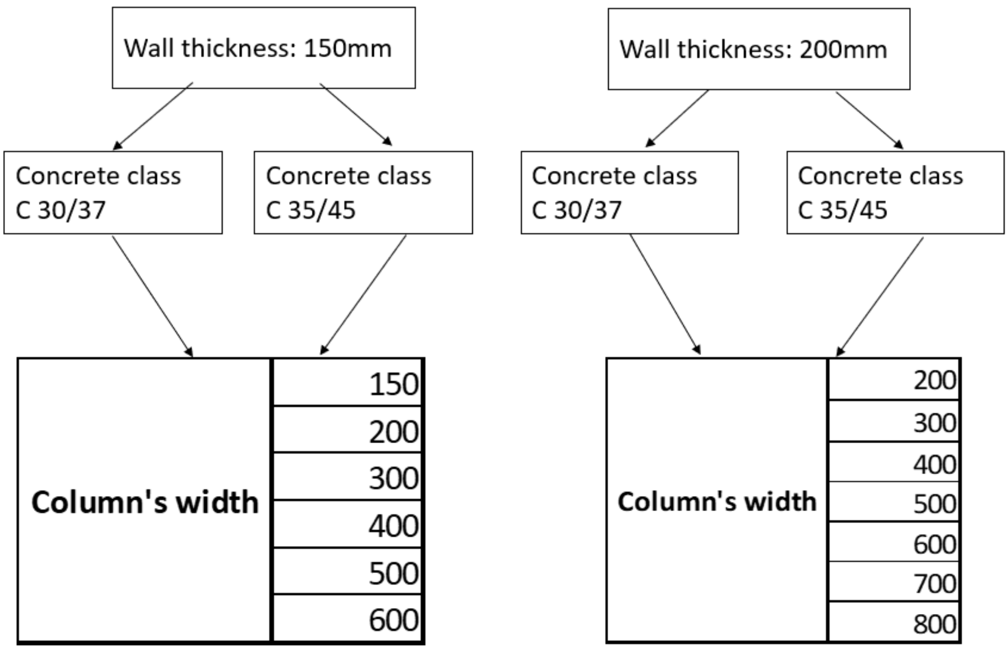


**Figure 9.** The widths of columns.

The height of columns varies from 500mm to 3500 mm.

## 3.2 **Actions on the element**

The chosen slab is P37. The span of the slab is considered to be 6 m. The load of concrete finishing ($n_1$) is 1,5 kN/m$^2$. The load of the hanging equipment ($n_2$) is 0,5 kN/m$^2$. According to these values, dead load $G_k$ equals:

$$G_k=21 \text{ kN/m}$$

Live load is considered according to the EN(1991-1-1) as 2kN/m$^2$ . The weight of lightweight concrete walls ($n_3$) is 0,5 kN/m$^2$. According to these values, live load $Q_k$ is:

$$Q_k=7{,}5 \text{ kN/m}$$

Then the load is combined according to EN(1990). As there is no other live load but $Q_k$, the value of $Q_d$ is:

$$Q_d=35{,}4 \text{ kN/m}$$

The load on the beam is 46,4 kN/m. Based on research, the length of the opening was initially chosen to be 2m. Consequently, the $M_{ed}$ is calculated:

$$M_{ed}= 11{,}8 \text{ kN*m}$$

Eccentricity around weaker axis. Support length is generally taken as 80mm. Therefore, the eccentricity for 150mm wall is calculated as:

$$e_{150}=35 \text{ mm}$$

Similarly, the eccentricity for 200mm wall is calculated as:

$$e_{200}=60 \text{ mm}$$

The moment around weaker axis ($M_{zz}$) varies as the width of column also varies. Moment is then calculated as:

$$M_{zz150}= 1.24 \text{ kN*m/m} * B_{column}$$

$$M_{zz200}= 2.124 \text{ kN*m/m} * B_{column}$$

## 3.3 **Reinforcing**

### 3.3.1 Main reinforcement

As columns are so thin, the diameters of the main reinforcements will be limited to 10mm, 12mm and 16mm. As those bars can fit inside the column.

### 3.3.2 Spacing of main reinforcement

For T10, T12, T16 reinforcements spacing of bars is 20mm, as the maximum value.

### 3.3.3 Anchorage length

Anchorage length for the main reinforcement is shown in Table 3.

