Risk-Consistency of Force-Based and Displacement-Based Design of Reinforced Concrete Moment Frames

A Dissertation Submitted in Partial Fulfilment of the Requirements
for the Master Degree in

Earthquake Engineering

By

Carlos Andrés Mora Castrillo

Supervisors: Dr. Ricardo Monteiro
Dr. Gerard O'Reilly

February, 2019

Istituto Universitario di Studi Superiori di Pavia

The dissertation entitled “Risk-Consistency of Force-Based and Displacement-Based Design of Reinforced Concrete Moment Frames”, by Carlos Andrés Mora Castrillo, has been approved in partial fulfilment of the requirements for the Master Degree in Earthquake Engineering.

Dr. Ricardo Monteiro_________________

Dr. Gerard O’Reilly_________________

ABSTRACT

Current trends in seismic engineering are focusing on the quantification of risk to provide a better tool for engineers and stakeholders when making decisions in design and construction. Codes and standards around the world have focused their attention on hazard and have prescribed requirements and provisions to follow so that certain performance objectives (e.g. life-safety) are met. It is unclear, however, what is the associated risk of these designs and if there is risk-consistency among them.

The traditional methodology employed by codes is force-based design, as it is easy to implement in computer programs and it is easy to understand. However, through research, it has been argued that this method, in some cases, is inconsistent with the complex non-linear behaviour of the real structure. For this reason, there has been effort in developing new approaches like displacement-based design that tries to attend these inconsistencies. This study provides insight on the main differences between the methods and makes a comparison on the risk associated for both, specifically for reinforced concrete moment frames.

This study shows, for a set of building heights, how the design is performed for each method, and then provides a complete hazard assessment for a site in San José, Costa Rica. Finally, a risk assessment is performed to obtain the mean annual frequency of exceedance of a certain drift for all the designs. It is shown that the risk associated with both methods is very similar, but there is no risk-consistency amongst the heights of the buildings.

Keywords: seismic risk; displacement-based design; force-based design; risk assessment; collapse.

ACKNOWLEDGMENTS

To my wife, who has been my support and companion throughout this amazing journey.

I would also like to make a special acknowledgment to my supervisors, who have shared their knowledge and time.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS

A_{cp}: area enclosed by the outside perimeter of the concrete cross section.

A_g: gross cross-sectional area.

A_{sh}: area of shear reinforcement within the spacing s.

$A_{s,top,end\ 1}$: area of the longitudinal reinforcement at the top of the section.

$A_{s,middle,end\ 1}$: area of the longitudinal reinforcement at the middle of the section.

$A_{s,bot,end\ 1}$: area of the longitudinal reinforcement at the bottom of the section.

B: concrete section width.

β_{Sc}: dispersion due to record-to-record variability.

β_{USc}: dispersion due to epistemic uncertainty.

β_{TSc}: total dispersion.

C_d: deflection amplification factor.

C_s: seismic coefficient.

C_t: parameter used for the estimation of structural period as established in ASCE 7-16.

C_u: coefficient for upper limit on calculated period which is variable depending on the value of S_{D1}.

C_{vx}: vertical distribution factor.

d: distance from the extreme compression fibre to the centroid of the longitudinal tension reinforcement.

D: dead load.

$d_{b,longitudinal}$: longitudinal reinforcement bar diameter.

$d_{b,shear\ reinforcement}$: shear reinforcement bar diameter.

E: earthquake load.

E_c: elastic modulus of concrete

E_s: elastic modulus of steel.

E_h: horizonal earthquake load.

E_v: vertical earthquake load.

f'_c: concrete compressive strength at 28 days.

f_y: steel reinforcement tensile strength.

F_a: short period site coefficient as defined in table 11.4-1 of ASCE 7-16.

F_e: elastic seismic demand force.

F_i: seismic force in level i.

F_{im}: seismic force in level i for mode m.

F_v: long period site coefficient as defined in table 11.4-2 of ASCE 7-16.

F_x: force associated with base shear distribution in floor x.

g: acceleration of gravity.

H: section height.

H_e: system with an equivalent height.

$H(s)$: hazard curve.

$H(\hat{s}_c)$: median hazard value.

h_i and h_x: height (ft or m) from the base to level i or x.

H_n , h_n: height of the building in meters.

x: exponent that depend on the structural typology of the building, as defined by ASCE 7-16.

I_e: importance factor as defined in ASCE 7-16.

I_g: gross moment of inertia.

k: an exponent related to the structure period as defined in ASCE 7-16.

k_1 and k_o: slope and the intercept of the line that is being fitted for the hazard curve in the log-log space.

k_2: local hazard curvature.

K_e: equivalent stiffness.

L: live load.

L_r: roof live load.

m_e: equivalent mass.

m_i: mass associated with mode shape i.

M_{pr}: probable flexural strength.

M_u and M_n: factored moment force and the nominal moment strength respectively.

$\sum M_{nc}$: sum of nominal flexural strengths of the columns framing into the joint.

$\sum M_{nb}$: sum of nominal flexural strengths of the beams framing into the joint.

p_{cp}: outside perimeter of the concrete cross section.

P_u: factored axial force from the analysis.

R: response modification factor as defined in ASCE 7-16.

R_ξ: modification factor for displacement spectrum.

Sa_m: acceleration corresponding to each mode.

s: shear reinforcement spacing in the hinge region.

S: snow load.

S_1 : spectral acceleration for 1s period for maximum credible earthquake with a return period of 2500 years.

S_{DS} : design spectral response acceleration parameter in the short period range defined in ASCE 7-16.

S_{D1}: design spectral response acceleration parameter at a period of 1.0 s as defined in ASCE 7-16.

S_{M1}: S_1 modified by site class effects.

S_{MS}: S_s modified by site class effects.

S_s: spectral acceleration for short period (0.2s) for maximum credible earthquake with a return period of 2500 years.

T: period of the structure.

T_a: estimated period of structure.

T_e: equivalent period.

T_L: long-period transition period as defined in ASCE 7-16.

T_m: period corresponding to specific m mode.

T_u: factored torsion force from the analysis.

V : total design lateral force or shear at the base of the structure.

V_b: base shear.

V_n: nominal shear strength of the section.

V_t: modal base shear.

V_u: factored shear force.

W: effective seismic weight as defined in ASCE 7-16.

w_i and w_x : the portion of the total effective seismic weight of the structure (W) located or assigned to level *i* or *x*.

x: parameter used for the estimation of structural period.

ρ_m: mass participation factor.

ρ_s: ratio of A_s to the gross section area.

ϕ: strength reduction factor.

ϕ_{im}: mode shape for the m mode.

θ_i: inter-storey drift.

θ_c: drift limit targeted at the beginning of the design.

ω_θ: higher mode drift reduction factor.

ξ: equivalent viscous damping.

ξ_{eq}: equivalent viscous damping for DDBD.

δ_x: inelastic deflection.

δ_{xe}: elastic deflection from the analysis.

Δ_a: allowable storey drift according to ASCE 7-16.

Δ_d: ductility demand of the system can be calculated as the ratio between the characteristic displacement.

Δ_{i+1} and Δ_i: are the maximum displacement in consecutive levels.

Δ_y: yield displacement.

λ: concrete modification factor for light-weight concrete.

λ_{LS}: MAFE of a certain limit state.

Ω_o: system overstrength factor.

1 INTRODUCTION

Current building codes and standards provide guidelines for performance-based seismic design. One of the main performance objectives is to ensure life safety (i.e. the non-collapse of the building) during extreme seismic events. To this end, structures are designed following the provisions stated in the codes and standards, using a site-specific response spectrum that is obtained from probabilistic seismic hazard analysis as input. In this sense, building codes are hazard-oriented but do not necessarily provide a measure for the implicit performance in terms of risk. Additionally, current codes are mostly based on linear analysis procedures modified by system response factors, which convert the complex non-linear dynamic behaviour of structures into a simpler linear one. These response factors vary from one code to the next, and often are decided by code committees.

In recent years, efforts have been made in order to reduce the implicit risk considerations in design codes. For example, FEMA P695 (FEMA & ATC, 2009) recommends a methodology to quantify, in a reliable way, the system response factors so that there is a clear and quantifiable relation between these and the performance objectives of the code. Moreover, seismic assessment methodologies have improved and performance objectives are checked in a more detailed manner using non-linear analysis, such as FEMA P-58 (Federal Emergency Management Agency, 2012), for example. In Europe, improvements are being implemented in Eurocode 8's new revision (European Committee for Standardisation, 2017), that introduces some reliability-based verifications. In New Zealand, knowing the seismic risk of buildings has been a priority, since poor building performance in recent earthquakes was observed. Approximately 16% of the reinforced concrete (RC) buildings were severely damaged in the central business district of Christchurch during the 2011 earthquake and more than 150 fatalities were reported (Kam & Pampanin, 2011). In response to this, a system for managing and classifying buildings as a function of their seismic risk has been developed, and applied using the EPB methodology (Ministry of Business Innovation and Employment of New Zealand, 2017) which gives a rating as a percent of the new building standard achieved (%NBS).

Traditional methods, such as force-based design (FBD), should be examined further in order to have some idea on the risk associated with structures designed this way. Furthermore, other methods, such as displacement-based design (DBD), should also be evaluated in a similar manner, as this approach can give a better understanding of structural behaviour and may provide more risk-consistent solutions than traditional methods, as a number of inherent limitations in the design philosophy of FBD have been addressed by Priestley (2003) .

Evaluating the implicit seismic risk of the different design methods is a complex and extensive task. This study is a first step to evaluate and compare the risk-consistency of code compliant buildings, specifically, high ductility moment resisting RC frames, designed according to the two main methodologies available: force-based and displacement-based design. FBD design was performed following the provisions of ASCE 7-16 (ASCE, 2017) since this is the design code adopted in Costa Rica, the location chosen for this case study application, while DBD was performed following the model code DBD12 (Sullivan et al., 2012). In both cases, for consistency, the member detailing was performed following the provisions of ACI 318-14 (ACI, 2015) for the design member forces identified using the respective methods.

This study scope is limited to only one site, the city of San José, Costa Rica, for which a specific hazard assessment was carried out and is presented in Chapter 5. This document is divided into the following chapters:

Chapter 2 provides a general background on the seismic design and assessment methodologies. The main design concepts for both FBD and DBD are explained. A general section on the theory used for seismic performance assessment is provided, as well as an overview of some relevant recent studies regarding risk assessment of similar building typologies.

Chapter 3 presents a general overview of the analysed buildings, particularly their geometry, materials and structural system. Design loads are assigned in this chapter following the minimum values proposed by ASCE 7-16 (ASCE, 2017), as this is the main reference for the Costa Rican seismic code (CFIA, 2011).

Chapter 4 provides insight on the specific building designs for both methods. It presents the structural models that were used, the applied forces and the element design for both methods. Additionally, a comparison is made between both design methods and a preliminary assessment of the design is performed via static pushover analysis.

Chapter 5 explains how the site-specific hazard information was obtained in order to carry out seismic design and assessment in a hazard-consistent manner. A probabilistic seismic hazard assessment is performed, and ground motion records were selected to then perform the risk assessment.

Chapter 6 assesses the implicit seismic risk for all the designs through incremental dynamic analysis and provides a comparison of the mean annual frequency of exceedance of predefined drifts in the buildings for both the structures designed using FBD and DBD.

The main objective of the study is to evaluate and compare FBD and DBD in order to understand their main design differences and the implications of this on the implicit risk associated with each design. Furthermore, risk-consistency for both methods and different building height can be analysed.

2 LITERATURE REVIEW

2.1 Introduction

Two main design methodologies (FBD and DBD) are proposed in this study, recognising that these are the more developed ones, where research has focused in the last decades and are typically encountered in the literature. FBD is widely used in codes and standards around the world, while DBD has been more confined to academic research. Recently, a model code, DBD12 (Sullivan et al., 2012), has been proposed with a view to implementing such a design approach in building codes in the near future. This section provides the general theoretical background for both methods and their code implementation. Furthermore, a general background for the tools and methods used during the risk assessment is presented, as well as previous studies surrounding the topic.

2.2 Design methodologies

2.2.1 Forced-based design

FBD is the most common methodology adopted by building codes. The design can be performed using the Equivalent Lateral Force Method (ELFM) or the Response Spectrum Method (RSM). The general procedure for force-based design is shown in Figure 2-1. As a first step, all the structural dimensions, including general geometry and member sizes are chosen. With this, the member stiffness is calculated considering the cracked section properties, typically 70% and 35% of the gross moment of inertia for columns and beams respectively, as prescribed by ACI 318-14 (ACI, 2015). Based on these characteristics, an estimation of the fundamental period of the structure is performed by empirical formulations (used for the ELFM) or by eigenvalue analysis (used for the RSM). Having the fundamental period, the base shear and elastic forces can be derived using the elastic response spectrum for the site. These forces are affected by a reduction factor that takes into account the structure type and expected ductility. The structure is then analysed using these reduced forces and the displacements are checked against the limits stated by the design code. This is an iterative procedure to optimise the structure and comply with the requirements. As a final step, capacity design is performed for brittle failure modes (i.e. shear failure) and ensure that all non-dissipative elements must remain elastic during the inelastic response of the structure, and also to ensure a stable mechanism that can maintain the vertical capacity during the inelastic response.

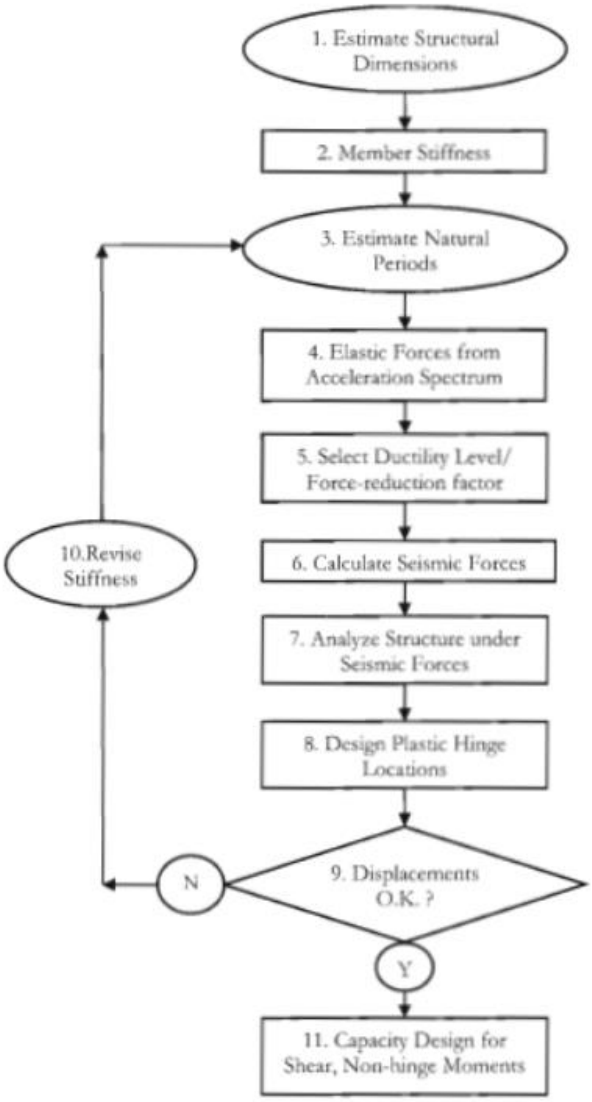


Figure 2-1. Force-based design procedure. (Priestley et al., 2007)

The ELFM can be used when the buildings are fairly regular in plan and height as this method considers a dynamic model consisting of a linear single degree of freedom system and estimates the seismic forces for the fundamental period only, as it can be shown in Figure 2-2. As a first step, the estimated period for the structure, T_a, is calculated in seconds as:

$$T_a = C_t h_n^x \qquad [2\text{-}1]$$

where h_n is the height of the building in meters, while C_t and x are parameters that depend on the structural typology of the building, as shown in **Error! Reference source not found.**. Afterwards, the seismic coefficient, C_s, can be calculated by using Equation [2-2]:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \qquad [2\text{-}2]$$

Table 2-1. ASCE 7-16 table for C_t and x parameters to determine the approximate period of the structure. (ASCE, 2017)

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028 (0.0724)[a]	0.8
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1	0.03 (0.0731)[a]	0.75
Steel buckling-restrained braced frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

[a]Metric equivalents are shown in parentheses.

This value of C_s is limited by Equations [2-3] and [2-4], that depend on the estimated period of the structure:

$$C_s = \frac{S_{D1}}{T_a\left(\frac{R}{I_e}\right)} \quad for \quad T_a \leq T_L \qquad [2\text{-}3]$$

$$C_s = \frac{S_{D1}T_L}{T_a^2\left(\frac{R}{I_e}\right)} \quad for \quad T_a > T_L \qquad [2\text{-}4]$$

where:

S_{DS} = design spectral response acceleration parameter in the short period range defined in ASCE 7-16.

S_{D1} = design spectral response acceleration parameter at a period of 1.0s as defined in ASCE 7-16.

R = response modification factor as defined in ASCE 7-16. This factor is used to obtain reduced forces for the inelastic response of the structure and is equivalent to the q factor shown in Figure 2-2 (c).

I_e = importance factor as defined in ASCE 7-16.

T_a = estimated period of the structure.

T_L= long-period transition period as defined in ASCE 7-16.

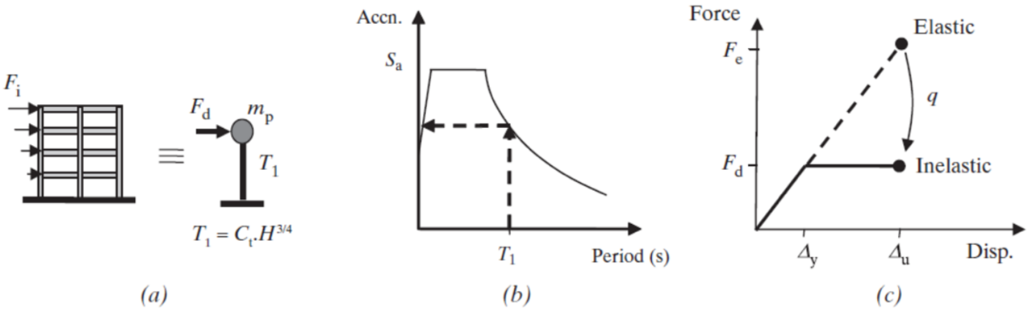


Figure 2-2. Conceptual diagram for ELFM. (Sullivan, 2012)

Having defined the seismic coefficient, the seismic base shear in a given direction is determined using Equation [2-5]:

$$V_{ELFM} = C_s W \qquad [2\text{-}5]$$

where W is the effective seismic weight as defined in ASCE 7-16. Finally, this base shear is distributed vertically along the structure in the following manner:

$$F_x = C_{vx} V_{ELFM} \qquad [2\text{-}6]$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n}(w_i h_i^k)} \qquad [2\text{-}7]$$

where:

C_{vx} = vertical distribution factor.

V_{ELFM} = total design lateral force or shear at the base of the structure for the ELFM.

w_i and w_x = the portion of the total effective seismic weight of the structure (W) located or assigned to level i and x, respectively.

h_i and h_x = the height (ft or m) from the base to level i and x, respectively.

k = an exponent related to the structure period as defined in ASCE 7-16.

As a more refined approach, the RSM can also be used. In this case, a multi-modal analysis is performed, and higher modes are directly considered. According to ASCE 7-16, the number of modes used for the analysis should be sufficient to obtain a cumulative modal mass participation of at least 90% of the actual mass.

The general procedure in this case is to calculate, through eigenvalue analysis, the mode shapes (ϕ_{im}) and the corresponding periods (T_m), where m corresponds to different mode shapes and i corresponds to the mass locations. Then, the modal mass participation factor is calculated as:

$$\rho_m = \frac{\left(\sum_{i,m=1}^{N} \phi_{im} m_i\right)^2}{\sum_{i,m=1}^{N} \phi_{im}^2 m_i \sum_{i=1}^{N} m_i} \qquad [2\text{-}8]$$

Then, the modal base shear for each of the modes is calculated as:

$$V_t = Sa_m g \left(\rho_m \sum_{i=1}^{N} m_i\right) \qquad [2\text{-}9]$$

where Sa_m is the acceleration corresponding to each mode and g is the acceleration due to gravity. Finally, the forces for each of the modes are distributed along the structure as follows:

$$F_{im} = V_t \frac{\phi_{im} m_i}{\sum_{i,m=1}^{N} \phi_{im} m_i} \qquad [2\text{-}10]$$

Having this distribution of forces for each mode, the internal forces in the elements or interest parameters can be calculated. The combined response of the parameters can then be obtained by using the square root of the sum of the squares (SRSS) method, the complete quadratic combination (CQC) method or any other approved method prescribed in ASCE 7-16.

For the case of ASCE 7-16, the forces obtained using RSM must be scaled so that when the combined response for the combined modal base shear is less than 100% of the calculated base shear obtained by ELFM, the forces are multiplied by V/V_{RSM} where:

V_{ELFM} = base shear obtained by ELFM.

V_{RSM} = combined base shear obtained by RSM.

The scaling is done to provide a minimum base shear for design and ASCE 7-16 explains that this scaling is due to studies showing that using less than 100% of the base shear calculated by ELFM can result in larger probabilities of collapse than the targeted 10% in 50 years. Doing this scaling is intended to mitigate this increased collapse vulnerability. It is noted, however, that the deformations are not scaled in a similar manner. This essentially means that the force reduction factor is being reduced to give the same base shear as the ELFM.

One of the main reasons to use a force-based method, as described before, is that it is easy to implement from a computational point of view. However, this method has several drawbacks that can cause inconsistency from a mechanics-based viewpoint. Sullivan (2012) presents the following shortcomings to the force-based method:

- Force reduction factors should not be set independent of expected ductility demand: non-structural displacement limits can often limit the ductility demand in the structure, and therefore setting reductions factors based solely on the ductility capacity of the members appears to be misleading in terms of actual building performance;

- The use of elastic stiffness for the prediction of inelastic force distribution: because of the formation of plastic hinges in some elements at different stages of the seismic loading, the stiffness can drastically change in the inelastic range, causing the distribution of forces in the building to differ from that of a purely elastic system;

- The relationships used to relate elastic displacement to inelastic displacement response: as Figure 2-2 (c) shows, in force-based methods it is generally assumed that the displacement of the inelastic system is equal or similar to the displacement of the elastic system under the elastic seismic demand force (F_e). This relationship, in reality, depends on the hysteretic properties of the structure, and this is not well-addressed in the force-based methods.

2.2.2 Displacement-based design

Large research efforts have been made to develop alternative methodologies that can address the deficiencies of force-based methods of design. Many displacement-based methods have been proposed, however, the most developed one is the direct displacement-based design (DDBD) method described by Priestley et al. (2007), which will be the one used in this study. The DDBD method consists of transforming the structure into an equivalent single degree of freedom (SDOF) system with an equivalent height (H_e), an equivalent mass (m_e), and equivalent stiffness (K_e) and an equivalent viscous damping (ξ), as shown in Figure 2-3.

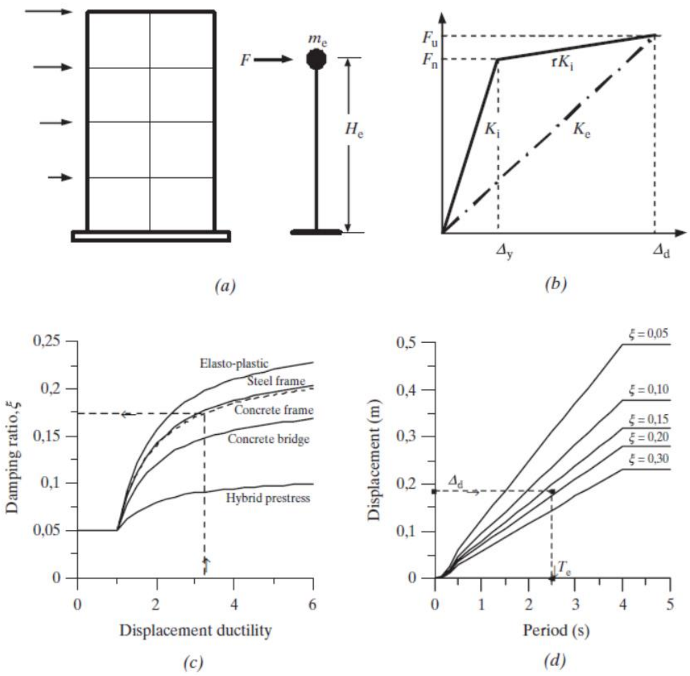


Figure 2-3. Conceptual diagram for DDBD. (Priestley et al., 2007)

Contrary to force-based methods, where the design is performed in an iterative way, checking the displacements at the end of the process, the first step in DDBD method is to establish a performance criterion (usually storey drift) in the following form:

$$\theta_i = \frac{(\Delta_{i+1} + \Delta_i)}{(x_{i+1} - x_i)} \qquad [2\text{-}11]$$

where Δ_{i+1} and Δ_i are the maximum displacement in consecutive levels and $(x_{i+1} - x_i)$ is the height of each level. For the case of RC frames, the displacement demands are expressed in the following way:

$$\Delta_i = \omega_\theta \theta_c h_i \left(\frac{4H_n - h_i}{4H_n - h_1}\right) \qquad [2\text{-}12]$$

where h_i is the height of the level in study, H_n is the total height of the building, ω_θ is a higher mode drift reduction factor and θ_c is the drift limit targeted at the beginning of the design. The displacements obtained from Equation [2-12] are used to calculate the equivalent SDOF system characteristics as follows:

$$\Delta_d = \frac{\sum_{i=1}^{n}\left(m_i \Delta_i^2\right)}{\sum_{i=1}^{n}\left(m_i \Delta_i\right)} \quad [2\text{-}13]$$

$$m_e = \sum_{i=1}^{n} (m_i \Delta_i)/\Delta_d \quad [2\text{-}14]$$

$$H_e = \sum_{i=1}^{n}(m_i \Delta_i h_i) \Bigg/ \sum_{i=1}^{n}(m_i \Delta_i) \quad [2\text{-}15]$$

The ductility demand of the system can be calculated as the ratio between the characteristic displacement (Δ_d) and the yield displacement (Δ_y) as shown in Figure 2-3(b). Then, the equivalent viscous damping, which is defined as a function of the ductility demand of the system (as opposed as what is done in force-based methods), can be obtained as shown in Figure 2-3(c), depending on the type of structure in study. Having this equivalent damping, one can scale the design displacement spectrum using a spectral modification factor. The DBD12 Model Code provides the following expression for the modification factor:

$$\mathrm{R}_\xi = \left(\frac{0.07}{0.02 + \xi}\right)^{0.5} \quad [2\text{-}16]$$

With this scaled displacement spectrum, it is possible to read the required effective period (T_e) as shown in Figure 2-3(d). Then, the effective stiffness and design base shear can be obtained from the following expressions:

$$\mathrm{K_e} = 4\pi^2 \frac{\mathrm{m_e}}{\mathrm{T_e^2}} \quad [2\text{-}17]$$

$$\mathrm{V_b} = \mathrm{K_e}\Delta_\mathrm{d} + \mathrm{C}\frac{\mathrm{m_e g}\Delta_\mathrm{d}}{\mathrm{H_e}} \quad [2\text{-}18]$$

It is important to notice that the right-side term of Equation [2-18] takes into account P-Delta effects, where C is a coefficient that depends on the structural typology utilised. The base shear (V_b) can be distributed along the height of the structure as equivalent lateral forces in the following form:

$$\mathrm{F_i} = \mathrm{kV_b}\,(\mathrm{m_i}\Delta_\mathrm{i}) \Bigg/ \sum_{\mathrm{i}=1}^{\mathrm{n}}(\mathrm{m_i}\Delta_\mathrm{i}) \quad [2\text{-}19]$$

It is recommended that 10% of the base shear is lumped at the roof level for structures that develop plastic hinges throughout the height of the building, such as frame buildings. The k factor in Equation [2-19] is introduced to take this into account, i.e., in the case of lumping 10% of the base shear in the roof level, then $\mathrm{k} = 0.9$. In this way, 90% of the base shear is distributed throughout the height and 10% is lumped at the roof level.

2.3 Risk assessment methodology

As discussed earlier, current codes intend to produce building designs that can meet certain performance objectives (e.g. maximum inelastic drift for a certain return period), but they are prescriptive by nature and do not provide a methodology to properly evaluate if these performance levels are met and if the associated risk is acceptable for the design.

Evaluating the implicit risk of code-conforming buildings requires a detailed assessment that can provide a quantitative reference on the amount of safety of a structure for a given limit state. As the variables involved in such an analysis (e.g. hazard assessment, modelling uncertainty) are probabilistic in nature, then this amount of safety can be expressed as the mean annual frequency of exceeding a certain limit state threshold of a chosen engineering demand parameter (EDP), which can be defined in a number of ways, usually in terms of storey drift.

After observing the poor performance of moment-resisting steel frame buildings in the 1994 Northridge earthquake, the SAC/FEMA project was founded to study and improve the performance of these building typologies. One of the main results of this project was the concept of seismic performance assessment through the implementation of the mean annual frequency of exceedance (MAFE) of a limit-state proposed by (Cornell et al., 2002). This approach links the probabilistic seismic hazard analysis (Cornell, 1968) with the structural response through an EDP via the hazard curve for a specific site. By convolution of the hazard curve, $H(s)$, the MAFE of a certain limit state (λ_{LS}) can be estimated as proposed by:

$$\lambda_{LS} = \int_0^{+\infty} \frac{dP(C < D|s)}{ds} H(s) ds \qquad [2\text{-}20]$$

where C and D are the capacity and demand, respectively, of a certain structure.

In order to find a closed-form solution (Cornell et al., 2002) proposed a linear approximation of the hazard curve in the log-log space as follows:

$$H(s) = P = k_o s^{-k_1} = k_o \exp(-k_1 lns) \qquad [2\text{-}21]$$

where k_1 and k_o represent the slope and the intercept of the line that is being fitted in log-log space.

Bradley & Dhakal (2008) demonstrated that this solution is very sensitive to the selected values of k_1 and k_o, and can induce rather large calculation errors of the MAFE and proposed an alternative hyperbolic model fit. For this reason, Vamvatsikos (2013) proposed a fitting solution that is more reliable and gives a better fitting throughout the curve by making a second-order approximation in the log space as follows:

$$H(s) = k_o \exp(-k_2 ln^2 s - k_1 lns) \qquad [2\text{-}22]$$

where k_2 represents the local hazard curvature, which is essentially an extension to the functional form of the linear model outlined in Equation [2-21]. Then, the final closed-form solution for the MAFE with a 50% confidence interval proposed by Vamvatsikos is given as follows:

$$\lambda_{LS} = \sqrt{p}\ \ k_o^{1-p}\ [H(\hat{s}_c)]^p \exp\left[\frac{k_1^2}{4k_2}(1-p)\right] \qquad [2\text{-}23]$$

where $H(\hat{s}_c)$ is the median hazard value and p can be expressed in the following way:

$$p = \frac{1}{1 + 2k_2\beta_{Sc}^2} \qquad [2\text{-}24]$$

where β_{Sc} is the intensity measure (IM) dispersion due to record-to-record variability (i.e. aleatory uncertainty). In addition to this, it is possible to take into account the epistemic uncertainty (e.g. modelling uncertainty) by introducing additional dispersion β_{USc} due to epistemic sources. It is possible to obtain the total dispersion β_{TSc} by using the square root of the sum of squares (SRRS) as follows:

$$\beta_{TSc}^2 = \beta_{Sc}^2 + \beta_{USc}^2 \qquad [2\text{-}25]$$

Figure 2-4 shows, in a graphical way, the process explained above to obtain a MAFE curve. Figure 2-4(a) shows the median and the 16% and 84% fractile of the IDA distribution. For a given level of drift θ_1 it is possible to obtain the median $\hat{s}$ and the dispersion β_{Sc} for the considered IM. Then, with the fitted hazard curve $H(s)$ shown in Figure 2-4(b) it is possible to obtain the MAFE λ_{LS} of the limit state defined by the drift θ_1. This process may be repeated for different levels of drift (i.e. different limit states) to obtain the final MAFE curve shown in Figure 2-4(c), where it is possible to read, for a given drift θ_1, the corresponding MAFE λ_{LS}.

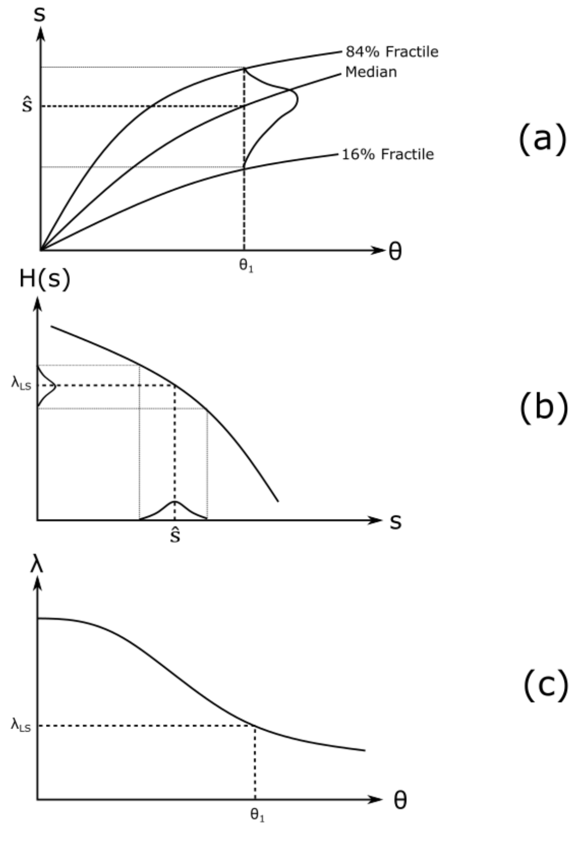


Figure 2-4. Graphical representation of risk assessment process for the MAFE.

2.4 Previous studies on risk consistency on code compliant buildings

Knowing the risk and being able to quantify it has been the scope of important projects around the world, such as FEMA P-58 (Federal Emergency Management Agency, 2012) that has created a complete methodology that is aimed to assess the seismic performance of new and existing buildings. Additionally, some effort has been focused on obtaining better response parameters as shown in the report FEMA P-695 (FEMA & ATC, 2009). These response parameters correspond to the response modification coefficient R, the system overstrength factor Ω_o, and the deflection amplification factor C_d that are currently used to modify the elastic analysis done during the FBD methodology proposed in ASCE 7-16 (ASCE, 2017). These factors are particularly important, as they are the basis for the estimation of non-linear behaviour in the FBD method and can affect greatly the seismic performance of buildings designed in this manner, as well as their implicit risk.

Although the aforementioned studies try to improve the seismic performance of the buildings, this does not mean that the result will be risk-consistent when assessed using more robust methodologies. In Italy, the RINTC project (RINTC Workgroup, 2018) has been working on analysing code-conforming structures to assess their implicit seismic risk when designed using the current regulations to examine what the implicitly accepted risk is of these buildings for modern design. Some of the findings have shown that the risk is not consistent, not only for different hazard levels, but for different structural typologies as shown in Figure 2-5.

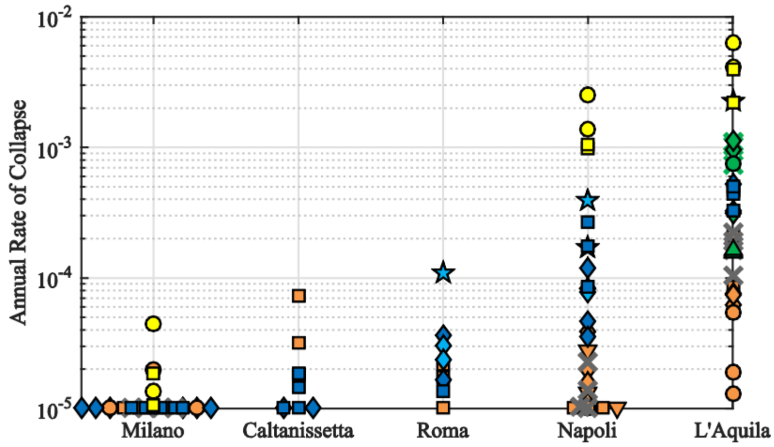


Figure 2-5. Annual rate of collapse for structures subjected to different hazard levels increasing from left to right. (Iervolino, Spillatura, & Bazzurro, 2017)

As an attempt to provide a more uniform risk for the designs, codes, as ASCE 7-16, have implemented risk-targeted hazard maps. These maps aim to establish a design ground motion that can produce a uniform level of risk for different designs. Silva et al. (2014) present a framework from which these risk-targeted hazard maps can be implemented in Europe.

2.5 Summary

This chapter explains the theoretical background for FBD and DBD methodologies that will be used for design of the case study buildings. It is shown how both methodologies deal with the inelastic behaviour of the structures during a seismic event. In the case of FBD, the elastic spectrum is transformed into an “inelastic” spectrum by means of the R factor, and subsequently the design is performed using an elastic model where the forces and displacements need to be modified by the Ω_o and C_d factors as specified in ASCE 7-16. On the other hand, DBD methodology transformed the entire structural system into an equivalent single degree of freedom system where the hysteretic behaviour of the particular structural typology is taken into account to generate a structure with the necessary stiffness to target the limit state of interest, in this case drift. Chapter 3 and Chapter 4 refer to the case study buildings and its design for both these methodologies and which are the main resulting differences.

Additionally, the risk assessment methodology and previous studies referring to this topic where presented. The risk assessment is developed, using this theoretical background, in Chapter 6.

3 GENERAL OVERVIEW OF ANALYSED BUILDINGS

3.1 Introduction

This study focuses on RC frames structures located in Costa Rica. According to site-hazard characteristics explained in Chapter 5, ASCE 7-16 (ASCE, 2017) classifies the seismic design as category D (see Chapter 4), which means that the site is in an area of high seismic hazard. As such, the only typology permitted corresponds to special moment frames as defined by the ACI 318-14 (ACI, 2015).

The risk assessment is performed for 3, 6 and 9 storey buildings with heights of 9 m, 18 m and 24 m respectively. Figure 3-1 shows the general elevation geometry for the 3-storey building, whereas for the other building heights, the same configuration was adopted.

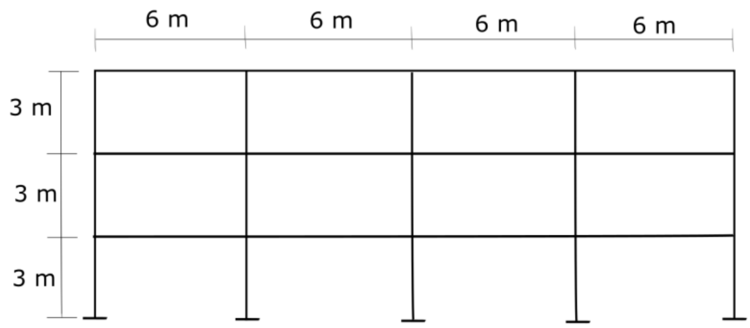


Figure 3-1. Three storey building elevation geometry.

Buildings are fully symmetrical in plan and regular in both height and plan. Each building is designed using DDBD method and RSM, for a total of 6 buildings.

3.2 Materials

The materials used in the design and assessment of the buildings are concrete and steel reinforcement, whose properties are defined in ACI 318-14 (ACI, 2015). Table 3-1 provides the mechanical characteristics used for concrete, where f_c' is the concrete compression strength at 28 days, E_c is the elastic modulus of concrete and λ is a reduction factor for lightweight concrete.

Table 3-1. Concrete main mechanical characteristics.

Parameter	Value	Code requirement	Reference
f_c'	28 MPa	Minimum 21 MPa	ACI Table 19.2.1.1
E_c	24870 MPa	$4700\sqrt{(f_c')}$	ACI 19.2.2.1
λ	1	Depends on type of concrete, 1 is for normal-weight.	ACI Table 19.2.4.2

The characteristics for steel reinforcement are shown in Table 3-2, where f_y is the strength of steel at yield and E_s is the elastic modulus of steel.

Table 3-2. Steel reinforcement characteristics.

Parameter	Value	Code requirement	Reference
f_y	420 MPa	420 MPa for special seismic applications	ACI Table 20.2.2.4a
E_s	200 GPa	200 GPa	ACI 20.2.2.2

3.3 Design loads

The considered loads for design are dead, live and seismic loads, according to ASCE 7-16 (ASCE, 2017) standard for minimum design loads for buildings and other structures. For this study, the loads correspond to a normal office building and its main components. Dead loads are composed of self-weight of structural components such as columns, beams, floor system, and additional loads such as floor finish, ceiling system, lightweight divisions and electromechanical pipes, ducts and equipment. The specific values for each component are shown in Table 3-3. A live load of 2.40 kN/m^2 is used for the design, as this is the minimum value proposed by ASCE 7-16.

Table 3-3. Floor loading (other than self-weight).

Load Type	Component	Weight [kN/m^2]
Dead loads	Floor system	3.20
	Floor finish	0.50
	Ceiling system	0.20
	Electromechanical	0.10
	Light-weight divisions	0.76
	Sub-total	4.76
Live loads	Minimum required by code	2.40
	Sub-total	2.40
Total		7.16

Seismic loading is obtained from probabilistic seismic hazard assessment (PSHA) specifically done for San José, Costa Rica. The results of PSHA will be discussed in Chapter 6.

3.4 Structural system

The structural system used is a special moment frame as defined in Chapter 18 of ACI 318-14 (ACI, 2015) for earthquake-resistant structures. The provisions used for the design provide a structure that can resist earthquake motions through ductile inelastic response of beams and columns. In the case of beams, plastic hinges are formed in both edges of the members, while columns only form plastic hinges at the base of the building. Capacity design is implemented so that the failure mechanism of the frame is ductile and there is no brittle failure due to shear demands in the members.

The study comprises eight buildings: (i) one set of three buildings designed using the FBD approach (ASCE 7-16); (ii) another set of three buildings using the DBD method proposed in the model code DBD12; and (iii) a set of two three-storey buildings (FBD and DBD) designed without taking into account gravity loads in the beams to study the impact in the behaviour and MAFE. The main geometrical characteristics of the buildings are shown in Table 3-4. Figure 3-2 through Figure 3-4 show the main geometrical dimension of the buildings.

Table 3-4. Frame building geometrical characteristics.

Building No.	Design method	Bay span [m]	Storey height [m]	No. storeys
3-FB	FBD	6	3	3
6-FB	FBD	6	3	6
9-FB	FBD	6	3	9
3-DB	DBD	6	3	3
6-DB	DBD	6	3	6
9-DB	DBD	6	3	9
3-FB-NG	FBD	6	3	3
3-DB-NG	DBD	6	3	3

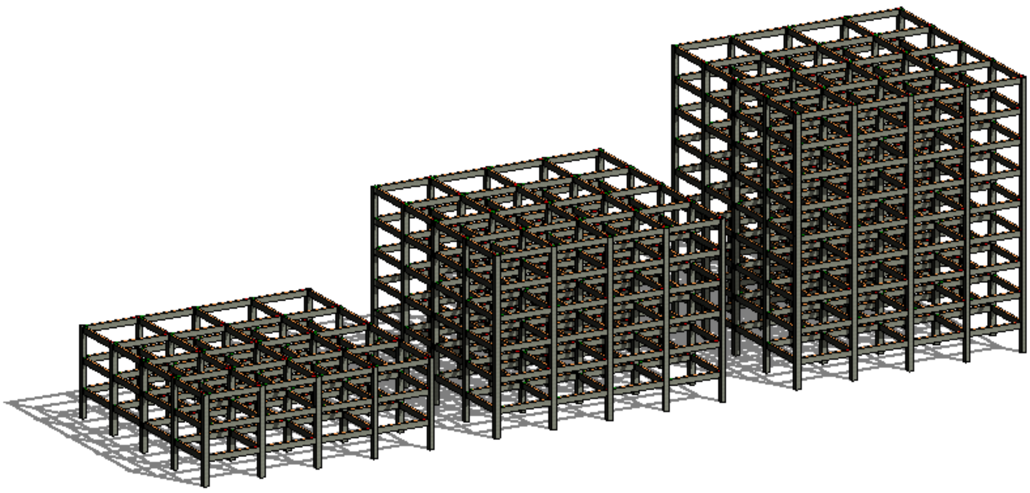


Figure 3-2. 3D view of the buildings.

The analysed buildings are space frames, which means that all of the columns and beams form part of the lateral load resisting system. Figure 3-3 shows the plan configuration of the frame, where it is possible to observe that all columns are connected by beams that form the frames in both directions.

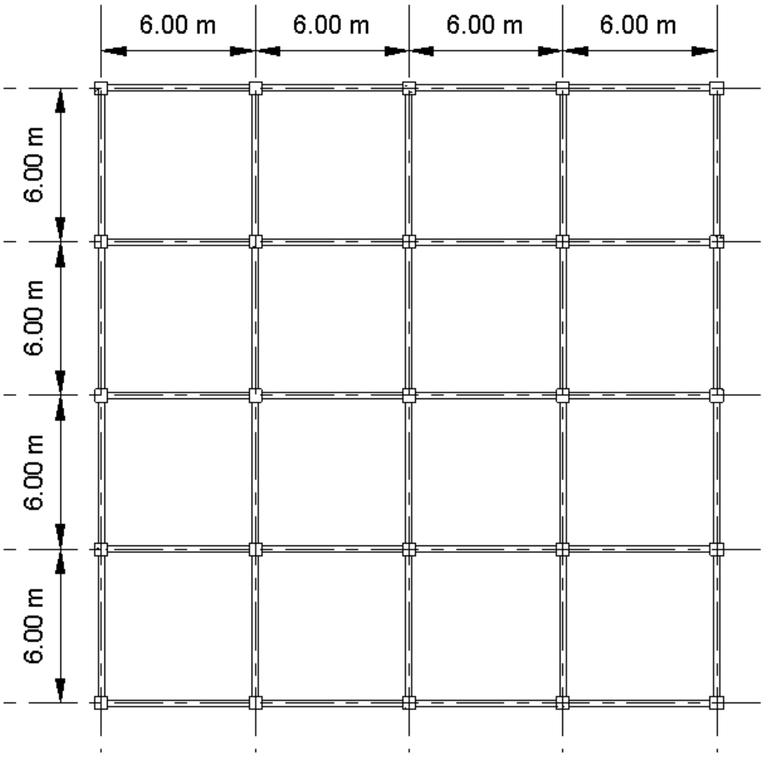


Figure 3-3. Typical floor geometry for the buildings.

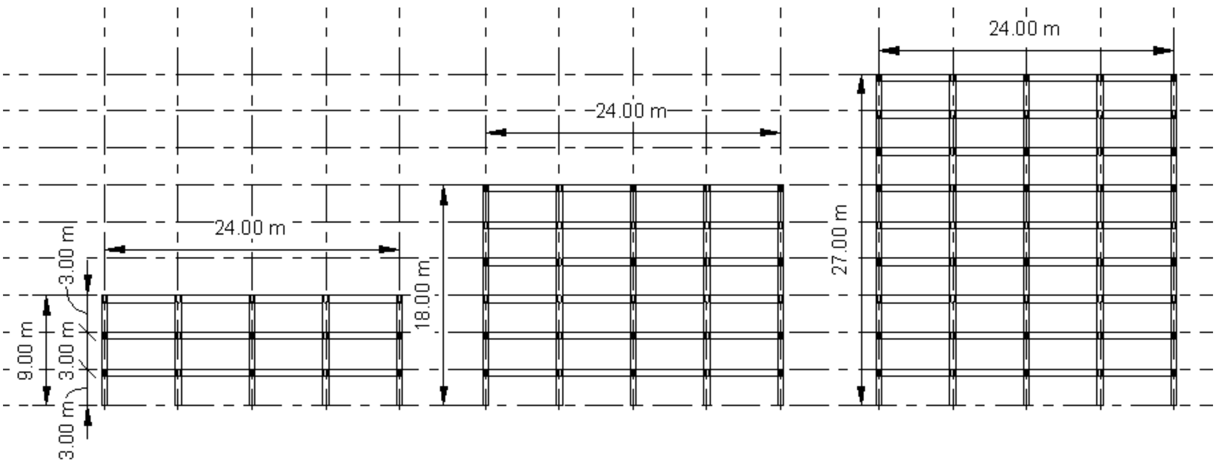


Figure 3-4. Elevation for 3,6 and 9 storey buildings.

3.5 Summary

The main geometrical and material characteristics where of the analysed buildings where presented in this chapter. The following chapter explains in detail the aspects of design for each of the buildings of the case study and makes a comparison between methodologies.