Table 3. The anchorage length of main reinforcement.

| | T10 | T12 | T16 |
|---|---|---|---|
| **C30/37** | 360 | 430 | 575 |
| **C35/45** | 325 | 390 | 520 |

### 3.3.4 Transverse reinforcement

The diameter of the stirrups is chosen to be 8 mm, as this value fulfills the EN(1992-1-1) requirements. It is bigger than 2.5mm, 3mm, 4mm, 6mm.

### 3.3.5 Spacing of transverse reinforcement

Spacing of transverse reinforcement is shown in Table 4.

Table 4. Spacings of stirrups.

| | **Main reinforcements' diameter mm** | | |
|---|---|---|---|
| **Columns' thicknesses mm** | **10** | **12** | **16** |
| **150** | 150 | 150 | 150 |
| **200** | 150 | 180 | 200 |

### 3.3.6 Mesh reinforcement

Mesh reinforcement is considered to be M8 150 mm x 150 mm. However, mesh reinforcement should be calculated accordingly for each case, as the diameter can vary according to necessity.

### 3.3.7 Nominal cover

According to the table 4.1N of (EN 1992-1-2), concrete exposure class was chosen as XC1, as the structure is dry or permanently wet.

After that, according to Table 1 shown in Chapter 2.3 in this thesis, minimum cover $C_{min}$ was picked as 10mm. The allowance in the design for deviation $\Delta C_{dev}$ =10mm. The diameter of the mesh $\varphi_{mesh}$ is 8mm. Therefore, $C_{nom}$=28mm.

### 3.3.8 Fire class

Precast concrete building structures normally have a fire resistance of 60 to 120 minutes. The fire class in this study is R60. According to Table 2 shown in Chapter 2.3 of this thesis, distance from the surface of the concrete to the center of the main bar **a (mm)** should be not less than 40 mm. (Figure 10)

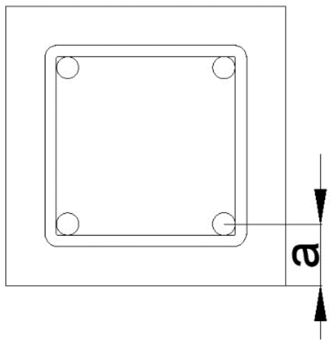


**Figure 10.** The minimum distance for fire protection.

Check according to fire classes:

For T10 reinforcement— $C_{nom} + \varphi_{stirr} + 0.5\ \varphi_{mainbar}$ = 41 mm > 40 mm — OK.
For T12 reinforcement— $C_{nom} + \varphi_{stirr} + 0.5\ \varphi_{mainbar}$ = 42 mm > 40 mm — OK.
For T16 reinforcement— $C_{nom} + \varphi_{stirr} + 0.5\ \varphi_{mainbar}$ = 44 mm > 40 mm — OK.

## 3.4 **Calculation procedure**

In order to facilitate the calculations, the calculation of the reinforcements was made, based on the readymade table of Sweco OY, which is made for the internal use only. This table covers the calculation procedure explained in Chapter 2.4 of this thesis. The procedure of calculating with the table is explained below.

At first, cross-section and height should be chosen. Then, the minimum concrete cover $C_{nom}$ is set. Concrete class and reinforcement type should be selected. Then, the stirrups diameter and spacing need to be chosen. After that, the maximum amount of main reinforcement is placed, according to geometrical properties.

Then, the $M_{yy}$ and $M_{zz}$ should be set. Those values will be used as constants for the further calculation.

After having all the values set, the calculation procedure can be started. Based on geometrical properties and applied loads, the maximum capacity of reinforcement is determined by entering the applied force $N_{ed}$ and checking the limit for the column. When the limit value is reached, the maximum capacity of the column is determined.

# 4 RESULTS

After the estimation of columns' capacities, the tables of reinforcement and graphs were created. On the horizontal axis of the chart, the heights of columns are written. On the vertical axis of the chart, the loadings are written. One table is created for one unique cross-section of the column.

## 4.1 How to use the table

In order to use such a table, one should pick a suitable table, according to the cross-section. After that, the designer needs to pick the height of the column.
Then one should find the matching number on the horizontal axis of the graph. The designer can interpolate the values on the axis if needed. Afterwards, one should pick the load and find the matching number on the vertical axis of the graph. The designer can interpolate the values on the axis if needed. By intersecting the two values, the point on the graph is found. The corresponding reinforcement for this specific column is then the curve right above the point. If the point is over all the curves, that means that there is no suitable reinforcement for that specific column with that specific load.

## 4.2 Restrictions on use

During the calculating process it was found out that some columns with certain reinforcement cannot withstand the moment from the opening beam. It was decided to restrict the width of the openings (beam span) from 2 m to 1,5 m. That means that those columns can be used only for the openings with the width not more than 1,5 m.

The list of designed columns with corresponding reinforcements for 1,5 m opening is shown in Table 5.

Table 5. Columns designed for 1,5 m opening.

| Opening: 1,5 m | |
|---|---|
| **Column's cross-section** | **Reinforcement** |
| 150x150 | T10, T12, T16 |
| 150x200 | T10, T12, T16 |
| 200x200 | T10, T12, T16 |

If the same columns mentioned above were designed with an opening width of 2 m, the reinforcement restrictions would be as shown in Table 6.

Table 6. Columns with restricted reinforcement.

| Opening: 2 m | |
|---|---|
| **Column's cross-section** | **Reinforcement** |
| 150x150 | T16 |
| 150x200 | T12, T16 |
| 200x200 | T12, T16 |

All the other columns are designed without restriction of reinforcements. All of them can be used for the openings with the width less than 2 m.

## 4.3 Table examples

Some examples of final tables (Tables 7 and 8) and graphs (Figures 11 and 12) are shown below.

Table 7. Calculated reinforcement for the 150x150 column with 1,5 m opening.

| | **4T10** | **4T12** | **4T16** |
|---|---|---|---|
| **Height (mm)** | **Load (kN)** | **Load (kN)** | **Load (kN)** |
| **3500** | 26 | 59 | 85 |
| **3000** | 34 | 85 | 114 |
| **2750** | 40 | 101 | 135 |
| **2500** | 47 | 121 | 161 |
| **2250** | 56 | 148 | 196 |
| **2000** | 143 | 184 | 244 |
| **1500** | 236 | 279 | 365 |
| **1000** | 250 | 294 | 380 |
| **500** | 265 | 309 | 380 |