4 BUILDING DESIGN

4.1 Introduction

The design for the complete set of six buildings is performed in this section. Distribution of forces and displacements along the building height are presented for each of the design cases, as well as specific member (columns and beams) design, including geometry and reinforcement areas for both flexure and shear forces.

A comparison between both design results (FBD and DBD) is performed through direct comparison of shear and drift distribution, and through nonlinear static pushover analysis.

4.2 Analysis model

4.2.1 Load factors and combinations

As established in ASCE 7-16 (ASCE, 2017), the structure and its components shall be designed so that their strength equals or exceeds the effects of the factored loads of the following combinations:

- Combination 1 : 1.4D
- Combination 2 : 1.2D + 1.6L + 0.5(L_r or S or R)
- Combination 3 : 1.2D + 1.6(L_r or S or R) + (L or 0.5W)
- Combination 4 : 1.2D + 1.0W + L + 0.5(L_r or S or R)
- Combination 5 : 0.9D + 1.0W
- Combination 6 : 1.2D + E_v + E_h + L + 0.2S
- Combination 7 : 0.9D - E_v + E_h

where D is the dead load, L is the live load, L_r is the roof life load, S is the snow load, R is the rain load, W is the wind load, E_v is the vertical earthquake load and E_h is the horizonal earthquake load. For the purpose of this study, no rain, snow or wind loads were considered.

4.2.2 Model geometry

A two-dimensional elastic analysis was performed in order to obtain the internal forces of the elements according to the force distribution given for both FBD and DBD methods. Figure 4-1 through Figure 4-3 show the geometry of each of the models for 3, 6 and 9 storeys.

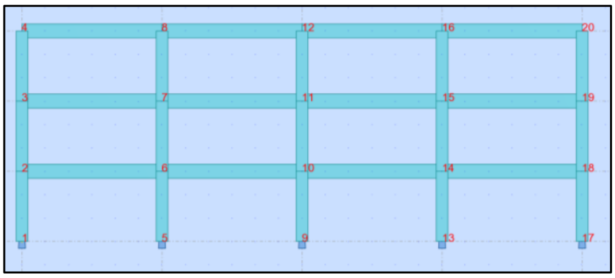


Figure 4-1. Model geometry for 3 storey building.

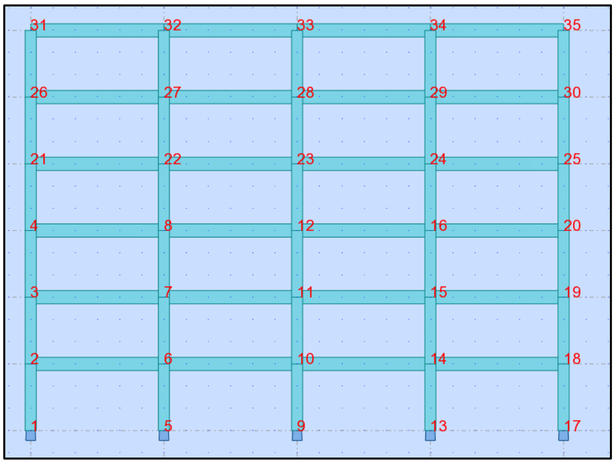


Figure 4-2. Model geometry for 6 storey building.

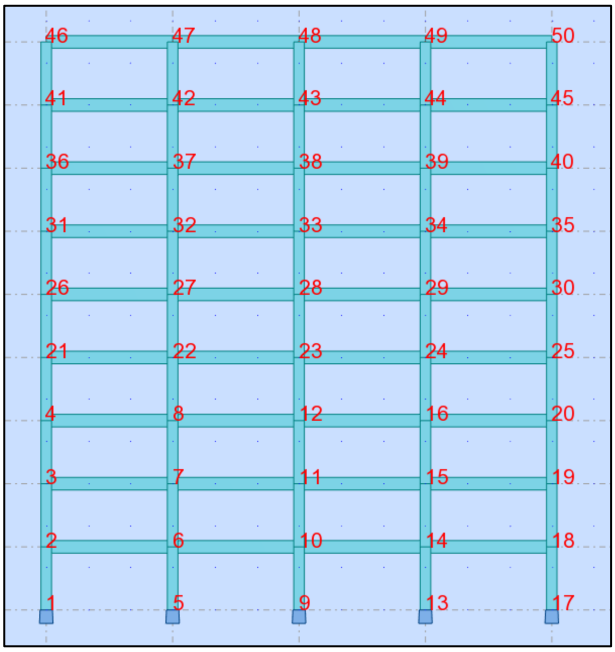


Figure 4-3. Model geometry for 9 storey building.

4.2.3 Sectional properties of elements

To take into account the cracked stiffness of concrete sections, ACI 318-14 (ACI, 2015) foresees that for elastic analysis, the values for cross-sectional area and moment inertia be taken as stated in Table 4-1, where I_g is the gross moment of inertia and A_g is the gross cross-sectional area.

Table 4-1. Moment of inertia and cross-sectional area for elastic analysis.

Member and condition		Moment of Inertia	Cross-sectional area
Columns		$0.70I_g$	$1.0A_g$
Walls	Uncracked	$0.70I_g$	
	Cracked	$0.35I_g$	
Beams		$0.35I_g$	
Flat plates and flat slabs		$0.25I_g$	

4.3 Force-based design

Forced-based design is done using the approach proposed in the standard ASCE 7-16 (ASCE, 2017). For this case study, the response spectrum is obtained using the uniform hazard spectrum (UHS) from the probabilistic seismic hazard assessment done for a specific site in San José, Costa Rica, discussed further in Chapter 5. ASCE 7-16 (ASCE, 2017) defines the steps to define the seismic design category, which will be used to define the seismic design requirements for the buildings designed with FBD method. For this purpose, the parameters in Table 4-2 are defined using the UHS and considering a soil characterised by $V_{s,30}$=265 m/s. The selection of this specific $V_{s,30}$ is related to the analysis being performed for a generic site in San José, for which there is no specific soil testing. As such, the Costa Rican Seismic Code (CFIA, 2011) defines that where the soil properties are not well defined, the soil type should be taken as Type S_3, which has an average value of $V_{s,30}$=265 m/s. As defined by the code, this type of soil has a soil profile of 6-12 m of clay with soft to medium stiffness or non-cohesive soils of low to medium density.

Table 4-2. Seismic parameters for constructing the design response spectrum according to ASCE 7-16.

Parameter	Value	Definition
S_1	0.81g	Spectral acceleration for 1s period for maximum credible earthquake with a return period of 2500 years.
S_s	2.44g	Spectral acceleration for short period (0.2s) for maximum credible earthquake with a return period of 2500 years.
F_a	1	Short period site coefficient as defined in table 11.4-1 of ASCE 7-16.
F_v	1.5	Long period site coefficient as defined in table 11.4-2 of ASCE 7-16.
S_{MS}	2.44g	S_s modified by site class effects.
S_{M1}	1.215g	S_1 modified by site class effects.
S_{DS}	1.63g	Design spectral acceleration for short period (Return period of 475 years).
S_{D1}	0.54g	Design spectral acceleration for long period (Return period of 475 years).

Having the above-mentioned parameters, it is possible to define the elastic design response spectrum as shown in Figure 4-4. This spectrum corresponds to the elastic demand, then, for the design of the ductile moment frame the obtained forces are reduced a factor R equal to 8, value specified by ASCE 7-16 (ASCE, 2017) for RC special moment frames.

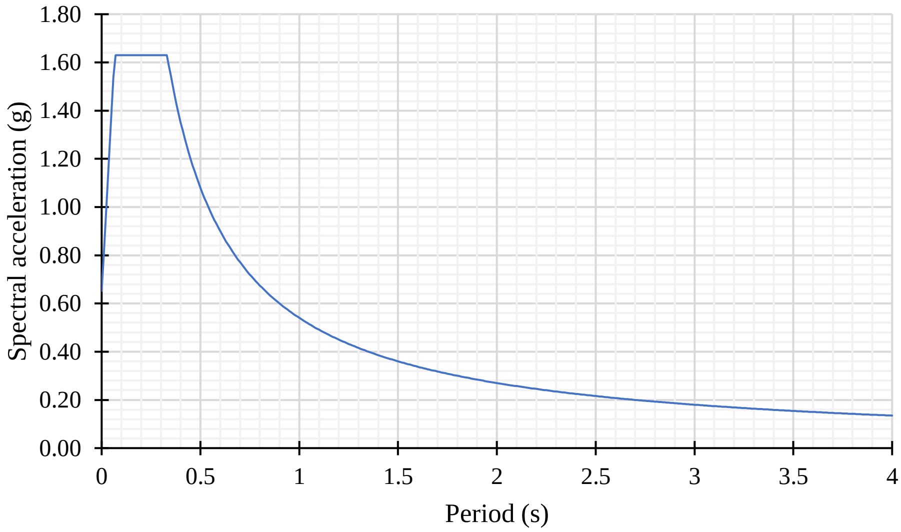


Figure 4-4. Design response spectrum according to ASCE 7-16 calibrated with the UHS data for a 475-year return period.

With these parameters, the following seismic design category for the buildings can be defined.

Table 4-3. Seismic design category definition.

Parameter	Value
Importance factor, I_e	1.0
Risk Category	II
Site Class	D
Seismic Design Category	D

4.3.1 Equivalent lateral force method (ELFM)

An equivalent lateral force analysis was performed according to the ASCE 7-16 (ASCE, 2017) procedure. As a first step, the estimated fundamental period is calculated using Equation [4-1].

$$T_a = C_t h_n^x \qquad [4\text{-}1]$$

where C_t and x are coefficients that depend on the structural typology, for RC moment frames the values are 0.0466 and 0.9 respectively, and h_n corresponds to the height of the structure.

Then, the seismic coefficient, C_s, can be calculated by using general expression in Equation [4-2]:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \qquad [4\text{-}2]$$

This value of C_s is limited by Equations [4-3] and [4-4], that depend on the estimated period of the structure:

$$C_s = \frac{S_{D1}}{T_a\left(\frac{R}{I_e}\right)} \quad for \ \ T_a \leq T_L \tag{4-3}$$

$$C_s = \frac{S_{D1}T_L}{T_a^2\left(\frac{R}{I_e}\right)} \quad for \ \ T_a > T_L \tag{4-4}$$

where S_{DS} is the design spectral acceleration for short period, R is the response modification factor and I_e is the importance; for this case the values correspond to 1.63, 8 and 1 respectively. This seismic coefficient has a lower and upper bound as previously explained in Section 2.2.1.
Table 4-4 shows in a summarised way the results for the ELFM. Calculation details can be found in Appendix B.

Table 4-4. Summary of calculations for ELFM.

Building	T_a (s)	C_s	W [kN]	V_{ELFM} [kN]
3-FB	0.34	0.20	2572	523
6-FB	0.63	0.16	5154	827
9-FB	0.90	0.11	7717	868

4.3.2 Response spectrum method (RSM)

The final design of the buildings was performed using RSM, as it is believed that this is a more detailed method and can capture the effects of higher modes. However, the RSM is governed by a clause in the design that stipulates that the ELFM, described and conducted in the previous section, should be used to determine a minimum base shear for the design when the fundamental period of the structure, calculated by eigenvalue analysis, exceeds the limit C_uT_a. Therefore, a scaling factor should be used when RSM results in a design base shear lower than the ELFM, meaning that the design base shear should be at least that of the ELFM, stated as follows:

"*12.9.1.4.1 Scaling of Forces. Where the calculated fundamental period exceeds C_uT_a in a given direction, C_uT_a shall be used in lieu of T in that direction. Where the combined response for the modal base shear (V_t) is less than 100% of the calculated base shear (V) using the equivalent lateral force procedure, the forces shall be multiplied by V/V_t.*" (ASCE, 2017)

In this case, C_u refers to the coefficient for upper limit on the calculated period, which is variable depending on the value of S_{D1} and T_a is the previously estimated period of the structure. It is important to mention that for the scaling, the ELFM base shear V is obtained by performing an ELFM analysis with the period C_uT_a.

A modal analysis was performed for the three buildings. Figure 4-5 through Figure 4-7 show the mode shapes for the first three modes of the three buildings and, as expected, the first mode (i.e. fundamental mode of vibration) is translational with a high mass participation ratio as discussed afterwards.

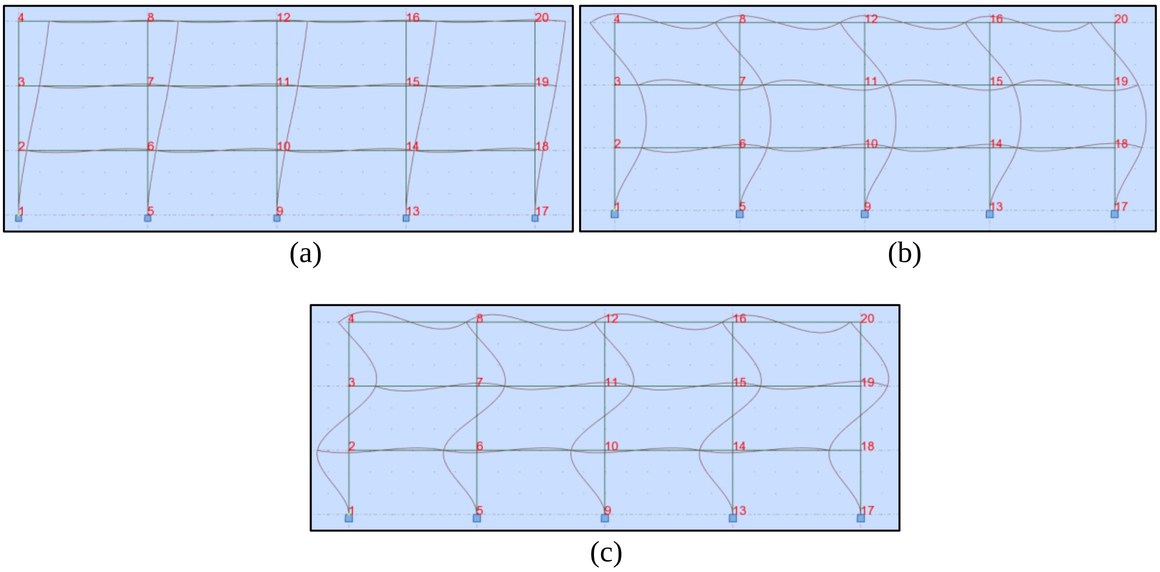


Figure 4-5. First three modes of vibration for 3 storey building, first mode (a), second mode (b) and third mode (c).

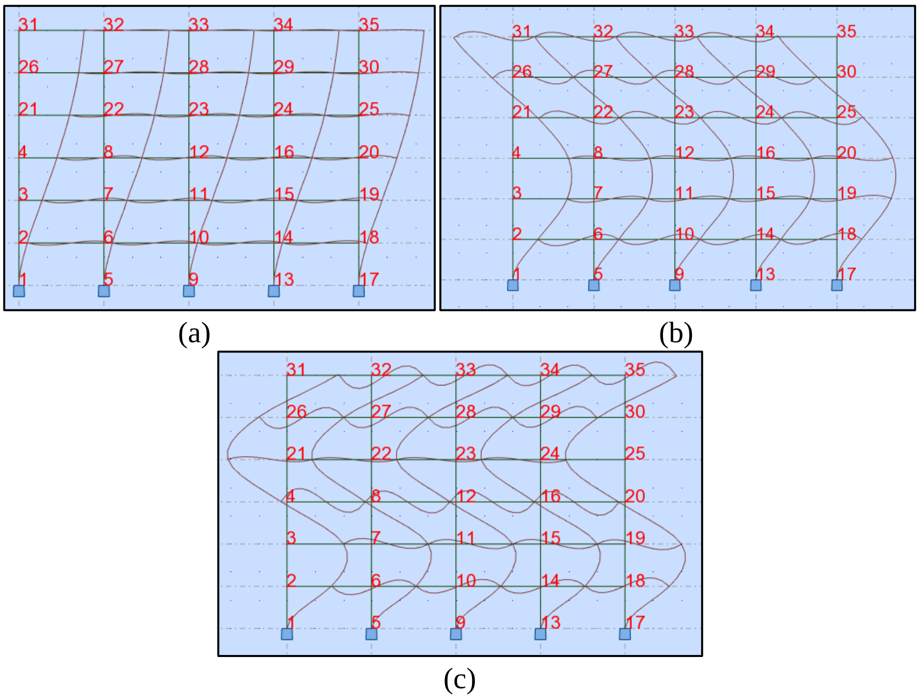
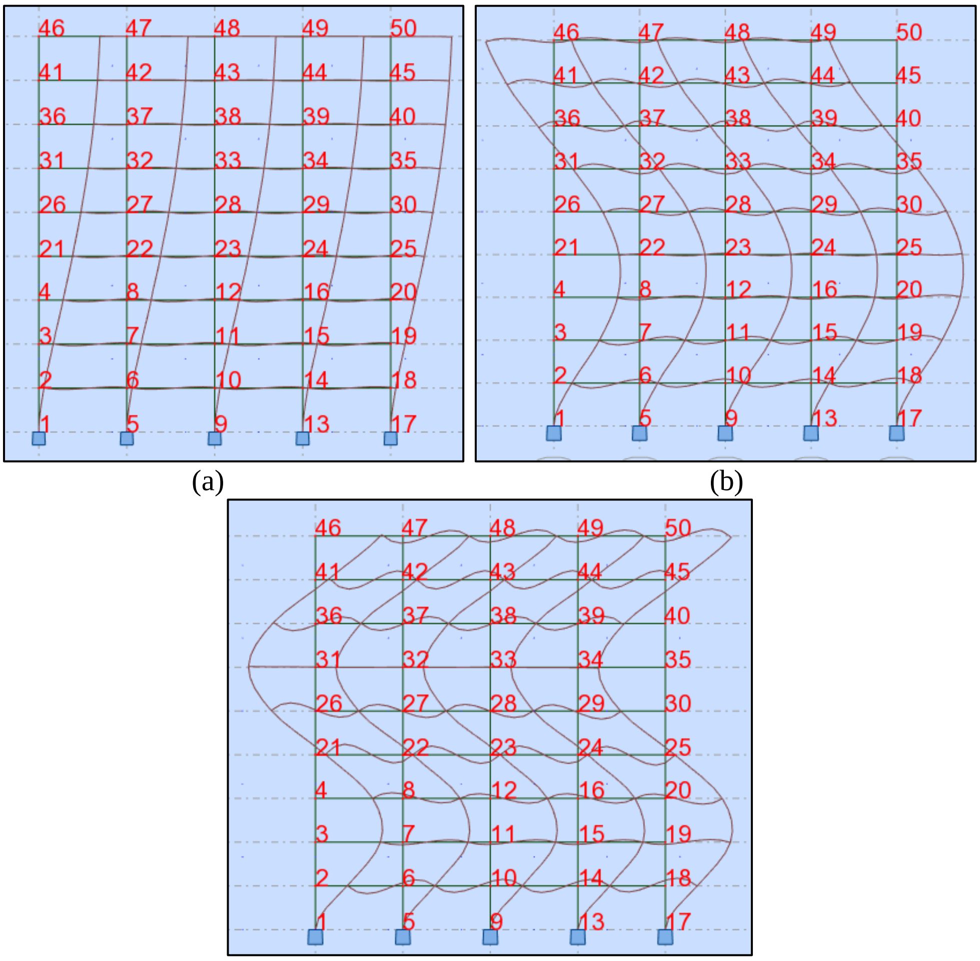


Figure 4-6. First three modes of vibration for 6 storey building, first mode (a), second mode (b) and third mode (c).



(a) (b) (c)

Figure 4-7. First three modes of vibration for 3 storey building, first mode (a), second mode (b) and third mode (c).

Table 4-5 through

Table 4-7 present the main results for the modal analysis. As it can be observed, the cumulative mass participation ratio is higher than 90% for mode 2, which is sufficient to comply with ASCE 7-16 (ASCE, 2017).

Table 4-5. Modal analysis results for 3 storey building.

Mode	Period [s]	M [%]	Cumulative mass [kg]
1	0.61	82.34	212646
2	0.17	95.87	247588
3	0.09	100.00	258254

Table 4-6. Modal analysis results for 6 storey building.

Mode	Period [s]	M [%]	Cumulative mass [kg]
1	1.25	80.46	419213
2	0.39	91.15	474910
3	0.20	95.72	498721
4	0.13	98.23	511798
5	0.09	99.56	518728
6	0.07	100.00	521021

Table 4-7. Modal analysis results for 9 storey building.

Mode	Period [s]	M [%]	Cumulative mass [kg]
1	1.92	80.15	628206
2	0.61	90.07	705958
3	0.34	94.10	737545
4	0.22	96.39	755494
5	0.16	97.88	767172
6	0.12	98.89	775088

As mentioned before, ASCE 7-16 (ASCE, 2017) requires a force scaling when the fundamental period calculated in the eigenvalue analysis exceeds $C_u T_a$ and if base shear of the modal analysis is less than the base shear calculated by the ELFM using the upper bound period $C_u T_a$. Table 4-8 shows a comparison between the fundamental mode calculated in the modal analysis and the upper bound period. As observed, in all the cases the upper bound period is less than the calculated with the modal analysis hence the base shear calculated for ELFM should use the upper bound period.

Table 4-8. Comparison between fundamental period of modal analysis and upper bound of ASCE 7-16.

Building	C_u	T_a (s)	$C_u T_a$ (s)	$T_{1,\,modal}$ (s)
3-FB	1.40	0.34	0.47	0.61
6-FB	1.40	0.63	0.88	1.25
9-FB	1.40	0.90	1.27	1.92

Table 4-9 shows the calculated base shear using ELFM (with the upper bound period) and the modal base shear V_{RSM}. The V_{RSM} was obtained by modal superposition using SRSS. The force scaling factor is then calculated as the ratio between both values. All forces are amplified in this way to obtain the design forces.

Table 4-9. Force scaling factor according to ASCE 7-16.

Building	V_{ELFM} [kN]	V_{RSM} [kN]	V_{FBD} [kN]	Force scaling factor
3-FB	523	268	523	1.95
6-FB	592	249	592	2.38
9-FB	617	217	617	2.85

4.3.3 Building Design

When using the FBD method, ASCE 7-16 establishes as a performance objective that the storey drift is maintained under a certain allowable drift that depends on the structure typology and the risk category, as shown in

Table 4-10. In this case, the structures fall under the definition of "All other structures" and risk category II as previously stated. Additionally, for RC moment frames that fall under the Seismic Design Category D, the value of allowable drift is reduced if the building is not regular and has little structural redundancy, which is not the case for this case study. Then, the maximum permitted drift is identified as Δ_a=0.020.

Table 4-10. Allowable storey drift according to ASCE 7-16. (ASCE, 2017)

Table 12.12-1 Allowable Story Drift, $\Delta_a^{a,b}$

Structure	Risk Category I or II	Risk Category III	Risk Category IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below level x.
[b] For seismic force-resisting systems solely comprising moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.

ASCE 7-16 establishes the following formula for the calculation of the inelastic deflection, δ_x, at any level, x:

$$\delta_x = \frac{C_d \delta_{xe}}{I_e}$$

Where C_d is the deflection amplification factor, δ_{xe} is the elastic deflection from the analysis and I_e is the importance factor. The deflection amplification factor is associated with the typology of the structure and for special RC moment resisting frames the value of $C_d = 5.5$.

The storey drift is computed as the largest difference of deflections of vertically aligned points at the top and the bottom of the storey along any of the edges of the structure. Figure 4-8 shows the design displacements and the corresponding drift profiles per storey for the three-storey building (3-FB). As it can be observed the largest drift is at the second floor and corresponds to a value of 1.10% which is lower than the allowable limit of 2%. In this case the structure could be optimised by reducing the cross section of columns and beams, but it is not possible, as these dimensions are the lower bound to resist gravitational loads, where the Combination 2 governs the design in the case of the beams. The columns cannot be reduced as the actual dimension (500 mm x 500 mm) is the minimum to provide enough development length and confinement for the reinforcement bars of the beam and also because the columns must comply with the capacity design that is explained in further detail in Section 4.6.3.

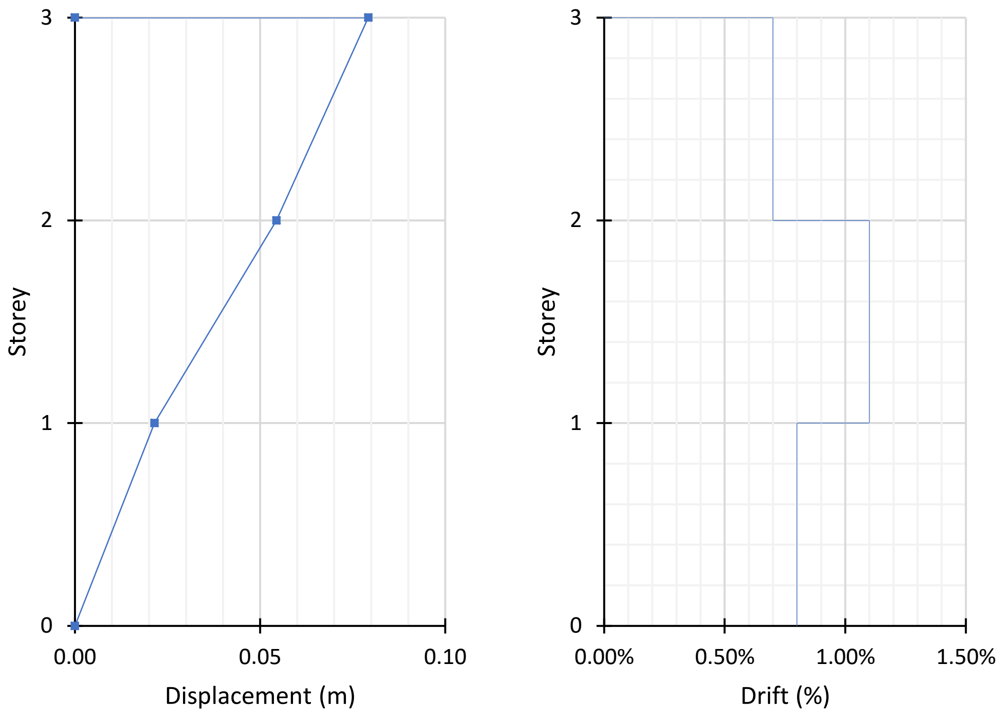


Figure 4-8. Displacement and drift profile for RSM, 3 storey building.

Figure 4-9 shows the shear force profile for the three-storey building. As it can be observed, the dashed line corresponds to the base shear calculated by the modal analysis and the continuous line corresponds to the scaled base shear using the factor required to maintain the minimum base shear given by the ELFM, as described in the previous section.

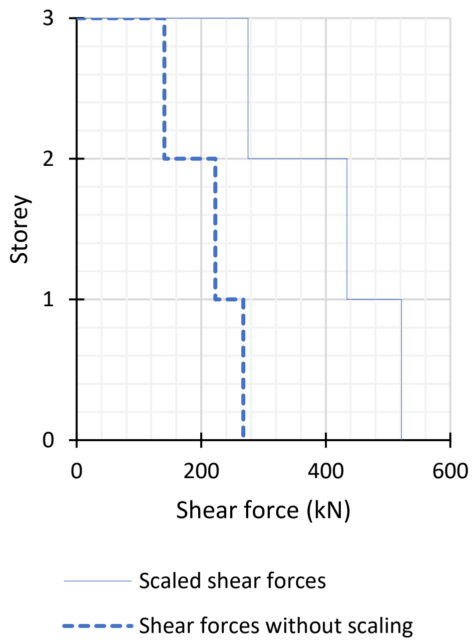


Figure 4-9. Shear profile for RSM, 3 storey building.

Figure 4-10 shows the displacement profile and the drift profile for the six-storey building (6-FB). In this case, the maximum drift is at level two with a value of 1.20%, which is closer to the allowable drift of 2%. Drift wise, the design is again not optimal, it would be possible to accommodate larger deformations, but the same issue, as discussed before, of minimum cross-sectional dimensions, would arise.

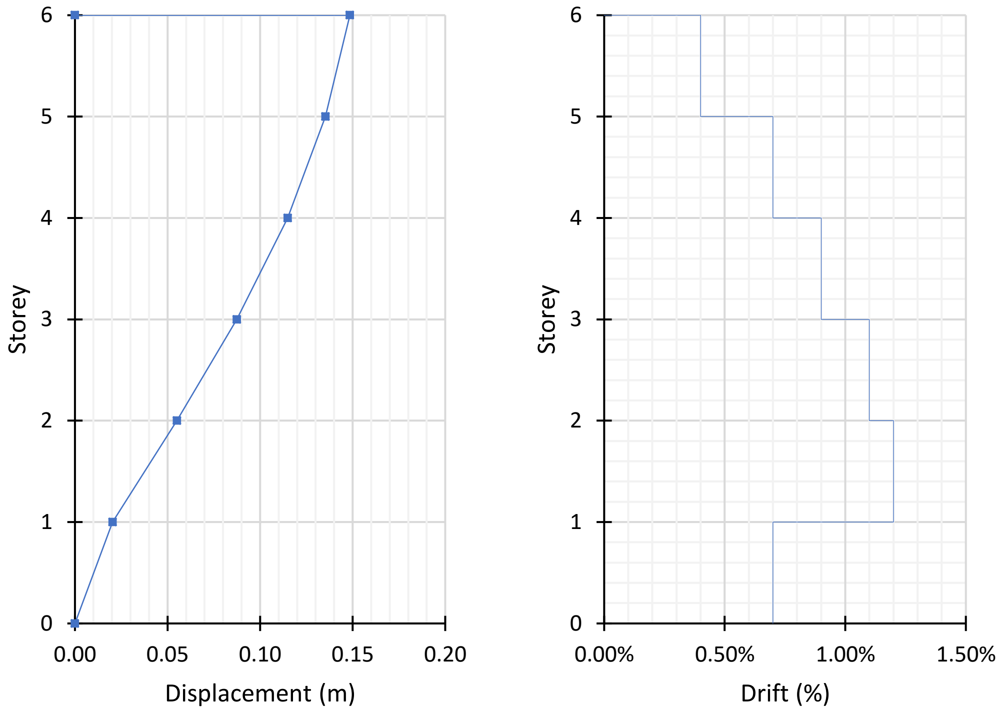


Figure 4-10. Displacement and drift profile for RSM, 6 storey building.

Figure 4-11 shows the shear profile for the six-storey building (6-FB) for the un-scaled and scaled case. It can be observed that the amplification produced by the scale factor is very considerable as the base shear more than doubles. ASCE 7-16 (ASCE, 2017), in its commentary, mentions that this scaling has to be done because studies have shown that considering a lower base shear can result in very flexible buildings that have a higher than expected risk of collapse. Consequently, incrementing the base shear results in a stiffer building with a lower lateral deformation.

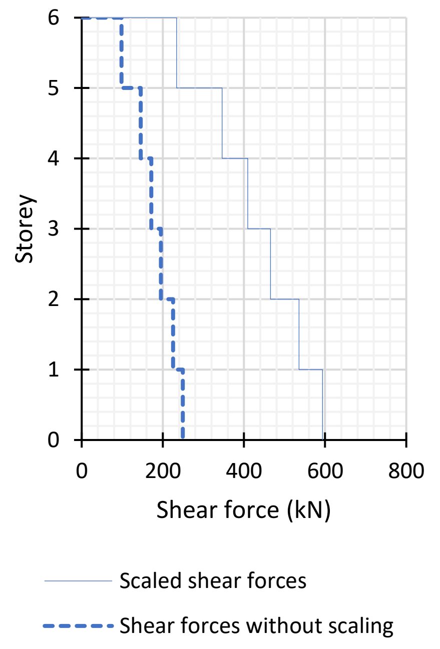


Figure 4-11. Shear profile for RSM, 6 storey building.

Figure 4-12 shows the displacement profile and drift profile for the nine-storey building (9-FB). It can be noticed that the maximum drift is approximately 1.00% in this case and it is lower than the allowable. The maximum drift is also slightly lower than the one for the six-storey, although the base shear shown in Figure 4-13 is slightly higher than the for the six-storey building. This can occur because the drift is higher in levels other than the critical one.

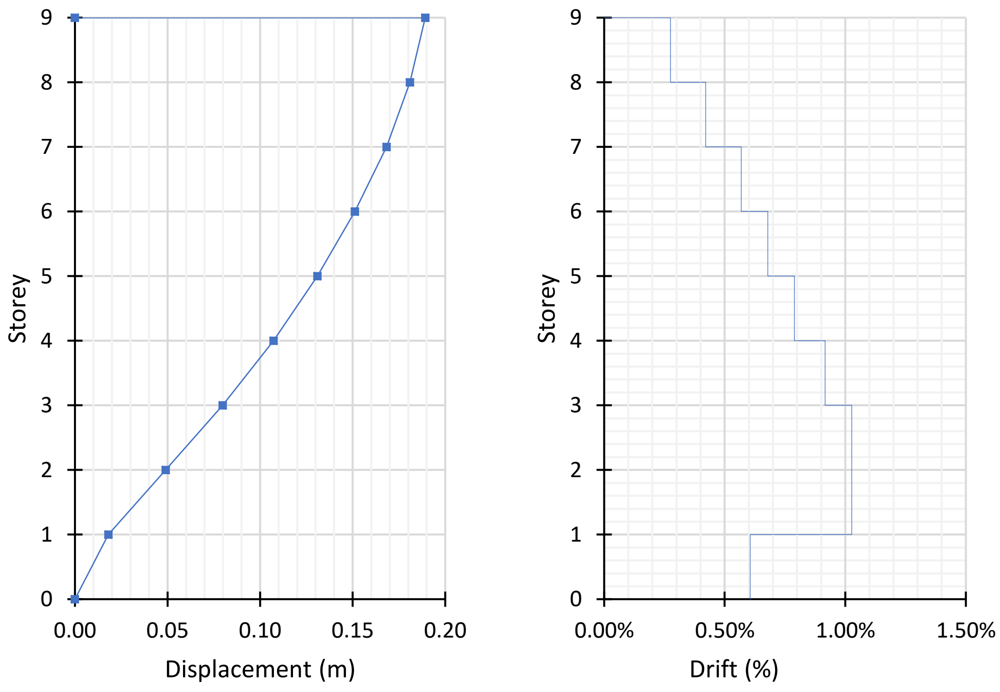


Figure 4-12. Displacement and drift profile for RSM, 9 storey building.

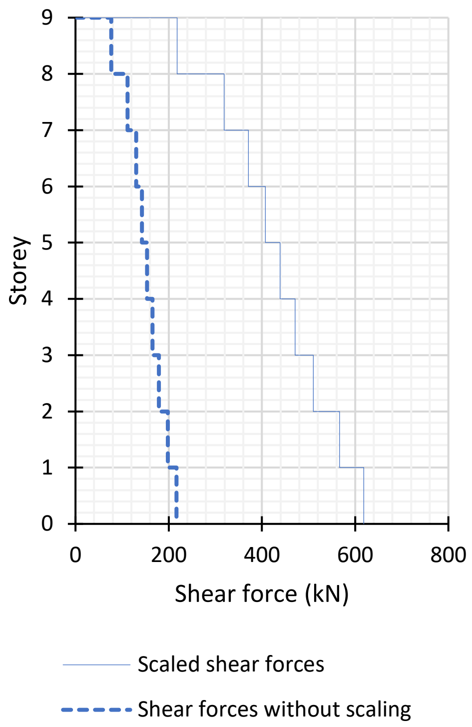


Figure 4-13. Shear profile for RSM, 9 storey building.

As observed in all the above cases, the base shear given by the RSM without scaling is considerably low when compared to the base shear with the scaling. These low values of base shear are due to two main factors. The first is the moment of inertia reduction factors given by ACI for the cracked concrete sections, which lowers the stiffness of the structure and as a result the period increases considerably when compared to the estimated periods obtained during the ELFM design. Secondly, all of the buildings are governed mostly by the first mode, which is the one with the longest period. As the majority of the shear demand comes from the first mode response, which has a long period, thus a low spectral acceleration, the total base shear is low.

Considering this displacements and shear force profiles it is possible at this point to perform the design for the individual members of the building. Specific element design is detailed in Section 4.6 of this document.

4.4 Displacement-based design

A displacement-based design was performed for the three buildings with different heights in order to compare the results with the FBD, following the DB12 model code (Sullivan et al., 2012).

4.4.1 Design parameters

As discussed in Section 2.2.2 of this document, the first step in DBD is to establish a performance objective. In order to match and being able to compare both methods (FBD and DBD), the same performance objective was selected. This performance objective is a maximum drift of 2.00% in any level. Then, an equivalent SDOF is obtained to target the performance objective. Table 4-11 shows the equivalent SDOF system parameters for each of the buildings, where Δ_d is the design displacement, m_e is the equivalent mass, H_e is the equivalent height, ξ_{eq} is the equivalent viscous damping, T_e is the equivalent period, K_e is the equivalent stiffness and V_b is the total base shear for design.

Table 4-11. Equivalent SDOF system parameters for DBD.

Parameter	Building		
	3-DB	6-DB	9-DB
Drift limit [%]	2%	2%	2%
Δ_d [m]	0.12	0.20	0.25
m_e [kg]	235377	478328	791156
H_e [m]	6.83	12.66	18.51
ξ_{eq} [%]	10.92%	9.63%	7.79%
T_e [s]	1.29	2.57	3.62
K_e [kN/m]	5584	2859	2383
V_b [kN]	676	600	656

4.4.2 Building design

The displacement profile and the targeted drift profile for the three-storey building (3-DB) can be observed in Figure 4-14. The expected maximum drift is at the first floor.

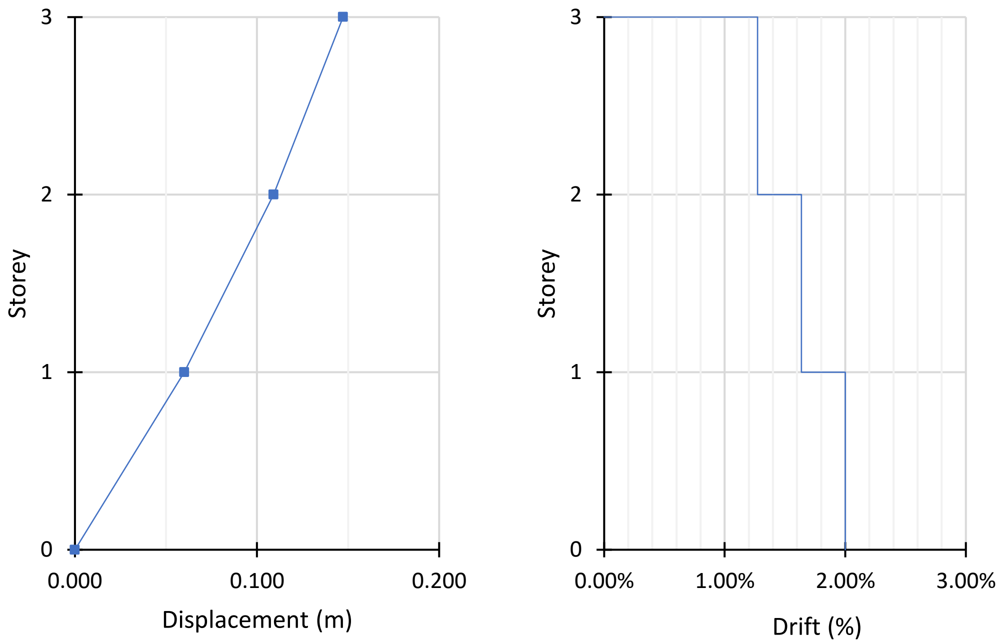


Figure 4-14. Displacement shape and drift for 3 storey building.

Once the base shear is obtained, it is possible to distribute it at each level as shown in Figure 4-15. It is important to notice that in the latter force distribution a 10% of the base shear is lumped at the last level, as recommended in the DBD12 model code.

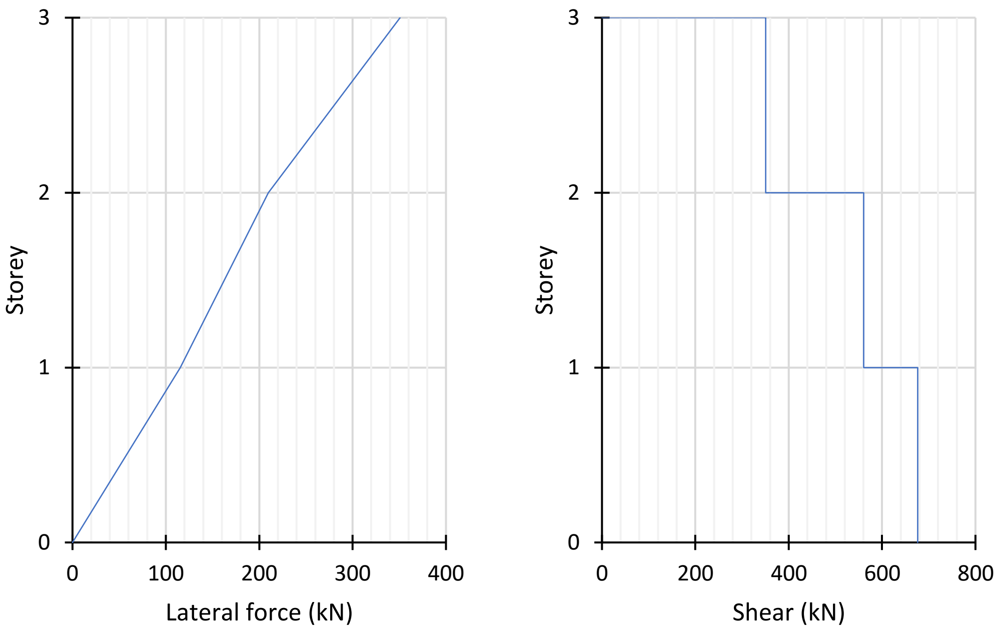


Figure 4-15. Lateral force and base shear for 3 storey building.

Figure 4-16 shows the displacement and targeted drift profile for the six-storey building (6-DB). Figure 4-17 shows the lateral force distribution (with the 10% base shear lumped at the 6th storey) and the shear force profile.

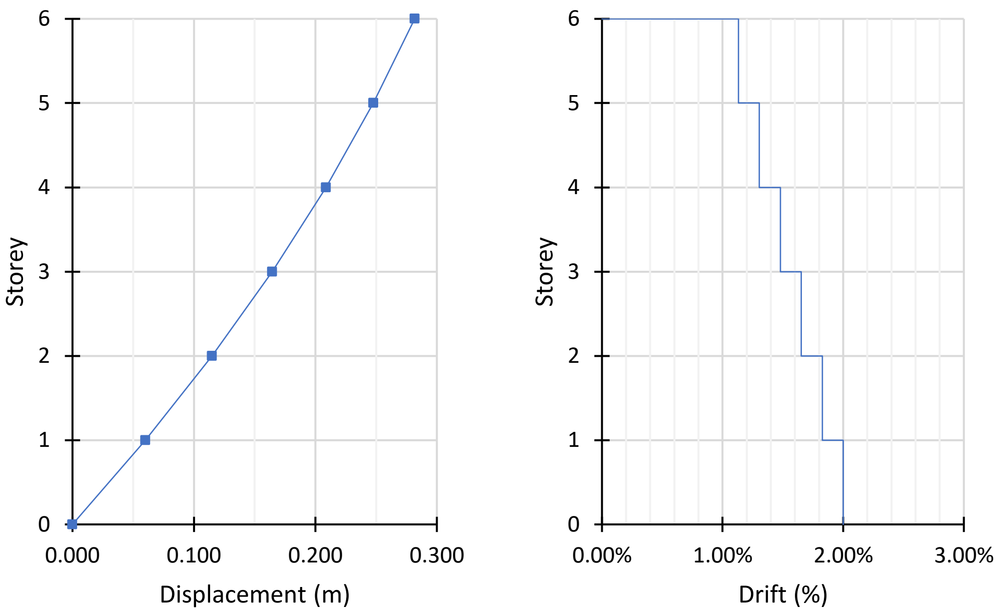


Figure 4-16. Displacement shape and drift for 6 storey building.

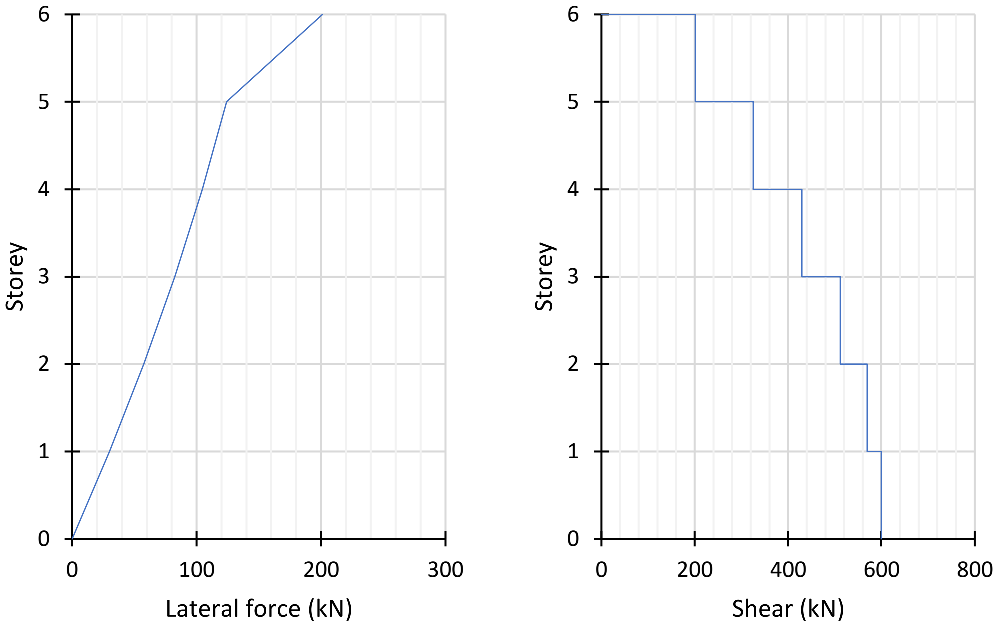


Figure 4-17. Lateral force and base shear for 6 storey building.

The displacement profile and the targeted drift profile for the nine-storey building (9-DB) are shown in Figure 4-18, while the lateral force and shear force profiles are shown in Figure 4-19.

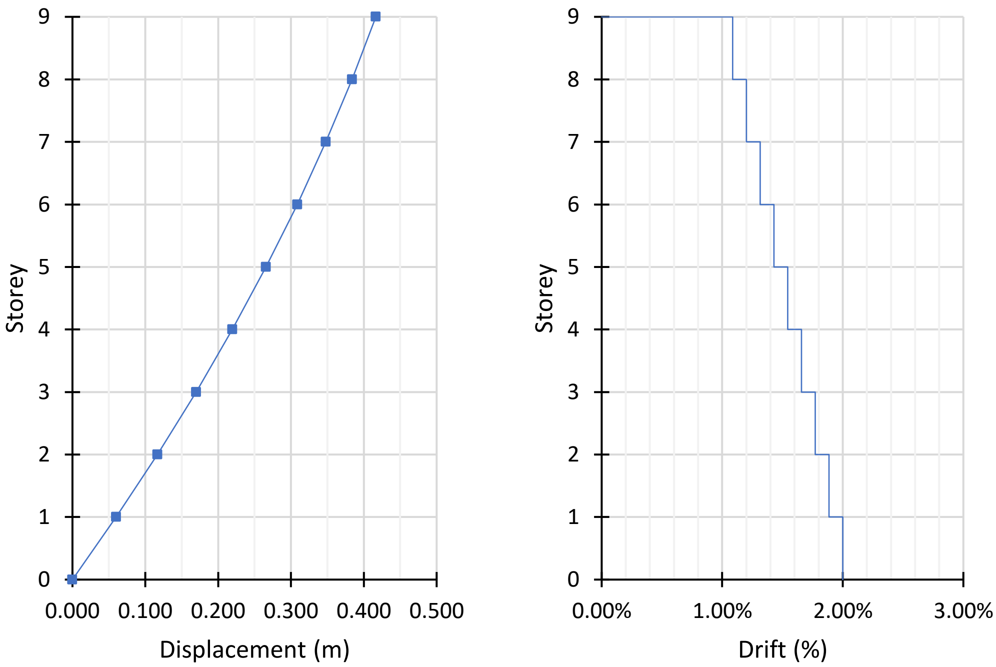


Figure 4-18. Displacement shape and drift for 9 storey building.

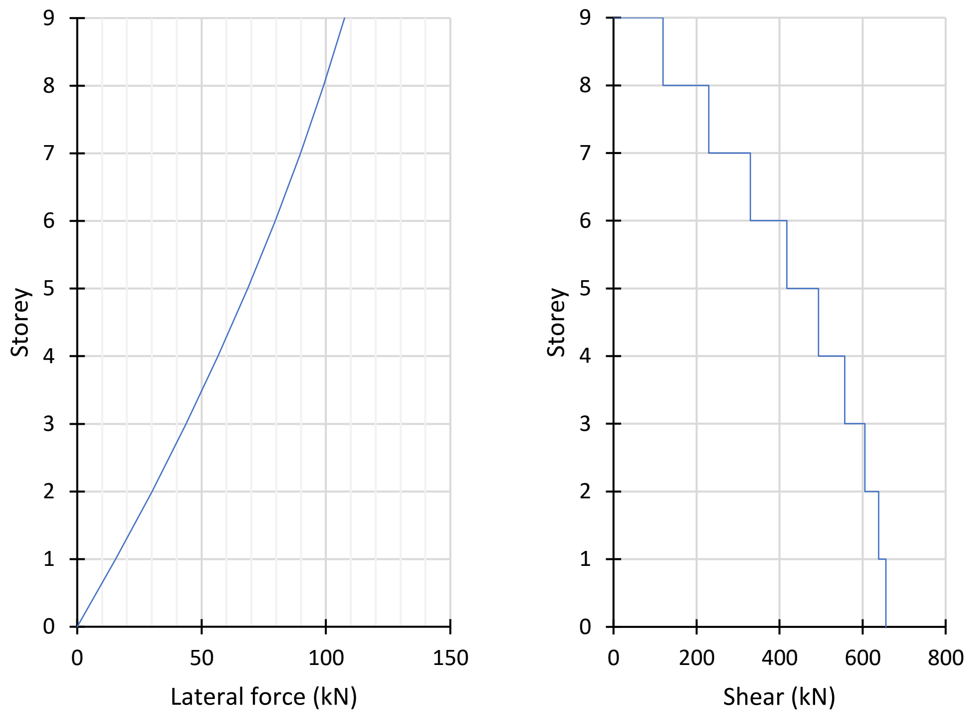


Figure 4-19. Lateral force and base shear for 9 storey building.

It is possible to observe from the design that the three-storey building is the one that has the maximum total base shear. This occurs because the six and nine-storey building have a larger equivalent period with a lower equivalent stiffness, which results in a lower total base shear.

4.5 Design comparison between methods

The following figures show a comparison between the designs for the three building heights for FBD and DBD. Additionally, the design for the un-scaled RSM case in FBD is presented. Figure 4-20 shows the design comparison for the three-storey building. It can be observed that the DBD method yields a higher base shear than the FBD method, and it is important to notice how the scaling done to the RSM has a considerable repercussion in the design base shear.

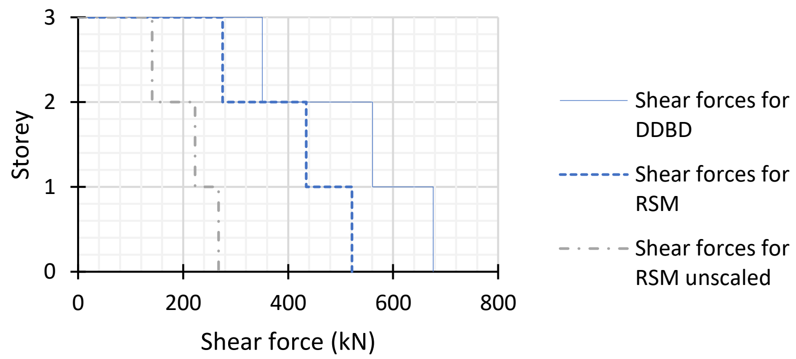


Figure 4-20. Shear profile comparison between DBD and RSM for 3 storey building.

For the case of the six-storey buildings, the base shear, and in general, the shear profile, is practically the same for both methods (FBD and DBD), as shown in Figure 4-21. If no scaling would be used, the difference between the two methods would be considerable.

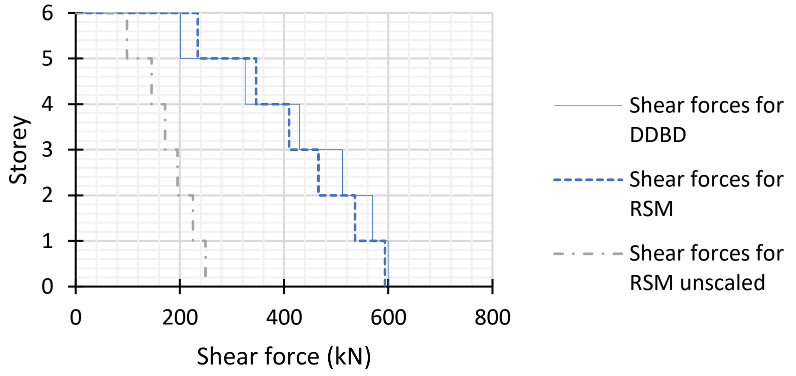


Figure 4-21. Shear profile comparison between DBD and RSM for 6 storey building.

In the nine-storey buildings, the base shear is slightly higher for the DBD method as shown in Figure 4-22. Also, it can be observed that DBD gives higher shear forces until approximately the middle of the height (storey 5), where a shift occurs and FDB gives the higher shears for the last floors.

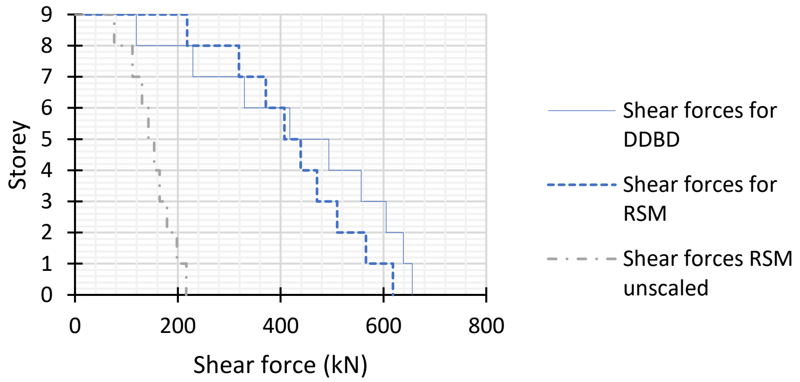


Figure 4-22. Shear profile comparison between DBD and RSM for 9 storey building.

Table 4-12 shows the ratio between the design base shear, V, and the total building weight, W, for each of the design cases. It is possible to observe that for all the cases (FBD and DBD), as the height increases, this ratio gets considerably lower. This is due to the fact that, as the height increases, the period increases and then the spectral acceleration is lower, finally leading to a lower base shear demand.

Table 4-12. Base shear ratio for the different design methods and heights.

Design case	Design base shear V (kN)	Total weight W (kN)	V/W ratio
3-DB / 3-DB-NG	676	2532	0.27
3-FB / 3-DB-NG	523	2532	0.21
6-DB	600	5108	0.12
6-FB	592	5108	0.12
9-DB	656	7685	0.09
9-FB	617	7685	0.08

4.6 Element design and detailing

Concrete element design and detailing is performed for all design cases following the ACI Standard "Building Code Requirements for Structural Concrete, ACI 318M-14" (American Concrete Institute, 2015). Concrete design properties (ACI 318M-14 19.2) and reinforcement steel properties (ACI 318M-14 20.2) are code compliant. The strength reduction factors, ϕ, used for the design are as shown in Table 4-13.

Table 4-13. Strength reduction factors for RC concrete members. (ACI, 2015)

Action or structural element		ϕ	Exceptions
(a)	Moment, axial force, or combined moment and axial force	0.65 to 0.90 in accordance with 21.2.2	Near ends of pretensioned members where strands are not fully developed, ϕ shall be in accordance with 21.2.3.
(b)	Shear	0.75	Additional requirements are given in 21.2.4 for structures designed to resist earthquake effects.
(c)	Torsion	0.75	—
(d)	Bearing	0.65	—
(e)	Post-tensioned anchorage zones	0.85	—
(f)	Brackets and corbels	0.75	—
(g)	Struts, ties, nodal zones, and bearing areas designed in accordance with strut-and-tie method in Chapter 23	0.75	—
(h)	Components of connections of precast members controlled by yielding of steel elements in tension	0.90	—
(i)	Plain concrete elements	0.60	—
(j)	Anchors in concrete elements	0.45 to 0.75 in accordance with Chapter 17	—

4.6.1 Beam design

RC beams are designed according to ACI 318-14 (ACI, 2015) following the provisions for beams of special moment frames. All the dimensional limits for the concrete section and reinforcement steel ratios comply with the requirements stated in the code. In all the cases, torsion and axial loads can be ignored in the design as their values are lower than the following limits:

$$P_u \leq 0.1\, f_c' A_g \qquad [4\text{-}5]$$

$$T_u \leq 0.083\lambda\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right) \qquad [4\text{-}6]$$

where:

P_u : is the factored axial force from the analysis.

T_u : is the factored torsion force from the analysis.

f_c': is the concrete compressive strength at 28 days.

A_g: is the cross-sectional gross area.

λ: is the reduction factor for lightweight concrete.

A_{cp}: area enclosed by the outside perimeter of the concrete cross section.

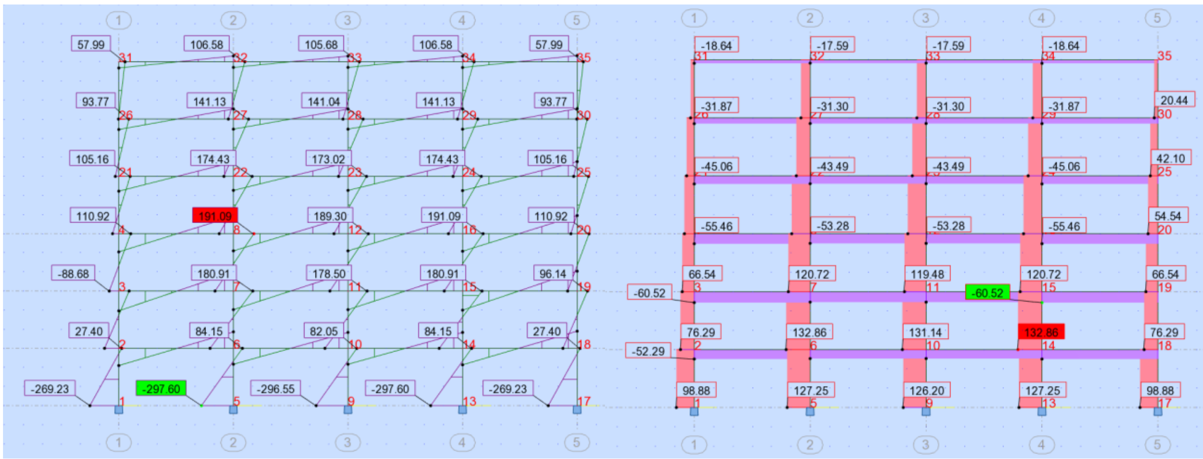
p_{cp}: outside perimeter of the concrete cross section.

Flexural and shear strength of the concrete sections are calculated according to the provisions of ACI 318-14, and have the sufficient strength to comply with:

$$M_u \leq \phi M_n \qquad [4\text{-}7]$$

$$V_u \leq \phi V_n \qquad [4\text{-}8]$$

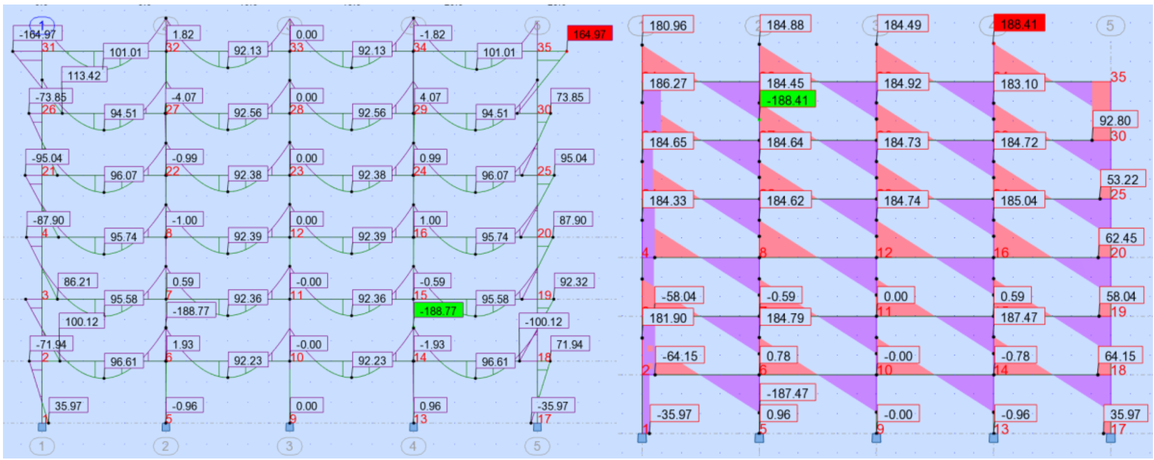
where M_u and M_n are the factored moment force and the nominal moment strength respectively. V_u is the factored shear force and V_n is the nominal shear strength of the section. These forces are obtained from an elastic analysis on a two-dimensional model described in section 4.2. The seismic force is distributed along the height of the building as stated in the previous section, and then finding the corresponding internal forces for each element. Figure 4-23 shows an example of the internal forces (moment and shear) produced due to lateral seismic loads in the six-storey DBD building.



(a) (b)

Figure 4-23. (a)Moment diagram and (b)shear diagram for lateral seismic loads in the 6-DB building.

Additionally, the gravitational loads (live and dead) are added in each floor as a distributed load, then the internal forces for each element are computed. Figure 4-24 shows an example of the internal forces (moment and shear) that are produced due to the live load in the six-storey DBD building.



(a) (b)

Figure 4-24. (a)Moment diagram and (b)shear diagram for live load in the 6-DB building.

After the internal forces are computed for all the cases (seismic and gravitational), they are combined according to the approach described in section 4.2.1. Each combination represents the ultimate forces M_u and V_u that need to be verified according to Equations [4-7] and [4-8].

4.6.2 Column design

RC columns are designed according to ACI 318-14 (ACI, 2015) following the provisions for columns of special moment frames. Torsion forces are ignored since they are lower than the threshold presented before. Axial and flexural strength is analysed as a combined action through an interaction diagram as shown in Figure 4-25.

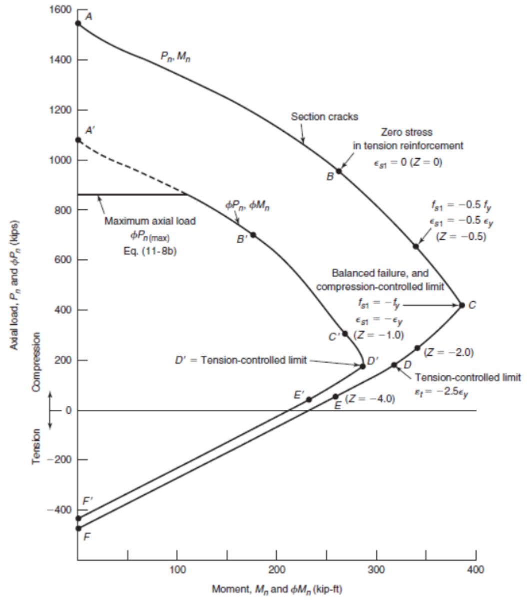


Figure 4-25. Interaction diagram for RC column design. (Wight & MacGregor, 2012)

Shear strength is calculated so that Equation [4-9] complies:

$$V_u \leq \phi V_n \qquad [4\text{-}9]$$

4.6.3 Capacity design

Capacity design is performed to prevent brittle shear failures in the RC frame elements. The design shear force is calculated as the maximum shear force that can be generated at the extreme of the elements due to the probable flexural strength, M_{pr}. ACI 318-14 establishes that it shall be assumed that moments of opposite sign corresponding to M_{pr} act at the joint faces and that the beam is loaded with the factored tributary gravity load along its span, as shown in Figure 4-26.

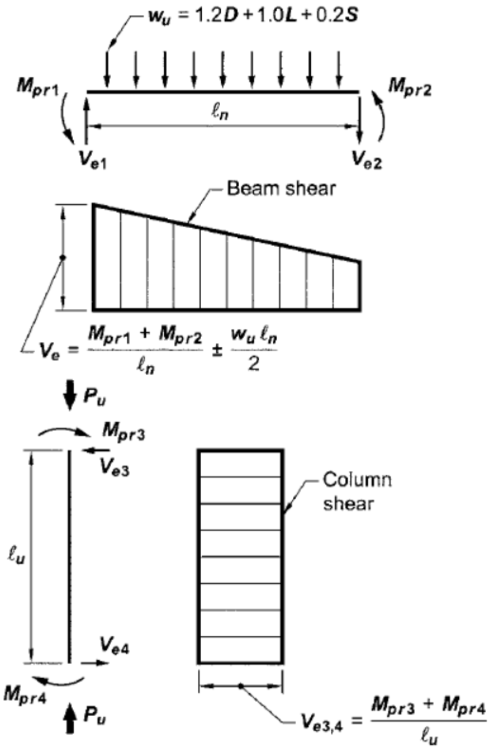


Figure 4-26. Required shear strength due to capacity design for columns and beams. (ACI, 2015)

Additionally, it is required that the columns be stronger than the beams to reduce the likelihood of column yielding during the inelastic response of the structural system. ACI 318-14 requires satisfying the following:

$$\sum M_{nc} \geq (6/5) \sum M_{nb} \qquad [4\text{-}10]$$

where $\sum M_{nc}$ is the sum of nominal flexural strengths of the columns framing into the joint and $\sum M_{nb}$ is the sum of nominal flexural strengths of the beams framing into the joint.

4.6.4 Element design for force-based method

All elements of the frame are designed as explained in the previous section and the detailed calculations are shown in Appendix C. Table 4-14 and Table 4-15 present the member design characteristics and reinforcement ratios for each of the buildings for FBD, where:

H: section height.

B: section width.

s: shear reinforcement spacing in the hinge region.

d: distance from the extreme compression fibre to the centroid of the longitudinal tension reinforcement.

$d_{b,longitudinal}$: longitudinal reinforcement bar diameter.

$d_{b,shear\ reinforcement}$: shear reinforcement bar diameter.

A_{sh}: area of shear reinforcement within the spacing s.

$A_{s,top,end\ 1}$: area of the longitudinal reinforcement at the top of the section.

$A_{s,middle,end\ 1}$: area of the longitudinal reinforcement at the middle of the section.

$A_{s,bot,end\ 1}$: area of the longitudinal reinforcement at the bottom of the section.

ρ_s: ratio of A_s to the gross section area.

Figure 4-27 through Figure 4-29 show the member labelling system for the force-based designs. As it can be observed, for the three-storey building, the same sections for columns and beams were used along the height of the building, while for the six and nine-storey buildings the sections were varied along the height because the demands were lower in the upper floors.

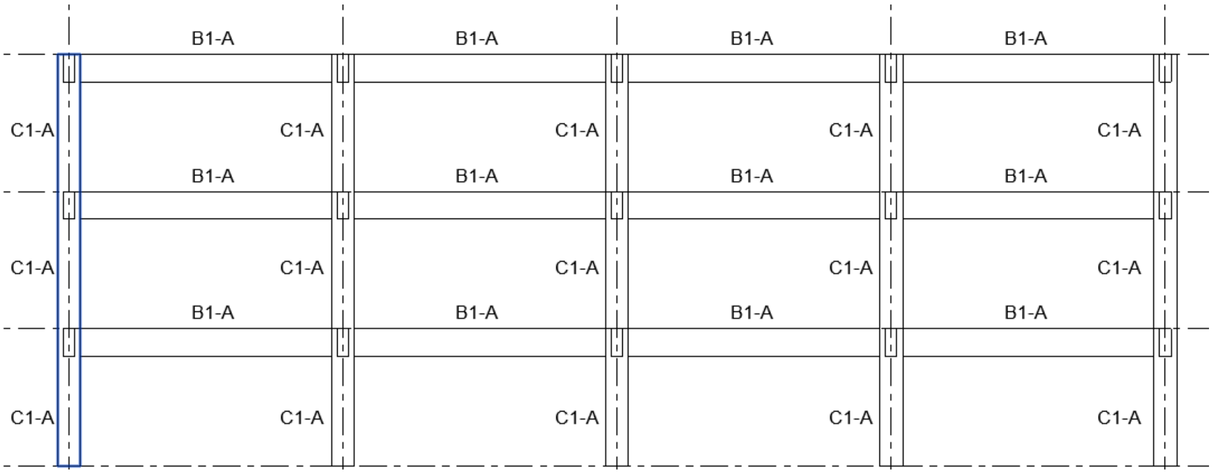


Figure 4-27. Member labels for 3-FB and 3-FB-NG buildings.

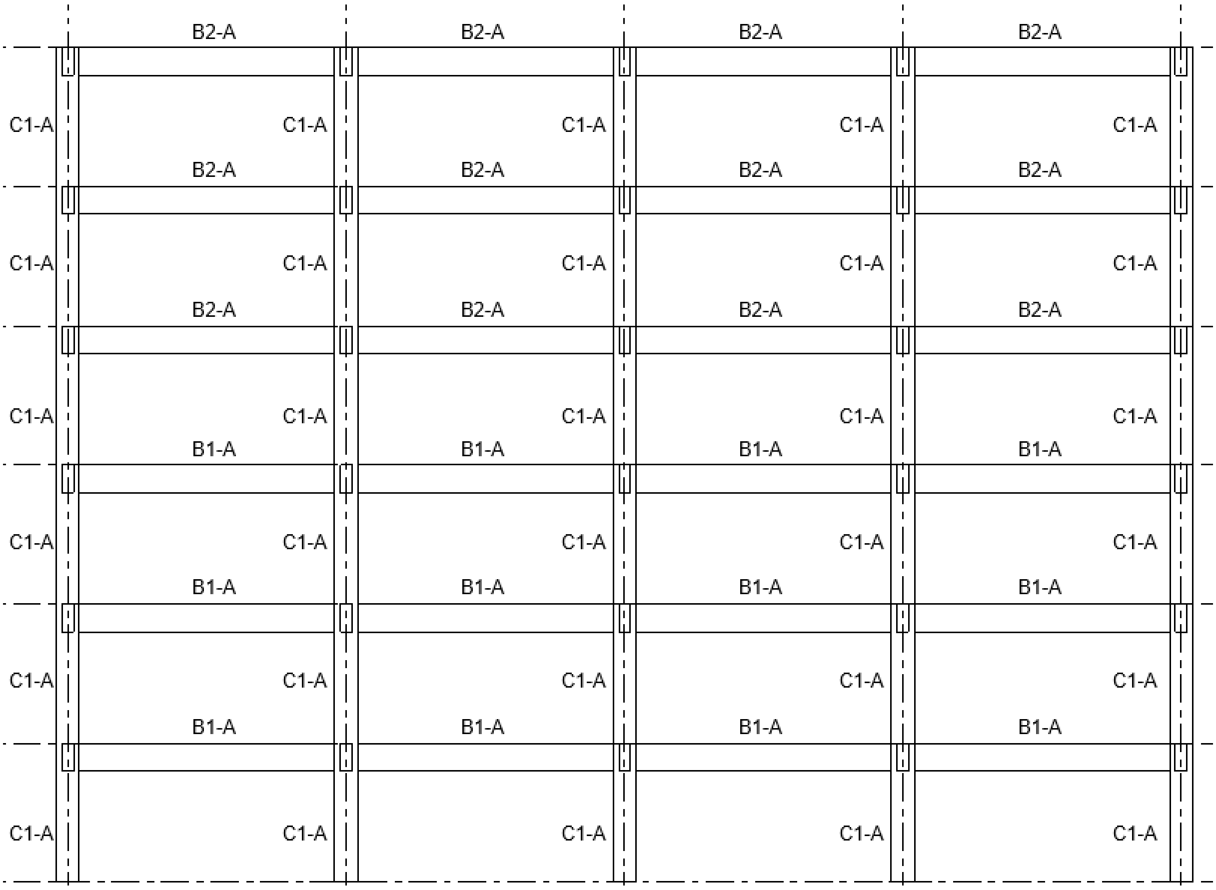


Figure 4-28. Member labels for 6-FB building.

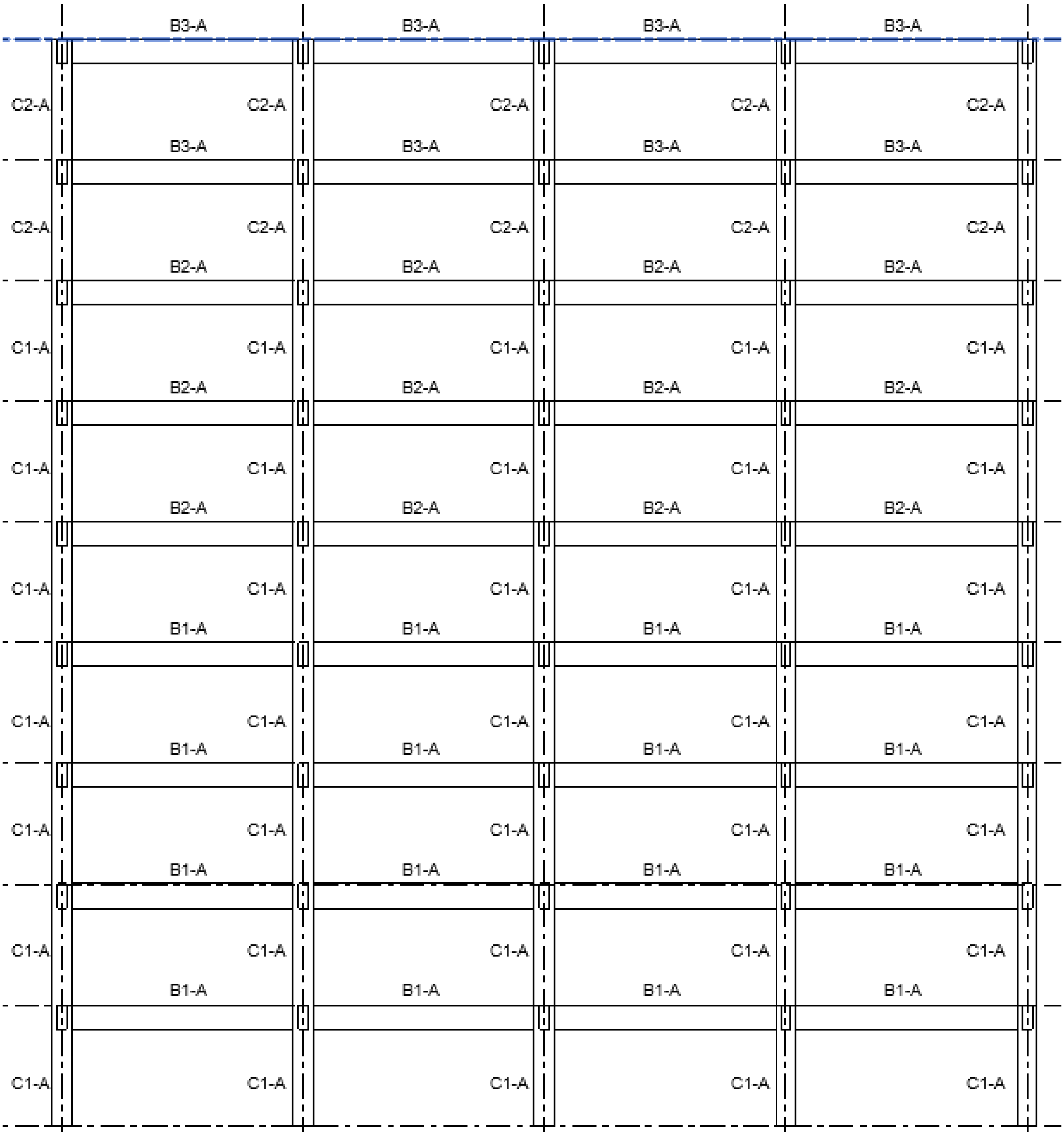


Figure 4-29. Member labels for 9-FB building.

Table 4-14. Main design characteristics for force-based design.

Parameter	3-FB / 3-FB-NG		6-FB			9-FB				
	C1-A	B1-A	C1-A	B1-A	B2-A	C1-A	C2-A	B1-A	B2-A	B3-A
H (m) =	0.5	0.6	0.5	0.6	0.6	0.5	0.5	0.6	0.6	0.6
B (m) =	0.5	0.25	0.5	0.25	0.25	0.5	0.5	0.25	0.25	0.25
s (mm) =	100	75	50	75	75	65	50	75	75	75
d (mm) =	450	550	450	550	550	450	450	550	550	550
$d_{b,\ longitudinal}$ (mm) =	19.1	19.1	22.2	22.2	22.2	22.2	22.2	19.1	22.2	22.2
$d_{b,\ shear\ reinforcement}$ (mm) =	12.7	9.5	9.5	9.5	9.5	12.7	9.5	9.5	9.5	9.5
A_{sh} (mm^2) =	507.00	142.00	213.00	142.00	142.00	380.00	213.00	142.00	142.00	142.00
$A_{s,top,end\ 1}$ (mm^2)=	1595.40	1374/462	1161.20	1478.80	1291.60	1161.20	1161.20	1562.20	1476.80	1291.60
$A_{s,mid,end\ 1}$ (mm^2)=	0.00	0/253	774.10	0.00	0.00	774.10	774.10	0.00	0.00	0.00
$A_{s,bot,end\ 1}$ (mm^2)=	1595.40	859/462	1161.20	774.15	774.15	1161.20	1161.20	774.15	774.15	774.15

Table 4-15. Reinforcement ratios for elements designed using forced-based design.

Parameter	3-FB / 3-FB-NG		6-FB			9-FB				
	C1-A	B1-A	C1-A	B1-A	B2-A	C1-A	C2-A	B1-A	B2-A	B3-A
$\rho_{s,top}$ (%)=	0.71%	1.00% / 0.34%	0.52%	1.08%	0.94%	0.52%	0.52%	1.14%	1.07%	0.94%
$\rho_{s,middle}$ (%)=	0.00%	0.00% / 0.18%	0.34%	0.00%	0.00%	0.34%	0.34%	0.00%	0.00%	0.00%
$\rho_{s,bottom}$ (%)=	0.71%	0.63% / 0.34%	0.52%	0.56%	0.56%	0.52%	0.52%	0.56%	0.56%	0.56%
$\rho_{s,total}$ (%)=	1.42%	1.62% / 0.86%	1.38%	1.64%	1.50%	1.38%	1.38%	1.70%	1.64%	1.50%

4.6.5 Element design for displacement-based method

Figure 4-30 through Figure 4-32 show the member labelling system for the displacement-based designs. As in the case of FBD, the three-storey building has the same sections for columns and beams along the height of the building, while for the six and nine-storey buildings the sections were varied along the height because the demands were lower in the upper floors.

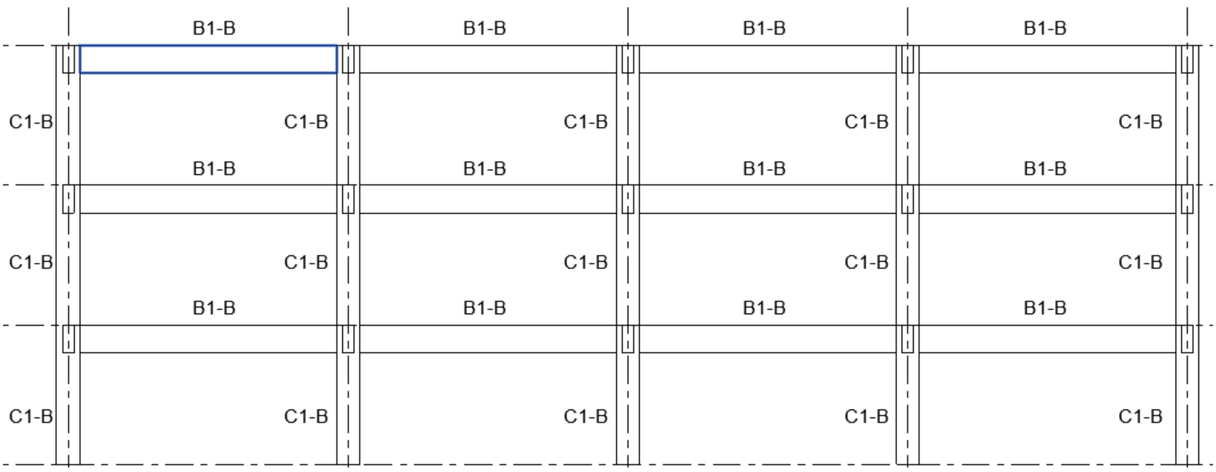


Figure 4-30. Member labels for 3-DB and 3-DB-NG buildings.

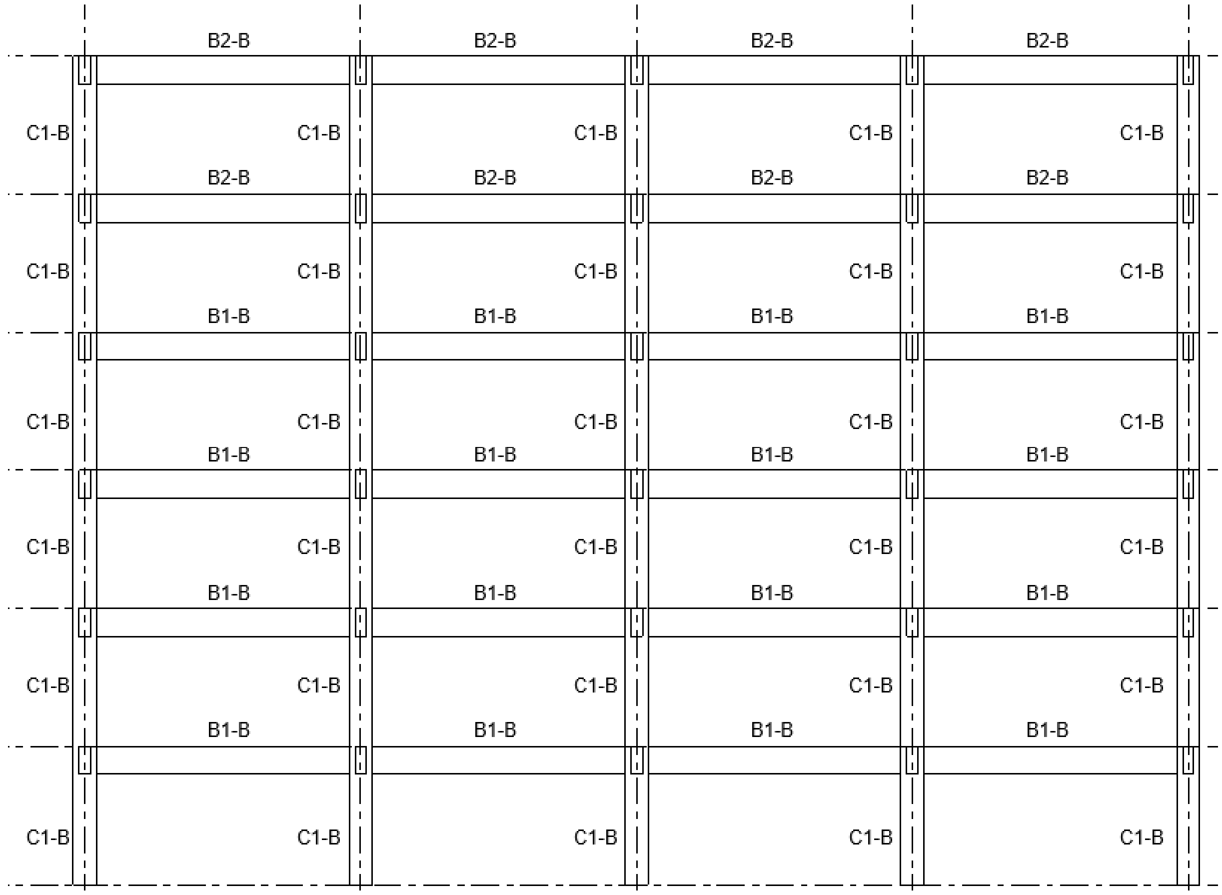


Figure 4-31. Member labels for 6-DB building.

.

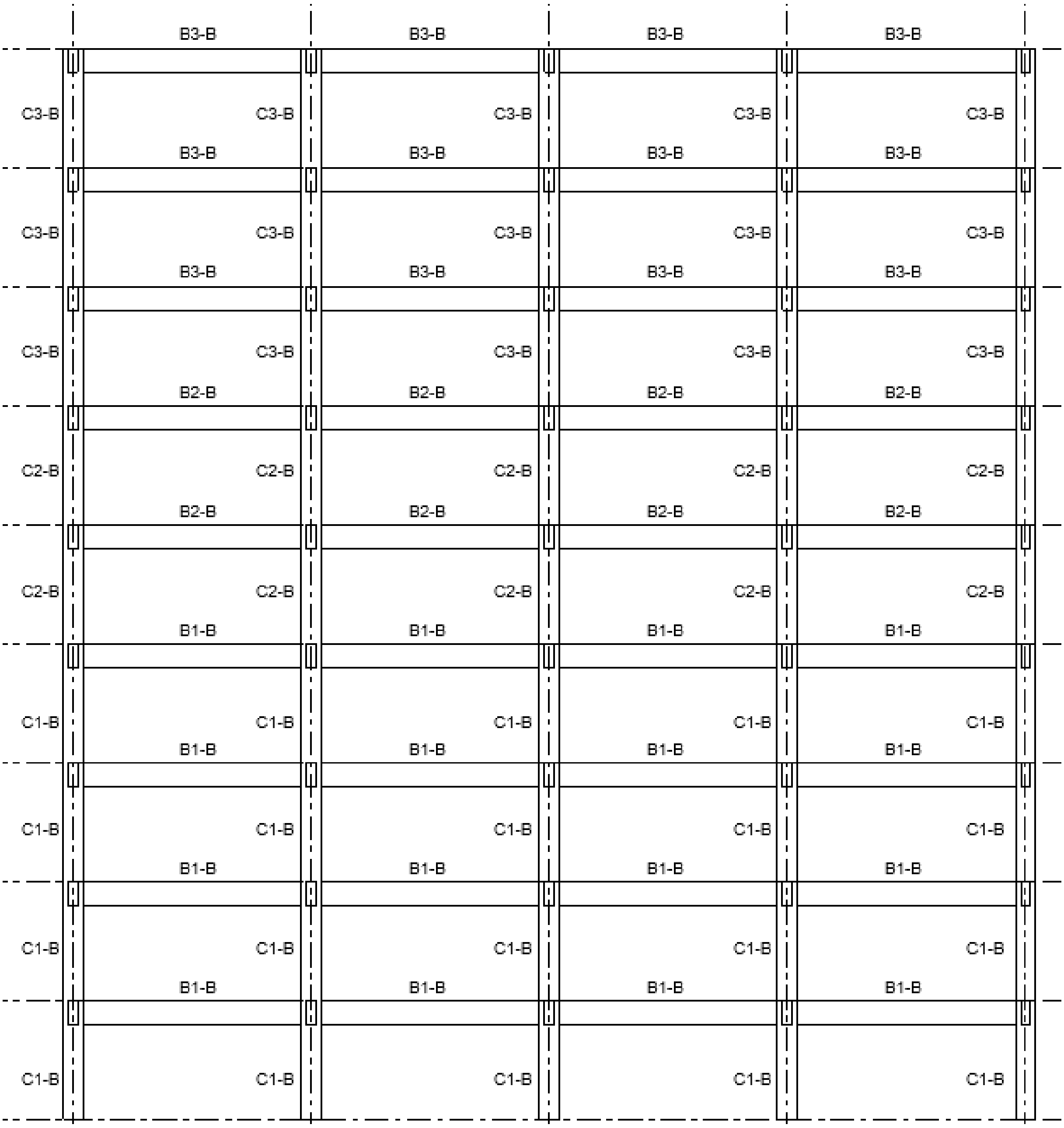


Figure 4-32. Member labels for 9-DB building.

Table 4-16 and Table 4-17 present the member design characteristics and reinforcement ratios for each of the buildings for DBD. This detailing has been carried out in the same manner as for the FBD cases, and the detailed calculations are listed in Appendix C.

Table 4-16. Main design characteristics for displacement-based design.

Parameter	3-DB / 3-DB-NG		6-DB			9-DB					
	C1-B	B1-B	C1-B	B1-B	B2-B	C1-B	C2-B	C3-B	B1-A	B2-A	B3-A
H (m) =	0.5	0.6	0.5	0.6	0.6	0.5	0.5	0.5	0.6	0.6	0.6
B (m) =	0.5	0.25	0.5	0.25	0.25	0.5	0.5	0.5	0.25	0.25	0.25
s (mm) =	100	75	50	75	75	65	50	50	75	75	75
d (mm) =	450	550	450	550	550	450	450	450	550	550	550
$d_{b,\ longitudinal}$ (mm) =	22.2	22.2	22.2	22.2	19.1	22.2	22.2	22.2	22.2	22.2	22.2
$d_{b,\ shear\ reinforcement}$ (mm) =	12.7	9.5	9.5	9.5	9.5	12.7	9.5	9.5	9.5	9.5	9.5
A_{sh} (mm^2) =	507.00	142.00	213.00	142.00	142.00	380.00	213.00	213.00	142.00	142.00	142.00
$A_{s,top,end\ 1}$ (mm^2)=	1161.00	1161/603	1161.20	1548.00	1146.00	1161.20	1161.20	1060.60	1548.00	1445.20	1161.00
$A_{s,mid,end\ 1}$ (mm^2)=	774.00	567/253	774.10	142.00	142.00	774.10	774.10	573.00	142.00	0.00	142.00
$A_{s,bot,end\ 1}$ (mm^2)=	1161.00	860/603	1161.20	550.00	573.00	1161.20	1161.20	1060.60	774.00	872.15	573.00

Table 4-17. Reinforcement ratios for elements designed using displacement-based design.

Parameter	3-DB / 3-DB-NG		6-DB			9-DB					
	C1-B	B1-B	C1-B	B1-B	B2-B	C1-B	C2-B	C3-B	B1-A	B2-A	B3-A
$\rho_{s,top}$ (%)=	0.52%	0.84% / 0.44%	0.52%	1.13%	0.83%	0.52%	0.52%	0.47%	1.13%	1.05%	0.84%
$\rho_{s,middle}$ (%)=	0.34%	0.41% / 0.18%	0.34%	0.10%	0.10%	0.34%	0.34%	0.25%	0.10%	0.00%	0.10%
$\rho_{s,bottom}$ (%)=	0.52%	0.63% / 0.44%	0.52%	0.40%	0.42%	0.52%	0.52%	0.47%	0.56%	0.63%	0.42%
$\rho_{s,total}$ (%)=	1.38%	1.88% / 1.06%	1.38%	1.63%	1.35%	1.38%	1.38%	1.20%	1.79%	1.69%	1.36%

4.7 Design assessment

4.7.1 Non-linear model

A non-linear structural model was created using the structural analysis program OpenSees (The Regents of the University of California, 2006). The model is a two-dimensional model with nonlinearity modelled through lumped plasticity elements for the RC columns and beams. Ibarra & Krawinkler (2005) developed a deteriorating hysteretic model for global collapse assessment of structures with a backbone curve and cyclic behaviour as shown in Figure 4-33, where K_e is the initial stiffness, K_s is the hardening stiffness, K_c is the post-capping stiffness, M_y is the yield moment, M_c is the moment at capping, θ_y is the yield rotation and θ_{cap} is the rotation at capping. As observed in Figure 4-33, one of the important characteristics of this model is the negative stiffness introduced for the post-capping residual strength and the cyclic deterioration that is based on hysteretic energy-dissipation due to cyclic loading.

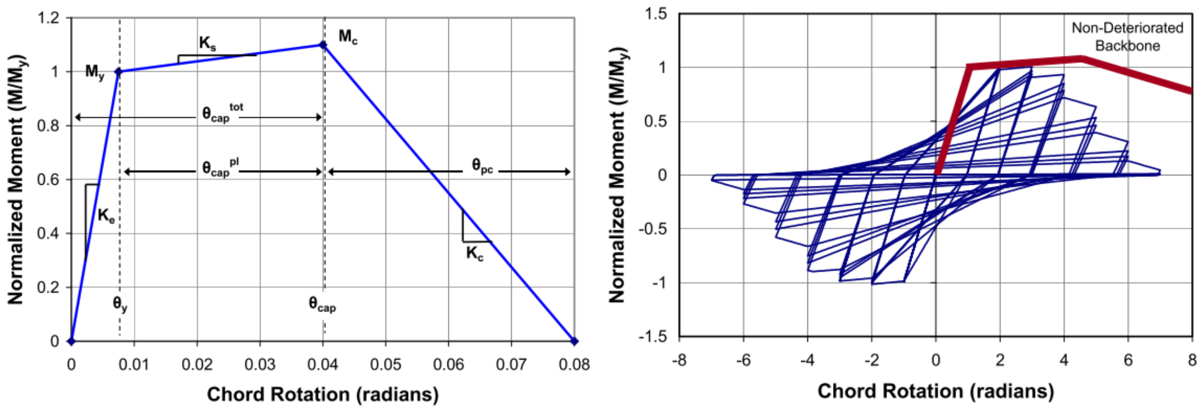


Figure 4-33. Behaviour of nonlinear hinge element. (Ibarra & Krawinkler, 2005)

For this study, the calibration of the lumped plasticity model was performed following the work of Haselton et al. (2008), where the model was calibrated to account for deterioration that precipitates the side-sway collapse as shown in Figure 4-34.

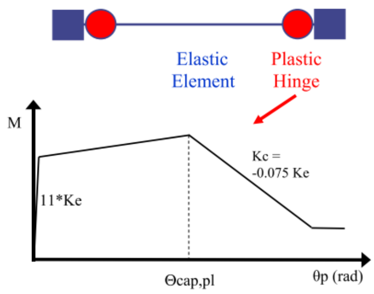


Figure 4-34. Calibration of hysteretic model.(Haselton et al., 2008)

The assembled structural model is fixed in the base of the columns and joints are modelled as infinitely rigid. A modal analysis was performed for each of the buildings, knowing the exact characteristics of section dimension, mechanical properties of materials and reinforcement ratios. Table 4-18 shows the result for the fundamental period of each of the buildings. Additionally, the average fundamental period for each of the building heights is shown.

Table 4-18. Fundamental period of the buildings computed through modal analysis.

Building	Fundamental Period T_1 (s)	Average T_1 (s)
3-DB	0.79	0.80
3-FB	0.81	
6-DB	1.90	1.82
6-FB	1.74	
9-DB	2.60	2.62
9-FB	2.64	

4.7.2 Preliminary design assessment through a pushover analysis

To evaluate the performed designs for both FBD and DBD, a pushover analysis was carried out for each of the buildings. Figure 4-35 through Figure 4-38 show the pushover curves for each of the buildings. It can be observed that for all the cases the maximum base shear is higher than the design base shear (showed in dashed lines). In general, both FBD and DBD have a similar behaviour, with some difference being observed for the three and six-storey buildings, where the maximum base shear is higher, as expected, for the FBD method.

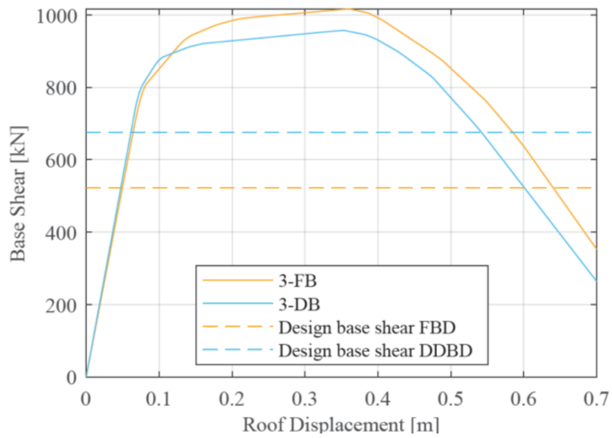


Figure 4-35. Pushover curve for 3-storey building.

As discussed previously, a set of two three-storey buildings (DBD and FBD) were also designed for seismic loading only, without considering the gravitational loading or other loading combinations for the beams of the frame. Figure 4-36 shows the pushover curve for this case. It can be observed that the maximum base shear is much lower for both cases when compared to the designs with gravitational loads. This is expected, as the governing design combination in these buildings was Combination 2, which only considers gravitational loads. When this combination was eliminated for the design for only seismic loads, the ratio of reinforcement of the beams is reduced as shown in sections 4.6.4 and 4.6.5 for FBD and DBD respectively. The decrease in the reinforcement ratio produces lower moment capacity, which during lateral loading forms hinges in the beams earlier than the design that considers the gravitational loads.