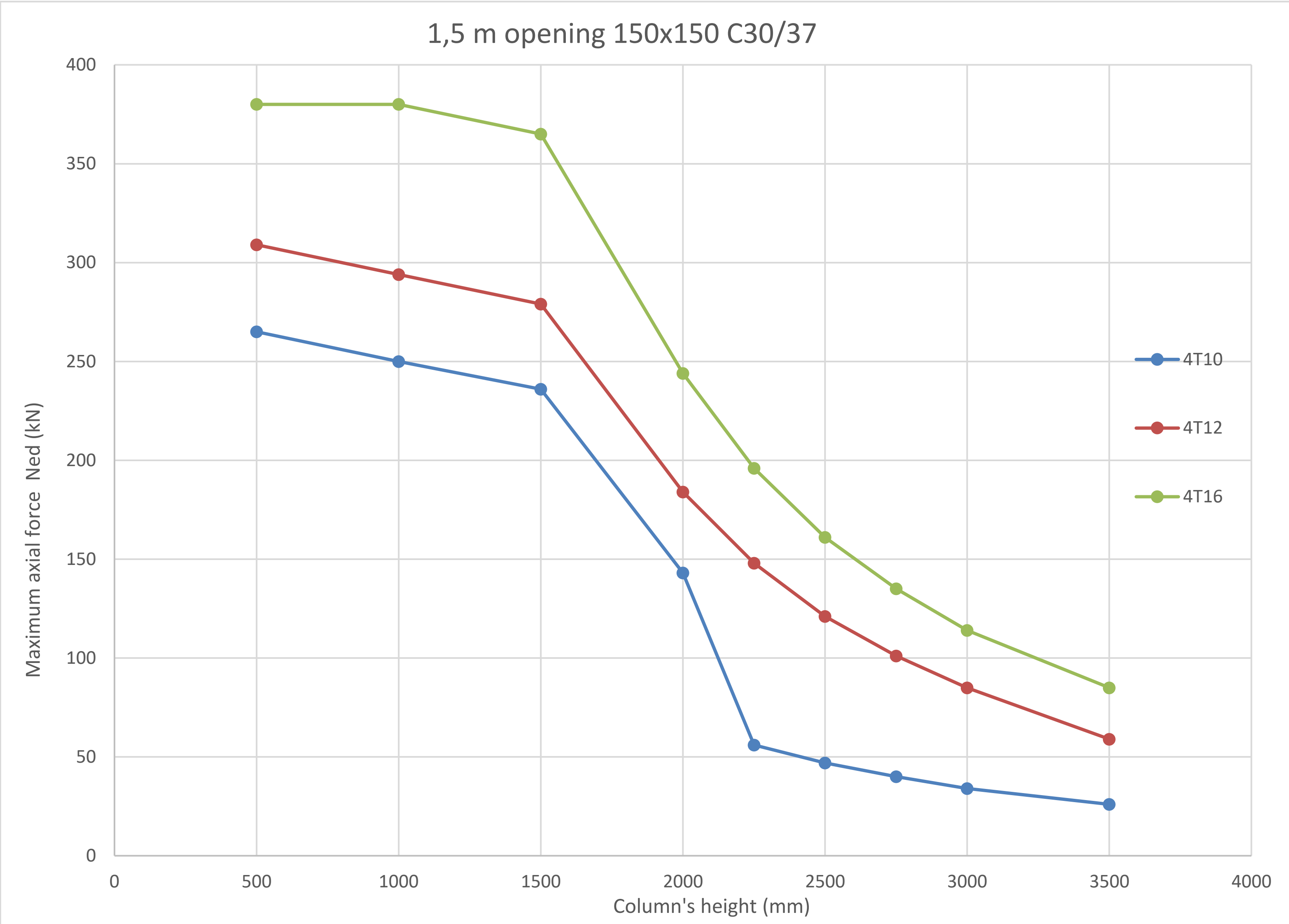


**Figure 11.** Graph of reinforcement for 150x150 column with 1,5m opening.

It is visible in Table 8 what are the maximum loads for different types of reinforcement at different column heights.

In Figure 11 we can see the behavior of a reinforced column under different loads and with different heights. Each curve refers to different reinforcement setting, which can differ in diameter of bars and in the amount of them. Each point of the curves corresponds to the critical loading of the column at the specific height.

Table 8. Calculated reinforcement for the 150x300 column with 2 m opening.

| | **4T10** | **4T12** | **4T16** | **6T10** | **6T12** | **6T16** |
|---|---|---|---|---|---|---|
| **Height (mm)** | **Load (kN)** | **Load (kN)** | **Load (kN)** | **Load (kN)** | **Load (kN)** | **Load (kN)** |
| **3500** | 131 | 219 | 331 | 222 | 294 | 409 |
| **3000** | 254 | 335 | 454 | 330 | 412 | 542 |
| **2750** | 343 | 428 | 550 | 422 | 501 | 645 |
| **2500** | 496 | 570 | 699 | 561 | 649 | 801 |
| **2250** | 696 | 755 | 870 | 746 | 822 | 952 |
| **2000** | 746 | 785 | 870 | 784 | 834 | 952 |
| **1500** | 750 | 785 | 870 | 784 | 834 | 952 |
| **1000** | 750 | 785 | 870 | 784 | 834 | 952 |
| **500** | 750 | 785 | 870 | 784 | 834 | 952 |