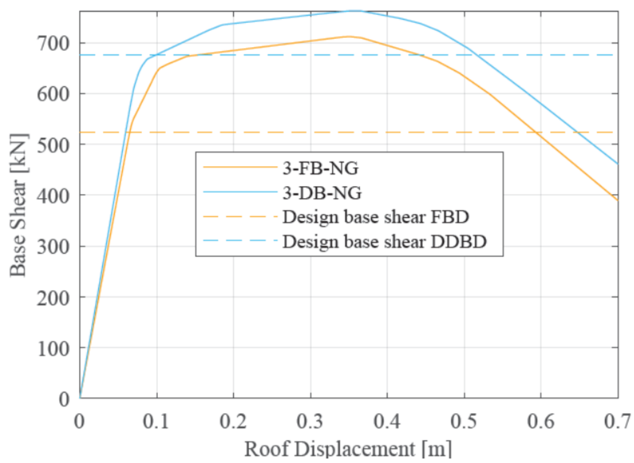


Figure 4-36. Pushover curve for 3-storey building (only seismic loading).

In all the cases, it is possible to observe that there is an additional base shear capacity. This additional base shear capacity of the buildings (when compared to the design base shear) can be attributed to the fact that gravity loads produce an overstrength in the frame, as they need to be added to the seismic load during the combination performed for the design. Additionally, as seen in Figure 4-36, the gravity loads have amplification load factors that in some cases make that a gravity combination governs the design.

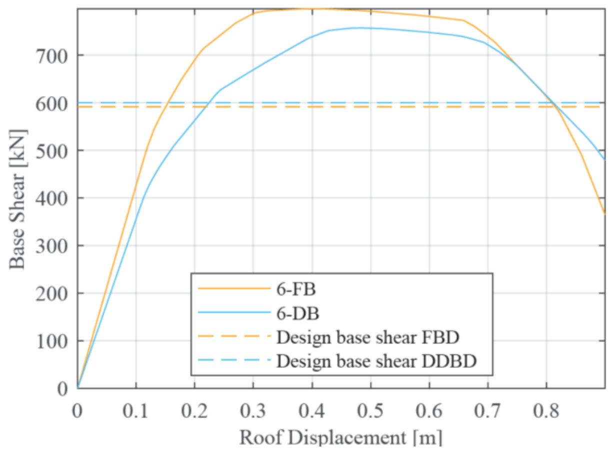


Figure 4-37. Pushover curve for 6 storey building.

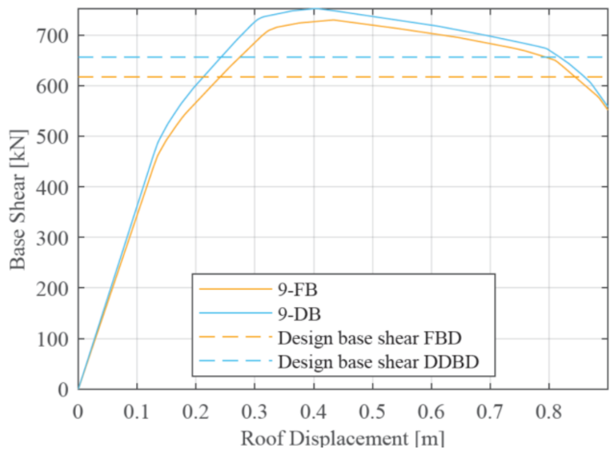


Figure 4-38. Pushover curve for 9 storey building.

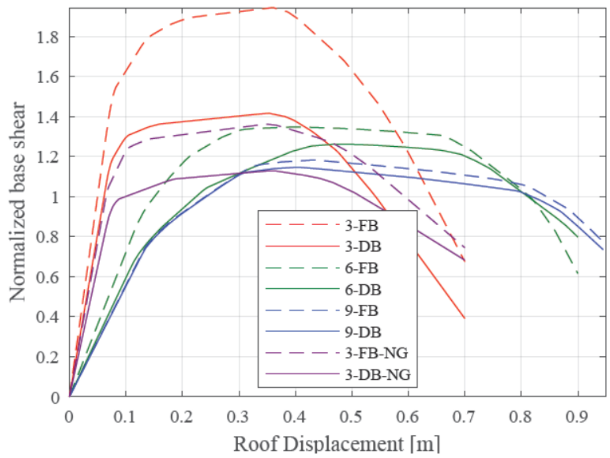


Figure 4-39. Pushover curves normalized to design base shear.

As it can be observed from the previous figures there is a factor of overstrength that can be quantified as the ratio between the maximum base shear capacity from the pushover curve and the design base shear. Figure 4-39 shows the pushover curves normalised to the design base shear, so that a clear comparison can be made. The overstrength factor ranges between 1.1 and 2.0, and it can be observed that as the height of the building increases, the overstrength factor tends to decrease. Additionally, DBD cases gives lower overstrength factors than FBD ones, which is particularly true for the case of the three-storey building. This difference in overstrength is due to a better estimation of the design base shear in the case of DBD method, because in the FBD the ductility is assumed by using the R factor and for the case of DBD the ductility is calculated based on the hysteretic behaviour of the structure and the corresponding displacement demand for the displacement spectrum.

Figure 4-39 also shows the difference between a design that considers gravity loads (3-FB and 3-DB) and the seismic load only case (3-FB-NG and 3-DB-NG). It can be observed that the overstrength is much lower for the case of only seismic loading, as the overstrength varies between 1.2 and 1.4. This overstrength factors are now similar to those of the six and nine-storey buildings, which is an indication that for the three-storey buildings the design is mostly driven by gravitational loading, and as the height increases the seismic loading starts driving the design. This is expected because the internal forces do not change in the beams with the height of the building, but the internal forces produced by the seismic loading increase with the increase of height.

4.8 Summary

This chapter presented the main design features for all of the buildings in the case study, starting with the loads used during the design and its combinations, along with the sectional properties used in the structural model that was used to determine the internal forces and displacements of the structure during the design phase. Afterwards, all the buildings are designed using both methods (FBD and DBD), showing the most important characteristics as total displacement, drifts and seismic force distribution along the height. A comparison between methods was performed, where it is possible to observe that the DBD case yields a higher base shear in all the cases and shows the importance of the scaling factor used in the RSM, which provides an amplification of the forces and stiffness of the designed buildings. Subsequently, the member design and detailing are presented for each of the buildings.

Moreover, the designs are assessed in a preliminary manner through pushover analysis performed to a non-linear model. It is possible to show that in every case, there is some degree of overstrength that comes from the gravitational loading, making the actual base shear capacity of the building higher than the design base shear.

The next chapter explores in detail the hazard characteristics of the study site, in order to perform an appropriate record selection for the risk assessment of the buildings design in this section.

5 HAZARD ASSESSMENT AND GROUND MOTION SELECTION

5.1 Introduction

In order to provide a consistent collapse risk assessment, it is fundamental that the ground shaking at the site of interest is well characterised. By performing a seismic hazard analysis, it is possible to estimate the expected ground motion caused by the occurrence of earthquakes that influence the site under study. Valuable information can be obtained from the hazard analysis such as seismic hazard curves, response spectrum and hazard disaggregation specific for the site and return period of interest.

Ground motion records are one of the main inputs for the collapse risk assessment, and for the risk estimate to be representative, the selected records must be hazard-consistent. Regarding this, record selection is done first by disaggregating (Bazzurro & Cornell, 1999) the contributions of magnitude, distance and epsilon (number of standard deviations from the median ground motion) to the hazard of the site.

The selection of a proper intensity measure (IM) is important to provide an unbiased estimate of the risk of collapse. To do so, the IM should achieve sufficiency and efficiency. Sufficiency is achieved when a set of different ground motions with the same IM value can provide an unbiased engineering demand parameter (EDP) distribution. On the other hand, efficiency indicates that the variability of the EDP is small for a set of ground motions with the same IM. In the case of this study, the chosen IM is the spectral acceleration at the first mode of vibration of the structure, $Sa(T_1)$, taking into account the cracked stiffness of the reinforced concrete sections. The selection of $Sa(T_1)$ as a sufficient and efficient IM is due to the fact that the response that is being evaluated is the collapse of the structure, which is mainly affected by the first mode. Also, as it can be observed in the modal analysis section, the first mode is dominant in the response of all the buildings with participating mass ratios of over 80%. Additionally, the hazard is disaggregated, as described before, in order to account for all other seismic parameters (such as magnitude, distance, duration, etc.) that the scalar IM is not able to consider.

5.2 Probabilistic seismic hazard assessment

The selected site for this study is San José, which is the capital city of Costa Rica and the area with the largest exposure in the country. A probabilistic seismic hazard assessment (PSHA) for San José was performed to obtain the uniform hazard spectrum, hazard curves and hazard disaggregation needed for the design and collapse risk assessment of the buildings. RESIS II (Á. Climent, Rojas, Alvarado, & Benito, 2008) Central American model was used to perform the assessment via the OpenQuake platform (Global Earthquake Model Foundation, 2018).

Seismicity in Costa Rica is considered to be medium to high with seismic events ranging between magnitude moment of 5.0 to 7.8. Most seismic events are due to the subduction zone and volcanic arch formed between Cocos and Caribbean plates, as observed in Figure 5-1.

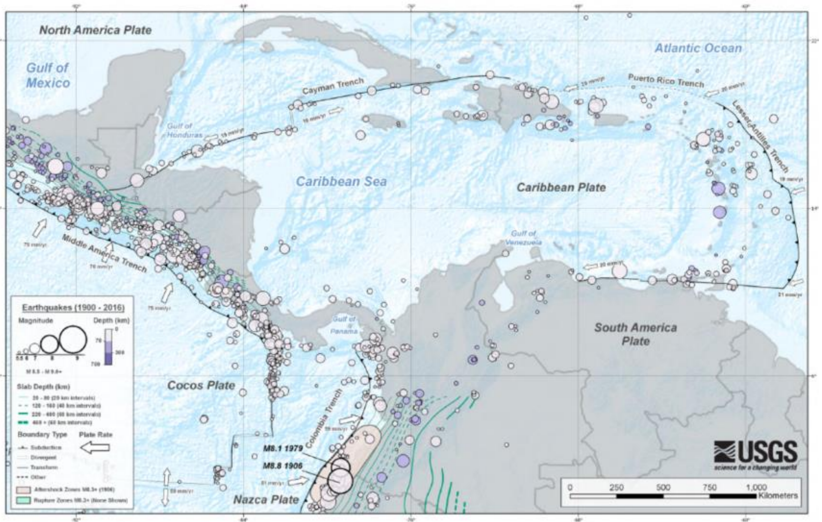


Figure 5-1. Tectonic configuration and seismicity in Central America and the Caribbean.(USGS, 2018)

Figure 5-2 shows, specifically for Costa Rica, a map of identified faults including the subduction zone. For the case of San José, the most important sources of seismicity are the faults near the volcanic arch, which extends through the middle of the territory near the city.

Figure 5-2. Fault map for Costa Rica. (Á. Climent et al., 2008).

For the purpose of obtaining representative results regarding the expected accelerations in the site, a soil with a $V_{s,30}$=265 m/s was used, as explained in Section 4.3. Figure 5-3 shows the uniform hazard spectrum for a return period of 475 years for this site and the design response spectrum according to ASCE 7-16.

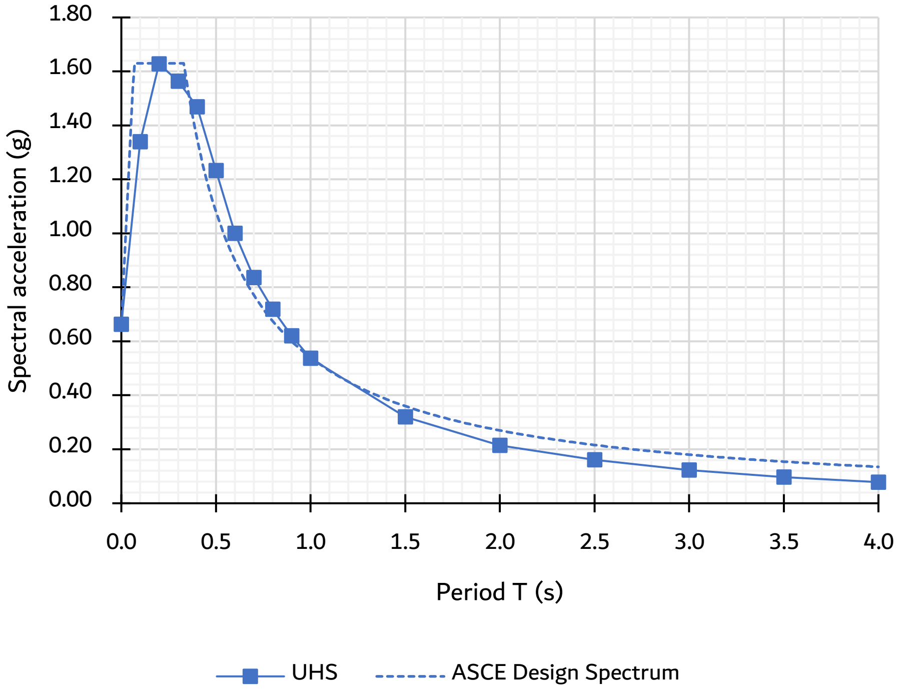


Figure 5-3. Uniform hazard spectrum with a return period of 475 years for San José.

The uniform hazard spectrum was used for the design stage for both force-based and displacement-based methodologies, so that the hazard level matches and a fair comparison can be performed for collapse risk assessment.

In order to perform the risk assessment, it is necessary to integrate the results of the incremental dynamic analysis with the corresponding hazard curve. Hazard curves represent the mean annual frequency of exceeding certain intensity levels of ground shaking, which in this case is the spectral acceleration at a given period of vibration. Equation [5-1] (McGuire, 1995) outlined the mathematical formulation that expresses the frequency of exceedance, λ, of a ground motion amplitude y, including the ground motion randomness, ε.

$$\lambda(y) = \sum_i v_i \iiint f_M(m) f_R(r) f_\varepsilon(\varepsilon) P[Y > y | m, r, \varepsilon]\, dm\, dr\, d\varepsilon \qquad [5\text{-}1]$$

For this specific study comparing the performance of RC frame structures design using FBD and DBD, hazard curves were computed for the specific periods (0.80s, 1.82s and 2.62s) corresponding to the average of the first mode of the 3, 6 and 9-storey buildings, computed from a modal analysis for the non-linear model as shown in Table 4-18. Three ground-motion prediction equations (GMPEs) were used for the PSHA, as RESIS II (Á. Climent et al., 2008) model indicates for Costa Rica. (A. Climent et al., 1994) and (Zhao et al., 2006) GMPEs were used for superficial faults, (Youngs, Chiou, Silva, & Humphrey, 1997) was used for inter-plate faults and finally (Zhao et al., 2006) and (Youngs et al., 1997) were used for intra-plate faults. Figure 5-4 shows the resulting hazard curves for each case.

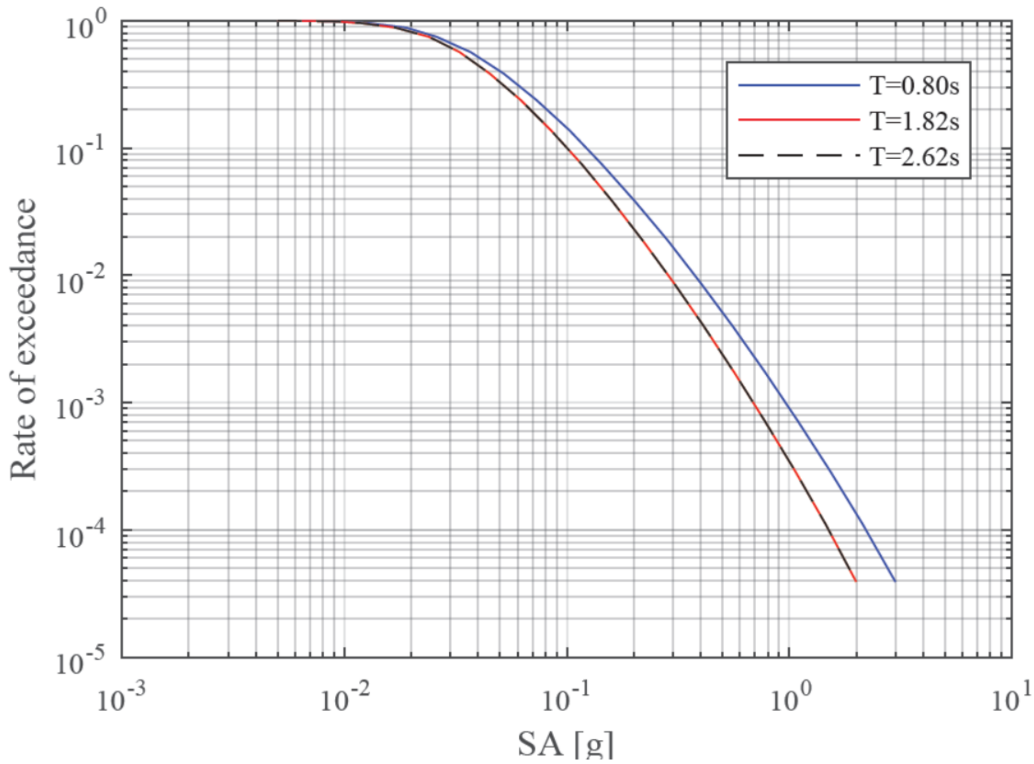


Figure 5-4. Hazard Curve for spectral acceleration at the three periods of vibration identified.

In order to proceed with ground motion record selection, disaggregation was performed for $Sa(T_1)$ of each building, and for the design return period of 475 years. As outlined by Bazzurro & Cornell (1999), disaggregation provides the conditional probability of observing an earthquake scenario (certain magnitude, M, distance, R, and epsilon, ε) given a ground motion exceedance. Figure 5-5 through Figure 5-7 show the results of disaggregation for each case. As it can be observed, for all three cases the modal value of magnitude and distance, as defined by Lin et al. (2013), is 6.75 M_w at a 10 km distance with an epsilon of 1.5.

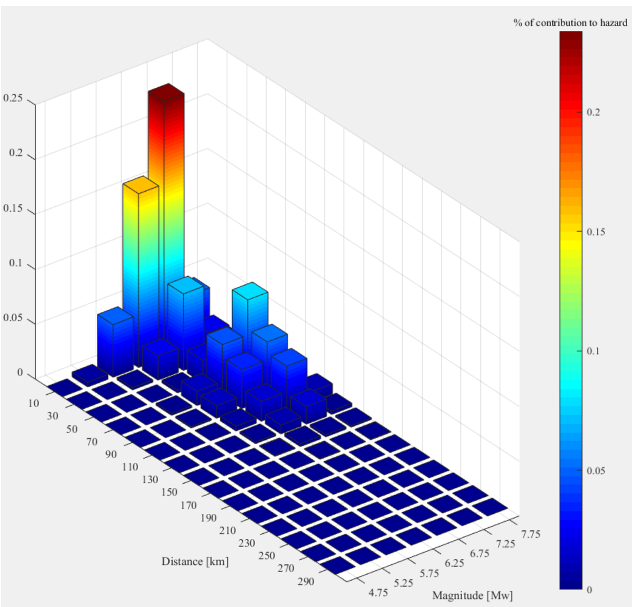


Figure 5-5. Disaggregation for Sa(0.80s), 475 years return period.

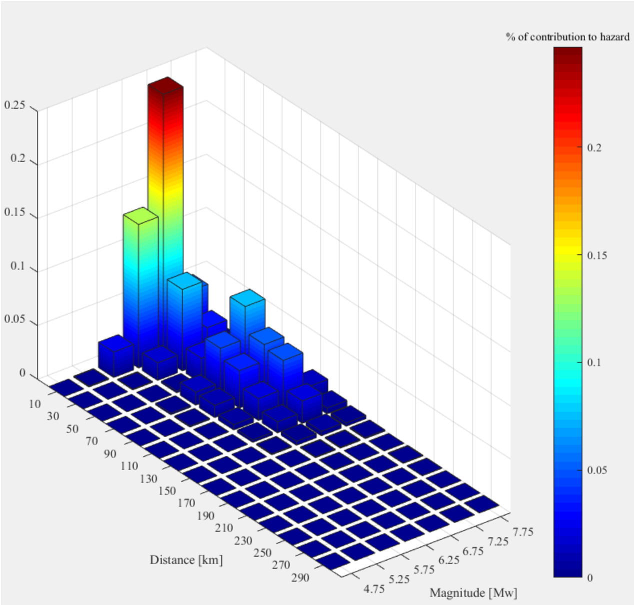


Figure 5-6. Disaggregation for Sa(1.82s), 475 years return period.

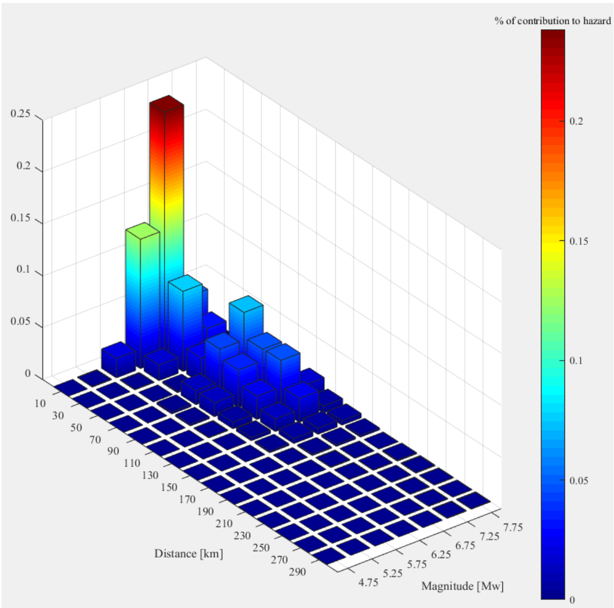


Figure 5-7. Disaggregation for Sa(2.62s), 475 years return period.

Table 5-1 shows the main parameters (magnitude, distance and epsilon) for the highest contribution event for each of the performed disaggregation analyses.

5.3 Record Selection

The selection of records was performed using the conditional mean spectrum approach (Baker, 2011) based on the results obtained from the PSHA for the three building heights. As it was observed in the previous section, the results for disaggregation show that the main earthquakes contributing to hazard are of magnitude M_w=6.75 at a distance of 10 km. Additionally, the selection was performed taking into account the value of epsilon, which is a property of the ground motion, defined as:

$$\varepsilon = \frac{\ln(S_a)_{data} - \ln(\mu_{Sa})}{\sigma_{Sa}}$$

Where $(S_a)_{data}$ is the spectral acceleration of the recording and μ_{Sa} and σ_{Sa} are predicted values of the median and logarithmic standard deviation. This parameter was proposed by Baker & Cornell (2005) as an indicator of the spectral shape that can improve the selection of records to obtain better results for calculating the mean annual frequency of exceedance of EDPs such as maximum inter-storey drift.

Selecting the records through the conditional mean spectrum (CMS) approach provides consistency with the hazard analysis performed in the previous section. As suggested by Baker (2011), CMS provides the mean spectral shape associated with the $Sa(T_1)$ target. In this way, representative ground motions can be selected so that they match the target spectral shape. Furthermore, the record selection was performed so that the maximum scaling factor to be used is 5.0. Figure 5-8 through Figure 5-10 show the response spectrum for the 30 scaled records selected using the CMS approach. As it can be observed, different record selections were performed for the spectral accelerations at the three fundamental periods of the analysed structures. The main parameters for the record selection are shown in Table 5-1, where T_1 is the fundamental period, M_w is the moment-magnitude and R is the distance.

Table 5-1. Input values for record selection using conditional spectrum approach.

No. of stories	T_1 (s)	M_w	R (km)	Epsilon
3	0.80	6.75	10	1.5
6	1.82	6.75	10	1.5
9	2.62	6.75	10	1.5

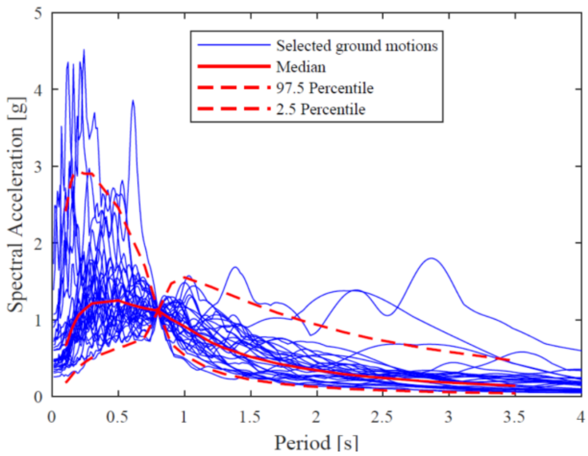


Figure 5-8. Response spectrum for selected records (3 storey buildings with T_1=0.80 s)

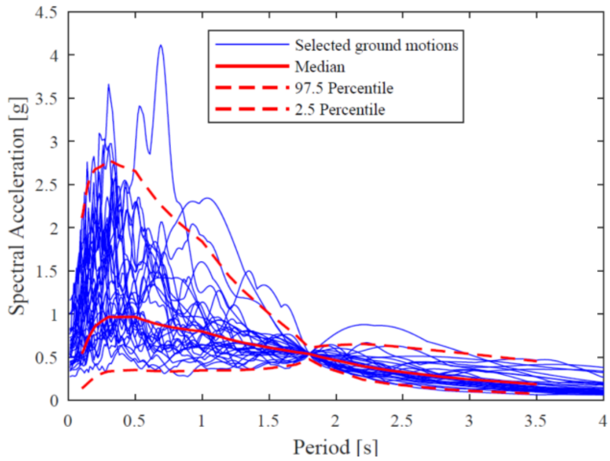


Figure 5-9. Response spectrum for selected records (6 storey buildings with T_1=1.82 s)

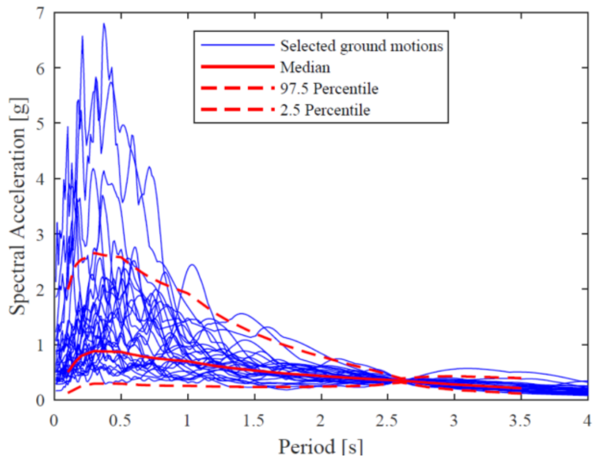


Figure 5-10. Response spectrum for selected records (9 storey buildings with T_1=2.62 s)

The record selection was performed using the conditional mean spectrum tool for ground motion selection (Baker & Lee, 2017). The records where obtained from the NGA W2 database.

5.4 Summary

A PSHA was performed for the study site, using the RESIS II model created specifically for Costa Rica. With this, it was possible to obtain the UHS for a return period of 475 years, which is the one used the design. Additionally, the hazard curves where obtained for the three building fundamental periods, which are going to be used in the risk assessment to make a convolution to obtain the mean annual frequency of exceedance.

In order to perform a reliable IDA, a record selection was performed. For this purpose, the disaggregation technique was used to obtain the modal value of magnitude, distance and epsilon for each of the building fundamental periods and for a return period of 475 years. It was observed that in all the cases the modal value was a magnitude M_w=6.75 at a distance of 10 km with an epsilon of 1.5. Afterwards, the conditional spectrum approach was used to make a record selection from the NGA W2 database. This ground motion records are used in Chapter 6, to perform a non-linear time history analysis of all of the buildings using the IDA methodology.

6 RISK ASSESSMENT

6.1 Introduction

The main objective of this study is to provide a comparison between the implicit seismic collapse risk of RC special moment frames designed using DBD and FBD approaches. As part of the study, a probabilistic seismic hazard assessment was performed and used as the basis for both the design and assessment, so that the end result is hazard-consistent.

To perform the collapse risk assessment, a suitable engineering demand parameter (EDP) must be defined. For this study, the chosen EDP was the maximum value along the building height of the peak storey drift over the duration of each ground motion, given that the structural failure mechanism is dependent on drift. Additionally, a consistent EDP was chosen with the designs that where done targeting an allowable storey drift as discussed in Chapter 4.

6.2 Incremental dynamic analysis

Collapse performance was evaluated by means of incremental dynamic analysis (IDA) (Vamvatsikos & Cornell, 2002). IDA was performed for each building with the 30 individual accelerogram records selected in the previous section. For IDA, the analysis was performed scaling each of the records until collapse was reached, yielding the curves as shown in Figure 6-1 through Figure 6-4 for each building.

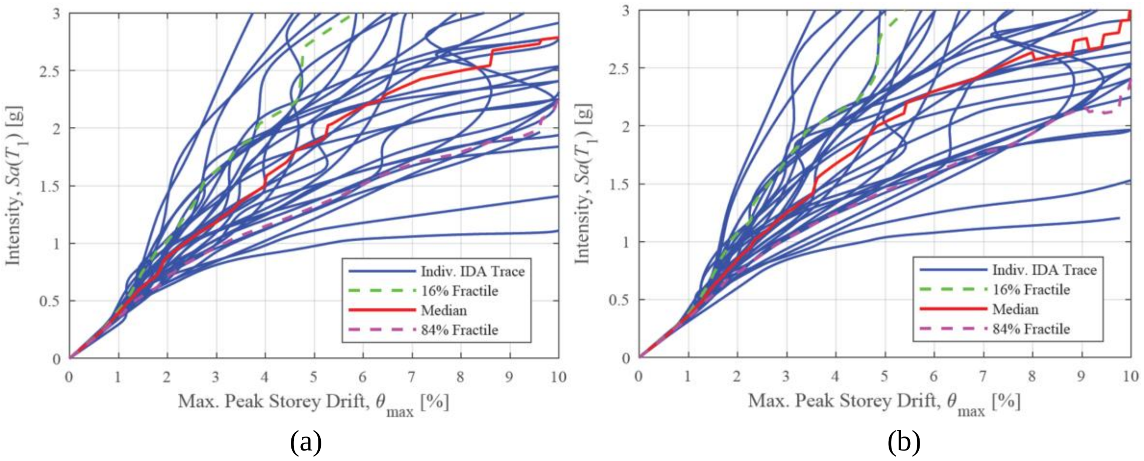


Figure 6-1. IDA results for: (a) 3 storey building designed with displacement-based method, (b) 3 storey building design with force-based design method.

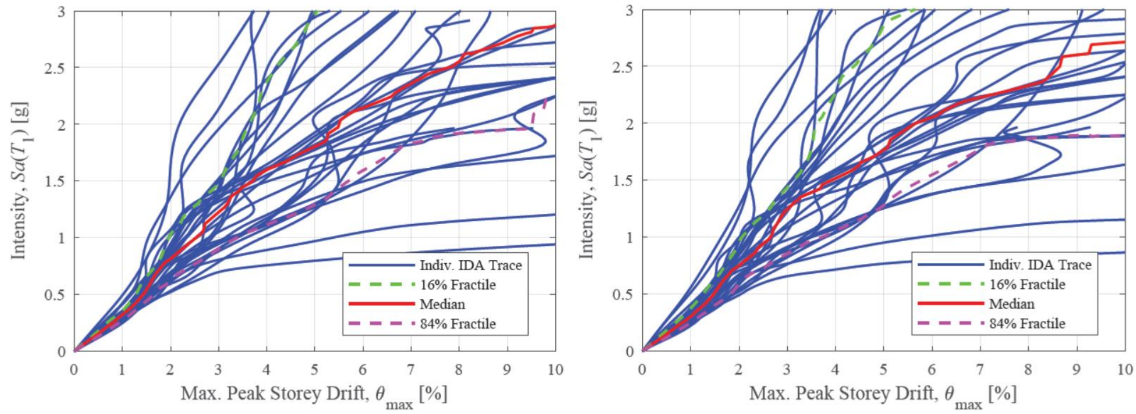
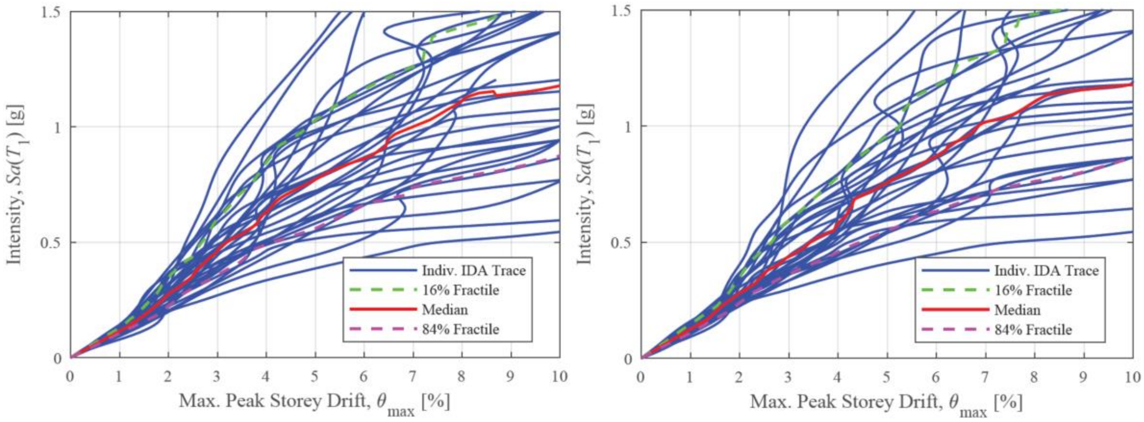
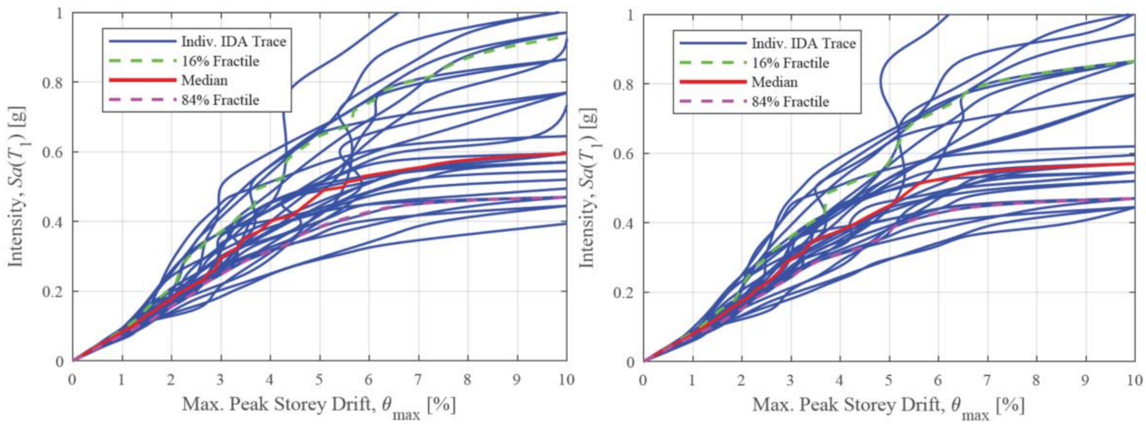


Figure 6-2. IDA results for: (a) 3 storey building designed with displacement-based method for only seismic loading, (b) 3 storey building design with force-based design method for only seismic loading.



(a) (b)

Figure 6-3. IDA results for: (a) 6 storey building designed with displacement-based method, (b) 6 storey building design with force-based design method.



(a) (b)

Figure 6-4. IDA results for: (a) 9 storey building designed with displacement-based method, (b) 9 storey building design with force-based design method.

It can be observed from the IDA curves that, as the building height increases, the intensity causing collapse tends to decrease, which is seen as the IDA traces begin to flatten off to the right. This can be associated with the fact that P-Delta effects are higher as the height increases. Additionally, the actual base shear (i.e. the maximum base shear obtained from the pushover curve) over weight ratio is lower as the height increases, as shown in Figure 6-5.

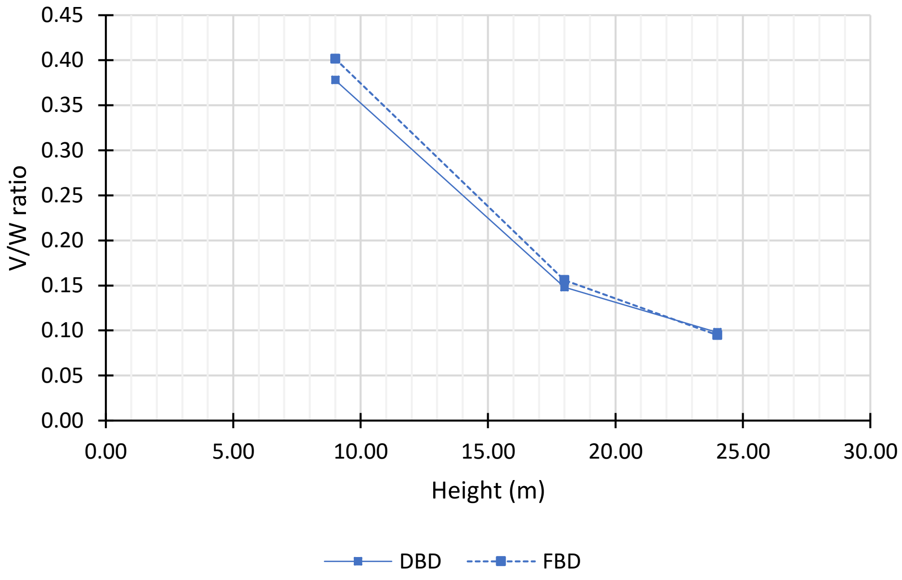


Figure 6-5. Base shear versus weight ratio for different building heights.

6.3 Hazard curve fitting

To obtain the mean annual frequency of exceedance in a closed-form manner, as described in Section 2.3, it is necessary to fit the hazard curve to the PSHA results outlined in Chapter 5. The fitting is done making a second order approximation of the hazard curve in logspace as follows:

$$H(s) = k_o \exp(-k_2 ln^2 s - k_1 ln s)$$

The fitting parameters established for each hazard curve are presented in Table 6-1.

Table 6-1. Hazard curve fitting parameters.

Number of storeys	k_o	k_1	k_2
3	0.0012	2.8402	0.2986
6	3.8624 e^{-5}	3.0841	0.2098
9	1.1572 e^{-5}	3.0549	0.1778

Figure 6-6 shows the hazard curve fitting for all the cases. As it can be observed, there is a good fitting throughout all the intensity values of the curve.

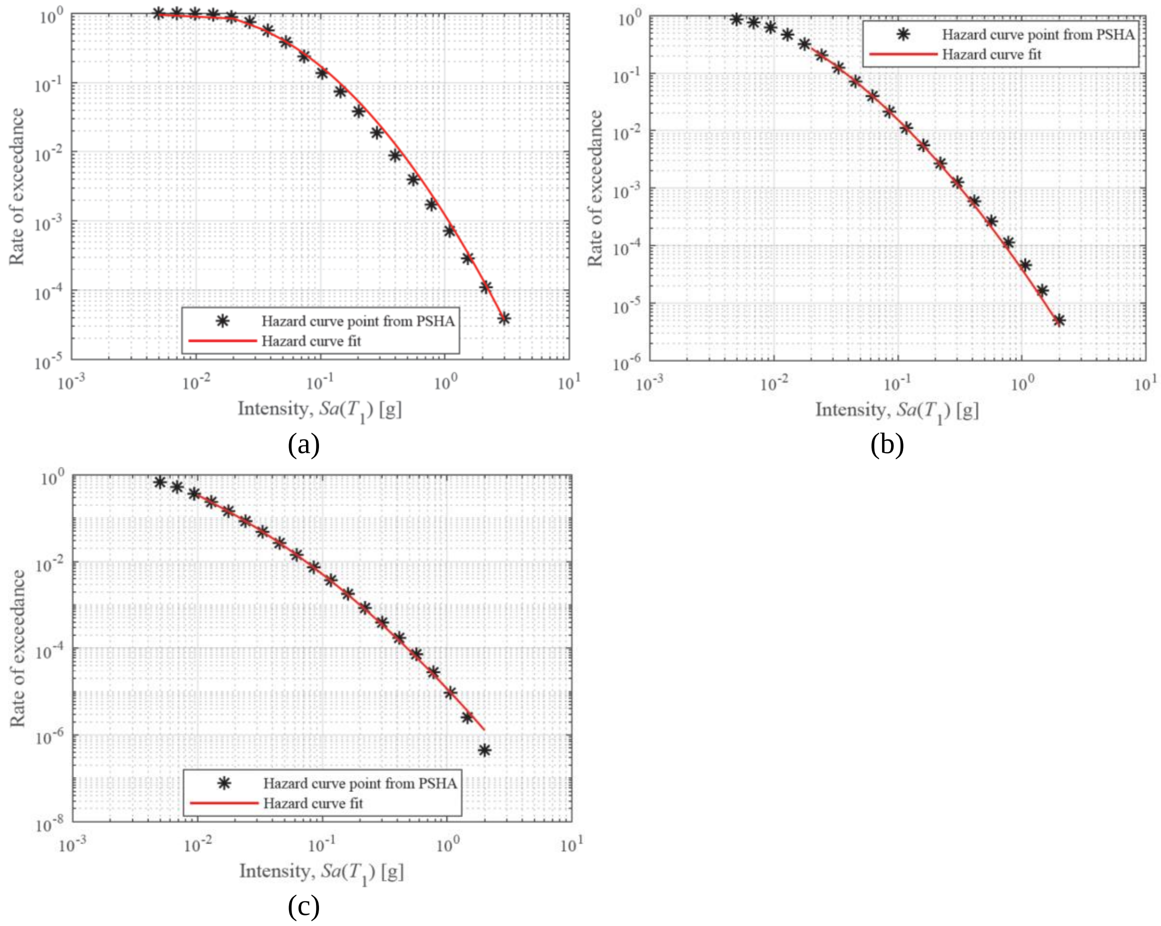


Figure 6-6. Hazard curve fit for the following periods: 0.80s (a), 1.82s (b) and 2.62s (c).

6.4 Risk assessment

An evaluation of the MAFE is performed for each of the building heights and design methods. Both aleatory and epistemic uncertainty are taken into account through dispersion from record-to-record variability and dispersion from the modelling uncertainty. Figure 6-7 shows the dispersion β_{Sc} due to record-to-record variability as a function of increasing drift. It is possible to observe that as the drift increases, the dispersion increases up to a value of approximately $\beta_{Sc} = 0.4$ whereas there is no discernible difference between the dispersion that is produced by the two design methods for drifts above 2.00%. For lower drifts it can be observed that there is a higher dispersion in FBD for the case of 3 and 9 storey buildings, and a lower dispersion in the case of the 6 storey building.

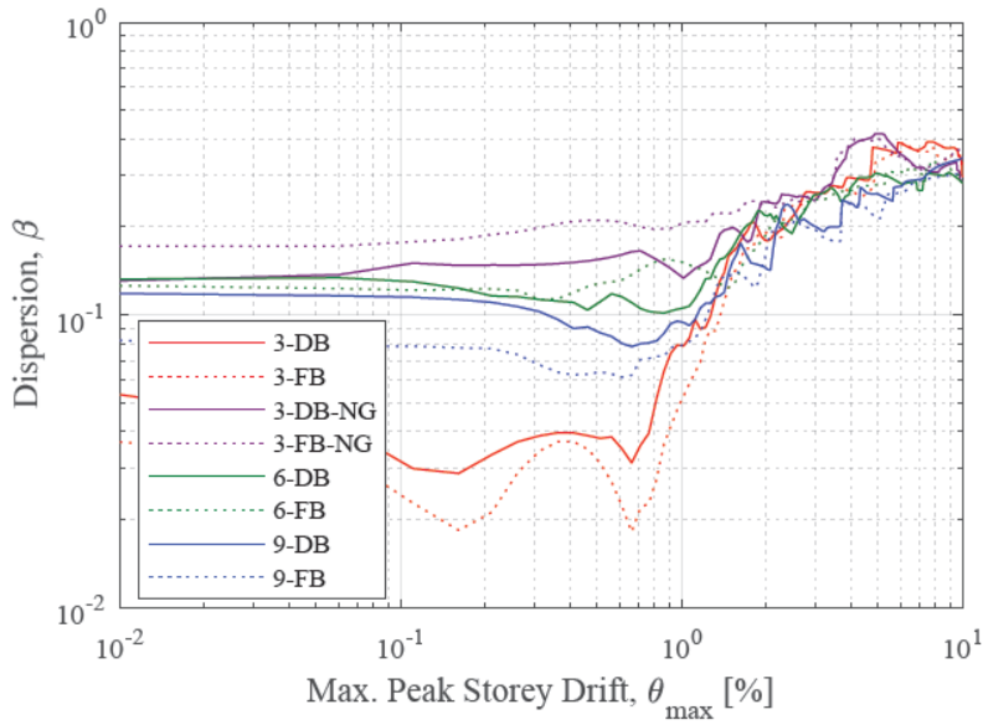


Figure 6-7. Dispersion due to record-to-record variability.

Kosič et al. (2016) studied the aleatory and epistemic uncertainty for RC moment resisting frames. The mean value for dispersion due to modelling in ten code-conforming RC moment frames was found to be of $\beta_{USc} = 0.24$. This value will be used in this study to include, in an approximate manner, the modelling uncertainty and thus having more representative MAFE results that incorporate most typical sources of uncertainty.

The result of the integration can be observed in Figure 6-8, which shows the MAFE curves for all the building heights and design methodologies. As a first observation, it can be noted that the difference of the MAFE between FBD and DBD methods is barely visible, as both methods produce comparable levels of risk with increasing drift. One of the main reasons for this result is the fact that the actual base shear for both methods was very similar for both methodologies. In the case of the three-storey building, the difference in actual base shear is approximately 6%, while for the six-storey and nine-storey buildings is around 5% and 1% respectively, for the values shown in Table 4-12. With this in mind, it is possible to say that for all the buildings, the difference is very low, and thus, the design for both methods leads to a similar structural behaviour, as seen in the pushover curves and the final MAFE result. For the case of the three-storey building, where the difference of the design base shear between both methods is high (approximately 22%), it would be expected to show different results, but it is important to highlight that in this case the gravitational forces play an important role and govern the design, which can be clearly seen in the normalised pushover curves of Figure 4-39, where it is possible to observe that the 3-DB and 3-FB buildings have a much higher actual base shear capacity and less difference between methods (6% for actual base shear instead of 22% for the design base shear) due to the gravitational loads. Then, as these particular designs are driven by the gravitational forces, which are the same in both methodologies, the designs lead again to a very similar structural behaviour. To study the influence of the gravitational forces in the seismic behaviour of the building, two three-storey buildings where designed considering only seismic loading (3-FB-NG and 3-DB-NG). As observed in

Figure 6-8, there are very small differences between both methods (FBD and DBD) despite the fact that the design base shear has a 25% difference. It can also be observed that not considering the gravitational loads results in a higher, but not significant, risk between 0.1% and 2.0% of drift.

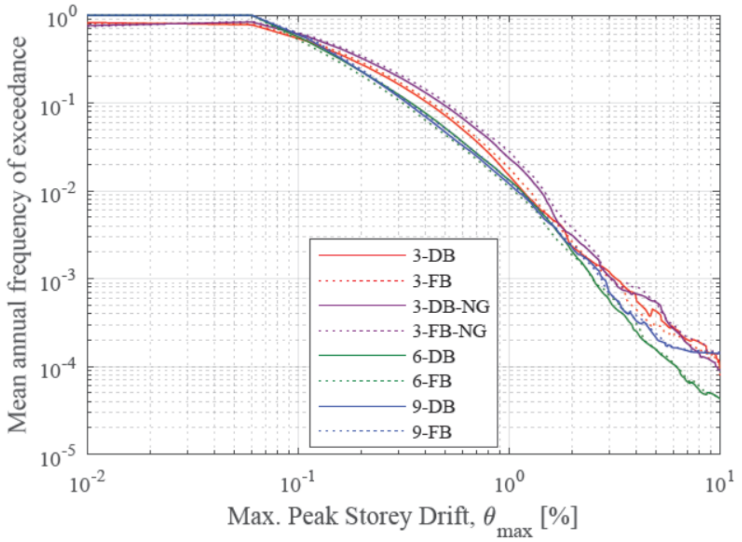


Figure 6-8. Mean annual frequency of exceedance (MAFE) for all the buildings.