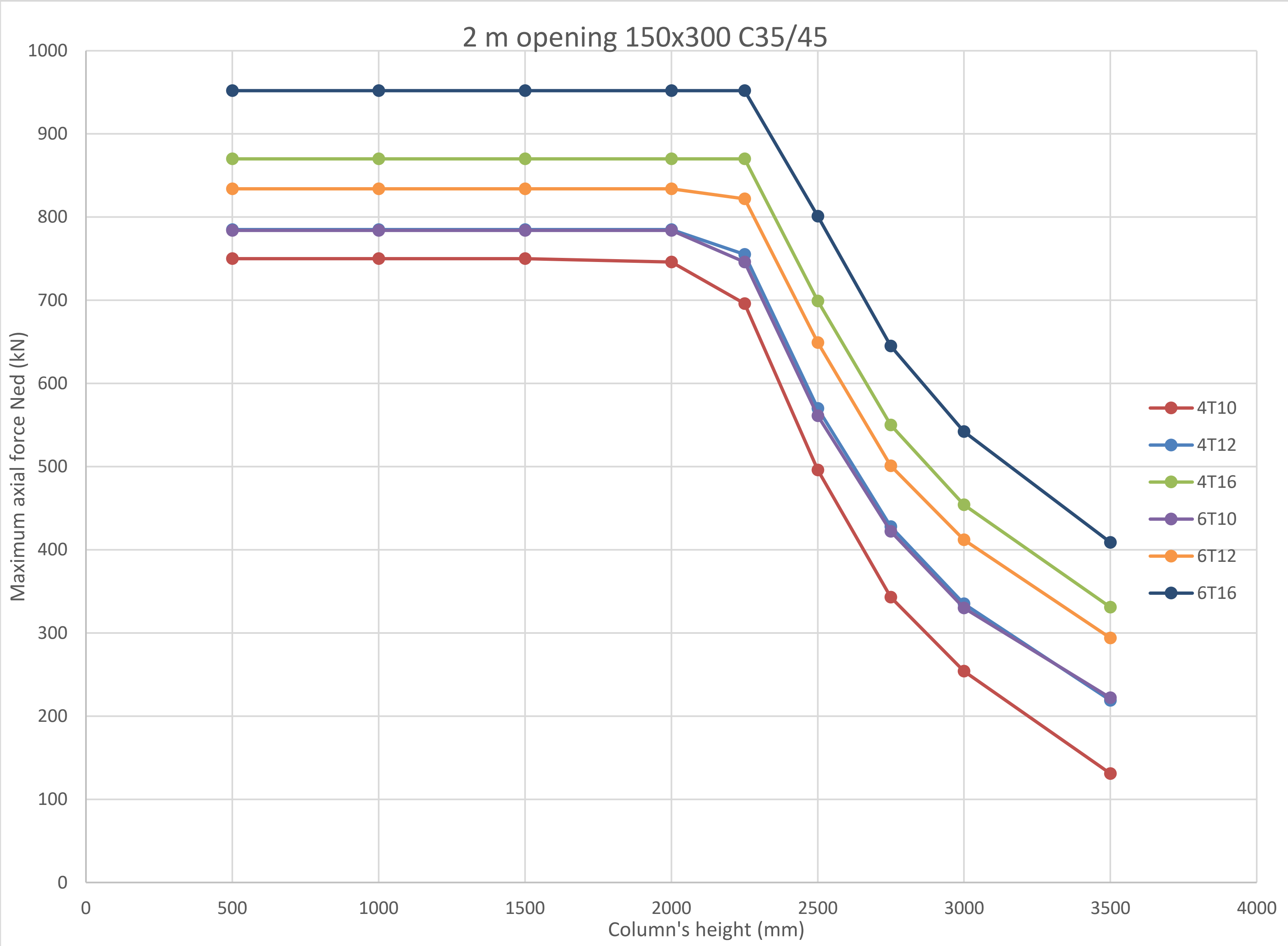


**Figure 12.** Graph of reinforcement for 150x300 column with 2 m opening.

In Table 8 it is visible what are the maximum loads for different types of reinforcement at different column heights.

In Figure 12 we can see the behavior of a reinforced column under different loads and with different heights. Each curve refers to different reinforcement setting, which can differ in diameter of bars and in the amount of them. Each point of the curves corresponds to the critical loading of the column at the specific height.

## 4.4 Result overview

During the calculation of columns' capacities, it was noticed that at some heights columns' capacities do not change. (Table 8.) In the graph in Figure

12 it is visible that before a certain height curves are straight. This happens because columns are short, and the cross-section is behaving fully under compression, therefore the critical load is the compression resistance of the column. After that a sudden drop takes place, when the column becomes slender and buckling is driving the results.

# 5 CONCLUSION

The implementation of standardized reinforcement tables will save a considerable amount of time for engineers that would design the precast wall structure elements. The usage of created tables will eliminate possible mistakes in simple design and will save time of the company.

However, the created tables are suitable only for a limited number of cases. If the span of the floor or the width of the opening is bigger than the ones considered in this study, or concrete class or reinforcement type is different, tables are not suitable for use. However, as this study was based on the real projects, the tables will be suitable for most of the cases.

It was discovered that short columns have maximum resistance based on compression. At some point, which corresponds to a certain height of column, the reinforcement curve goes down. At that height the column becomes slender and capacity decreases.

Lastly, the tables are handy and easy to use. Throughout the process of conducting this study, it was noticed that the calculation of these columns was at some level a repetitive process, where one can easily miss some information which can lead to a mistake. Therefore, the usage of standardized graphs for reinforcement, created in this study, will be indeed a good asset for designing of reinforced concrete elements.

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