Figure 6-8 also shows that there is risk-consistency between the building heights. Initially it was expected that an increase of the height would mean an increase in the risk, because as the height increases the ratio between the actual base shear and the weight of the building decreased as shown in Figure 6-5. However, this aspect is apparently balanced with a low spectral acceleration response, as the period increases because of the height of the building. Similar results were found by (Haselton et al., 2010), where it was shown that the collapse risk tends to be consistent over the building heights for RC moment frames.

It is also possible that the risk is consistent between methods (FBD and DBD) because one of the main factors that influence the behaviour of RC moment frames is the P-Delta effects, due to large deformations. The different designs give different initial structural stiffness as shown in Figure 4-39, but once the plastic hinges are formed in the beams, the behaviour is very similar for every design and it is mostly affected by P-Delta effects, that eventually drive the buildings to collapse.

At design level, the targeted limit state was a maximum drift of 2%. Table 6-2 presents a comparison for the results of the MAFE at this level of drift so that the differences between the two design methods can be observed. It can be observed that the actual base shear over weight ratio is higher than the design base shear over weight ratio for all the cases, as it was determined from the pushover curves presented in Chapter 4 due to the overstrength that comes from the gravitational forces. When comparing the MAFE, it can be observed that there is consistency (the values vary between 0.0020 to 0.0024) along the design methods and along the height for this level of drift. This is not the case for the buildings designed only for seismic loading, where it can be observed that the MAFE increases approximately a 50% when compared to the rest of the cases. In terms of dispersion, it can be observed that the dispersion coming from aleatory uncertainty is similar for all the cases but increases in the case of the buildings where only seismic loads were considered.

Table 6-2. Results comparison for 2% drift.

Design case	Design V/W ratio	Actual V/W ratio	T_1 [s]	$Sa(T)_{median}$	β_{Sc}	β_{USc}	$\lambda_{\theta=2\%}$
3-DB	0.27	0.38	0.79	1.80	0.18	0.24	0.0021
3-FB	0.21	0.40	0.81	1.70	0.19	0.24	0.0024
3-DB-NG	0.27	0.38	0.88	1.65	0.25	0.24	0.0032
3-FB-NG	0.21	0.40	0.92	1.50	0.25	0.24	0.0039
6-DB	0.12	0.15	1.90	0.25	0.21	0.24	0.0020
6-FB	0.12	0.16	1.74	0.28	0.19	0.24	0.0020
9-DB	0.09	0.10	2.60	0.19	0.16	0.24	0.0022
9-FB	0.08	0.09	2.64	0.18	0.20	0.24	0.0022

In order to make a complete comparison between both design methods, it is necessary to compare the design efficiency in terms of cost. Table 6-3 shows, for each of the buildings, the total weight of steel reinforcement and the associated cost for one frame. The cost is calculated in United States Dollars (USD) at a price of 0.94 USD per kilogram. This value represents an average of reinforcement steel price in Costa Rica.

Table 6-3. Comparison of steel weight and cost for buildings.

Building	W_{steel} [tonne]	Cost [USD]
3-DB	2.81	2646
3-FB	2.63	2475
3-DB-NG	2.12	1990
3-FB-NG	1.98	1862
6-DB	5.04	4741
6-FB	5.10	4794
9-DB	7.63	7175
9-FB	7.81	7345

Figure 6-9 and Figure 6-10 show a comparison between both methods in terms of reinforcement steel weight and cost respectively. For the case of weight, it can be observed that in the case of the 3-storey building, FBD produces a lighter design with approximately a 7% difference, but in the case of the 6 and 9 storey buildings, DBD is the method that produces a more efficient design with a 2% difference approximately.

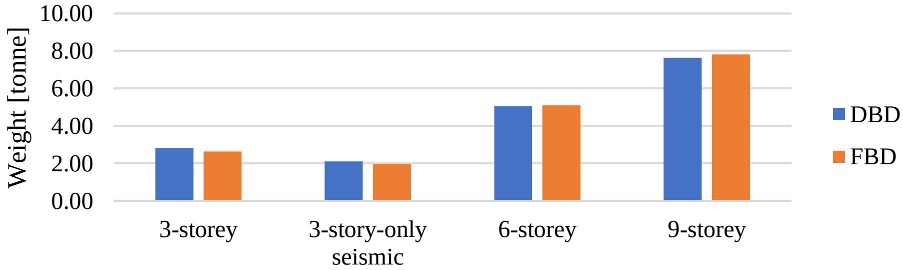


Figure 6-9. Reinforcement steel weight comparison between DBD and FBD.

Cost is directly proportional to the weight of steel, and as so, it is possible to observe from Figure 6-10 that the same trend follows. FBD produces a lower cost for the three storey building and DBD produces a lower cost for the 6 and 9-storey buildings.

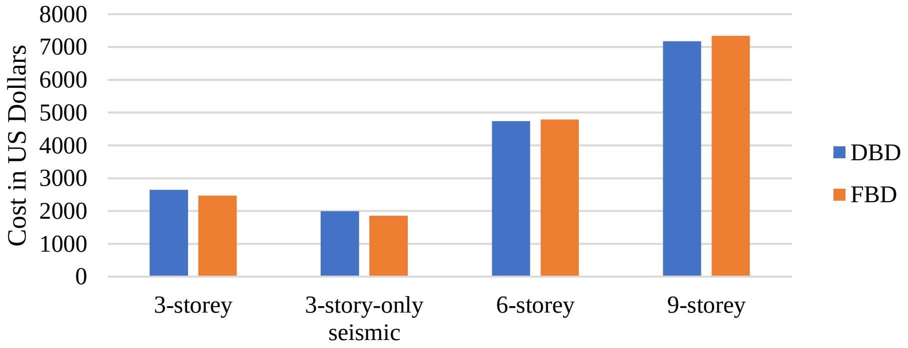


Figure 6-10. Reinforcement steel cost comparison between DBD and FBD.

In absolute value, if it is considered that the building consists of approximately 10 individual frames, the total cost for the steel reinforcement in a 3-storey building is 26460 USD for DBD and 24750 USD for FBD, then for this case the difference between both methods is 1710 USD. In the case of the 6-storey building the steel cost is 47410 USD and 47940 USD for DBD and FBD respectively, with a difference of 530 USD. Finally, for the case of the 9-storey building the steel cost is 71750 USD and 73450 USD for DBD and FBD respectively, with a difference of 1700 USD. Then, in terms of absolute value, the most significant difference can be observe in the 3 and 9-storey buildings, where in the first case the FBD method is more cost-efficient and in the second case the DBD method is more cost-efficient.

6.5 Summary

In order to perform a risk assessment, an IDA was performed for each of the buildings in the case study. From the IDA, statistical measurements of performance were obtained, specifically the median, the 16 and 84 percentiles and the dispersion for different values of drift. Then, a convolution between a fitted hazard curve and the IDA results was computed to obtain the MAFE at every value of drift from 0% to 10%. The results were then analysed and compared for each of the design methods and building heights. It was determined that there exists risk consistency between both the methods and the buildings heights. Additionally, a comparison between both methods in terms of cost-efficient designs was performed using an average price for reinforcement steel in Costa Rica.

7 SUMMARY, CONCLUSIONS AND FUTURE STUDIES

7.1 Summary

A risk assessment study was performed for a set of six RC moment resisting frames designed through the FBD and DBD methodologies. FBD was done following the provisions of ASCE 7-17 (ASCE, 2017), while DBD was done following the provisions of DBD12 Model Code (Sullivan et al., 2012). The design and detailing of the concrete members was carried out following the requirements of ACI 318-14 (ACI, 2015).

To obtain hazard-consistent results, a site-specific probabilistic hazard assessment was computed for San José, Costa Rica. The resulting uniform hazard spectrum was used to perform the designs with both methodologies (FBD and DBD). Conditional mean spectrum approach (Baker, 2011) was used to carry out ground motion record selection for non-linear dynamic analysis.

A non-linear two-dimensional model was created to evaluate the performance of the buildings by implementing an IDA (Vamvatsikos & Cornell, 2002). Through the convolution with the hazard curve, it was possible to obtain the MAFE. The obtained results were used to make a risk-based comparison between both methods and evaluate if the designs are risk-consistent.

7.2 Conclusions

The following conclusions can be extracted from the performed risk assessment:

- The results of the study suggested that there is not a major difference in risk for a 2% drift exceedance for buildings designed for FBD or DBD when following the respective design codes and detailing rules.

- There was very small influence of building height and the associated seismic risk. The results showed that there was consistency of the risk over the building heights.

- The force scaling that is required by ASCE 7-16 had an important effect on the overall performance of the buildings designed using the RSM. As an effect of this scaling, the base shear amplified more than double in some cases which directly affected the associated collapse performance.

- The ELFM is the most conservative approach as it gave the highest design base shear amongst FBD and DBD methods. Although no risk assessment was performed for the ELFM, it is reasonable to expect a lower risk of collapse for buildings designed in this manner due to its conservative nature.

- Overstrength due to gravitational loads and the load combinations involved in the design governed the design of the beams in many cases. A risk assessment was performed for the case of three-storey buildings in which only seismic loading was considered. The results showed that there is a slight increase in the risk between values of 0.1% and 2% of drift when compared to the standard design that considers gravitational loads.

- One of the components of capacity design is the strong-column weak-beam (SCWB) design established in ACI 318-14, which was explained in section 4.6.3. This provision of the code may have had a strong influence on the results, as the ratio of steel in the column design was in many cases governed by this. Haselton et al. (2010) showed that the SCWB has a very important effect in the collapse probability. The main effect of this provision over the structural behaviour is that having stronger columns increases the probability of having hinging distributed across the height of the building.

- Although the results suggest that the risk associated with the design using both methods is essentially the same, it is important to note that using FBD was very time consuming when compared to the DBD method, which can be an advantage from a designer's perspective.

- When comparing the methods from a cost-efficient point of view, it can be observed that for the three-storey building, the FBD method produces a better design, but for taller buildings (six and nine storeys), DBD produces a more cost-efficient design.

7.3 Future studies

As a first attempt to compare the risk consistency of FBD and DBD designs, this study has limitations that are worth noting and that could be studied in the future:

- Variables involving the design of the buildings are numerous and can produce very different outcomes. Future studies should include other building heights and different bay spacings.

- Additional risk assessment should be performed for sites with different hazard levels, as this study is limited to one site.

- The analysis was performed on a two-dimensional model of frame without any irregularities. Performing a three-dimensional analysis could offer better insight as it is possible to model some real building features, such as staircases and lift ducts, that induce irregularities.

- Other building typologies should be considered, as the structural behaviour is influenced depending on the hysteretic behaviour of the structural elements. In the case of this study, the ductility comes from the plastic hinges formed at the beam ends, while in the case of other typologies such as walls, the ductility comes from the plastic hinge formed at the base of the wall. These two cases will show different behaviour not only because the hysteresis is different for a wall than for a beam, and also, because the position of the hinging is different and thus the deformation of the structure will change.

8 REFERENCES

ACI. (2015). *Building Code Requirements for Structural Concrete and Commentary*. Michigan: American Concrete Institute.

ASCE. (2017). *ASCE/SEI 7-16 Minimum design loads for buildings and other structures*. *ASCE standard*. American Society of Civil Engineers. https://doi.org/10.1061/9780784412916

Baker, J. W. (2011). The Conditinal Mean Spectrum: A Tool for Ground Motion Selection. *ASCE Journal of Structural Engineering*, *137*(3), 322–331. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000215

Baker, J. W., & Cornell, C. A. (2005). A vector-valued ground motion intensity measure consisting of spectral acceleration and epsilon. *Earthquake Engineering and Structural Dynamics*, (October 2004), 1193–1217. https://doi.org/10.1002/eqe.474

Baker, J. W., & Lee, C. (2017). An Improved Algorithm for Selecting Ground Motions to Match a Conditional Spectrum. *Journal of Earthquake Engineering*, *2469*(January). https://doi.org/10.1080/13632469.2016.1264334

Bazzurro, P., & Cornell, C. A. (1999). Disaggregation of Seismic Hazard, (April), 501–520.

Bradley, B. A., & Dhakal, R. P. (2008). Error estimation of closed-form solution for annual rate of structural collapse. *Earthquake Engineering and Structural Dynamics*, (July), 1721–1737. https://doi.org/10.1002/eqe

CFIA. (2011). *Código Sísmico de Costa Rica 2010* (Editorial). Cartago: Colegio Federado de Ingenieros y Arquitectos de Costa Rica.

Climent, Á., Rojas, W., Alvarado, G. E., & Benito, B. (2008). Proyecto Resis II: Evaluación de la amenaza sísmica en Costa Rica. *Red Sismológica Nacional, Universidad de Costa Rica, Instituo Costarricense de Electricidad, Universidad Politécnica de Madrid*, 122.

Climent, A., Taylor, W., Ciudad Real, M., Strauch, W., Villagram, M., Dahle, A., & Bungum, H. (1994). *Spectral strong-motion attenuation in Central America*, *NORSAR Technical Report, No. 2-17*.

Cornell, C. A. (1968). ENGINEERING SEISMIC RISK ANALYSIS. *Bulletin of the Seismological Society of America*, *58*(5), 1583–1606.

Cornell, C. A., Jalayer, F., Hamburger, R. O., & Foutch, D. A. (2002). Probabilistic Basis for 2000 SAC Federal Emergency Management Agency Steel Moment Frame Guidelines. *Journal of Structural Engineering*, *128*(April 2001), 526–533.

European Committee for Standardisation. (2017). *Final Draft EN1998-1 NEN SC8 PT1*.

Federal Emergency Management Agency. (2012). *Seismic Performance Assessment of Buildings - Volume 1 – Methodology FEMA P-58-1*. *Fema P-58-1*.

FEMA, & ATC. (2009). *Quantification of Building Seismic Performance Factors*. *FEMA P695*. https://doi.org/10.1016/j.compstruc.2009.08.001

Global Earthquake Model Foundation. (2018). OpenQuake Engine. Retrieved from https://github.com/gem/oq-engine/tree/engine-3.3

Haselton, C. B., Goulet, C. A., Mitrani-reiser, J., Beck, J. L., Deierlein, G. G., Porter, K. A., & Stewart, J. P. (2008). *An Assessment to Benchmark the Seismic Performance of a Code-Conforming Reinforced Concrete Moment-Frame Building*.

Haselton, C. B., Liel, A. B., Deierlein, G. G., Dean, B. S., & Chou, J. H. (2010). Seismic Collapse Safety of Reinforced Concrete Buildings . I : Assessment of Ductile Moment Frames. *Journal of Structural Engineering*, *137*(April), 481–491. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000318.

Ibarra, L. F., & Krawinkler, H. (2005). *Global Collapse of Frame Structures under Seismic Excitations*. https://doi.org/10.1016/j.acra.2012.09.008

Iervolino, I., Spillatura, A., & Bazzurro, P. (2017). Rintc Project - Assessing the (Implicit) Seismic Risk of Code-Conforming Structures in Italy. In *Proceedings of the 6th International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering (COMPDYN 2015)* (pp. 1545–1557). https://doi.org/10.7712/120117.5512.17282

Kam, W. Y., & Pampanin, S. (2011). The seismic performance of RC buildings in the 22 February 2011 Christchurch earthquake. *Structural Concrete*. https://doi.org/10.1002/suco.201100044

Kosič, M., Dolšek, M., & Fajfar, P. (2016). Dispersions for the pushover-based risk assessment of reinforced concrete frames and cantilever walls. *Earthquake Engineering and Structural Dynamics*. https://doi.org/10.1002/eqe.2753

Lin, T., Harmsen, S. C., Baker, J. W., & Luco, N. (2013). Conditional Spectrum Computation Incorporating Multiple Causal Earthquakes and Ground-Motion Prediction Models. *Bulletin of the Seismological Society of America*, *103*(2), 1103–1116. https://doi.org/10.1785/0120110293

McGuire, R. K. (1995). Probabilistic Seismic Hazard Analysis and Design Earthquakes : Closing the Loop. *Bulletin of the Seismological Society of America*, *85*(5), 1275–1284.

Ministry of Business Innovation and Employment of New Zealand. (2017). *EPB Methodology: the methodology to identify earthquake-prone buildings*.

Priestley, M. (2003). Myths and Fallacies in Earthquake Engineering. Revisited: The Ninth Mallet Milne Lecture 2003. In *Myths and Fallacies in Earthquake Engineering. Revisited: The Ninth Mallet Milne Lecture 2003*. Pavia.

Priestley, M., Calvi, G., & Kowalsky, M. (2007). *Displacement-Based Seismic Design of Structures* (1st editio). Pavia: IUSS Press.

RINTC Workgroup. (2018). *Results of the 2015-2017 Implicit seismic risk of code- conforming structures in Italy (RINTC) project*. Naples, Italy. Retrieved from http://www.reluis.it/

Silva, V., Crowley, H., & Bazzurro, P. (2014). RISK-TARGETED HAZARD MAPS FOR EUROPE. In *Second European Conference on Earthquake Engineering and Seismology* (pp. 1–12). Istanbul.

Sullivan, T. (2012). Highlighting Differences between Force-Based and Displacement-Based Design Solutions for Reinforced Concrete Frame Structures. *Structural Engineering International*, (2), 122–131. https://doi.org/10.2749/101686613X13439149156958

Sullivan, T., Priestley, M., & Calvi, G. (2012). *DBD12 A Model Code for Displacement-Based Seismic Design of Structures*. IUSS Press.

The Regents of the University of California. (2006). OpenSees. Berkeley. Retrieved from http://opensees.berkeley.edu

USGS. (2018). Caribbean_tsum. Retrieved from https://earthquake.usgs.gov/earthquakes/tectonic/images/caribbean_tsum.pdf

Vamvatsikos, D. (2013). Derivation of new SAC/FEMA performance evaluation solutions with second-order hazard approximation. *Earthquake Engineering and Structural Dynamics*, *42*, 1171–1188. https://doi.org/10.1002/eqe.2265

Vamvatsikos, D., & Cornell, C. A. (2002). Incremental dynamic analysis. *Earthquake Engineering and Structural Dynamics*, *514*(January 2001), 491–514. https://doi.org/10.1002/eqe.141

Wight, J. K., & MacGregor, J. G. (2012). *Reinforced Concrete Mechanics and Design*. Pearson.

Youngs, R., Chiou, S., Silva, W., & Humphrey, J. (1997). Strong ground motion attenuation relationships for subduction zone earthquakes. *Seismological Research Letters*, *68*(1), 58–73.

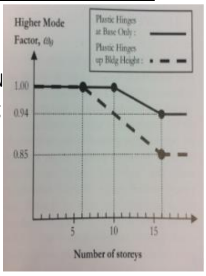
Zhao, J., Zhang, J., Asano, A., Ohno, Y., Oouchi, T., Takahashi, T., … Somerville, P. (2006). Attenuation Relations of Strong Ground Motion in Japan Using Site Classification Based on Predominant Period. *Bulletin of the Seismological Society of America*, *96*, 898–913.

APPENDIX A – DISPLACEMENT BASED DESIGN

Displacement based design for 3 storey building:

Displacement based design RF Moment Resistant Frames

No. of storeys =	3	
Storey height =	3.0	m
Storey weight=	858	kN
Storey mass=	87462	kg
Drift limit=	0.020	
ω_θ=	1.00	



Frame Properties

Hn=	9	m
α1=	2	
α2=	2	
B1=	6	m
B2=	6	m
hb1=	0.6	m
hb2=	0.6	m
M1=	1	
M2=	1	
θy1=	0.012	
θy1=	0.012	

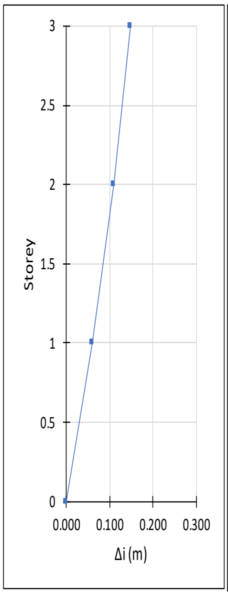
Material Properties

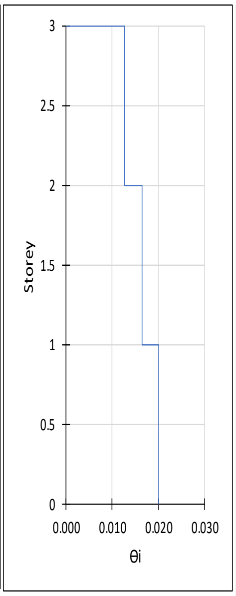
f'_c=	28	MPa
E_c=	25	Gpa
E_s=	200	GPa
f_y=	420	MPa
f_u=	525	MPa
f_{ye}=	462	MPa
f_u/f_{ye}=	1.14	
d_b=	22	mm
ϵ_y=	0.00231	
ϵ_{ls}=	0.08	

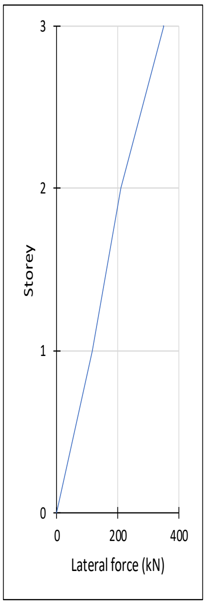
Displacement profile

Storey	H_i (m)	δi	θi	Δi (m)	m_i (kg)
0	0	0.000	0	0.00000	0
1	3	0.407	0.02	0.06000	87462
2	6	0.741	0.016364	0.10909	87462
3	9	1.000	0.012727	0.14727	87462
0	0	0.000	0	0.00000	0
0	0	0.000	0	0.00000	0
0	0	0.000	0	0.00000	0

$m_i*\Delta_i$	$m_i*\Delta_i^2$	$m_i*\Delta_i*Hi$	$m_i*g*\Delta_i$	F_i (kN)	V_{Di}	$\theta_{P-\Delta,i}$
0	0	0	0	0.00	676	0
5247.706	314.8624	15743.12	51453.76	115.44	676	25.36068
9541.284	1040.867	57247.71	93552.29	209.88	561	12.51018
12880.73	1896.981	115926.6	126295.6	350.97	351	10.36589
0	0	0	0	0.00	0	0
0	0	0	0	0.00	0	0
0	0	0	0	0.00	0	0
27670	3253	188917	271302	676		

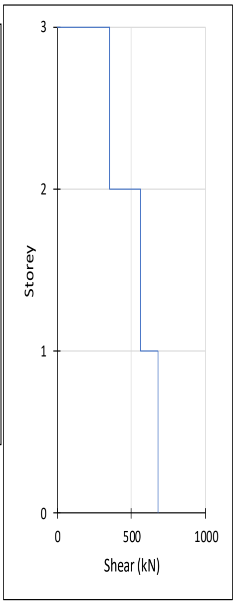






Equivalent SDOF

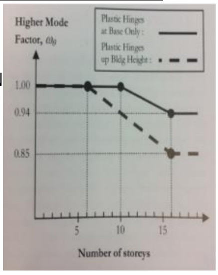
Δ_d=	0.118	m
m_e=	235377.1	kg
H_e=	6.83	m
Δ_y=	0.079	m
μ=	1.49	
ξ_{eq}=	10.92%	
η=	0.74	
T_e=	1.29	s
K_e=	5584	kN/m
V_o=	656	kN
β=	0.5	
$V_{P-\Delta}$=	20	kN
V_b=	676	kN



Displacement based design for 6 storey building:

Displacement based design RF Moment Resistant Frames

No. of storeys =	6	
Storey height =	3.0	m
Storey weight=	858	kN
Storey mass=	87462	kg
Drift limit=	0.020	
ω_θ=	1.00	



Frame Properties

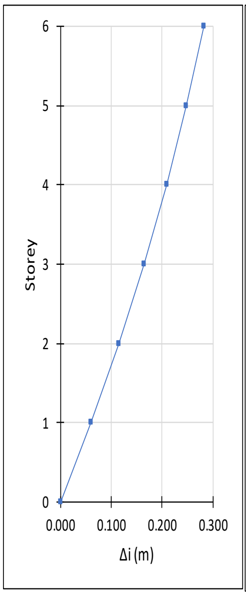
Hn=	18	m
$\alpha1$=	2	
$\alpha2$=	2	
B1=	6	m
B2=	6	m
hb1=	0.6	m
hb2=	0.6	m
M1=	1	
M2=	1	
$\theta y1$=	0.012	
$\theta y1$=	0.012	

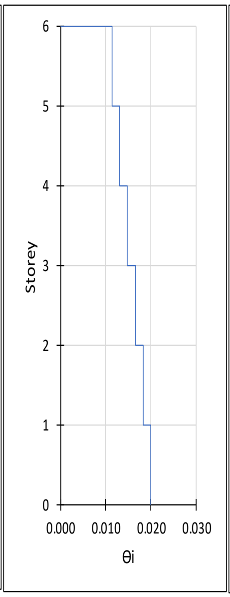
Material Properties

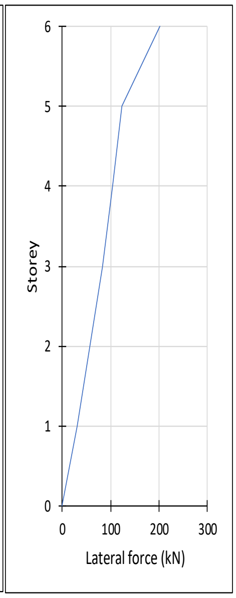
f'_c=	28	MPa
E_c=	25	Gpa
E_s=	200	GPa
f_y=	420	MPa
f_u=	525	MPa
f_{ye}=	462	MPa
f_u/f_{ye}=	1.14	
d_b=	22	mm
ϵ_y=	0.00231	
ϵ_{ls}=	0.08	

Displacement profile

Storey	H_i (m)	δi	θi	Δi (m)	m_i (kg)	$m_i*\Delta_i$	$m_i*\Delta_i^2$	$m_i*\Delta_i*Hi$	$m_i*g*\Delta_i$	F_i (kN)	V_{Di}	$\theta_{P-\Delta,i}$
0	0	0.000	0	0.00000	0	0	0	0	0	0.00	600	0
1	3	0.213	0.02	0.06000	87462	5247.706	314.8624	15743.12	51453.76	30.06	600	28.59894
2	6	0.407	0.018261	0.11478	87462	10039.09	1152.313	60234.54	98433.28	57.50	570	13.74495
3	9	0.583	0.016522	0.16435	87462	14374.15	2362.361	129367.4	140938.6	82.33	512	9.221445
4	12	0.741	0.014783	0.20870	87462	18252.89	3809.299	219034.7	178969.6	104.55	430	7.373426
5	15	0.880	0.013043	0.24783	87462	21675.31	5371.707	325129.6	212526.4	124.15	325	6.877733
6	18	1.000	0.011304	0.28174	87462	24641.4	6942.448	443545.3	241609	201.12	201	8.033678
						94231	19953	1193055	923931	600		

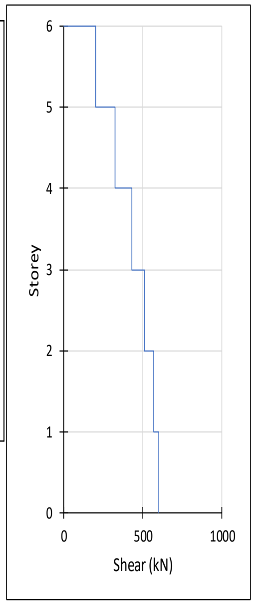






Equivalent SDOF

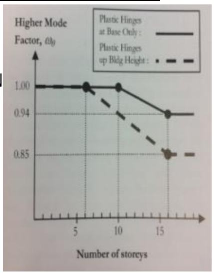
Δ_d=	0.197	m
m_e=	478327.7	kg
H_e=	12.66	m
Δ_y=	0.146	m
μ=	1.35	
ξ_{eq}=	9.63%	
η=	0.78	
T_e=	2.57	s
K_e=	2859	kN/m
V_o=	563	kN
β=	0.5	
$V_{P-\Delta}$=	36	kN
V_b=	600	kN



Displacement based design for 9 storey building:

Displacement based design RF Moment Resistant Frames

No. of storeys =	9	
Storey height =	3.0	m
Storey weight=	858	kN
Storey mass=	87462	kg
Drift limit=	0.020	
ω_θ=	0.95	



Frame Properties

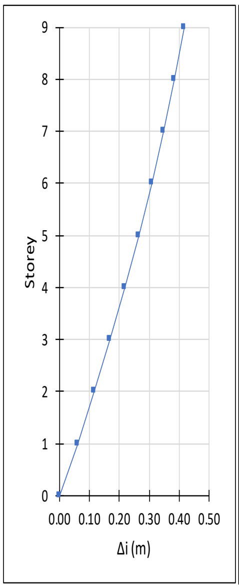
Hn=	27	m
α1=	2	
α2=	2	
B1=	6	m
B2=	6	m
hb1=	0.6	m
hb2=	0.6	m
M1=	1	
M2=	1	
θy1=	0.012	
θy1=	0.012	

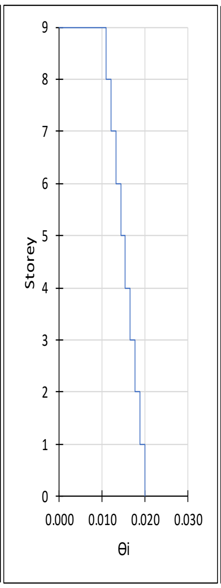
Material Properties

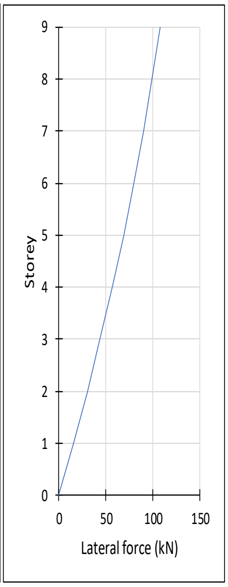
f'_c=	28	MPa
E_c=	25	Gpa
E_s=	200	GPa
f_y=	420	MPa
f_u=	525	MPa
f_{ye}=	462	MPa
f_u/f_{ye}=	1.14	
d_b=	22	mm
ϵ_y=	0.00231	
ϵ_{ls}=	0.08	

Displacement profile

Storey	H_i (m)	δi	θi	Δi (m)	m_i (kg)	$m_i*\Delta_i$	$m_i*\Delta_i^2$	$m_i*\Delta_i*Hi$	$m_i*g*\Delta_i$	F_i (kN)	V_{Di}	$\theta_{P-\Delta,i}$
0	0	0.000	0	0.00000	0	0	0	0	0	0.00	590	0
1	3	0.144	0.02	0.06000	87462	5247.706	314.8624	15743.12	51453.76	15.48	590	29.04854
2	6	0.280	0.018857	0.11657	87462	10195.54	1188.509	61173.26	99967.31	30.07	575	14.063
3	9	0.407	0.017714	0.16971	87462	14843.51	2519.156	133591.6	145540.6	43.79	545	9.293242
4	12	0.527	0.016571	0.21943	87462	19191.61	4211.188	230299.3	188173.8	56.61	501	7.08999
5	15	0.638	0.015429	0.26571	87462	23239.84	6175.158	348597.6	227866.7	68.55	444	5.953402
6	18	0.741	0.014286	0.30857	87462	26988.2	8327.789	485787.7	264619.3	79.61	376	5.431344
7	21	0.835	0.013143	0.34800	87462	30436.7	10591.97	639170.6	298431.8	89.78	296	5.433661
8	24	0.922	0.012	0.38400	87462	33585.32	12896.76	806047.7	329304.1	99.07	207	6.228019
9	27	1.000	0.010857	0.41657	87462	36434.08	15177.4	983720.1	357236.1	107.47	107	9.625922
						200163	61403	3704131	1962593	590		

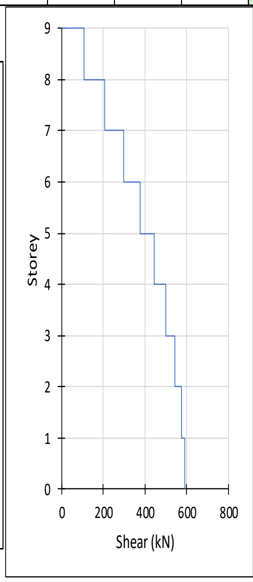






Equivalent SDOF

Δ_d=	0.253	m
$\Delta_{D,\xi}$=	0.261	m
T_D=	4.000	s
m_e=	791156.2	kg
H_e=	18.51	m
Δ_y=	0.214	m
μ=	1.18	
ξ_{eq}=	7.79%	
η=	0.85	
T_e=	3.62	s
K_e=	2383	kN/m
V_o=	603	kN
β=	0.5	
$V_{P-\Delta}$=	53	kN
V_b=	656	kN



APPENDIX B – EQUIVALENT LATERAL FORCE DESIGN

Force-based design for 3 storey building:

Effective seismic weight (ASCE 7-16 12.7.2)	
Element	**Weight (kN/m2)**
Floor system	3.23
Floor finish	0.5
Ceiling	0.2
Electromechanical components	0.1
Lightweight divisions	0.73
Total	4.76
Beams self weight (kN/m) =	3.5
Columns self weight (kN/floor)=	88
Seismic weight (kN) =	2572
Live load (kN/m2)=	2.4
Live load per floor (kN)=	345.6

Seismic Parameters	
S_1=	0.81
S_s=	2.44
F_a=	1.00
F_v=	1.50
S_{MS}=	2.44
S_{M1}=	1.22
S_{DS}=	1.63
S_{D1}=	0.81
R=	8
I_e=	1
C_d=	5.5
Ω_o=	3
$C_{s,calc}$=	0.20
$C_{s,max}$=	0.21
$C_{s,min}$=	0.07
C_s=	0.20
V=	523
Factor=	1.95

Categories according to ASCE 7-16	
Risk Category	II
Site Class	D
Seismic Design Category	D

Structure Charactertistics	
T=	0.47
Effective width=	6
Frame length=	24
Number of stories=	3
Storie height=	3
hn=	9
T,modal=	0.61
Vt=	268
k=	1

Approximate fundamental period	
hn=	9
Ct=	0.05
x=	0.90
Ta=	0.34
Cu=	1.40
T,max=	0.47

ELFM design for 6 storey building:

Effective seismic weight (ASCE 7-16 12.7.2)	
Element	**Weight (kN/m2)**
Floor system	3.23
Floor finish	0.5
Ceiling	0.2
Electromechanical components	0.1
Lightweight divisions	0.73
Total	4.76
Beams self weight (kN/m) =	3.5
Columns self weight (kN/floor)=	88
Seismic weight (kN) =	5145
Live load (kN/m2)=	2.4
Live load per floor (kN)=	345.6

Seismic Parameters	
S_1=	0.81
S_s=	2.44
F_a=	1.00
F_v=	1.50
S_{MS}=	2.44
S_{M1}=	1.22
S_{DS}=	1.63
S_{D1}=	0.81
R=	8
I_e=	1
C_d=	5.5
Ω_o=	3
$C_{s,calc}$=	0.20
$C_{s,max}$=	0.12
$C_{s,min}$=	0.07
C_s=	0.12
V=	592
Factor=	2.38

Categories according to ASCE 7-16	
Risk Category	II
Site Class	D
Seismic Design Category	D

Structure Charactertistics	
T=	0.88
Effective width=	6
Frame length=	24
Number of stories=	6
Storie height=	3
hn=	18
T,modal=	1.25
Vt=	249
k=	1.189774

Approximate fundamental period	
hn=	18
Ct=	0.05
x=	0.90
Ta=	0.63
Cu=	1.40
T,max=	0.88

ELFM design for storey building:

Effective seismic weight (ASCE 7-16 12.7.2)	
Element	**Weight (kN/m2)**
Floor system	3.23
Floor finish	0.5
Ceiling	0.2
Electromechanical components	0.1
Lightweight divisions	0.73
Total	4.76
Beams self weight (kN/m) =	3.5
Columns self weight (kN/floor)=	88
Seismic weight (kN) =	7717
Live load (kN/m2)=	2.4
Live load per floor (kN)=	345.6

Seismic Parameters	
S_1=	0.81
S_s=	2.44
F_a=	1.00
F_v=	1.50
S_{MS}=	2.44
S_{M1}=	1.22
S_{DS}=	1.63
S_{D1}=	0.81
R=	8
I_e=	1
C_d=	5.5
Ω_o=	3
$C_{s,calc}$=	0.20
$C_{s,max}$=	0.08
$C_{s,min}$=	0.07
C_s=	0.08
V=	617
Factor=	2.85

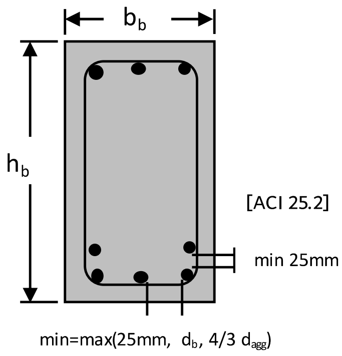
Categories according to ASCE 7-16	
Risk Category	II
Site Class	D
Seismic Design Category	D

Structure Charactertistics	
T=	1.27
Effective width=	6
Frame length=	24
Number of stories=	9
Storie height=	3
hn=	27
T,modal=	1.92
Vt=	217
k=	1.38

Approximate fundamental period	
hn=	27
Ct=	0.05
x=	0.90
Ta=	0.90
Cu=	1.40
T,max=	1.27

APPENDIX C – REINFORCED CONCRETE MEMBER DESIGN

Identification		Section Properties			Material Properties	
ID =	3-DB	h_b=	600 mm	OK!	f'_c=	28 MPa
Storey =	All	b_b=	250 mm	OK!	E_c=	24870 MPa
Grid, x=	1 - 5	L_b=	5500 mm		f_{yl}=	420 MPa
Grid, y=	NA	cv =	40 mm		f_{yt}=	420 MPa
		A_g=	150000 mm2		E_s=	200 GPa



Other input parameters	
λ=	1
min d 0	19.1 mm
min $d_{b,END2}$=	19.1 mm
ω_u=	58 kN/m

Proposed Shear Reinforcement					
END 1 (Plastic hinge 1)		CENTER (Elastic)		END 1 (Plastic hinge 1)	
ϕ =	9.5 mm	ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	2	No. legs=	2	No. legs=	2
s=	75 mm	s=	125 mm	s=	75 mm

Important notes:

-If using compression reinforcement, check requirements at the end of the spreadsheet.

-Check for requirements of max. spacing of longitudinal bars at the end of the spreadsheet.

-Lap splice locations must follow the requirements mentioned at the end of the spreadsheet.

-Check for necessary development lenghts at the end of the spreadsheet.

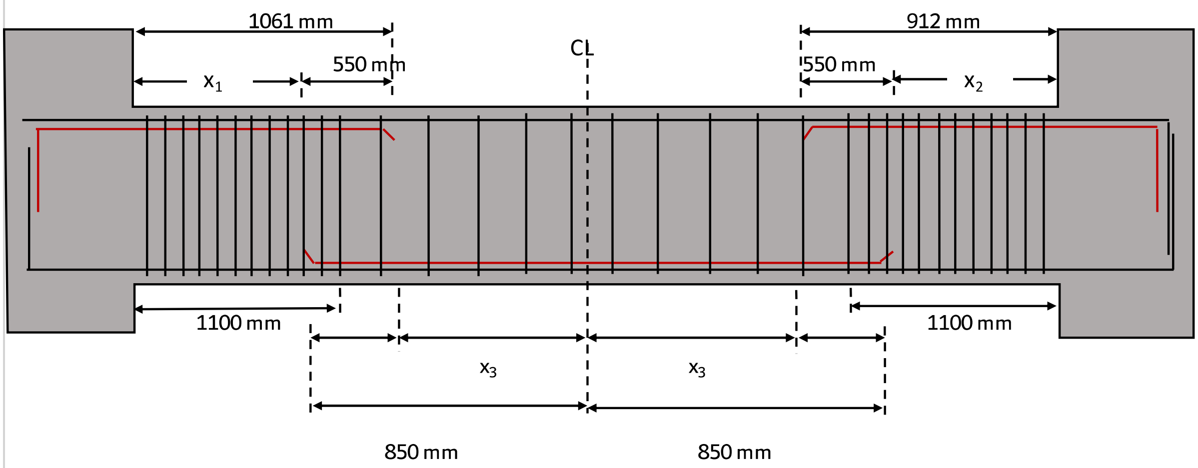
FORCES ACTING IN EACH SECTION														
END 1 =			1100 mm		CENTER =			3300 mm		END 2 =			1100 mm	
M_u^+=	100.00 kN-m				M_u^+=	115.00 kN-m				M_u^+=	100.00 kN-m			
M_u^-=	318.00 kN-m				M_u^-=	100.00 kN-m				M_u^-=	318.00 kN-m			
V_u=	214.00 kN			OK!	V_u=	214.00 kN			OK!	V_u=	214.00 kN			OK!
P_u=	13.00 kN			OK!	P_u=	13.00 kN			OK!	P_u=	13.00 kN			OK!
T_u=	1.00 kN-m			OK!	T_u=	1.00 kN			OK!	T_u=	1.00 kN			OK!
Flexural Reinforcement:					Flexural Reinforcement:					Flexural Reinforcement:				
no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)
3	22	0	0	50	3	19	0	0	50	3	22	0	0	50
2	19	0	0	300	3	19	0	0	550	2	19	0	0	300
3	19	0	0	550	0	0	0	0	0	3	19	0	0	550
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

MOMENT DESIGN								
END 1			CENTER			END 2		
Press for equilibrium					OK! All sections in equilibrium			
ϕ^+ =	0.9		ϕ^+ =	0.9		ϕ^+ =	0.9	
ϕ^- =	0.9		ϕ^- =	0.9		ϕ^- =	0.9	
ϕM_n^+ =	247 kN-m	OK!	ϕM_n^+ =	188 kN-m	OK!	ϕM_n^+ =	247 kN-m	OK!
ϕM_n^- =	308 kN-m	OK!	ϕM_n^- =	188 kN-m	OK!	ϕM_n^- =	308 kN-m	OK!
$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2	

END 1			CENTER			END 2		
$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2	
$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2	
A_s^+=	1427 mm2	OK!	A_s^+=	860 mm2	OK!	A_s^+=	1427 mm2	OK!
A_s^-=	1728 mm2	OK!	A_s^-=	860 mm2	OK!	A_s^-=	1728 mm2	OK!
$M_u^+/\phi M_n^+$ =	0.41	OK!	$M_u^+/\phi M_n^+$ =	0.61	OK!	$M_u^+/\phi M_n^+$ =	0.41	OK!
$M_u^-/\phi M_n^-$ =	1.03	OK!	$M_u^-/\phi M_n^-$ =	0.53	OK!	$M_u^-/\phi M_n^-$ =	1.03	OK!

SHEAR DESIGN								
END 1			CENTER			END 2		
ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2
A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!
s=	75 mm	OK!	s=	125 mm	OK!	s=	75 mm	OK!
s_{max}=	115 mm		s_{max}=	138 mm		s_{max}=	115 mm	
λ=	1		λ=	1		λ=	1	
V_c=	0 kN		V_c=	131 kN		V_c=	0 kN	
V_s=	437 kN		V_s=	262 kN		V_s=	437 kN	
ϕV_n=	327 kN		ϕV_n=	295 kN		ϕV_n=	327 kN	
ω_u=	58 kN/m		ω_u=	58 kN/m		ω_u=	58 kN/m	
$V_{e,max}$=	298 kN		$V_{e,max}$=	234 kN		$V_{e,max}$=	298 kN	
$V_{e,min}$=	-23 kN		$V_{e,min}$=	41 kN		$V_{e,min}$=	-23 kN	
V_{dis}=	298 kN		V_{dis}=	234 kN		V_{dis}=	298 kN	
$V_{dis}/\phi V_n$=	0.91	OK!	$V_{dis}/\phi V_n$=	0.79	OK!	$V_{dis}/\phi V_n$=	0.91	OK!

DEVELOPMENT LENGHTS AND SPLICES					
END 1		CENTER		END 2	
1. Additional reinforcement		**1. Additional reinforcement**		**1. Additional reinforcement**	
d_b=	22.2 mm	d_b=	19.1 mm	d_b=	19.1 mm
x_1=	300 mm	x_3=	300 mm	x_2=	300 mm
λ=	1	λ=	1	λ=	1
ψ_e=	1	ψ_e=	1	ψ_e=	1
ψ_s=	1	ψ_s=	0.8	ψ_s=	0.8
ψ_t=	1.3	ψ_t=	1	ψ_t=	1.3
A_{tr}=	142 mm2	A_{tr}=	142 mm2	A_{tr}=	142 mm2
c_b=	50 mm	c_b=	50 mm	c_b=	50 mm
s=	75 mm	s=	125 mm	s=	75 mm
n=	2	n=	1	n=	2
k_{tr}=	38	k_{tr}=	45	k_{tr}=	38
$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5
ξ=	3.25	ξ=	2.5	ξ=	3.25
l_d=	1061 mm	l_d=	702 mm	l_d=	912 mm
l_{bast}=	1061 mm	l_{bast}=	850 mm	l_{bast}=	912 mm
2. Development length into column		**2. Development length into column**		**2. Development length into column**	
max d_b=	22.2 mm	max d_b=	19.1 mm	max d_b=	22.2 mm
l_{dh}=	326 mm	l_{dh}=	281 mm	l_{dh}=	326 mm



MAXIMUN SPACING OF REINFORCEMENT IN TENSION FACE AND SKIN REINFORCEMENT				
Tension Face		Skin reinforcement		
c_c=	39 mm	c_c=	40 mm	
f_s=	280 MPa	f_s=	280 MPa	
max spacing=	283 mm	max spacing=	280 mm	

LATERAL SUPPORT OF COMPRESSION REINFORCEMENT

Notes for support of compression reinforcement (this applies only when compression is taken into account):

1. Minimun size:

   No.10 if longitudinal bars are No32 or less.

   No.13 if longitudinal bars are No36 or more, and for bundles.

2. Maximun spacing:

   $d_{b,long}$= 22.2 mm

   $d_{b,trans}$= 9.5 mm

   s_{max}= 250 mm

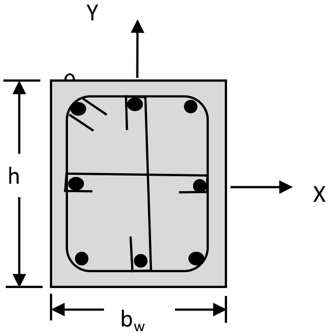
3. Longitudinal compression reinforcement shall be arranged in such a manner that every corner and alternate bar be enclosed by the corner of the transverse reinforcement with an included angle of 135 degrees or less. No bar shall be farther than 150mm clear on each side along the transverse reinforcement from such an enclosed bar.

Identification	
ID =	3-DB
Storey =	All
Grid, x=	1-5
Grid, y=	NA
0	

Material Prop.	
f'_c=	28 MPa
E_c=	24870 MPa
f_{yl}=	420 MPa
f_{yt}=	420 MPa
E_s=	200 GPa
λ=	1

Section Properties	
h=	500 mm
b_w=	500 mm
l_w=	3000 mm
c_c=	40 mm
A_g=	250000 mm2
I_{xx}=	5E+09 mm4
I_{yy}=	5E+09 mm4
d_{xx}=	450.00 mm
d_{yy}=	450.00 mm
h_x=	140 mm
ϕ_{min}=	19 mm
n_l=	12
A_{ch}=	176400 mm2
d_{agg}=	25 mm

Proposed Shear Reinforcement			
XX-Axis		YY-Axis	
Plastic hinge region		Plastic hinge region	
ϕ =	12.7 mm	ϕ =	12.7 mm
No. legs=	4	No. legs=	4
s=	100 mm	s=	100 mm
Ash=	507 mm2	Ash=	507 mm2
Other regions		Other regions	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	4	No. legs=	4
s=	100 mm	s=	100 mm
Ash=	284 mm2	Ash=	284 mm2
Pu and Mu corresponding to Vu in x and y			
Pu=	1032 kN	Pu=	0 kN
Mu=	338 kN	Mu=	0 kN



Axial Force		Shear Force in x		Shear Force in y		Axial Force from EQ	
P_D=	637 kN	V_{Dx}=	1 kN	V_{Dy}=	0 kN	P^+_{Ex}=	134 kN
P_L=	262 kN	V_{Lx}=	1 kN	V_{Ly}=	0 kN	P^-_{Ex}=	-134 kN
P_u=	1032 kN	V_{Ex}=	147 kN	V_{Ey}=	0 kN	P^+_{Ey}=	0 kN
		V_{ux}=	149 kN	V_{uy}=	0 kN	P^-_{Ey}=	0 kN

In equilibrium!

PRESS BUTTON FOR EQUILIBRIUM

Flexural Reinforcement xx:						Flexural Reinforcement yy:					
no.	ϕ	no.	ϕ	d (mm)	Ast	no.	ϕ	no.	ϕ	d (mm)	Ast
2	22	1	22	50	###	2	22	1	22	50	###
2	22	0	0	250	774	2	22	0	0	250	774
0	0	0	0	0	0	0	0	0	0	0	0
2	22	1	22	450	###	2	22	1	22	450	###
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Moment capacity of columns and beams		
Top joint	xx-axis	yy-axis
Mprc1=	417	439
Mprc2=	417	439
Mprb1=	378	378
Mprb2=	302	302
Bot joint	xx-axis	yy-axis
Mprc1=	417	439
Mprc2=	417	439
Mprb1=	378	378
Mprb2=	302	302

Dimension and longitudinal reinforcement limits

$b_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
$h_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
b_w/h_w or h_w/b_w=		1.00	>=	0.4	OK!	[ACI 18.7.2.1 (b)]
A_{st}=	3097 mm2				OK!	[ACI 18.7.4.1]

Transverse reinforcement in plastic hinge region

l_o=	500 mm		[ACI 18.7.5.1]
$h_{x,max}$=	350 mm	OK!	[ACI 18.7.5.2, e and f]
s_{max}=	115 mm	OK!	[ACI 18.7.5.3]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
$A_{sh}/sb_{c,x}$=	1.01%	OK!	[ACI 18.7.5.4]
$A_{sh}/sb_{c,y}$=	1.01%	OK!	[ACI 18.7.5.4]

Transverse reinforcement in other regions

s_{max}=	115 mm	OK!	[ACI 18.7.5.5]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
ϕ_{min}=	9.5 mm	OK!	[ACI 25.7.2.2]

Shear force check

Vux max=	973 kN	OK!	[ACI 22.5.1.2]
Vuy max=	848 kN	OK!	[ACI 22.5.1.2]

Bi-axial moment capacity

$$\left(\frac{Mu,xx}{Mn,xx}\right)^2 + \left(\frac{Mu,yy}{Mn,yy}\right)^2 = \quad 1 \quad \text{OK!}$$

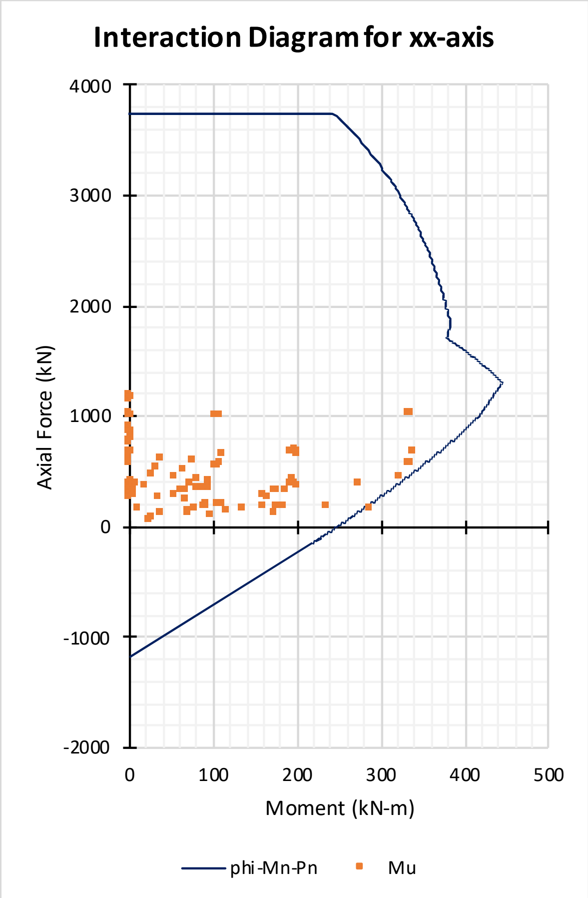
Shear capacity

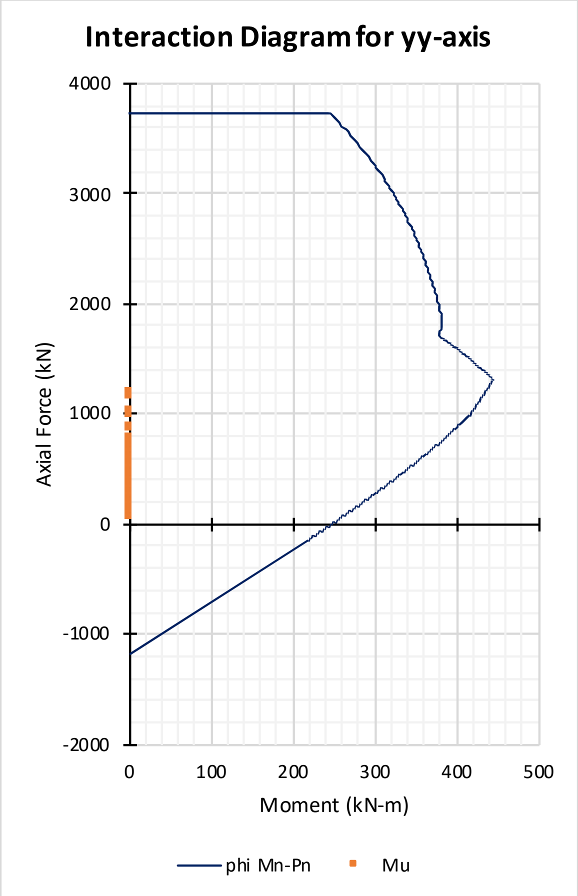
xx-axis			yy-axis		
	Plastic hinge region	Other		Plastic hinge region	Other
ϕVn=	1102 kN	786 kN	Vs=	977 kN	661 kN
Vu/ϕVn=	0.30	0.43	Vu/ϕVn=	0.34	0.50

Check
OK!

Strong column-weak beam check [ACI 18.7.3.2]

Joint	xx-axis				yy-axis			
	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check
Top	834	680	1.23	OK!	878	680	1.29	OK!
Bottom	834	680	1.23	OK!	878	680	1.29	OK!





Important notes:

-Combinations used for earthquake: 1.2D+1.0L+/-1.0E and 0.9D+/-1.0E **If different change Cap sheets accordingly

-Only symmetrical reinforcement is supported in the same axis.

-Lap splices outside lo region, the zone with lap-splice should comply with confinement reinforcement as in hinge region.

-Special provisions apply if column supports reactions from discontinued stiff members. [ACI 18.7.5.6]

-When using mechanical or welded splices check [ACI 18.7.4.3].

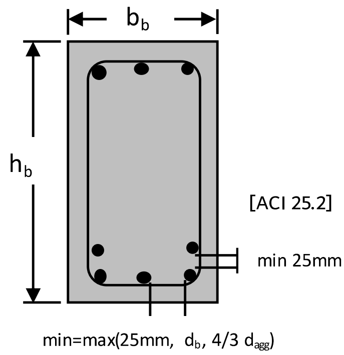
-Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees.

-No unsupported bar shall be farther than 150mm clear on each side along the tie form a laterally supported bar.

-If concrete cover exceeds 100mm, additional transverse reinforcement having cover not exceeding 100mm and spacing not exceeding 300mm shall be provided [ACI 18.7.5.7]

-If f'c exceeds 70MPa, special provisions apply [ACI 22.5.3.1]

Identification		Section Properties			Material Properties	
ID =	6-DB	h_b=	600 mm	OK!	f'_c=	28 MPa
Storey =	1,2,3,4	b_b=	250 mm	OK!	E_c=	24870 MPa
Grid, x=	1 - 5	L_b=	5500 mm		f_{yl}=	420 MPa
Grid, y=	NA	cv =	40 mm		f_{yt}=	420 MPa
		A_g=	150000 mm2		E_s=	200 GPa



0

Other input parameters	
λ=	1
min d 0	22.2 mm
min $d_{b,END2}$=	22.2 mm
ω_u=	58 kN/m

Proposed Shear Reinforcement					
END 1 (Plastic hinge 1)		CENTER (Elastic)		END 1 (Plastic hinge 1)	
ϕ =	9.5 mm	ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	2	No. legs=	2	No. legs=	2
s=	75 mm	s=	125 mm	s=	75 mm

Important notes:

-If using compression reinforcement, check requirements at the end of the spreadsheet.

-Check for requirements of max. spacing of longitudinal bars at the end of the spreadsheet.

-Lap splice locations must follow the requirements mentioned at the end of the spreadsheet.

-Check for necessary development lenghts at the end of the spreadsheet.

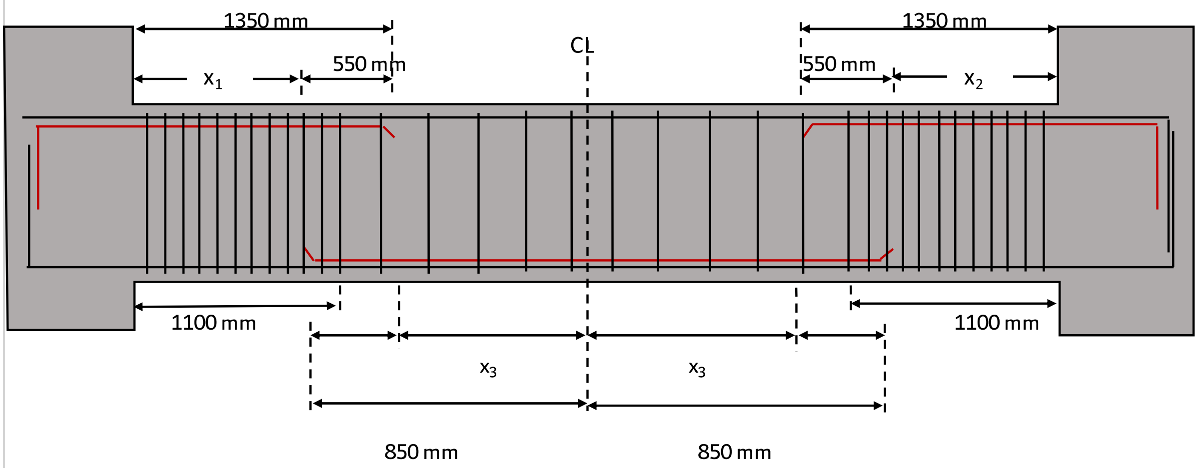
FORCES ACTING IN EACH SECTION														
END 1 =			1100 mm		CENTER =			3300 mm		END 2 =			1100 mm	
M_u^+=	88.00 kN-m				M_u^+=	120.00 kN-m				M_u^+=	88.00 kN-m			
M_u^-=	338.00 kN-m				M_u^-=	0.01 kN-m				M_u^-=	338.00 kN-m			
V_u=	220.00 kN		OK!		V_u=	200.00 kN		OK!		V_u=	220.00 kN		OK!	
P_u=	88.00 kN		OK!		P_u=	88.00 kN		OK!		P_u=	88.00 kN		OK!	
T_u=	1.00 kN-m		OK!		T_u=	1.00 kN		OK!		T_u=	1.00 kN		OK!	
Flexural Reinforcement:					Flexural Reinforcement:					Flexural Reinforcement:				
no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)
4	22	0	0	50	2	22	0	0	50	4	22	0	0	50
2	9.5	0	0	300	2	22	0	0	550	2	9.5	0	0	300
2	22	0	0	550	0	0	0	0	0	2	22	0	0	550
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

MOMENT DESIGN								
END 1			CENTER			END 2		
Press for equilibrium					OK! All sections in equilibrium			
ϕ^+ =	0.9		ϕ^+ =	0.9		ϕ^+ =	0.9	
ϕ^- =	0.9		ϕ^- =	0.9		ϕ^- =	0.9	
ϕM_n^+ =	185 kN-m	OK!	ϕM_n^+ =	170 kN-m	OK!	ϕM_n^+ =	185 kN-m	OK!
ϕM_n^- =	344 kN-m	OK!	ϕM_n^- =	170 kN-m	OK!	ϕM_n^- =	344 kN-m	OK!
$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2	

END 1			CENTER			END 2		
$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2	
$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2	
A_s^+=	916 mm2	OK!	A_s^+=	774 mm2	OK!	A_s^+=	916 mm2	OK!
A_s^-=	1690 mm2	OK!	A_s^-=	774 mm2	OK!	A_s^-=	1690 mm2	OK!
$M_u^+/\phi M_n^+$ =	0.48	OK!	$M_u^+/\phi M_n^+$ =	0.71	OK!	$M_u^+/\phi M_n^+$ =	0.48	OK!
$M_u^-/\phi M_n^-$ =	0.98	OK!	$M_u^-/\phi M_n^-$ =	0.00	OK!	$M_u^-/\phi M_n^-$ =	0.98	OK!

SHEAR DESIGN											
END 1				CENTER				END 2			
ϕ =	9.5 mm	No. legs=	2	ϕ =	9.5 mm	No. legs=	2	ϕ =	9.5 mm	No. legs=	2
A_{sh}=	142 mm2		OK!	A_{sh}=	142 mm2		OK!	A_{sh}=	142 mm2		OK!
s=	75 mm		OK!	s=	125 mm		OK!	s=	75 mm		OK!
s_{max}=	133 mm			s_{max}=	138 mm			s_{max}=	133 mm		
λ=	1			λ=	1			λ=	1		
V_c=	0 kN			V_c=	130 kN			V_c=	0 kN		
V_s=	437 kN			V_s=	262 kN			V_s=	437 kN		
ϕV_n=	327 kN			ϕV_n=	294 kN			ϕV_n=	327 kN		
ω_u=	58 kN/m			ω_u=	58 kN/m			ω_u=	58 kN/m		
$V_{e,max}$=	315 kN			$V_{e,max}$=	250 kN			$V_{e,max}$=	315 kN		
$V_{e,min}$=	-7 kN			$V_{e,min}$=	57 kN			$V_{e,min}$=	-7 kN		
V_{dis}=	315 kN			V_{dis}=	250 kN			V_{dis}=	315 kN		
$V_{dis}/\phi V_n$=	0.96		OK!	$V_{dis}/\phi V_n$=	0.85		OK!	$V_{dis}/\phi V_n$=	0.96		OK!

DEVELOPMENT LENGHTS AND SPLICES					
END 1		CENTER		END 2	
1. Additional reinforcement		**1. Additional reinforcement**		**1. Additional reinforcement**	
d_b=	22.2 mm	d_b=	19.1 mm	d_b=	22.2 mm
x_1=	800 mm	x_3=	300 mm	x_2=	800 mm
λ=	1	λ=	1	λ=	1
ψ_e=	1	ψ_e=	1	ψ_e=	1
ψ_s=	1	ψ_s=	0.8	ψ_s=	1
ψ_t=	1.3	ψ_t=	1	ψ_t=	1.3
A_{tr}=	142 mm2	A_{tr}=	142 mm2	A_{tr}=	142 mm2
c_b=	50 mm	c_b=	50 mm	c_b=	50 mm
s=	75 mm	s=	125 mm	s=	75 mm
n=	2	n=	1	n=	2
k_{tr}=	38	k_{tr}=	45	k_{tr}=	38
$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5
ξ=	3.25	ξ=	2.5	ξ=	3.25
l_d=	1061 mm	l_d=	702 mm	l_d=	1061 mm
l_{bast}=	1350 mm	l_{bast}=	850 mm	l_{bast}=	1350 mm
2. Development length into column		**2. Development length into column**		**2. Development length into column**	
max d_b=	22.2 mm	max d_b=	22.2 mm	max d_b=	22.2 mm
l_{dh}=	326 mm	l_{dh}=	326 mm	l_{dh}=	326 mm



MAXIMUN SPACING OF REINFORCEMENT IN TENSION FACE AND SKIN REINFORCEMENT			
Tension Face		Skin reinforcement	
c_c=	39 mm	c_c=	40 mm
f_s=	280 MPa	f_s=	280 MPa
max spacing=	283 mm	max spacing=	280 mm

LATERAL SUPPORT OF COMPRESSION REINFORCEMENT

Notes for support of compression reinforcement (this applies only when compression is taken into account):

1. Minimun size:

   No.10 if longitudinal bars are No32 or less.

   No.13 if longitudinal bars are No36 or more, and for bundles.

2. Maximun spacing:

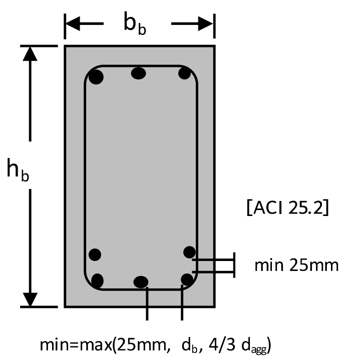
   $d_{b,long}$= 19.1 mm

   $d_{b,trans}$= 9.5 mm

   s_{max}= 250 mm

3. Longitudinal compression reinforcement shall be arranged in such a manner that every corner and alternate bar be enclosed by the corner of the transverse reinforcement with an included angle of 135 degrees or less. No bar shall be farther than 150mm clear on each side along the transverse reinforcement from such an enclosed bar.

Identification		Section Properties			Material Properties	
ID =	6-DB	h_b=	600 mm	OK!	f'_c=	28 MPa
Storey =	5,6	b_b=	250 mm	OK!	E_c=	24870 MPa
Grid, x=	1 - 5	L_b=	5500 mm		f_{yl}=	420 MPa
Grid, y=	NA	cv =	40 mm		f_{yt}=	420 MPa
		A_g=	150000 mm2		E_s=	200 GPa



0

Other input parameters		
λ=		1
min d	0	22.2 mm
min $d_{b,END2}$=		22.2 mm
ω_u=		58 kN/m

Proposed Shear Reinforcement					
END 1 (Plastic hinge 1)		CENTER (Elastic)		END 1 (Plastic hinge 1)	
ϕ =	9.5 mm	ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	2	No. legs=	2	No. legs=	2
s=	75 mm	s=	125 mm	s=	75 mm

Important notes:

-If using compression reinforcement, check requirements at the end of the spreadsheet.

-Check for requirements of max. spacing of longitudinal bars at the end of the spreadsheet.

-Lap splice locations must follow the requirements mentioned at the end of the spreadsheet.

-Check for necessary development lenghts at the end of the spreadsheet.

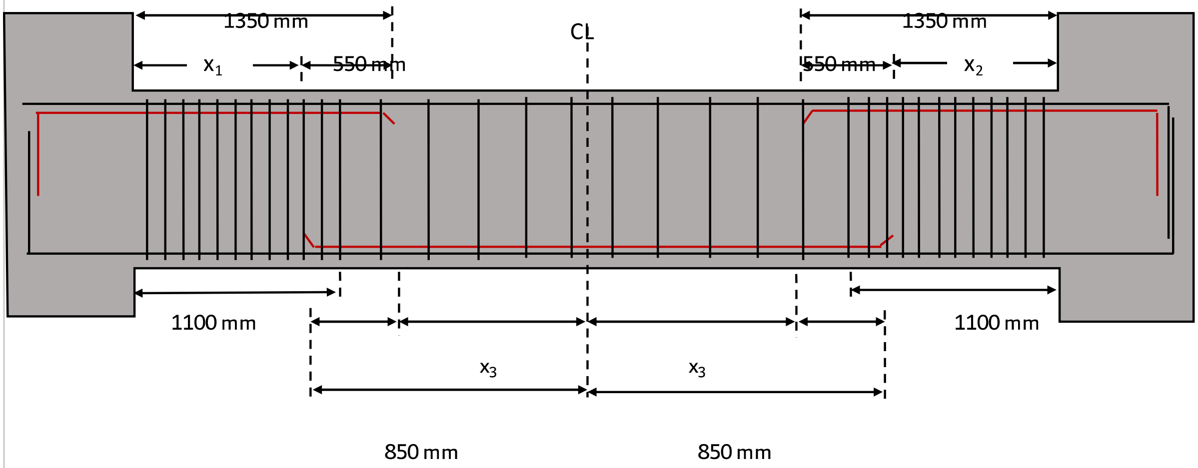
FORCES ACTING IN EACH SECTION														
END 1 =			1100 mm		CENTER =			3300 mm		END 2 =			1100 mm	
M_u^+=	50.00 kN-m				M_u^+=	101.00 kN-m				M_u^+=	50.00 kN-m			
M_u^-=	258.00 kN-m				M_u^-=	100.00 kN-m				M_u^-=	258.00 kN-m			
V_u=	220.00 kN			OK!	V_u=	200.00 kN			OK!	V_u=	220.00 kN			OK!
P_u=	88.00 kN			OK!	P_u=	88.00 kN			OK!	P_u=	88.00 kN			OK!
T_u=	1.00 kN-m			OK!	T_u=	1.00 kN			OK!	T_u=	1.00 kN			OK!
Flexural Reinforcement:					Flexural Reinforcement:					Flexural Reinforcement:				
no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)
4	19	0	0	50	2	19	0	0	50	4	19	0	0	50
2	9.5	0	0	300	2	19	0	0	550	2	9.5	0	0	300
2	19	0	0	550	0	0	0	0	0	2	19	0	0	550
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

MOMENT DESIGN								
END 1			CENTER			END 2		
Press for equilibrium						OK! All sections in equilibrium		
ϕ^+ =	0.9		ϕ^+ =	0.9		ϕ^+ =	0.9	
ϕ^- =	0.9		ϕ^- =	0.9		ϕ^- =	0.9	
ϕM_n^+ =	143 kN-m	OK!	ϕM_n^+ =	128 kN-m	OK!	ϕM_n^+ =	143 kN-m	OK!
ϕM_n^- =	262 kN-m	OK!	ϕM_n^- =	128 kN-m	OK!	ϕM_n^- =	262 kN-m	OK!
$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2	

END 1			CENTER			END 2		
$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2	
$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2	
A_s^+=	715 mm2	OK!	A_s^+=	1146 mm2	OK!	A_s^+=	715 mm2	OK!
A_s^-=	1288 mm2	OK!	A_s^-=	1146 mm2	OK!	A_s^-=	1288 mm2	OK!
$M_u^+/\phi M_n^+$ =	0.35	OK!	$M_u^+/\phi M_n^+$ =	0.79	OK!	$M_u^+/\phi M_n^+$ =	0.35	OK!
$M_u^-/\phi M_n^-$ =	0.99	OK!	$M_u^-/\phi M_n^-$ =	0.78	OK!	$M_u^-/\phi M_n^-$ =	0.99	OK!

SHEAR DESIGN											
END 1				CENTER				END 2			
ϕ =	9.5 mm	No. legs=	2	ϕ =	9.5 mm	No. legs=	2	ϕ =	9.5 mm	No. legs=	2
A_{sh}=	142 mm2		OK!	A_{sh}=	142 mm2		OK!	A_{sh}=	142 mm2		OK!
s=	75 mm		OK!	s=	125 mm		OK!	s=	75 mm		OK!
s_{max}=	133 mm			s_{max}=	138 mm			s_{max}=	133 mm		
λ=	1			λ=	1			λ=	1		
V_c=	0 kN			V_c=	136 kN			V_c=	0 kN		
V_s=	437 kN			V_s=	262 kN			V_s=	437 kN		
ϕV_n=	327 kN			ϕV_n=	298 kN			ϕV_n=	327 kN		
ω_u=	58 kN/m			ω_u=	58 kN/m			ω_u=	58 kN/m		
$V_{e,max}$=	278 kN			$V_{e,max}$=	214 kN			$V_{e,max}$=	278 kN		
$V_{e,min}$=	-43 kN			$V_{e,min}$=	21 kN			$V_{e,min}$=	-43 kN		
V_{dis}=	278 kN			V_{dis}=	214 kN			V_{dis}=	278 kN		
$V_{dis}/\phi V_n$=	0.85		OK!	$V_{dis}/\phi V_n$=	0.72		OK!	$V_{dis}/\phi V_n$=	0.85		OK!

DEVELOPMENT LENGHTS AND SPLICES					
END 1		CENTER		END 2	
1. Additional reinforcement		**1. Additional reinforcement**		**1. Additional reinforcement**	
d_b=	19.1 mm	d_b=	19.1 mm	d_b=	19.1 mm
x_1=	800 mm	x_3=	300 mm	x_2=	800 mm
λ=	1	λ=	1	λ=	1
ψ_e=	1	ψ_e=	1	ψ_e=	1
ψ_s=	0.8	ψ_s=	0.8	ψ_s=	0.8
ψ_t=	1.3	ψ_t=	1	ψ_t=	1.3
A_{tr}=	142 mm2	A_{tr}=	142 mm2	A_{tr}=	142 mm2
c_b=	50 mm	c_b=	50 mm	c_b=	50 mm
s=	75 mm	s=	125 mm	s=	75 mm
n=	2	n=	1	n=	2
k_{tr}=	38	k_{tr}=	45	k_{tr}=	38
$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5
ξ=	3.25	ξ=	2.5	ξ=	3.25
l_d=	912 mm	l_d=	702 mm	l_d=	912 mm
l_{bast}=	1350 mm	l_{bast}=	850 mm	l_{bast}=	1350 mm
2. Development length into column		**2. Development length into column**		**2. Development length into column**	
max d_b=	19.1 mm	max d_b=	19.1 mm	max d_b=	19.1 mm
l_{dh}=	281 mm	l_{dh}=	281 mm	l_{dh}=	281 mm



MAXIMUN SPACING OF REINFORCEMENT IN TENSION FACE AND SKIN REINFORCEMENT				
Tension Face		Skin reinforcement		
c_c=	40 mm	c_c=	40 mm	
f_s=	280 MPa	f_s=	280 MPa	
max spacing=	279 mm	max spacing=	280 mm	

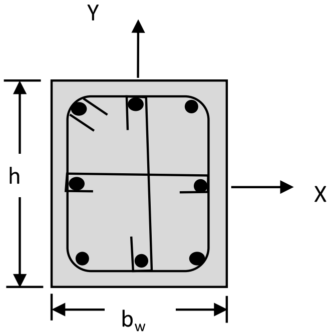
LATERAL SUPPORT OF COMPRESSION REINFORCEMENT
Notes for support of compression reinforcement (this applies only when compression is taken into account):
1. Minimun size:
No.10 if longitudinal bars are No32 or less.
No.13 if longitudinal bars are No36 or more, and for bundles.
2. Maximun spacing:
$d_{b,long}$= 19.1 mm
$d_{b,trans}$= 9.5 mm
s_{max}= 250 mm
3. Longitudinal compression reinforcement shall be arranged in such a manner that every corner and alternate bar be enclosed by the corner of the transverse reinforcement with an included angle of 135 degrees or less. No bar shall be farther than 150mm clear on each side along the transverse reinforcement from such an enclosed bar.

Identification	
ID =	6-DB
Storey =	1,2,3
Grid, x=	1-5
Grid, y=	NA

Material Prop.	
f'_c=	28 MPa
E_c=	24870 MPa
f_{yl}=	420 MPa
f_{yt}=	420 MPa
E_s=	200 GPa
λ=	1

Section Properties	
h=	500 mm
b_w=	500 mm
l_w=	3000 mm
c_c=	40 mm
A_g=	250000 mm2
I_{xx}=	5E+09 mm4
I_{yy}=	5E+09 mm4
d_{xx}=	450.00 mm
d_{yy}=	450.00 mm
h_x=	135 mm
ϕ_{min}=	22 mm
n_l=	12
A_{ch}=	176400 mm2
d_{agg}=	25 mm

Proposed Shear Reinforcement			
XX-Axis		YY-Axis	
Plastic hinge region		Plastic hinge region	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	50 mm	s=	50 mm
Ash=	213 mm2	Ash=	213 mm2
Other regions		Other regions	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	125 mm	s=	125 mm
Ash=	213 mm2	Ash=	213 mm2
Pu and Mu corresponding to Vu in x and y			
Pu=	1032 kN	Pu=	0 kN
Mu=	268 kN	Mu=	0 kN



Axial Force		Shear Force in x		Shear Force in y		Axial Force from EQ	
P_D=	1266 kN	V_{Dx}=	20 kN	V_{Dy}=	0 kN	P^+_{Ex}=	234 kN
P_L=	520 kN	V_{Lx}=	9 kN	V_{Ly}=	0 kN	P^-_{Ex}=	-234 kN
P_u=	2048 kN	V_{Ex}=	130 kN	V_{Ey}=	0 kN	P^+_{Ey}=	0 kN
		V_{ux}=	163 kN	V_{uy}=	0 kN	P^-_{Ey}=	0 kN

In equilibrium!

PRESS BUTTON FOR EQUILIBRIUM

Flexural Reinforcement xx:						Flexural Reinforcement yy:					
no.	ϕ	no.	ϕ	d (mm)	Ast	no.	ϕ	no.	ϕ	d (mm)	Ast
2	22	1	22	50	###	2	22	1	22	50	###
2	22	0	0	250	774	2	22	0	0	250	774
2	22	1	22	450	###	2	22	1	22	450	###
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Moment capacity of columns and beams		
Top joint	xx-axis	yy-axis
Mprc1=	480	501
Mprc2=	480	501
Mprb1=	390	390
Mprb2=	390	390
Bot joint	xx-axis	yy-axis
Mprc1=	480	501
Mprc2=	480	501
Mprb1=	390	390
Mprb2=	390	390

Dimension and longitudinal reinforcement limits

$b_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
$h_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
b_w/h_w or h_w/b_w=		1.00	>=	0.4	OK!	[ACI 18.7.2.1 (b)]
A_{st}=	3097 mm2				OK!	[ACI 18.7.4.1]

Transverse reinforcement in plastic hinge region

l_o=	500 mm		[ACI 18.7.5.1]
$h_{x,max}$=	350 mm	OK!	[ACI 18.7.5.2, e and f]
s_{max}=	125 mm	OK!	[ACI 18.7.5.3]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
$A_{sh}/sb_{c,x}$=	0.85%	OK!	[ACI 18.7.5.4]
$A_{sh}/sb_{c,y}$=	0.85%	OK!	[ACI 18.7.5.4]

Transverse reinforcement in other regions

s_{max}=	133 mm	OK!	[ACI 18.7.5.5]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
ϕ_{min}=	9.5 mm	OK!	[ACI 25.7.2.2]

Shear force check

Vux max=	973 kN	OK!	[ACI 22.5.1.2]
Vuy max=	848 kN	OK!	[ACI 22.5.1.2]

Bi-axial moment capacity

$$\left(\frac{Mu,xx}{Mn,xx}\right)^2 + \left(\frac{Mu,yy}{Mn,yy}\right)^2 = $$ 0.81085 OK!

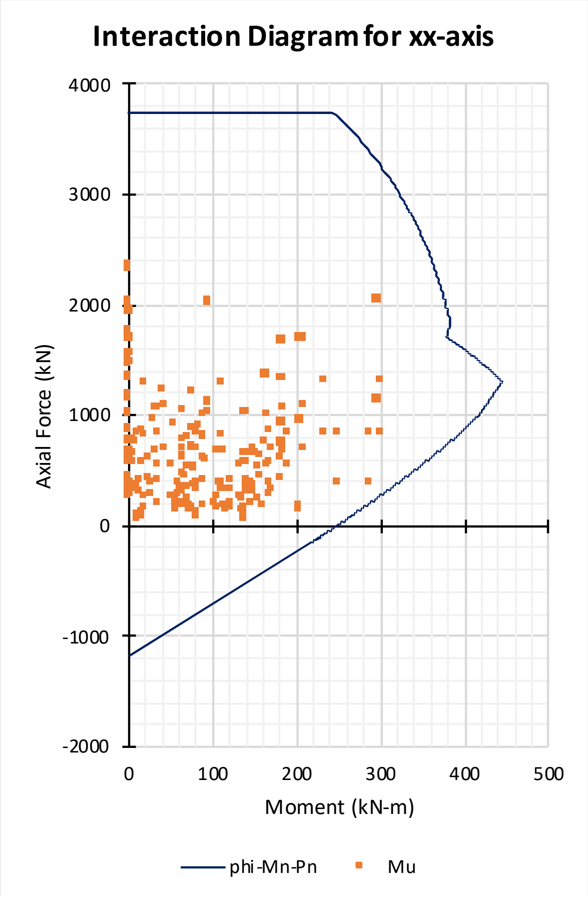
Shear capacity

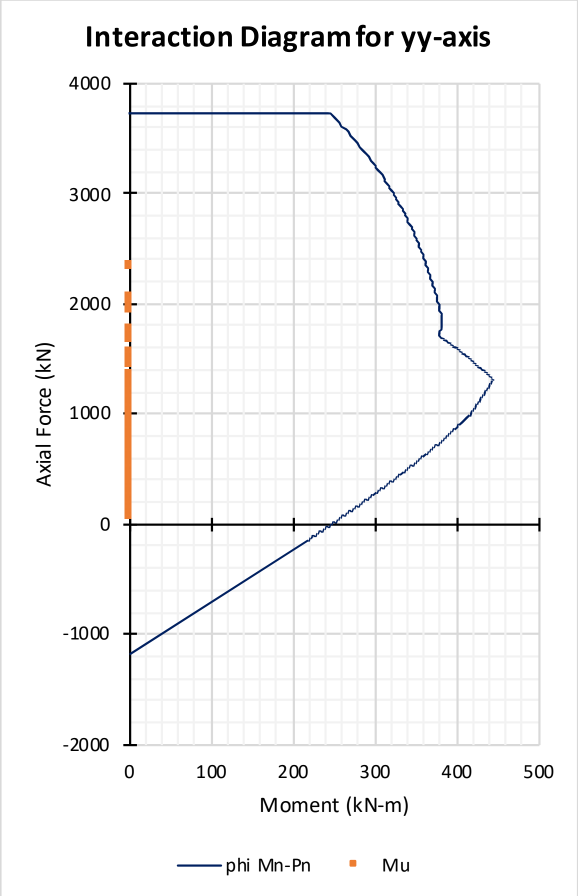
xx-axis			yy-axis		
Plastic hinge region		Other	Plastic hinge region		Other
ϕVn=	987 kN	625 kN	Vs=	862 kN	500 kN
Vu/ϕVn=	0.39	0.62	Vu/ϕVn=	0.44	0.76

Check
OK!

Strong column-weak beam check [ACI 18.7.3.2]

Joint	xx-axis				yy-axis			
	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check
Top	960	780	1.23	OK!	1002	780	1.29	OK!
Bottom	960	780	1.23	OK!	1002	780	1.29	OK!





Important notes:

-Combinations used for earthquake: 1.2D+1.0L+/-1.0E and 0.9D+/-1.0E **If different change Cap sheets accordingly

-Only symmetrical reinforcement is supported in the same axis.

-Lap splices outside lo region, the zone with lap-splice should comply with confinement reinforcement as in hinge region.

-Special provisions apply if column supports reactions from discontinued stiff members. [ACI 18.7.5.6]

-When using mechanical or welded splices check [ACI 18.7.4.3].

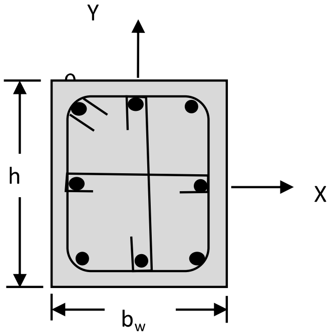
-Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees.

-No unsupported bar shall be farther than 150mm clear on each side along the tie form a laterally supported bar.

-If concrete cover exceeds 100mm, additional transverse reinforcement having cover not exceeding 100mm and spacing not exceeding 300mm shall be provided [ACI 18.7.5.7]

-If f'c exceeds 70MPa, special provisions apply [ACI 22.5.3.1]

Identification		Material Prop.		Section Properties	
ID =	6-DB	f'_c=	28 MPa	h=	500 mm
Storey =	4,5,6	E_c=	24870 MPa	b_w=	500 mm
Grid, x=	1-5	f_{yl}=	420 MPa	l_w=	3000 mm
Grid, y=	NA	f_{yt}=	420 MPa	c_c=	40 mm
		E_s=	200 GPa	A_g=	250000 mm2
0		λ=	1	I_{xx}=	5E+09 mm4
				I_{yy}=	5E+09 mm4
				d_{xx}=	450.00 mm
				d_{yy}=	450.00 mm
				h_x=	135 mm
				ϕ_{min}=	22 mm
				n_l=	12
				A_{ch}=	176400 mm2
				d_{agg}=	25 mm



Proposed Shear Reinforcement			
XX-Axis		YY-Axis	
Plastic hinge region		Plastic hinge region	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	50 mm	s=	50 mm
Ash=	213 mm2	Ash=	213 mm2
Other regions		Other regions	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	125 mm	s=	125 mm
Ash=	213 mm2	Ash=	213 mm2
Pu and Mu corresponding to Vu in x and y			
Pu=	1017 kN	Pu=	0 kN
Mu=	165 kN	Mu=	0 kN

Axial Force		Shear Force in x		Shear Force in y		Axial Force from EQ	
P_D=	652 kN	V_{Dx}=	1 kN	V_{Dy}=	0 kN	P^+_{Ex}=	129 kN
P_L=	260 kN	V_{Lx}=	1 kN	V_{Ly}=	0 kN	P^-_{Ex}=	-129 kN
P_u=	1017 kN	V_{Ex}=	99 kN	V_{Ey}=	0 kN	P^+_{Ey}=	0 kN
		V_{ux}=	101 kN	V_{uy}=	0 kN	P^-_{Ey}=	0 kN

In equilibrium!

PRESS BUTTON FOR EQUILIBRIUM

Flexural Reinforcement xx:						Flexural Reinforcement yy:					
no.	ϕ	no.	ϕ	d (mm)	Ast	no.	ϕ	no.	ϕ	d (mm)	Ast
2	22	1	22	50	###	2	22	1	22	50	###
2	22	0	0	250	774	2	22	0	0	250	774
2	22	1	22	450	###	2	22	1	22	450	###
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Moment capacity of columns and beams		
Top joint	xx-axis	yy-axis
Mprc1=	420	441
Mprc2=	420	441
Mprb1=	343	343
Mprb2=	343	343
Bot joint	xx-axis	yy-axis
Mprc1=	420	441
Mprc2=	420	441
Mprb1=	343	343
Mprb2=	343	343

Dimension and longitudinal reinforcement limits

$b_{w,min}$=	300 mm			OK!	[ACI 18.7.2.1 (a)]
$h_{w,min}$=	300 mm			OK!	[ACI 18.7.2.1 (a)]
b_w/h_w or h_w/b_w=	1.00	>=	0.4	OK!	[ACI 18.7.2.1 (b)]
A_{st}=	3097 mm2			OK!	[ACI 18.7.4.1]

Transverse reinforcement in plastic hinge region

l_o=	500 mm		[ACI 18.7.5.1]
$h_{x,max}$=	350 mm	OK!	[ACI 18.7.5.2, e and f]
s_{max}=	125 mm	OK!	[ACI 18.7.5.3]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
$A_{sh}/sb_{c,x}$=	0.85%	OK!	[ACI 18.7.5.4]
$A_{sh}/sb_{c,y}$=	0.85%	OK!	[ACI 18.7.5.4]

Transverse reinforcement in other regions

s_{max}=	133 mm	OK!	[ACI 18.7.5.5]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
ϕ_{min}=	9.5 mm	OK!	[ACI 25.7.2.2]

Shear force check

Vux max=	972 kN	OK!	[ACI 22.5.1.2]
Vuy max=	848 kN	OK!	[ACI 22.5.1.2]

Bi-axial moment capacity

$$\left(\frac{Mu,xx}{Mn,xx}\right)^2+\left(\frac{Mu,yy}{Mn,yy}\right)^2= \quad 0.81085 \quad \text{OK!}$$

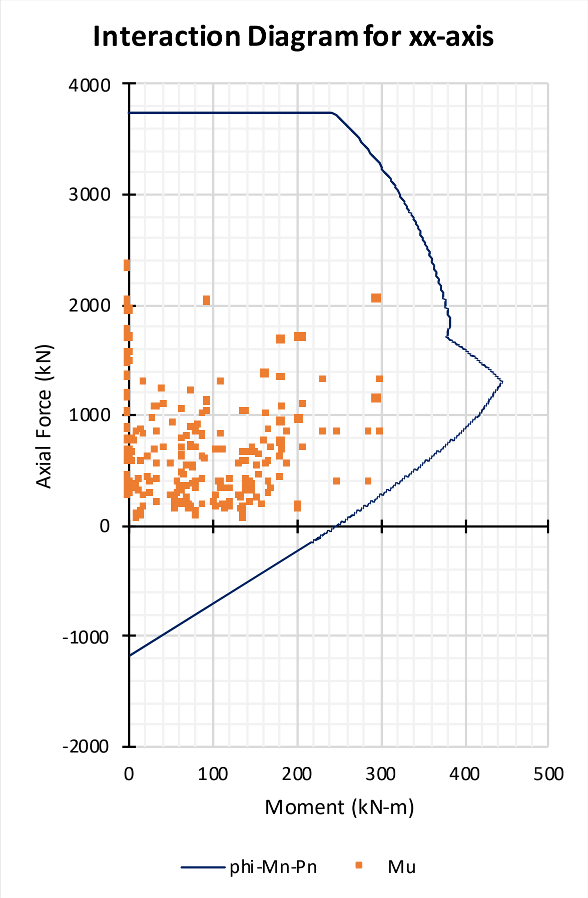
Shear capacity

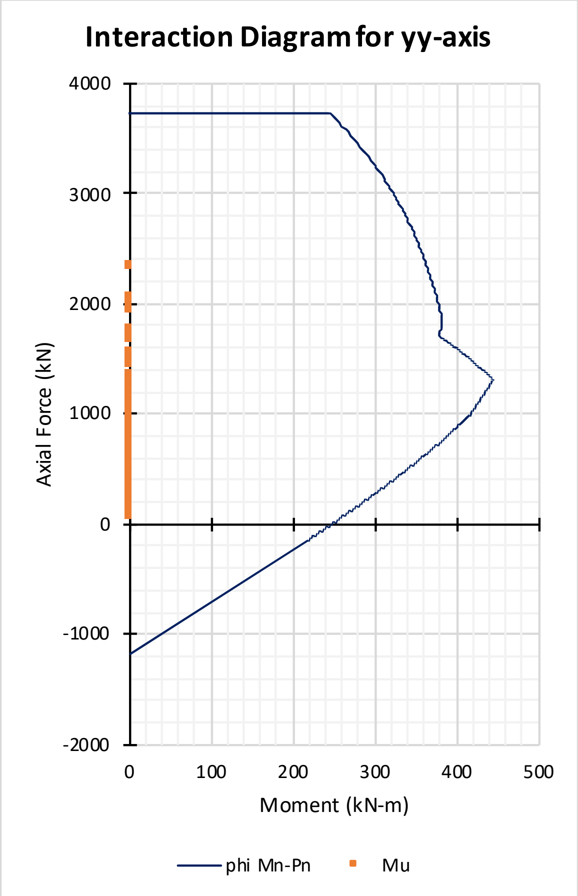
xx-axis			yy-axis		
Plastic hinge region		Other	Plastic hinge region		Other
ϕVn=	985 kN	623 kN	Vs=	862 kN	500 kN
Vu/ϕVn=	0.34	0.54	Vu/ϕVn=	0.38	0.66

Check
OK!

Strong column-weak beam check [ACI 18.7.3.2]

Joint	xx-axis				yy-axis			
	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check
Top	840	686	1.22	OK!	882	686	1.29	OK!
Bottom	840	686	1.22	OK!	882	686	1.29	OK!





Important notes:

-Combinations used for earthquake: 1.2D+1.0L+/-1.0E and 0.9D+/-1.0E **If different change Cap sheets accordingly

-Only symmetrical reinforcement is supported in the same axis.

-Lap splices outside lo region, the zone with lap-splice should comply with confinement reinforcement as in hinge region.

-Special provisions apply if column supports reactions from discontinued stiff members. [ACI 18.7.5.6]

-When using mechanical or welded splices check [ACI 18.7.4.3].

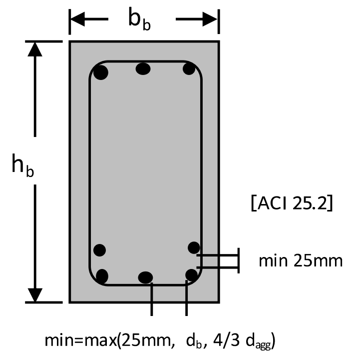
-Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees.

-No unsupported bar shall be farther than 150mm clear on each side along the tie form a laterally supported bar.

-If concrete cover exceeds 100mm, additional transverse reinforcement having cover not exceeding 100mm and spacing not exceeding 300mm shall be provided [ACI 18.7.5.7]

-If f'c exceeds 70MPa, special provisions apply [ACI 22.5.3.1]

Identification		Section Properties			Material Properties	
ID =	9-DB	h_b=	600 mm	OK!	f'_c=	28 MPa
Storey =	1,2,3,4	b_b=	250 mm	OK!	E_c=	24870 MPa
Grid, x=	1 - 5	L_b=	5500 mm		f_{yl}=	420 MPa
Grid, y=	NA	cv=	40 mm		f_{yt}=	420 MPa
		A_g=	150000 mm2		E_s=	200 GPa



0

Other input parameters	
λ=	1
min d 0	22.2 mm
min $d_{b,END2}$=	22.2 mm
ω_u=	58 kN/m

Proposed Shear Reinforcement					
END 1 (Plastic hinge 1)		CENTER (Elastic)		END 1 (Plastic hinge 1)	
ϕ =	9.5 mm	ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	2	No. legs=	2	No. legs=	2
s=	75 mm	s=	125 mm	s=	75 mm

Important notes:

-If using compression reinforcement, check requirements at the end of the spreadsheet.

-Check for requirements of max. spacing of longitudinal bars at the end of the spreadsheet.

-Lap splice locations must follow the requirements mentioned at the end of the spreadsheet.

-Check for necessary development lenghts at the end of the spreadsheet.

FORCES ACTING IN EACH SECTION								
END 1 =	1100 mm		CENTER =	3300 mm		END 2 =	1100 mm	
M_u^+=	110.00 kN-m		M_u^+=	123.00 kN-m		M_u^+=	110.00 kN-m	
M_u^-=	345.00 kN-m		M_u^-=	140.00 kN-m		M_u^-=	345.00 kN-m	
V_u=	221.00 kN	OK!	V_u=	170.00 kN	OK!	V_u=	221.00 kN	OK!
P_u=	0.00 kN	OK!	P_u=	0.00 kN	OK!	P_u=	0.00 kN	OK!
T_u=	0.00 kN-m	OK!	T_u=	0.00 kN	OK!	T_u=	0.00 kN	OK!

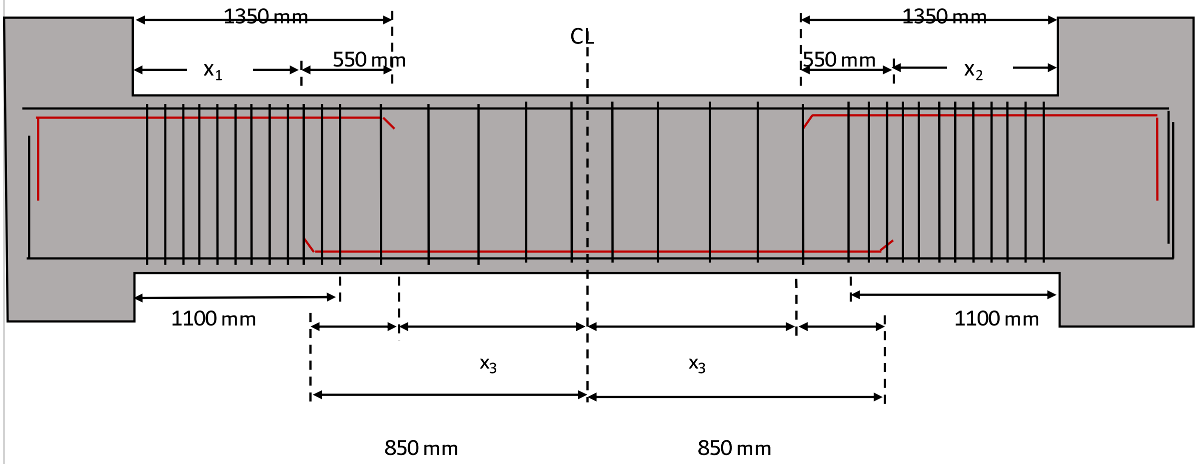
Flexural Reinforcement:					Flexural Reinforcement:					Flexural Reinforcement:				
no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)
4	22	0	0	50	2	22	0	0	50	4	22	0	0	50
2	9.5	0	0	300	2	19	0	0	550	2	9.5	0	0	300
2	22	0	0	550	0	0	0	0	0	2	22	0	0	550
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

MOMENT DESIGN								
END 1			CENTER			END 2		
Press for equilibrium						OK! All sections in equilibrium		
ϕ^+ =	0.9		ϕ^+ =	0.9		ϕ^+ =	0.9	
ϕ^- =	0.9		ϕ^- =	0.9		ϕ^- =	0.9	
ϕM_n^+ =	185 kN-m	OK!	ϕM_n^+ =	128 kN-m	OK!	ϕM_n^+ =	185 kN-m	OK!
ϕM_n^- =	344 kN-m	OK!	ϕM_n^- =	170 kN-m	OK!	ϕM_n^- =	344 kN-m	OK!
$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2	

END 1			CENTER			END 2		
$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2	
$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2	
A_s^+=	916 mm2	OK!	A_s^+=	1347 mm2	OK!	A_s^+=	916 mm2	OK!
A_s^-=	1690 mm2	OK!	A_s^-=	774 mm2	OK!	A_s^-=	1690 mm2	OK!
$M_u^+/\phi M_n^+$ =	0.59	OK!	$M_u^+/\phi M_n^+$ =	0.96	OK!	$M_u^+/\phi M_n^+$ =	0.59	OK!
$M_u^-/\phi M_n^-$ =	1.00	OK!	$M_u^-/\phi M_n^-$ =	0.82	OK!	$M_u^-/\phi M_n^-$ =	1.00	OK!

SHEAR DESIGN								
END 1			CENTER			END 2		
ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2
A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!
s=	75 mm	OK!	s=	125 mm	OK!	s=	75 mm	OK!
s_{max}=	133 mm		s_{max}=	138 mm		s_{max}=	133 mm	
λ=	1		λ=	1		λ=	1	
V_c=	0 kN		V_c=	130 kN		V_c=	0 kN	
V_s=	437 kN		V_s=	262 kN		V_s=	437 kN	
ϕV_n=	327 kN		ϕV_n=	294 kN		ϕV_n=	327 kN	
ω_u=	58 kN/m		ω_u=	58 kN/m		ω_u=	58 kN/m	
$V_{e,max}$=	315 kN		$V_{e,max}$=	250 kN		$V_{e,max}$=	315 kN	
$V_{e,min}$=	-7 kN		$V_{e,min}$=	57 kN		$V_{e,min}$=	-7 kN	
V_{dis}=	315 kN		V_{dis}=	250 kN		V_{dis}=	315 kN	
$V_{dis}/\phi V_n$=	0.96	OK!	$V_{dis}/\phi V_n$=	0.85	OK!	$V_{dis}/\phi V_n$=	0.96	OK!

DEVELOPMENT LENGHTS AND SPLICES					
END 1		CENTER		END 2	
1. Additional reinforcement		**1. Additional reinforcement**		**1. Additional reinforcement**	
d_b=	22.2 mm	d_b=	19.1 mm	d_b=	22.2 mm
x_1=	800 mm	x_3=	300 mm	x_2=	800 mm
λ=	1	λ=	1	λ=	1
ψ_e=	1	ψ_e=	1	ψ_e=	1
ψ_s=	1	ψ_s=	0.8	ψ_s=	1
ψ_t=	1.3	ψ_t=	1	ψ_t=	1.3
A_{tr}=	142 mm2	A_{tr}=	142 mm2	A_{tr}=	142 mm2
c_b=	50 mm	c_b=	50 mm	c_b=	50 mm
s=	75 mm	s=	125 mm	s=	75 mm
n=	2	n=	1	n=	2
k_{tr}=	38	k_{tr}=	45	k_{tr}=	38
$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5
ξ=	3.25	ξ=	2.5	ξ=	3.25
l_d=	1061 mm	l_d=	702 mm	l_d=	1061 mm
l_{bast}=	1350 mm	l_{bast}=	850 mm	l_{bast}=	1350 mm
2. Development length into column		**2. Development length into column**		**2. Development length into column**	
max d_b=	22.2 mm	max d_b=	22.2 mm	max d_b=	22.2 mm
l_{dh}=	326 mm	l_{dh}=	326 mm	l_{dh}=	326 mm



MAXIMUN SPACING OF REINFORCEMENT IN TENSION FACE AND SKIN REINFORCEMENT			
Tension Face		Skin reinforcement	
c_c=	39 mm	c_c=	40 mm
f_s=	280 MPa	f_s=	280 MPa
max spacing=	283 mm	max spacing=	280 mm

LATERAL SUPPORT OF COMPRESSION REINFORCEMENT

Notes for support of compression reinforcement (this applies only when compression is taken into account):

1. Minimun size:

No.10 if longitudinal bars are No32 or less.

No.13 if longitudinal bars are No36 or more, and for bundles.

2. Maximun spacing:

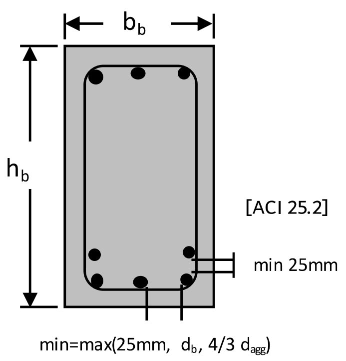
$d_{b,long}$= 19.1 mm

$d_{b,trans}$= 9.5 mm

s_{max}= 250 mm

3. Longitudinal compression reinforcement shall be arranged in such a manner that every corner and alternate bar be enclosed by the corner of the transverse reinforcement with an included angle of 135 degrees or less. No bar shall be farther than 150mm clear on each side along the transverse reinforcement from such an enclosed bar.

Identification		Section Properties			Material Properties	
ID =	9-DB	h_b=	600 mm	OK!	f'_c=	28 MPa
Storey =	5,6	b_b=	250 mm	OK!	E_c=	24870 MPa
Grid, x=	1 - 5	L_b=	5500 mm		f_{yl}=	420 MPa
Grid, y=	NA	cv=	40 mm		f_{yt}=	420 MPa
		A_g=	150000 mm2		E_s=	200 GPa



0

Other input parameters	
λ=	1
min d 0	22.2 mm
min $d_{b,END2}$=	22.2 mm
ω_u=	58 kN/m

Proposed Shear Reinforcement					
END 1 (Plastic hinge 1)		CENTER (Elastic)		END 1 (Plastic hinge 1)	
ϕ =	9.5 mm	ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	2	No. legs=	2	No. legs=	2
s=	75 mm	s=	125 mm	s=	75 mm

Important notes:

-If using compression reinforcement, check requirements at the end of the spreadsheet.

-Check for requirements of max. spacing of longitudinal bars at the end of the spreadsheet.

-Lap splice locations must follow the requirements mentioned at the end of the spreadsheet.

-Check for necessary development lenghts at the end of the spreadsheet.

FORCES ACTING IN EACH SECTION														
END 1 =			1100 mm		CENTER =			3300 mm		END 2 =			1100 mm	
M_u^+=	80.00 kN-m				M_u^+=	104.00 kN-m				M_u^+=	80.00 kN-m			
M_u^-=	310.00 kN-m				M_u^-=	115.00 kN-m				M_u^-=	310.00 kN-m			
V_u=	216.00 kN			OK!	V_u=	160.00 kN			OK!	V_u=	216.00 kN			OK!
P_u=	0.00 kN			OK!	P_u=	0.00 kN			OK!	P_u=	0.00 kN			OK!
T_u=	0.00 kN-m			OK!	T_u=	0.00 kN			OK!	T_u=	0.00 kN			OK!
Flexural Reinforcement:					Flexural Reinforcement:					Flexural Reinforcement:				
no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)
2	19	2	22	50	2	19	0	0	50	2	19	2	22	50
2	9.5	0	0	200	2	22	0	0	550	2	9.5	0	0	200
2	22	0	0	550	0	0	0	0	0	2	22	0	0	550
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

MOMENT DESIGN

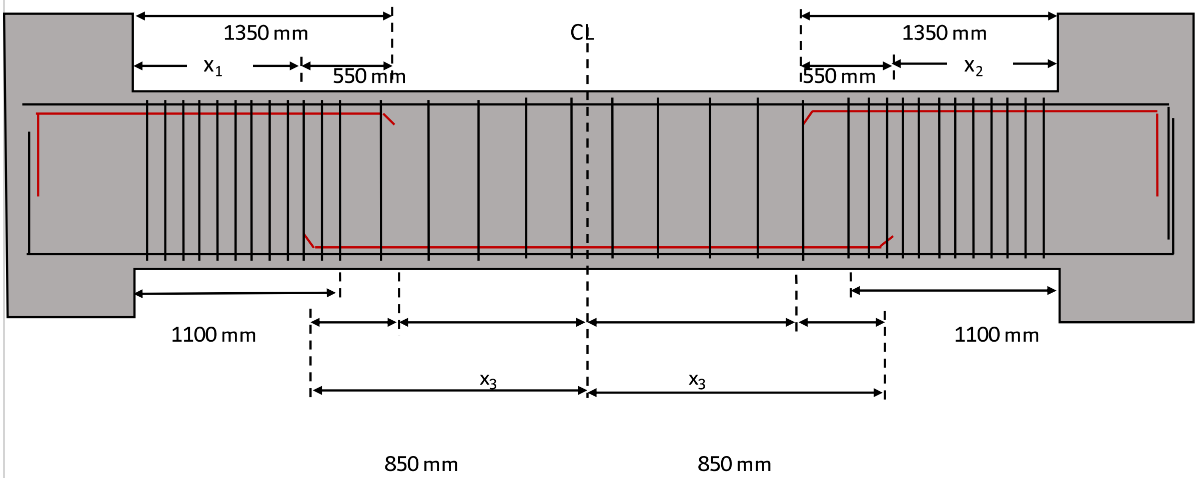
END 1			CENTER			END 2		
Press for equilibrium						OK! All sections in equilibrium		
ϕ^+ =	0.9		ϕ^+ =	0.9		ϕ^+ =	0.9	
ϕ^- =	0.9		ϕ^- =	0.9		ϕ^- =	0.9	
ϕM_n^+ =	179 kN-m	OK!	ϕM_n^+ =	170 kN-m	OK!	ϕM_n^+ =	179 kN-m	OK!
ϕM_n^- =	309 kN-m	OK!	ϕM_n^- =	128 kN-m	OK!	ϕM_n^- =	309 kN-m	OK!
$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2	

END 1			CENTER			END 2		
$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2	
$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2	
A_s^+=	916 mm2	OK!	A_s^+=	774 mm2	OK!	A_s^+=	916 mm2	OK!
A_s^-=	1489 mm2	OK!	A_s^-=	1347 mm2	OK!	A_s^-=	1489 mm2	OK!
$M_u^+/\phi M_n^+$ =	0.45	OK!	$M_u^+/\phi M_n^+$ =	0.61	OK!	$M_u^+/\phi M_n^+$ =	0.45	OK!
$M_u^-/\phi M_n^-$ =	1.00	OK!	$M_u^-/\phi M_n^-$ =	0.90	OK!	$M_u^-/\phi M_n^-$ =	1.00	OK!

SHEAR DESIGN

END 1			CENTER			END 2		
ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2
A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!
s=	75 mm	OK!	s=	125 mm	OK!	s=	75 mm	OK!
s_{max}=	133 mm		s_{max}=	138 mm		s_{max}=	133 mm	
λ=	1		λ=	1		λ=	1	
V_c=	0 kN		V_c=	139 kN		V_c=	0 kN	
V_s=	437 kN		V_s=	262 kN		V_s=	437 kN	
ϕV_n=	327 kN		ϕV_n=	301 kN		ϕV_n=	327 kN	
ω_u=	58 kN/m		ω_u=	58 kN/m		ω_u=	58 kN/m	
$V_{e,max}$=	300 kN		$V_{e,max}$=	235 kN		$V_{e,max}$=	300 kN	
$V_{e,min}$=	-22 kN		$V_{e,min}$=	42 kN		$V_{e,min}$=	-22 kN	
V_{dis}=	300 kN		V_{dis}=	235 kN		V_{dis}=	300 kN	
$V_{dis}/\phi V_n$=	0.91	OK!	$V_{dis}/\phi V_n$=	0.78	OK!	$V_{dis}/\phi V_n$=	0.91	OK!

DEVELOPMENT LENGHTS AND SPLICES					
END 1		CENTER		END 2	
1. Additional reinforcement		**1. Additional reinforcement**		**1. Additional reinforcement**	
d_b=	22.2 mm	d_b=	19.1 mm	d_b=	22.2 mm
x_1=	800 mm	x_3=	300 mm	x_2=	800 mm
λ=	1	λ=	1	λ=	1
ψ_e=	1	ψ_e=	1	ψ_e=	1
ψ_s=	1	ψ_s=	0.8	ψ_s=	1
ψ_t=	1.3	ψ_t=	1	ψ_t=	1.3
A_{tr}=	142 mm2	A_{tr}=	142 mm2	A_{tr}=	142 mm2
c_b=	50 mm	c_b=	50 mm	c_b=	50 mm
s=	75 mm	s=	125 mm	s=	75 mm
n=	2	n=	1	n=	2
k_{tr}=	38	k_{tr}=	45	k_{tr}=	38
$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5
ξ=	3.25	ξ=	2.5	ξ=	3.25
l_d=	1061 mm	l_d=	702 mm	l_d=	1061 mm
l_{bast}=	1350 mm	l_{bast}=	850 mm	l_{bast}=	1350 mm
2. Development length into column		**2. Development length into column**		**2. Development length into column**	
max d_b=	22.2 mm	max d_b=	22.2 mm	max d_b=	22.2 mm
l_{dh}=	326 mm	l_{dh}=	326 mm	l_{dh}=	326 mm



MAXIMUN SPACING OF REINFORCEMENT IN TENSION FACE AND SKIN REINFORCEMENT				
Tension Face		Skin reinforcement		
c_c=	40 mm	c_c=	40 mm	
f_s=	280 MPa	f_s=	280 MPa	
max spacing=	279 mm	max spacing=	280 mm	

LATERAL SUPPORT OF COMPRESSION REINFORCEMENT

Notes for support of compression reinforcement (this applies only when compression is taken into account):

1. Minimun size:

No.10 if longitudinal bars are No32 or less.

No.13 if longitudinal bars are No36 or more, and for bundles.

2. Maximun spacing:

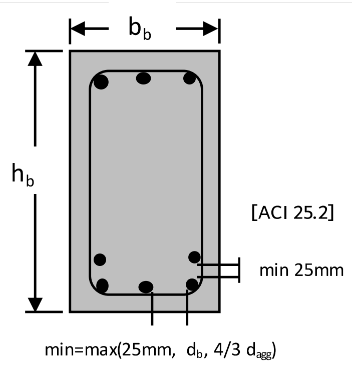
$d_{b,long}$= 19.1 mm

$d_{b,trans}$= 9.5 mm

s_{max}= 250 mm

3. Longitudinal compression reinforcement shall be arranged in such a manner that every corner and alternate bar be enclosed by the corner of the transverse reinforcement with an included angle of 135 degrees or less. No bar shall be farther than 150mm clear on each side along the transverse reinforcement from such an enclosed bar.

Identification		Section Properties			Material Properties	
ID =	9-DB	h_b=	600 mm	OK!	f'_c=	28 MPa
Storey =	7,8,9	b_b=	250 mm	OK!	E_c=	24870 MPa
Grid, x=	1 - 5	L_b=	5500 mm		f_{yl}=	420 MPa
Grid, y=	NA	cv=	40 mm		f_{yt}=	420 MPa
		A_g=	150000 mm2		E_s=	200 GPa



0

Other input parameters	
λ=	1
min d 0	22.2 mm
min $d_{b,END2}$=	22.2 mm
ω_u=	58 kN/m

Proposed Shear Reinforcement					
END 1 (Plastic hinge 1)		CENTER (Elastic)		END 1 (Plastic hinge 1)	
ϕ =	9.5 mm	ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	2	No. legs=	2	No. legs=	2
s=	75 mm	s=	125 mm	s=	75 mm

Important notes:

-If using compression reinforcement, check requirements at the end of the spreadsheet.

-Check for requirements of max. spacing of longitudinal bars at the end of the spreadsheet.

-Lap splice locations must follow the requirements mentioned at the end of the spreadsheet.

-Check for necessary development lenghts at the end of the spreadsheet.

FORCES ACTING IN EACH SECTION								
END 1 =	1100 mm		CENTER =	3300 mm		END 2 =	1100 mm	
M_u^+=	40.00 kN-m		M_u^+=	104.00 kN-m		M_u^+=	40.00 kN-m	
M_u^-=	254.00 kN-m		M_u^-=	73.00 kN-m		M_u^-=	254.00 kN-m	
V_u=	216.00 kN	OK!	V_u=	160.00 kN	OK!	V_u=	216.00 kN	OK!
P_u=	0.00 kN	OK!	P_u=	0.00 kN	OK!	P_u=	0.00 kN	OK!
T_u=	0.00 kN-m	OK!	T_u=	0.00 kN	OK!	T_u=	0.00 kN	OK!

Flexural Reinforcement:					Flexural Reinforcement:					Flexural Reinforcement:				
no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)
3	22	0	0	50	2	22	0	0	50	3	22	0	0	50
2	9.5	0	0	300	2	19	0	0	550	2	9.5	0	0	300
2	19	0	0	550	0	0	0	0	0	2	19	0	0	550
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

MOMENT DESIGN

Press for equilibrium

OK! All sections in equilibrium

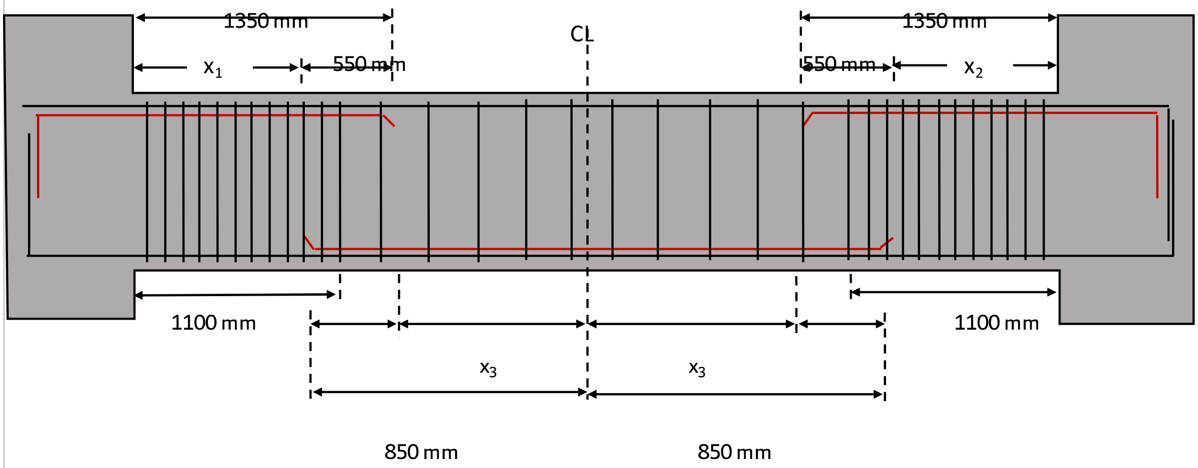
END 1			CENTER			END 2		
ϕ^{+} =	0.9		ϕ^{+} =	0.9		ϕ^{+} =	0.9	
ϕ^{-} =	0.9		ϕ^{-} =	0.9		ϕ^{-} =	0.9	
ϕM_n^{+} =	143 kN-m	OK!	ϕM_n^{+} =	128 kN-m	OK!	ϕM_n^{+} =	143 kN-m	OK!
ϕM_n^{-} =	265 kN-m	OK!	ϕM_n^{-} =	170 kN-m	OK!	ϕM_n^{-} =	265 kN-m	OK!
$A_s^{+}{}_{,min}$=	458 mm2		$A_s^{+}{}_{,min}$=	458 mm2		$A_s^{+}{}_{,min}$=	458 mm2	

END 1			CENTER			END 2		
$A_s^{-}{}_{,min}$=	458 mm2		$A_s^{-}{}_{,min}$=	458 mm2		$A_s^{-}{}_{,min}$=	458 mm2	
$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2	
A_s^{+}=	715 mm2	OK!	A_s^{+}=	1347 mm2	OK!	A_s^{+}=	715 mm2	OK!
A_s^{-}=	1303 mm2	OK!	A_s^{-}=	774 mm2	OK!	A_s^{-}=	1303 mm2	OK!
$M_u^{+}/\phi M_n^{+}$ =	0.28	OK!	$M_u^{+}/\phi M_n^{+}$ =	0.82	OK!	$M_u^{+}/\phi M_n^{+}$ =	0.28	OK!
$M_u^{-}/\phi M_n^{-}$ =	0.96	OK!	$M_u^{-}/\phi M_n^{-}$ =	0.43	OK!	$M_u^{-}/\phi M_n^{-}$ =	0.96	OK!

SHEAR DESIGN

END 1			CENTER			END 2		
ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2
A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!
s=	75 mm	OK!	s=	125 mm	OK!	s=	75 mm	OK!
s_{max}=	133 mm		s_{max}=	138 mm		s_{max}=	133 mm	
λ=	1		λ=	1		λ=	1	
V_c=	0 kN		V_c=	130 kN		V_c=	0 kN	
V_s=	437 kN		V_s=	262 kN		V_s=	437 kN	
ϕV_n=	327 kN		ϕV_n=	294 kN		ϕV_n=	327 kN	
ω_u=	58 kN/m		ω_u=	58 kN/m		ω_u=	58 kN/m	
$V_{e,max}$=	280 kN		$V_{e,max}$=	215 kN		$V_{e,max}$=	280 kN	
$V_{e,min}$=	-42 kN		$V_{e,min}$=	22 kN		$V_{e,min}$=	-42 kN	
V_{dis}=	280 kN		V_{dis}=	215 kN		V_{dis}=	280 kN	
$V_{dis}/\phi V_n$=	0.85	OK!	$V_{dis}/\phi V_n$=	0.73	OK!	$V_{dis}/\phi V_n$=	0.85	OK!

DEVELOPMENT LENGHTS AND SPLICES					
END 1		CENTER		END 2	
1. Additional reinforcement		**1. Additional reinforcement**		**1. Additional reinforcement**	
d_b=	22.2 mm	d_b=	19.1 mm	d_b=	22.2 mm
x_1=	800 mm	x_3=	300 mm	x_2=	800 mm
λ=	1	λ=	1	λ=	1
ψ_e=	1	ψ_e=	1	ψ_e=	1
ψ_s=	1	ψ_s=	0.8	ψ_s=	1
ψ_t=	1.3	ψ_t=	1	ψ_t=	1.3
A_{tr}=	142 mm2	A_{tr}=	142 mm2	A_{tr}=	142 mm2
c_b=	50 mm	c_b=	50 mm	c_b=	50 mm
s=	75 mm	s=	125 mm	s=	75 mm
n=	2	n=	1	n=	2
k_{tr}=	38	k_{tr}=	45	k_{tr}=	38
$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5
ξ=	3.25	ξ=	2.5	ξ=	3.25
l_d=	1061 mm	l_d=	702 mm	l_d=	1061 mm
l_{bast}=	1350 mm	l_{bast}=	850 mm	l_{bast}=	1350 mm
2. Development length into column		**2. Development length into column**		**2. Development length into column**	
max d_b=	22.2 mm	max d_b=	22.2 mm	max d_b=	22.2 mm
l_{dh}=	326 mm	l_{dh}=	326 mm	l_{dh}=	326 mm



MAXIMUN SPACING OF REINFORCEMENT IN TENSION FACE AND SKIN REINFORCEMENT				
Tension Face		Skin reinforcement		
c_c=	39 mm	c_c=	40 mm	
f_s=	280 MPa	f_s=	280 MPa	
max spacing=	283 mm	max spacing=	280 mm	

LATERAL SUPPORT OF COMPRESSION REINFORCEMENT

Notes for support of compression reinforcement (this applies only when compression is taken into account):

1. Minimun size:

    No.10 if longitudinal bars are No32 or less.

    No.13 if longitudinal bars are No36 or more, and for bundles.

2. Maximun spacing:

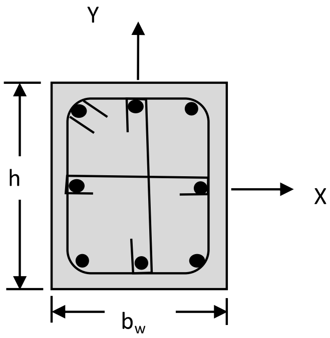
    $d_{b,long}$= 19.1 mm

    $d_{b,trans}$= 9.5 mm

    s_{max}= 250 mm

3. Longitudinal compression reinforcement shall be arranged in such a manner that every corner and alternate bar be enclosed by the corner of the transverse reinforcement with an included angle of 135 degrees or less. No bar shall be farther than 150mm clear on each side along the transverse reinforcement from such an enclosed bar.

Identification		Material Prop.		Section Properties	
ID =	9-DB	f'_c=	28 MPa	h=	500 mm
Storey =	1,2,3,4	E_c=	24870 MPa	b_w=	500 mm
Grid, x=	1-5	f_{yl}=	420 MPa	l_w=	3000 mm
Grid, y=	NA	f_{yt}=	420 MPa	c_c=	40 mm
		E_s=	200 GPa	A_g=	250000 mm2
		λ=	1	I_{xx}=	5E+09 mm4
				I_{yy}=	5E+09 mm4
				d_{xx}=	450.00 mm
				d_{yy}=	450.00 mm
				h_x=	197 mm
				ϕ_{min}=	22 mm
				n_l=	8
				A_{ch}=	176400 mm2
				d_{agg}=	25 mm



Proposed Shear Reinforcement			
XX-Axis		YY-Axis	
Plastic hinge region		Plastic hinge region	
ϕ =	12.7 mm	ϕ =	12.7 mm
No. legs=	3	No. legs=	3
s=	65 mm	s=	65 mm
Ash=	380 mm2	Ash=	380 mm2
Other regions		Other regions	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	125 mm	s=	125 mm
Ash=	213 mm2	Ash=	213 mm2
Pu and Mu corresponding to Vu in x and y			
Pu=	3051 kN	Pu=	0 kN
Mu=	299 kN	Mu=	0 kN

Axial Force		Shear Force in x		Shear Force in y		Axial Force from EQ	
P_D=	1894 kN	V_{Dx}=	20 kN	V_{Dy}=	0 kN	P^+_{Ex}=	350 kN
P_L=	778 kN	V_{Lx}=	9 kN	V_{Ly}=	0 kN	P^-_{Ex}=	-350 kN
P_u=	3051 kN	V_{Ex}=	134 kN	V_{Ey}=	0 kN	P^+_{Ey}=	0 kN
		V_{ux}=	167 kN	V_{uy}=	0 kN	P^-_{Ey}=	0 kN

In equilibrium!

PRESS BUTTON FOR EQUILIBRIUM

Flexural Reinforcement xx:						Flexural Reinforcement yy:					
no.	ϕ	no.	ϕ	d (mm)	Ast	no.	ϕ	no.	ϕ	d (mm)	Ast
2	22	1	22	50	###	2	22	1	22	50	###
2	22	0	0	250	774	2	22	0	0	250	774
2	22	1	22	450	###	2	22	1	22	450	###
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Moment capacity of columns and beams		
Top joint	xx-axis	yy-axis
Mprc1=	520	548
Mprc2=	520	548
Mprb1=	423	423
Mprb2=	423	423
Bot joint	xx-axis	yy-axis
Mprc1=	520	548
Mprc2=	520	548
Mprb1=	423	390
Mprb2=	423	423

Dimension and longitudinal reinforcement limits

$b_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
$h_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
b_w/h_w or h_w/b_w=		1.00	>=	0.4	OK!	[ACI 18.7.2.1 (b)]
A_{st}=	3097 mm2				OK!	[ACI 18.7.4.1]

Transverse reinforcement in plastic hinge region

l_o=	500 mm		[ACI 18.7.5.1]
$h_{x,max}$=	200 mm	OK!	[ACI 18.7.5.2, e and f]
s_{max}=	125 mm	OK!	[ACI 18.7.5.3]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
$A_{sh}/sb_{c,x}$=	1.17%	OK!	[ACI 18.7.5.4]
$A_{sh}/sb_{c,y}$=	1.17%	OK!	[ACI 18.7.5.4]

Transverse reinforcement in other regions

s_{max}=	133 mm	OK!	[ACI 18.7.5.5]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
ϕ_{min}=	9.5 mm	OK!	[ACI 25.7.2.2]

Shear force check

Vux max=	1141 kN	OK!	[ACI 22.5.1.2]
Vuy max=	848 kN	OK!	[ACI 22.5.1.2]

Bi-axial moment capacity

$$\left(\frac{Mu,xx}{Mn,xx}\right)^2 + \left(\frac{Mu,yy}{Mn,yy}\right)^2 = \quad 0.968665 \quad \text{OK!}$$

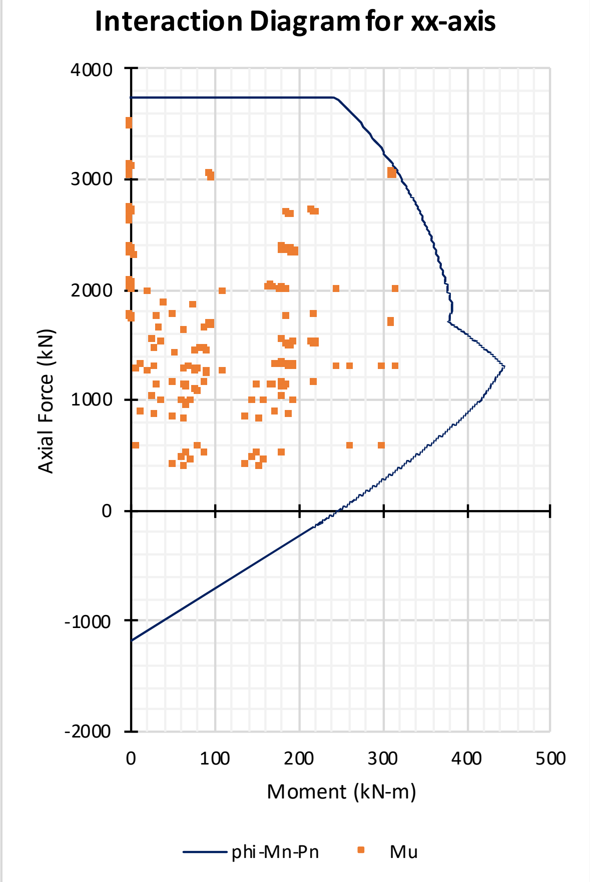
Shear capacity

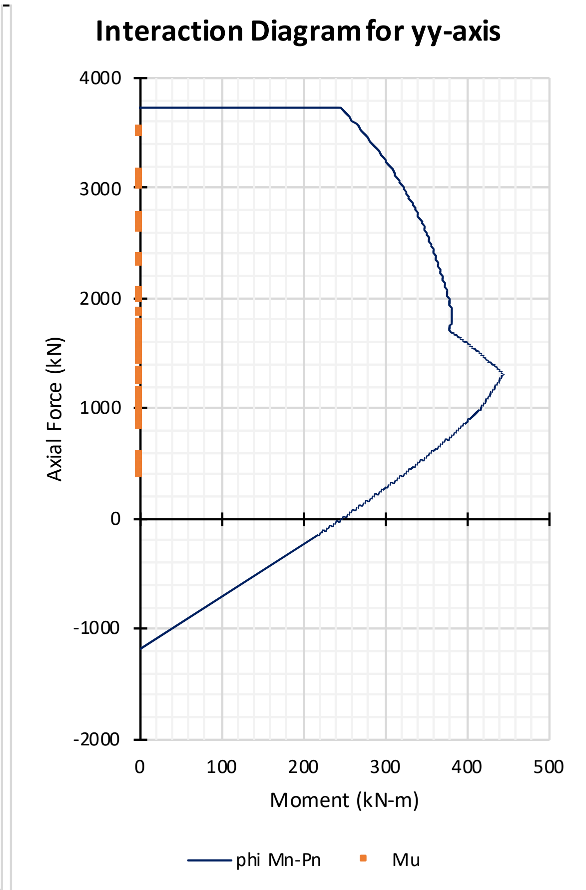
xx-axis			yy-axis		
	Plastic hinge region	Other		Plastic hinge region	Other
ϕVn=	1380 kN	793 kN	Vs=	1088 kN	500 kN
Vu/ϕVn=	0.28	0.48	Vu/ϕVn=	0.34	0.75

Check
OK!

Strong column-weak beam check [ACI 18.7.3.2]

Joint	xx-axis				yy-axis			
	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check
Top	1040	846	1.23	OK!	1096	846	1.30	OK!
Bottom	1040	846	1.23	OK!	1096	813	1.35	OK!





Important notes:

-Combinations used for earthquake: 1.2D+1.0L+/-1.0E and 0.9D+/-1.0E **If different change Cap sheets accordingly

-Only symmetrical reinforcement is supported in the same axis.

-Lap splices outside lo region, the zone with lap-splice should comply with confinement reinforcement as in hinge region.

-Special provisions apply if column supports reactions from discontinued stiff members. [ACI 18.7.5.6]

-When using mechanical or welded splices check [ACI 18.7.4.3].

-Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees.

-No unsupported bar shall be farther than 150mm clear on each side along the tie form a laterally supported bar.

-If concrete cover exceeds 100mm, additional transverse reinforcement having cover not exceeding 100mm and spacing not exceeding 300mm shall be provided [ACI 18.7.5.7]

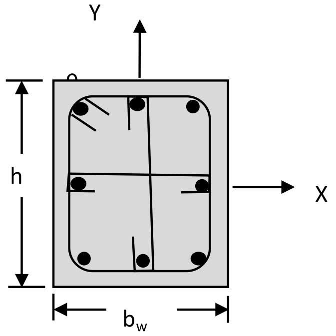
-If f'c exceeds 70MPa, special provisions apply [ACI 22.5.3.1]

Identification	
ID =	9-DB
Storey =	5,6
Grid, x=	1-5
Grid, y=	NA
0	

Material Prop.	
f'_c=	28 MPa
E_c=	24870 MPa
f_{yl}=	420 MPa
f_{yt}=	420 MPa
E_s=	200 GPa
λ=	1

Section Properties	
h=	500 mm
b_w=	500 mm
l_w=	3000 mm
c_c=	40 mm
A_g=	250000 mm2
I_{xx}=	5E+09 mm4
I_{yy}=	5E+09 mm4
d_{xx}=	450.00 mm
d_{yy}=	450.00 mm
h_x=	197 mm
ϕ_{min}=	22 mm
n_l=	8
A_{ch}=	176400 mm2
d_{agg}=	25 mm

Proposed Shear Reinforcement			
XX-Axis		YY-Axis	
Plastic hinge region		Plastic hinge region	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	50 mm	s=	50 mm
Ash=	213 mm2	Ash=	213 mm2
Other regions		Other regions	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	125 mm	s=	125 mm
Ash=	213 mm2	Ash=	213 mm2
Pu and Mu corresponding to Vu in x and y			
Pu=	1696 kN	Pu=	0 kN
Mu=	184 kN	Mu=	0 kN



Axial Force		Shear Force in x		Shear Force in y		Axial Force from EQ	
P_D=	1051 kN	V_{Dx}=	33 kN	V_{Dy}=	0 kN	P^+_{Ex}=	170 kN
P_L=	432 kN	V_{Lx}=	15 kN	V_{Ly}=	0 kN	P^-_{Ex}=	-133 kN
P_u=	1696 kN	V_{Ex}=	107 kN	V_{Ey}=	0 kN	P^+_{Ey}=	0 kN
		V_{ux}=	162 kN	V_{uy}=	0 kN	P^-_{Ey}=	0 kN

In equilibrium!

PRESS BUTTON FOR EQUILIBRIUM

Flexural Reinforcement xx:						Flexural Reinforcement yy:					
no.	ϕ	no.	ϕ	d (mm)	Ast	no.	ϕ	no.	ϕ	d (mm)	Ast
2	22	1	22	50	###	2	22	1	22	50	###
2	22	0	0	250	774	2	22	0	0	250	774
2	22	1	22	450	###	2	22	1	22	450	###
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Moment capacity of columns and beams		
Top joint	xx-axis	yy-axis
Mprc1=	471	484
Mprc2=	471	484
Mprb1=	382	382
Mprb2=	382	382
Bot joint	xx-axis	yy-axis
Mprc1=	471	484
Mprc2=	471	484
Mprb1=	382	382
Mprb2=	382	382

Dimension and longitudinal reinforcement limits

$b_{w,min}$=	300 mm			OK!	[ACI 18.7.2.1 (a)]
$h_{w,min}$=	300 mm			OK!	[ACI 18.7.2.1 (a)]
b_w/h_w or h_w/b_w=	1.00	>=	0.4	OK!	[ACI 18.7.2.1 (b)]
A_{st}=	3097 mm2			OK!	[ACI 18.7.4.1]

Transverse reinforcement in plastic hinge region

l_o=	500 mm		[ACI 18.7.5.1]
$h_{x,max}$=	350 mm	OK!	[ACI 18.7.5.2, e and f]
s_{max}=	125 mm	OK!	[ACI 18.7.5.3]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
$A_{sh}/sb_{c,x}$=	0.85%	OK!	[ACI 18.7.5.4]
$A_{sh}/sb_{c,y}$=	0.85%	OK!	[ACI 18.7.5.4]

Transverse reinforcement in other regions

s_{max}=	133 mm	OK!	[ACI 18.7.5.5]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
ϕ_{min}=	9.5 mm	OK!	[ACI 25.7.2.2]

Shear force check

Vux max=	1035 kN	OK!	[ACI 22.5.1.2]
Vuy max=	848 kN	OK!	[ACI 22.5.1.2]

Bi-axial moment capacity

$$\left(\frac{Mu,xx}{Mn,xx}\right)^2+\left(\frac{Mu,yy}{Mn,yy}\right)^2=$$

0.478448 OK!

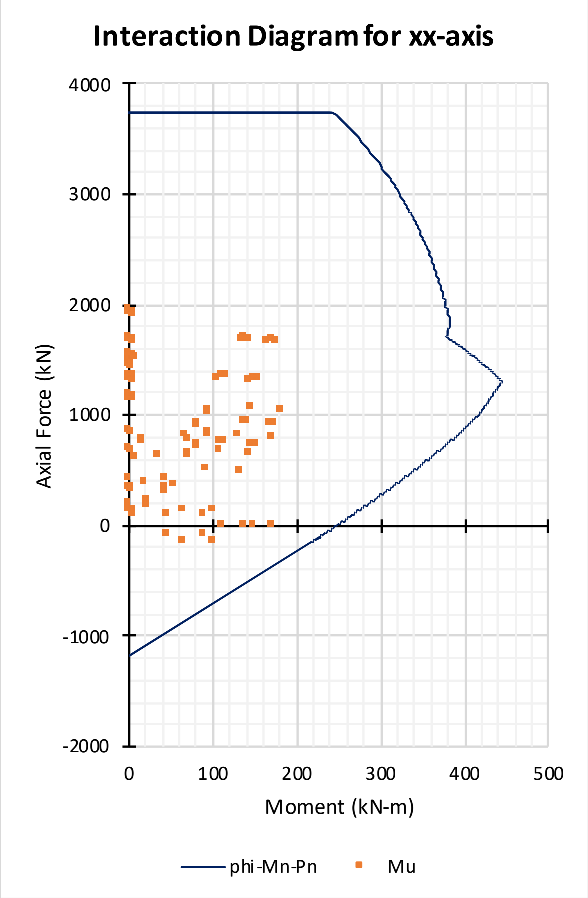
Shear capacity

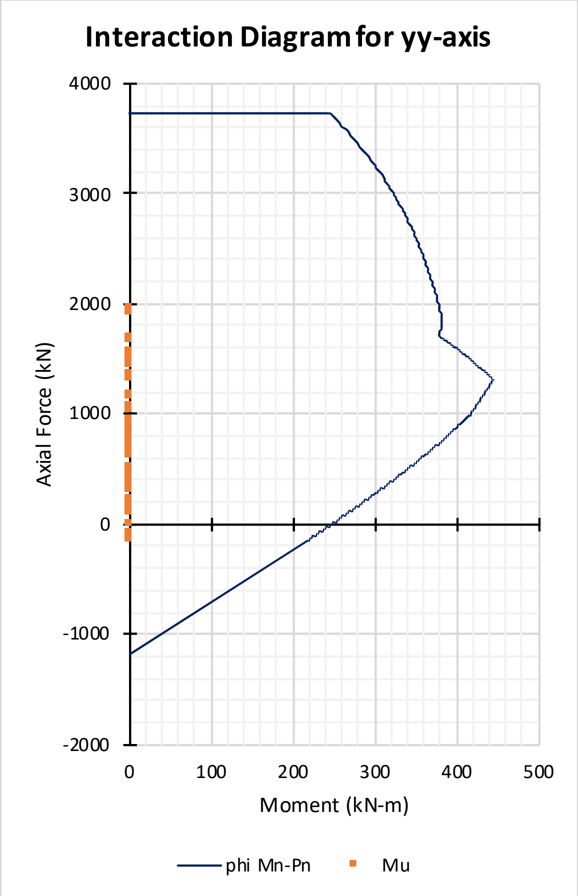
xx-axis			yy-axis		
	Plastic hinge region	Other		Plastic hinge region	Other
ϕVn=	1049 kN	687 kN	Vs=	862 kN	500 kN
Vu/ϕVn=	0.36	0.54	Vu/ϕVn=	0.42	0.73

Check
OK!

Strong column-weak beam check [ACI 18.7.3.2]

Joint	xx-axis				yy-axis			
	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check
Top	942	764	1.23	OK!	968	764	1.27	OK!
Bottom	942	764	1.23	OK!	968	764	1.27	OK!





Important notes:

-Combinations used for earthquake: 1.2D+1.0L+/-1.0E and 0.9D+/-1.0E **If different change Cap sheets accordingly

-Only symmetrical reinforcement is supported in the same axis.

-Lap splices outside lo region, the zone with lap-splice should comply with confinement reinforcement as in hinge region.

-Special provisions apply if column supports reactions from discontinued stiff members. [ACI 18.7.5.6]

-When using mechanical or welded splices check [ACI 18.7.4.3].

-Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees.

-No unsupported bar shall be farther than 150mm clear on each side along the tie form a laterally supported bar.

-If concrete cover exceeds 100mm, additional transverse reinforcement having cover not exceeding 100mm and spacing not exceeding 300mm shall be provided [ACI 18.7.5.7]

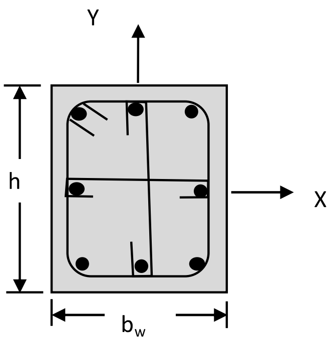
-If f'c exceeds 70MPa, special provisions apply [ACI 22.5.3.1]

Identification	
ID =	9-DB
Storey =	7.8.9
Grid, x=	1-5
Grid, y=	NA
0	

Material Prop.	
f'_c=	28 MPa
E_c=	24870 MPa
f_{yl}=	420 MPa
f_{yt}=	420 MPa
E_s=	200 GPa
λ=	1

Section Properties	
h=	500 mm
b_w=	500 mm
l_w=	3000 mm
c_c=	40 mm
A_g=	250000 mm2
I_{xx}=	5E+09 mm4
I_{yy}=	5E+09 mm4
d_{xx}=	450.00 mm
d_{yy}=	450.00 mm
h_x=	200 mm
ϕ_{min}=	19 mm
n_l=	8
A_{ch}=	176400 mm2
d_{agg}=	25 mm

Proposed Shear Reinforcement			
XX-Axis		YY-Axis	
Plastic hinge region		Plastic hinge region	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	50 mm	s=	50 mm
Ash=	213 mm2	Ash=	213 mm2
Other regions		Other regions	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	114 mm	s=	114 mm
Ash=	213 mm2	Ash=	213 mm2
Pu and Mu corresponding to Vu in x and y			
Pu=	1017 kN	Pu=	0 kN
Mu=	128 kN	Mu=	0 kN



Axial Force		Shear Force in x		Shear Force in y		Axial Force from EQ	
P_D=	627 kN	V_{Dx}=	50 kN	V_{Dy}=	0 kN	P^+_{Ex}=	55 kN
P_L=	259 kN	V_{Lx}=	20 kN	V_{Ly}=	0 kN	P^-_{Ex}=	-55 kN
P_u=	1017 kN	V_{Ex}=	68 kN	V_{Ey}=	0 kN	P^+_{Ey}=	0 kN
		V_{ux}=	148 kN	V_{uy}=	0 kN	P^-_{Ey}=	0 kN

In equilibrium!

PRESS BUTTON FOR EQUILIBRIUM

Flexural Reinforcement xx:						Flexural Reinforcement yy:					
no.	ϕ	no.	ϕ	d (mm)	Ast	no.	ϕ	no.	ϕ	d (mm)	Ast
2	22	1	19	50	###	2	22	1	19	50	###
2	19	0	0	250	573	2	19	0	0	250	573
2	22	1	19	450	###	2	22	1	19	450	###
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Moment capacity of columns and beams		
Top joint	xx-axis	yy-axis
Mprc1=	391	400
Mprc2=	391	400
Mprb1=	325	325
Mprb2=	325	325
Bot joint	xx-axis	yy-axis
Mprc1=	391	400
Mprc2=	391	400
Mprb1=	325	325
Mprb2=	325	325

Dimension and longitudinal reinforcement limits

$b_{w,min}$=	300 mm			OK!	[ACI 18.7.2.1 (a)]
$h_{w,min}$=	300 mm			OK!	[ACI 18.7.2.1 (a)]
b_w/h_w or h_w/b_w=	1.00	>=	0.4	OK!	[ACI 18.7.2.1 (b)]
A_{st}=	2694 mm2			OK!	[ACI 18.7.4.1]

Transverse reinforcement in plastic hinge region

l_o=	500 mm		[ACI 18.7.5.1]
$h_{x,max}$=	350 mm	OK!	[ACI 18.7.5.2, e and f]
s_{max}=	115 mm	OK!	[ACI 18.7.5.3]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
$A_{sh}/sb_{c,x}$=	0.85%	OK!	[ACI 18.7.5.4]
$A_{sh}/sb_{c,y}$=	0.85%	OK!	[ACI 18.7.5.4]

Transverse reinforcement in other regions

s_{max}=	115 mm	OK!	[ACI 18.7.5.5]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
ϕ_{min}=	9.5 mm	OK!	[ACI 25.7.2.2]

Shear force check

Vux max=	972 kN	OK!	[ACI 22.5.1.2]
Vuy max=	848 kN	OK!	[ACI 22.5.1.2]

Bi-axial moment capacity

$$\left(\frac{Mu,xx}{Mn,xx}\right)^2 + \left(\frac{Mu,yy}{Mn,yy}\right)^2 = \quad 0.526007 \quad \text{OK!}$$

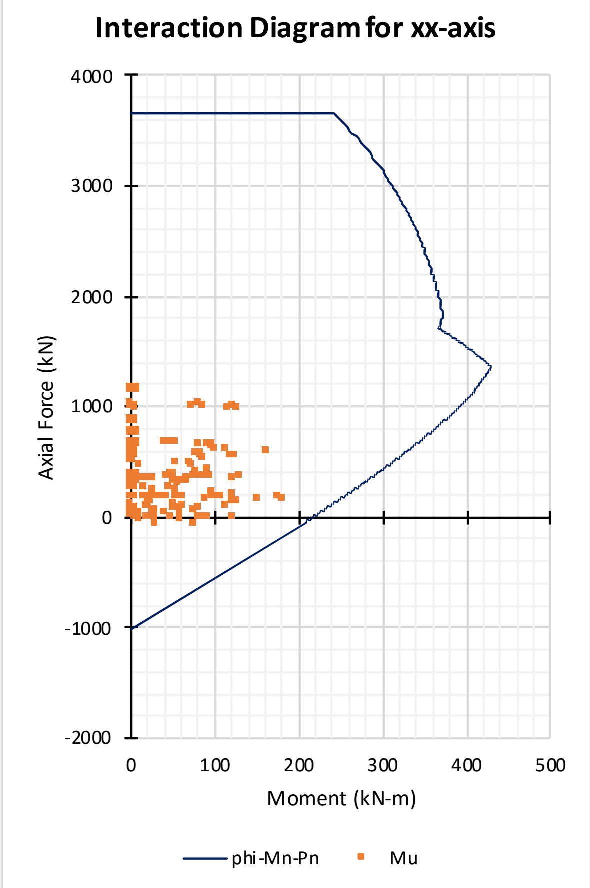
Shear capacity

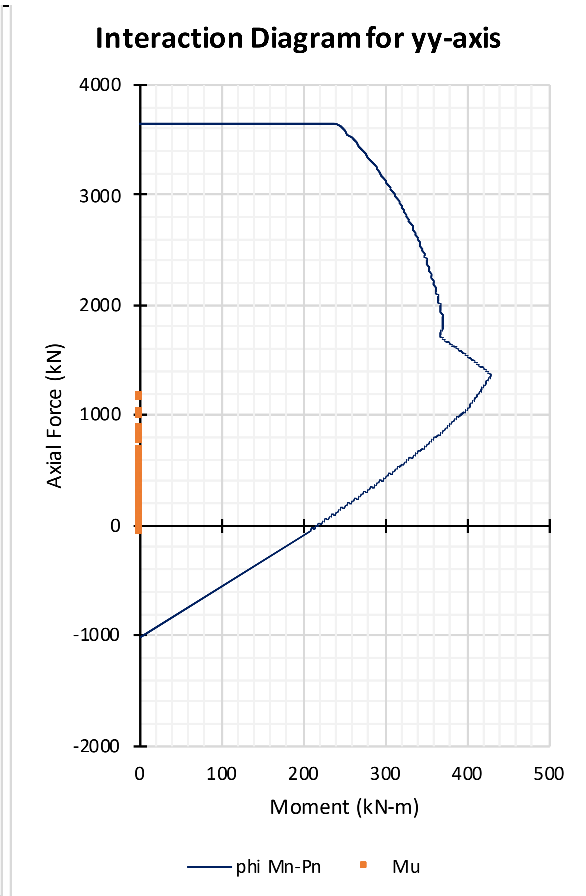
xx-axis			yy-axis		
Plastic hinge region		Other	Plastic hinge region		Other
ϕVn=	985 kN	647 kN	Vs=	862 kN	523 kN
Vu/ϕVn=	0.32	0.48	Vu/ϕVn=	0.36	0.59

Check
OK!

Strong column-weak beam check [ACI 18.7.3.2]

Joint	xx-axis				yy-axis			
	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check
Top	782	650	1.20	OK!	800	650	1.23	OK!
Bottom	782	650	1.20	OK!	800	650	1.23	OK!





Important notes:

-Combinations used for earthquake: 1.2D+1.0L+/-1.0E and 0.9D+/-1.0E **If different change Cap sheets accordingly

-Only symmetrical reinforcement is supported in the same axis.

-Lap splices outside lo region, the zone with lap-splice should comply with confinement reinforcement as in hinge region.

-Special provisions apply if column supports reactions from discontinued stiff members. [ACI 18.7.5.6]

-When using mechanical or welded splices check [ACI 18.7.4.3].

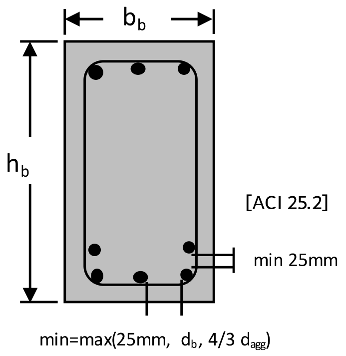
-Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees.

-No unsupported bar shall be farther than 150mm clear on each side along the tie form a laterally supported bar.

-If concrete cover exceeds 100mm, additional transverse reinforcement having cover not exceeding 100mm and spacing not exceeding 300mm shall be provided [ACI 18.7.5.7]

-If f'c exceeds 70MPa, special provisions apply [ACI 22.5.3.1]

Identification		Section Properties			Material Properties	
ID =	3-FB	h_b=	600 mm	OK!	f'_c=	28 MPa
Storey =	All	b_b=	250 mm	OK!	E_c=	24870 MPa
Grid, x=	1 - 5	L_b=	5500 mm		f_{yl}=	420 MPa
Grid, y=	NA	cv=	40 mm		f_{yt}=	420 MPa
		A_g=	150000 mm2		E_s=	200 GPa



0

Other input parameters		
λ=		1
min d	0	19.1 mm
min $d_{b,END2}$=		19.1 mm
ω_u=		58 kN/m

Proposed Shear Reinforcement					
END 1 (Plastic hinge 1)		CENTER (Elastic)		END 1 (Plastic hinge 1)	
ϕ =	9.5 mm	ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	2	No. legs=	2	No. legs=	2
s=	75 mm	s=	125 mm	s=	75 mm

Important notes:

-If using compression reinforcement, check requirements at the end of the spreadsheet.

-Check for requirements of max. spacing of longitudinal bars at the end of the spreadsheet.

-Lap splice locations must follow the requirements mentioned at the end of the spreadsheet.

-Check for necessary development lenghts at the end of the spreadsheet.

FORCES ACTING IN EACH SECTION								
END 1 =	1100 mm		CENTER =	3300 mm		END 2 =	1100 mm	
M_u^+=	25.00 kN-m		M_u^+=	102.00 kN-m		M_u^+=	25.00 kN-m	
M_u^-=	280.00 kN-m		M_u^-=	0.01 kN-m		M_u^-=	280.00 kN-m	
V_u=	201.00 kN	OK!	V_u=	201.00 kN	OK!	V_u=	201.00 kN	OK!
P_u=	13.00 kN	OK!	P_u=	13.00 kN	OK!	P_u=	13.00 kN	OK!
T_u=	1.00 kN-m	OK!	T_u=	1.00 kN	OK!	T_u=	1.00 kN	OK!

Flexural Reinforcement:					Flexural Reinforcement:					Flexural Reinforcement:				
no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)
3	19	0	0	50	3	19	0	0	50	3	19	0	0	50
2	19	0	0	94	3	19	0	0	550	2	19	0	0	94
3	19	0	0	550	0	0	0	0	0	3	19	0	0	550
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

MOMENT DESIGN

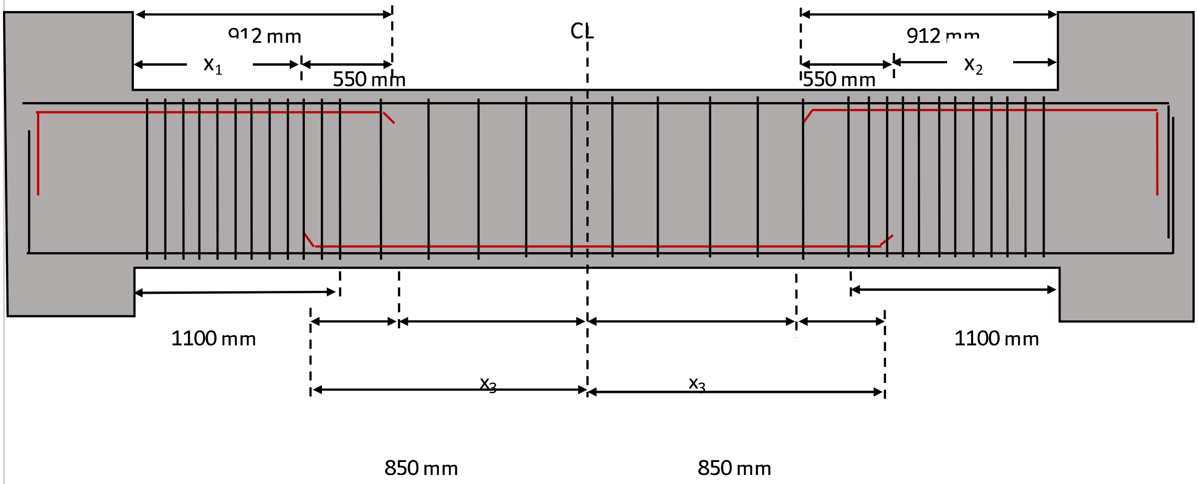
END 1			CENTER			END 2		
Press for equilibrium						OK! All sections in equilibrium		
ϕ^+ =	0.9		ϕ^+ =	0.9		ϕ^+ =	0.9	
ϕ^- =	0.9		ϕ^- =	0.9		ϕ^- =	0.9	
ϕM_n^+ =	193 kN-m	OK!	ϕM_n^+ =	188 kN-m	OK!	ϕM_n^+ =	193 kN-m	OK!
ϕM_n^- =	295 kN-m	OK!	ϕM_n^- =	188 kN-m	OK!	ϕM_n^- =	295 kN-m	OK!
$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2	

END 1			CENTER			END 2		
$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2	
$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2	
A_s^+=	1427 mm2	OK!	A_s^+=	860 mm2	OK!	A_s^+=	1427 mm2	OK!
A_s^-=	1427 mm2	OK!	A_s^-=	860 mm2	OK!	A_s^-=	1427 mm2	OK!
$M_u^+/\phi M_n^+$ =	0.13	OK!	$M_u^+/\phi M_n^+$ =	0.54	OK!	$M_u^+/\phi M_n^+$ =	0.13	OK!
$M_u^-/\phi M_n^-$ =	0.95	OK!	$M_u^-/\phi M_n^-$ =	0.00	OK!	$M_u^-/\phi M_n^-$ =	0.95	OK!

SHEAR DESIGN

END 1			CENTER			END 2		
ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2
A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!
s=	75 mm	OK!	s=	125 mm	OK!	s=	75 mm	OK!
s_{max}=	115 mm		s_{max}=	138 mm		s_{max}=	115 mm	
λ=	1		λ=	1		λ=	1	
V_c=	0 kN		V_c=	131 kN		V_c=	0 kN	
V_s=	437 kN		V_s=	262 kN		V_s=	437 kN	
ϕV_n=	327 kN		ϕV_n=	295 kN		ϕV_n=	327 kN	
ω_u=	58 kN/m		ω_u=	58 kN/m		ω_u=	58 kN/m	
$V_{e,max}$=	294 kN		$V_{e,max}$=	229 kN		$V_{e,max}$=	294 kN	
$V_{e,min}$=	-28 kN		$V_{e,min}$=	36 kN		$V_{e,min}$=	-28 kN	
V_{dis}=	294 kN		V_{dis}=	229 kN		V_{dis}=	294 kN	
$V_{dis}/\phi V_n$=	0.90	OK!	$V_{dis}/\phi V_n$=	0.78	OK!	$V_{dis}/\phi V_n$=	0.90	OK!

DEVELOPMENT LENGHTS AND SPLICES					
END 1		CENTER		END 2	
1. Additional reinforcement		**1. Additional reinforcement**		**1. Additional reinforcement**	
d_b=	19.1 mm	d_b=	19.1 mm	d_b=	19.1 mm
x_1=	300 mm	x_3=	300 mm	x_2=	300 mm
λ=	1	λ=	1	λ=	1
ψ_e=	1	ψ_e=	1	ψ_e=	1
ψ_s=	0.8	ψ_s=	0.8	ψ_s=	0.8
ψ_t=	1.3	ψ_t=	1	ψ_t=	1.3
A_{tr}=	142 mm2	A_{tr}=	142 mm2	A_{tr}=	142 mm2
c_b=	50 mm	c_b=	50 mm	c_b=	50 mm
s=	75 mm	s=	125 mm	s=	75 mm
n=	2	n=	1	n=	2
k_{tr}=	38	k_{tr}=	45	k_{tr}=	38
$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5
ξ=	3.25	ξ=	2.5	ξ=	3.25
l_d=	912 mm	l_d=	702 mm	l_d=	912 mm
l_{bast}=	912 mm	l_{bast}=	850 mm	l_{bast}=	912 mm
2. Development length into column		**2. Development length into column**		**2. Development length into column**	
max d_b=	19.1 mm	max d_b=	19.1 mm	max d_b=	19.1 mm
l_{dh}=	281 mm	l_{dh}=	281 mm	l_{dh}=	281 mm



MAXIMUN SPACING OF REINFORCEMENT IN TENSION FACE AND SKIN REINFORCEMENT			
Tension Face		Skin reinforcement	
c_c=	40 mm	c_c=	40 mm
f_s=	280 MPa	f_s=	280 MPa
max spacing=	279 mm	max spacing=	280 mm

LATERAL SUPPORT OF COMPRESSION REINFORCEMENT

Notes for support of compression reinforcement (this applies only when compression is taken into account):

1. Minimun size:

No.10 if longitudinal bars are No32 or less.

No.13 if longitudinal bars are No36 or more, and for bundles.

2. Maximun spacing:

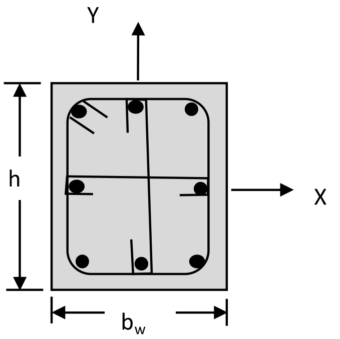
$d_{b,long}$= 19.1 mm

$d_{b,trans}$= 9.5 mm

s_{max}= 250 mm

3. Longitudinal compression reinforcement shall be arranged in such a manner that every corner and alternate bar be enclosed by the corner of the transverse reinforcement with an included angle of 135 degrees or less. No bar shall be farther than 150mm clear on each side along the transverse reinforcement from such an enclosed bar.

Identification		Material Prop.		Section Properties	
ID =	3-FB	f'_c=	28 MPa	h=	500 mm
Storey =	All	E_c=	24870 MPa	b_w=	500 mm
Grid, x=	1-5	f_{yl}=	420 MPa	l_w=	3000 mm
Grid, y=	NA	f_{yt}=	420 MPa	c_c=	40 mm
		E_s=	200 GPa	A_g=	250000 mm2
		λ=	1	I_{xx}=	5E+09 mm4
				I_{yy}=	5E+09 mm4
				d_{xx}=	450.00 mm
				d_{yy}=	450.00 mm
				h_x=	140 mm
				ϕ_{min}=	19 mm
				n_l=	12
				A_{ch}=	176400 mm2
				d_{agg}=	25 mm



Proposed Shear Reinforcement			
XX-Axis		YY-Axis	
Plastic hinge region		Plastic hinge region	
ϕ =	12.7 mm	ϕ =	12.7 mm
No. legs=	4	No. legs=	4
s=	100 mm	s=	100 mm
Ash=	507 mm2	Ash=	507 mm2
Other regions		Other regions	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	4	No. legs=	4
s=	100 mm	s=	100 mm
Ash=	284 mm2	Ash=	284 mm2
Pu and Mu corresponding to Vu in x and y			
Pu=	1032 kN	Pu=	0 kN
Mu=	268 kN	Mu=	0 kN

Axial Force		Shear Force in x		Shear Force in y		Axial Force from EQ	
P_D=	637 kN	V_{Dx}=	1 kN	V_{Dy}=	0 kN	P^+_{Ex}=	100 kN
P_L=	262 kN	V_{Lx}=	1 kN	V_{Ly}=	0 kN	P^-_{Ex}=	-100 kN
P_u=	1032 kN	V_{Ex}=	114 kN	V_{Ey}=	0 kN	P^+_{Ey}=	0 kN
		V_{ux}=	116 kN	V_{uy}=	0 kN	P^-_{Ey}=	0 kN

In equilibrium!

PRESS BUTTON FOR EQUILIBRIUM

Flexural Reinforcement xx:						Flexural Reinforcement yy:					
no.	ϕ	no.	ϕ	d (mm)	Ast	no.	ϕ	no.	ϕ	d (mm)	Ast
4	19	0	0	50	###	4	19	0	0	50	###
2	19	0	0	180	573	2	19	0	0	180	573
2	19	0	0	320	573	2	19	0	0	320	573
4	19	0	0	450	###	4	19	0	0	450	###
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Moment capacity of columns and beams		
Top joint	xx-axis	yy-axis
Mprc1=	440	454
Mprc2=	440	454
Mprb1=	365	365
Mprb2=	365	365
Bot joint	xx-axis	yy-axis
Mprc1=	440	454
Mprc2=	440	454
Mprb1=	365	365
Mprb2=	365	365

Dimension and longitudinal reinforcement limits

$b_{w,min}$=	300 mm			OK!	[ACI 18.7.2.1 (a)]
$h_{w,min}$=	300 mm			OK!	[ACI 18.7.2.1 (a)]
b_w/h_w or h_w/b_w=	1.00	>=	0.4	OK!	[ACI 18.7.2.1 (b)]
A_{st}=	3438 mm2			OK!	[ACI 18.7.4.1]

Transverse reinforcement in plastic hinge region

l_o=	500 mm		[ACI 18.7.5.1]
$h_{x,max}$=	350 mm	OK!	[ACI 18.7.5.2, e and f]
s_{max}=	115 mm	OK!	[ACI 18.7.5.3]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
$A_{sh}/sb_{c,x}$=	1.01%	OK!	[ACI 18.7.5.4]
$A_{sh}/sb_{c,y}$=	1.01%	OK!	[ACI 18.7.5.4]

Transverse reinforcement in other regions

s_{max}=	115 mm	OK!	[ACI 18.7.5.5]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
ϕ_{min}=	9.5 mm	OK!	[ACI 25.7.2.2]

Shear force check

Vux max=	973 kN	OK!	[ACI 22.5.1.2]
Vuy max=	848 kN	OK!	[ACI 22.5.1.2]

Bi-axial moment capacity

$$\left({}^{Mu,xx}/_{Mn,xx}\right)^2 + \left({}^{Mu,yy}/_{Mn,yy}\right)^2 = $$ 1 OK!

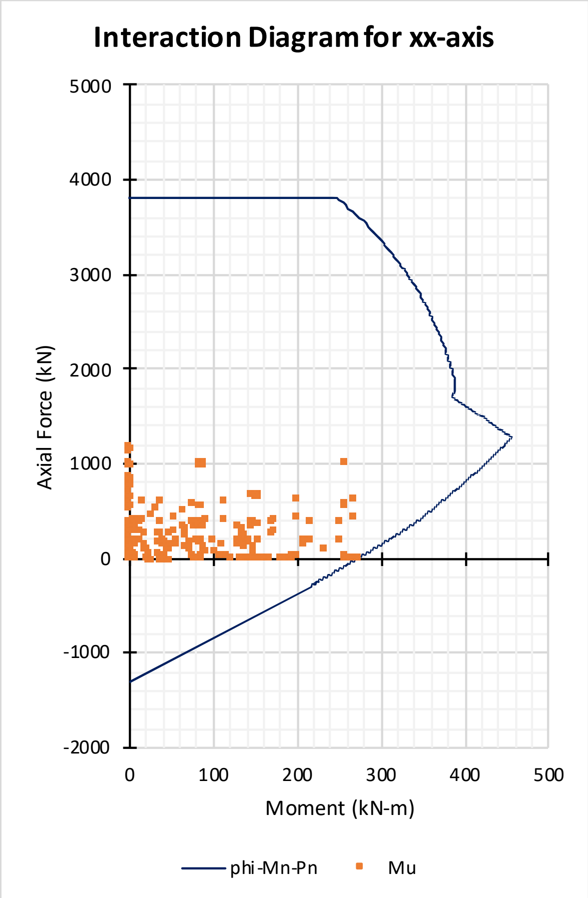
Shear capacity

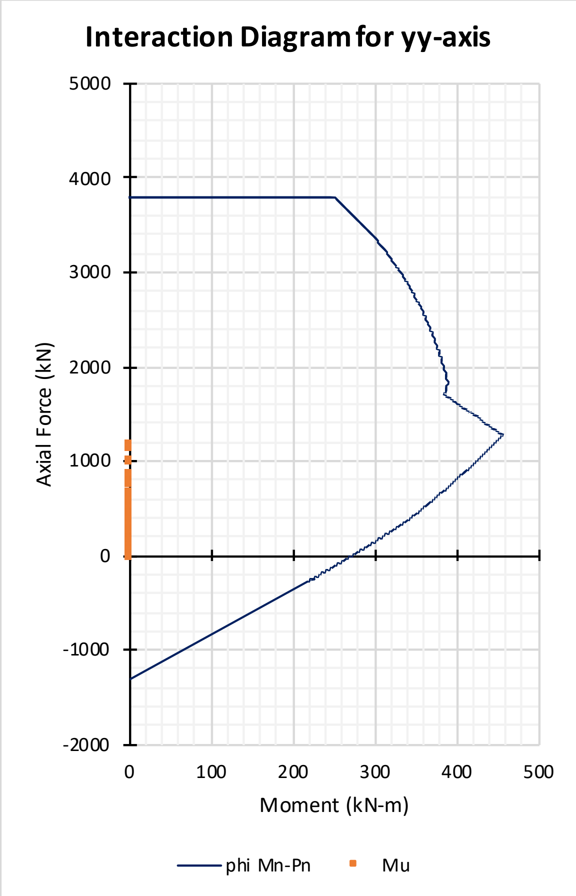
xx-axis			yy-axis		
	Plastic hinge region	Other		Plastic hinge region	Other
ϕVn=	1102 kN	786 kN	Vs=	977 kN	661 kN
Vu/ϕVn=	0.32	0.44	Vu/ϕVn=	0.35	0.52

Check
OK!

Strong column-weak beam check [ACI 18.7.3.2]

Joint	xx-axis				yy-axis			
	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check
Top	880	730	1.21	OK!	908	730	1.24	OK!
Bottom	880	730	1.21	OK!	908	730	1.24	OK!





Important notes:

-Combinations used for earthquake: 1.2D+1.0L+/-1.0E and 0.9D+/-1.0E **If different change Cap sheets accordingly

-Only symmetrical reinforcement is supported in the same axis.

-Lap splices outside lo region, the zone with lap-splice should comply with confinement reinforcement as in hinge region.

-Special provisions apply if column supports reactions from discontinued stiff members. [ACI 18.7.5.6]

-When using mechanical or welded splices check [ACI 18.7.4.3].

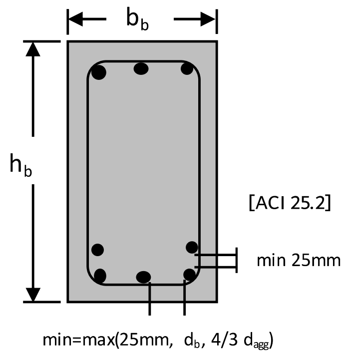
-Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees.

-No unsupported bar shall be farther than 150mm clear on each side along the tie form a laterally supported bar.

-If concrete cover exceeds 100mm, additional transverse reinforcement having cover not exceeding 100mm and spacing not exceeding 300mm shall be provided [ACI 18.7.5.7]

-If f'c exceeds 70MPa, special provisions apply [ACI 22.5.3.1]

Identification		Section Properties			Material Properties	
ID =	6-FB	h_b=	600 mm	OK!	f'_c=	28 MPa
Storey =	1,2,3	b_b=	250 mm	OK!	E_c=	24870 MPa
Grid, x=	1 - 5	L_b=	5500 mm		f_{yl}=	420 MPa
Grid, y=	NA	cv =	40 mm		f_{yt}=	420 MPa
		A_g=	150000 mm2		E_s=	200 GPa



0

Other input parameters		
λ=		1
min d	0	22.2 mm
min $d_{b,END2}$=		22.2 mm
ω_u=		58 kN/m

Proposed Shear Reinforcement					
END 1 (Plastic hinge 1)		CENTER (Elastic)		END 1 (Plastic hinge 1)	
ϕ =	9.5 mm	ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	2	No. legs=	2	No. legs=	2
s=	75 mm	s=	125 mm	s=	75 mm

Important notes:

-If using compression reinforcement, check requirements at the end of the spreadsheet.

-Check for requirements of max. spacing of longitudinal bars at the end of the spreadsheet.

-Lap splice locations must follow the requirements mentioned at the end of the spreadsheet.

-Check for necessary development lenghts at the end of the spreadsheet.

FORCES ACTING IN EACH SECTION								
END 1 =	1100 mm		CENTER =	3300 mm		END 2 =	1100 mm	
M_u^+=	88.00 kN-m		M_u^+=	110.00 kN-m		M_u^+=	88.00 kN-m	
M_u^-=	324.00 kN-m		M_u^-=	0.01 kN-m		M_u^-=	324.00 kN-m	
V_u=	216.00 kN	OK!	V_u=	216.00 kN	OK!	V_u=	216.00 kN	OK!
P_u=	88.00 kN	OK!	P_u=	88.00 kN	OK!	P_u=	88.00 kN	OK!
T_u=	1.00 kN-m	OK!	T_u=	1.00 kN	OK!	T_u=	1.00 kN	OK!

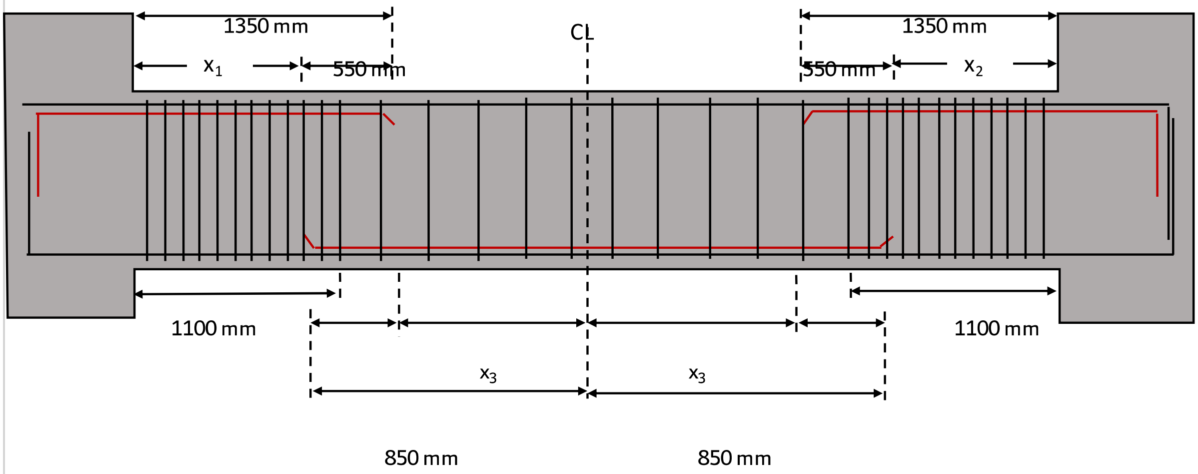
Flexural Reinforcement:					Flexural Reinforcement:					Flexural Reinforcement:				
no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)
2	22	0	0	50	2	22	0	0	50	2	22	0	0	50
2	22	0	0	94	2	22	0	0	550	2	22	0	0	94
2	22	0	0	550	0	0	0	0	0	2	22	0	0	550
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

MOMENT DESIGN								
END 1			CENTER			END 2		
Press for equilibrium						OK! All sections in equilibrium		
ϕ^+ =	0.9		ϕ^+ =	0.9		ϕ^+ =	0.9	
ϕ^- =	0.9		ϕ^- =	0.9		ϕ^- =	0.9	
ϕM_n^+ =	176 kN-m	OK!	ϕM_n^+ =	170 kN-m	OK!	ϕM_n^+ =	176 kN-m	OK!
ϕM_n^- =	316 kN-m	OK!	ϕM_n^- =	170 kN-m	OK!	ϕM_n^- =	316 kN-m	OK!
$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2	

END 1			CENTER			END 2		
$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2	
$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2	
A_s^+=	1548 mm2	OK!	A_s^+=	774 mm2	OK!	A_s^+=	1548 mm2	OK!
A_s^-=	1548 mm2	OK!	A_s^-=	774 mm2	OK!	A_s^-=	1548 mm2	OK!
$M_u^+/\phi M_n^+$ =	0.50	OK!	$M_u^+/\phi M_n^+$ =	0.65	OK!	$M_u^+/\phi M_n^+$ =	0.50	OK!
$M_u^-/\phi M_n^-$ =	1.03	OK!	$M_u^-/\phi M_n^-$ =	0.00	OK!	$M_u^-/\phi M_n^-$ =	1.03	OK!

SHEAR DESIGN								
END 1			CENTER			END 2		
ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2
A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!
s=	75 mm	OK!	s=	125 mm	OK!	s=	75 mm	OK!
s_{max}=	133 mm		s_{max}=	138 mm		s_{max}=	133 mm	
λ=	1		λ=	1		λ=	1	
V_c=	0 kN		V_c=	130 kN		V_c=	0 kN	
V_s=	437 kN		V_s=	262 kN		V_s=	437 kN	
ϕV_n=	327 kN		ϕV_n=	294 kN		ϕV_n=	327 kN	
ω_u=	58 kN/m		ω_u=	58 kN/m		ω_u=	58 kN/m	
$V_{e,max}$=	302 kN		$V_{e,max}$=	238 kN		$V_{e,max}$=	302 kN	
$V_{e,min}$=	-19 kN		$V_{e,min}$=	45 kN		$V_{e,min}$=	-19 kN	
V_{dis}=	302 kN		V_{dis}=	238 kN		V_{dis}=	302 kN	
$V_{dis}/\phi V_n$=	0.92	OK!	$V_{dis}/\phi V_n$=	0.81	OK!	$V_{dis}/\phi V_n$=	0.92	OK!

DEVELOPMENT LENGHTS AND SPLICES					
END 1		CENTER		END 2	
1. Additional reinforcement		**1. Additional reinforcement**		**1. Additional reinforcement**	
d_b=	22.2 mm	d_b=	19.1 mm	d_b=	22.2 mm
x_1=	800 mm	x_3=	300 mm	x_2=	800 mm
λ=	1	λ=	1	λ=	1
ψ_e=	1	ψ_e=	1	ψ_e=	1
ψ_s=	1	ψ_s=	0.8	ψ_s=	1
ψ_t=	1.3	ψ_t=	1	ψ_t=	1.3
A_{tr}=	142 mm2	A_{tr}=	142 mm2	A_{tr}=	142 mm2
c_b=	50 mm	c_b=	50 mm	c_b=	50 mm
s=	75 mm	s=	125 mm	s=	75 mm
n=	2	n=	1	n=	2
k_{tr}=	38	k_{tr}=	45	k_{tr}=	38
$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5
ξ=	3.25	ξ=	2.5	ξ=	3.25
l_d=	1061 mm	l_d=	702 mm	l_d=	1061 mm
l_{bast}=	1350 mm	l_{bast}=	850 mm	l_{bast}=	1350 mm
2. Development length into column		**2. Development length into column**		**2. Development length into column**	
max d_b=	22.2 mm	max d_b=	22.2 mm	max d_b=	22.2 mm
l_{dh}=	326 mm	l_{dh}=	326 mm	l_{dh}=	326 mm



MAXIMUN SPACING OF REINFORCEMENT IN TENSION FACE AND SKIN REINFORCEMENT				
Tension Face		Skin reinforcement		
c_c=	39 mm	c_c=	40 mm	
f_s=	280 MPa	f_s=	280 MPa	
max spacing=	283 mm	max spacing=	280 mm	

LATERAL SUPPORT OF COMPRESSION REINFORCEMENT

Notes for support of compression reinforcement (this applies only when compression is taken into account):

1. Minimun size:

No.10 if longitudinal bars are No32 or less.

No.13 if longitudinal bars are No36 or more, and for bundles.

2. Maximun spacing:

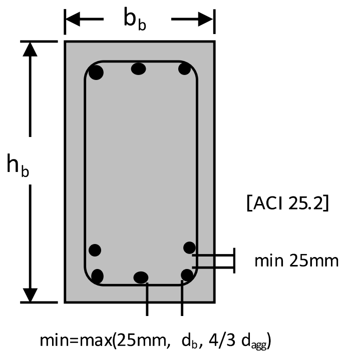
$d_{b,long}$= 19.1 mm

$d_{b,trans}$= 9.5 mm

s_{max}= 250 mm

3. Longitudinal compression reinforcement shall be arranged in such a manner that every corner and alternate bar be enclosed by the corner of the transverse reinforcement with an included angle of 135 degrees or less. No bar shall be farther than 150mm clear on each side along the transverse reinforcement from such an enclosed bar.

Identification		Section Properties			Material Properties	
ID =	6-FB	h_b=	600 mm	OK!	f'_c=	28 MPa
Storey =	4,5,6	b_b=	250 mm	OK!	E_c=	24870 MPa
Grid, x=	1 - 5	L_b=	5500 mm		f_{yl}=	420 MPa
Grid, y=	NA	cv=	40 mm		f_{yt}=	420 MPa
		A_g=	150000 mm2		E_s=	200 GPa



0

Other input parameters	
λ=	1
min d 0	19.1 mm
min $d_{b,END2}$=	19.1 mm
ω_u=	58 kN/m

Proposed Shear Reinforcement					
END 1 (Plastic hinge 1)		CENTER (Elastic)		END 1 (Plastic hinge 1)	
ϕ =	9.5 mm	ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	2	No. legs=	2	No. legs=	2
s=	75 mm	s=	125 mm	s=	75 mm

Important notes:

-If using compression reinforcement, check requirements at the end of the spreadsheet.

-Check for requirements of max. spacing of longitudinal bars at the end of the spreadsheet.

-Lap splice locations must follow the requirements mentioned at the end of the spreadsheet.

-Check for necessary development lenghts at the end of the spreadsheet.

FORCES ACTING IN EACH SECTION														
END 1 =			1100 mm		CENTER =			3300 mm		END 2 =			1100 mm	
M_u^+=	50.00 kN-m				M_u^+=	102.00 kN-m				M_u^+=	50.00 kN-m			
M_u^-=	289.00 kN-m				M_u^-=	0.01 kN-m				M_u^-=	289.00 kN-m			
V_u=	201.00 kN			OK!	V_u=	201.00 kN			OK!	V_u=	201.00 kN			OK!
P_u=	13.00 kN			OK!	P_u=	13.00 kN			OK!	P_u=	13.00 kN			OK!
T_u=	1.00 kN-m			OK!	T_u=	1.00 kN			OK!	T_u=	1.00 kN			OK!
Flexural Reinforcement:					Flexural Reinforcement:					Flexural Reinforcement:				
no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)
2	22	0	0	50	2	22	0	0	50	2	22	0	0	50
2	19	0	0	94	2	22	0	0	550	2	19	0	0	94
2	22	0	0	550	0	0	0	0	0	2	22	0	0	550
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

MOMENT DESIGN

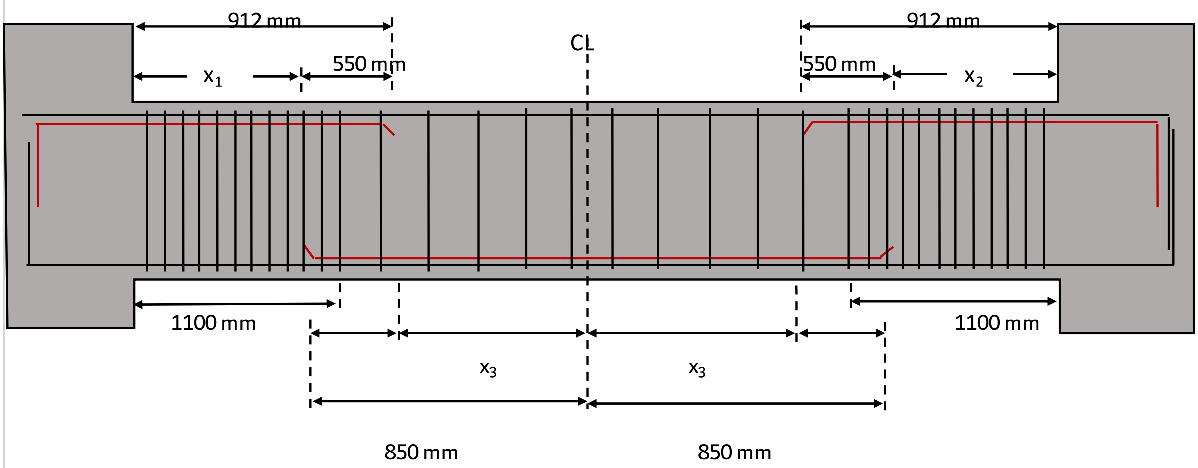
END 1			CENTER			END 2		
Press for equilibrium						OK! All sections in equilibrium		
ϕ^+ =	0.9		ϕ^+ =	0.9		ϕ^+ =	0.9	
ϕ^- =	0.9		ϕ^- =	0.9		ϕ^- =	0.9	
ϕM_n^+ =	175 kN-m	OK!	ϕM_n^+ =	170 kN-m	OK!	ϕM_n^+ =	175 kN-m	OK!
ϕM_n^- =	278 kN-m	OK!	ϕM_n^- =	170 kN-m	OK!	ϕM_n^- =	278 kN-m	OK!
$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2	

END 1			CENTER			END 2		
$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2	
$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2	
A_s^+=	1341 mm2	OK!	A_s^+=	774 mm2	OK!	A_s^+=	1341 mm2	OK!
A_s^-=	1341 mm2	OK!	A_s^-=	774 mm2	OK!	A_s^-=	1341 mm2	OK!
$M_u^+/\phi M_n^+$ =	0.29	OK!	$M_u^+/\phi M_n^+$ =	0.60	OK!	$M_u^+/\phi M_n^+$ =	0.29	OK!
$M_u^-/\phi M_n^-$ =	1.04	OK!	$M_u^-/\phi M_n^-$ =	0.00	OK!	$M_u^-/\phi M_n^-$ =	1.04	OK!

SHEAR DESIGN

END 1			CENTER			END 2		
ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2
A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!
s=	75 mm	OK!	s=	125 mm	OK!	s=	75 mm	OK!
s_{max}=	115 mm		s_{max}=	138 mm		s_{max}=	115 mm	
λ=	1		λ=	1		λ=	1	
V_c=	0 kN		V_c=	130 kN		V_c=	0 kN	
V_s=	437 kN		V_s=	262 kN		V_s=	437 kN	
ϕV_n=	327 kN		ϕV_n=	294 kN		ϕV_n=	327 kN	
ω_u=	58 kN/m		ω_u=	58 kN/m		ω_u=	58 kN/m	
$V_{e,max}$=	286 kN		$V_{e,max}$=	221 kN		$V_{e,max}$=	286 kN	
$V_{e,min}$=	-36 kN		$V_{e,min}$=	28 kN		$V_{e,min}$=	-36 kN	
V_{dis}=	286 kN		V_{dis}=	221 kN		V_{dis}=	286 kN	
$V_{dis}/\phi V_n$=	0.87	OK!	$V_{dis}/\phi V_n$=	0.75	OK!	$V_{dis}/\phi V_n$=	0.87	OK!

DEVELOPMENT LENGHTS AND SPLICES					
END 1		CENTER		END 2	
1. Additional reinforcement		**1. Additional reinforcement**		**1. Additional reinforcement**	
d_b=	19.1 mm	d_b=	19.1 mm	d_b=	19.1 mm
x_1=	300 mm	x_3=	300 mm	x_2=	300 mm
λ=	1	λ=	1	λ=	1
ψ_e=	1	ψ_e=	1	ψ_e=	1
ψ_s=	0.8	ψ_s=	0.8	ψ_s=	0.8
ψ_t=	1.3	ψ_t=	1	ψ_t=	1.3
A_{tr}=	142 mm2	A_{tr}=	142 mm2	A_{tr}=	142 mm2
c_b=	50 mm	c_b=	50 mm	c_b=	50 mm
s=	75 mm	s=	125 mm	s=	75 mm
n=	2	n=	1	n=	2
k_{tr}=	38	k_{tr}=	45	k_{tr}=	38
$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5
ξ=	3.25	ξ=	2.5	ξ=	3.25
l_d=	912 mm	l_d=	702 mm	l_d=	912 mm
l_{bast}=	912 mm	l_{bast}=	850 mm	l_{bast}=	912 mm
2. Development length into column		**2. Development length into column**		**2. Development length into column**	
max d_b=	22.2 mm	max d_b=	22.2 mm	max d_b=	22.2 mm
l_{dh}=	326 mm	l_{dh}=	326 mm	l_{dh}=	326 mm



MAXIMUN SPACING OF REINFORCEMENT IN TENSION FACE AND SKIN REINFORCEMENT				
Tension Face		Skin reinforcement		
c_c=	39 mm	c_c=	40 mm	
f_s=	280 MPa	f_s=	280 MPa	
max spacing=	283 mm	max spacing=	280 mm	

LATERAL SUPPORT OF COMPRESSION REINFORCEMENT

Notes for support of compression reinforcement (this applies only when compression is taken into account):

1. Minimun size:

No.10 if longitudinal bars are No32 or less.

No.13 if longitudinal bars are No36 or more, and for bundles.

2. Maximun spacing:

$d_{b,long}$= 19.1 mm

$d_{b,trans}$= 9.5 mm

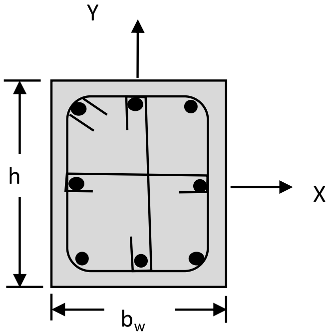
s_{max}= 250 mm

3. Longitudinal compression reinforcement shall be arranged in such a manner that every corner and alternate bar be enclosed by the corner of the transverse reinforcement with an included angle of 135 degrees or less. No bar shall be farther than 150mm clear on each side along the transverse reinforcement from such an enclosed bar.

Identification	
ID =	6-FB
Storey =	1,2,3
Grid, x=	1-5
Grid, y=	NA

Material Prop.	
f'_c=	28 MPa
E_c=	24870 MPa
f_{yl}=	420 MPa
f_{yt}=	420 MPa
E_s=	200 GPa
λ=	1

Section Properties	
h=	500 mm
b_w=	500 mm
l_w=	3000 mm
c_c=	40 mm
A_g=	250000 mm2
I_{xx}=	5E+09 mm4
I_{yy}=	5E+09 mm4
d_{xx}=	450.00 mm
d_{yy}=	450.00 mm
h_x=	135 mm
ϕ_{min}=	22 mm
n_l=	12
A_{ch}=	176400 mm2
d_{agg}=	25 mm



Proposed Shear Reinforcement			
XX-Axis		YY-Axis	
Plastic hinge region		Plastic hinge region	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	50 mm	s=	50 mm
Ash=	213 mm2	Ash=	213 mm2
Other regions		Other regions	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	125 mm	s=	125 mm
Ash=	213 mm2	Ash=	213 mm2
Pu and Mu corresponding to Vu in x and y			
Pu=	1032 kN	Pu=	0 kN
Mu=	268 kN	Mu=	0 kN

Axial Force		Shear Force in x		Shear Force in y		Axial Force from EQ	
P_D=	1266 kN	V_{Dx}=	20 kN	V_{Dy}=	0 kN	P^+_{Ex}=	234 kN
P_L=	520 kN	V_{Lx}=	9 kN	V_{Ly}=	0 kN	P^-_{Ex}=	-234 kN
P_u=	2048 kN	V_{Ex}=	130 kN	V_{Ey}=	0 kN	P^+_{Ey}=	0 kN
		V_{ux}=	163 kN	V_{uy}=	0 kN	P^-_{Ey}=	0 kN

In equilibrium!

PRESS BUTTON FOR EQUILIBRIUM

Flexural Reinforcement xx:						Flexural Reinforcement yy:					
no.	ϕ	no.	ϕ	d (mm)	Ast	no.	ϕ	no.	ϕ	d (mm)	Ast
2	22	1	22	50	###	2	22	1	22	50	###
2	22	0	0	250	774	2	22	0	0	250	774
2	22	1	22	450	###	2	22	1	22	450	###
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Moment capacity of columns and beams		
Top joint	xx-axis	yy-axis
Mprc1=	480	501
Mprc2=	480	501
Mprb1=	390	390
Mprb2=	390	390
Bot joint	xx-axis	yy-axis
Mprc1=	480	501
Mprc2=	480	501
Mprb1=	390	390
Mprb2=	390	390

Dimension and longitudinal reinforcement limits

$b_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
$h_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
b_w/h_w or h_w/b_w=		1.00	>=	0.4	OK!	[ACI 18.7.2.1 (b)]
A_{st}=	3097 mm2				OK!	[ACI 18.7.4.1]

Transverse reinforcement in plastic hinge region

l_o=	500 mm		[ACI 18.7.5.1]
$h_{x,max}$=	350 mm	OK!	[ACI 18.7.5.2, e and f]
s_{max}=	125 mm	OK!	[ACI 18.7.5.3]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
$A_{sh}/sb_{c,x}$=	0.85%	OK!	[ACI 18.7.5.4]
$A_{sh}/sb_{c,y}$=	0.85%	OK!	[ACI 18.7.5.4]

Transverse reinforcement in other regions

s_{max}=	133 mm	OK!	[ACI 18.7.5.5]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
ϕ_{min}=	9.5 mm	OK!	[ACI 25.7.2.2]

Shear force check

Vux max=	973 kN	OK!	[ACI 22.5.1.2]
Vuy max=	848 kN	OK!	[ACI 22.5.1.2]

Bi-axial moment capacity

$$\left(\frac{Mu,xx}{Mn,xx}\right)^2+\left(\frac{Mu,yy}{Mn,yy}\right)^2=$$ 0.81085 OK!

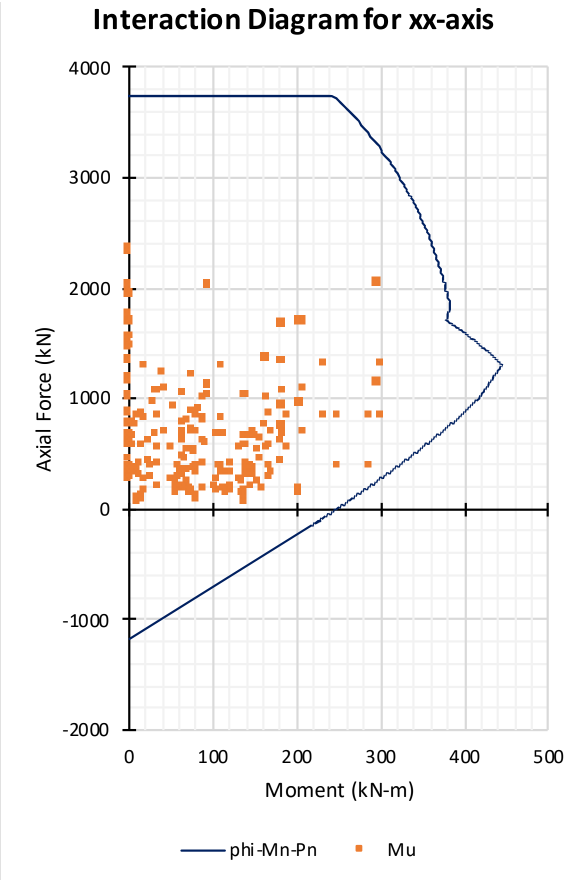
Shear capacity

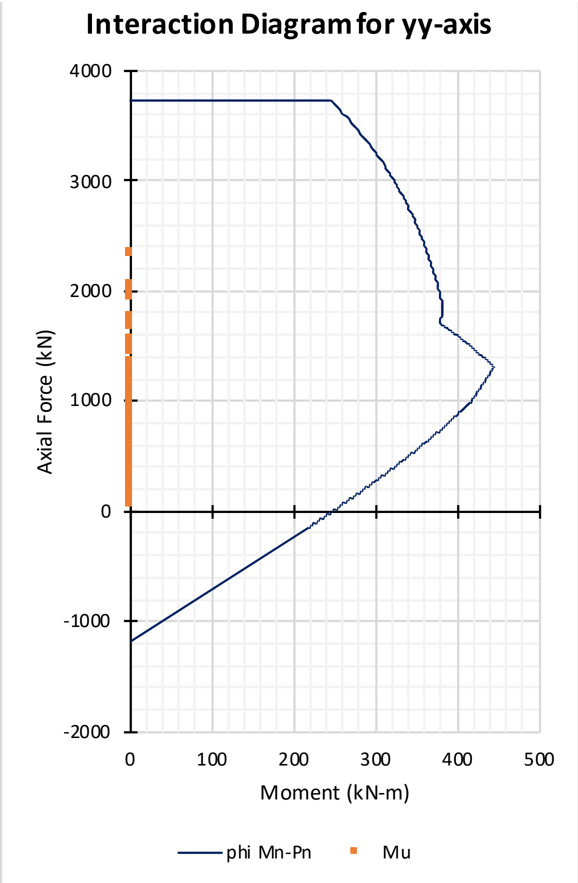
xx-axis				yy-axis			
Plastic hinge region		Other		Plastic hinge region		Other	
ϕVn=	987 kN	625 kN		Vs=	862 kN	500 kN	
Vu/ϕVn=	0.39	0.62		Vu/ϕVn=	0.44	0.76	

Check
OK!

Strong column-weak beam check [ACI 18.7.3.2]

Joint	xx-axis				yy-axis			
	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check
Top	960	780	1.23	OK!	1002	780	1.29	OK!
Bottom	960	780	1.23	OK!	1002	780	1.29	OK!





Important notes:

-Combinations used for earthquake: 1.2D+1.0L+/-1.0E and 0.9D+/-1.0E **If different change Cap sheets accordingly

-Only symmetrical reinforcement is supported in the same axis.

-Lap splices outside lo region, the zone with lap-splice should comply with confinement reinforcement as in hinge region.

-Special provisions apply if column supports reactions from discontinued stiff members. [ACI 18.7.5.6]

-When using mechanical or welded splices check [ACI 18.7.4.3].

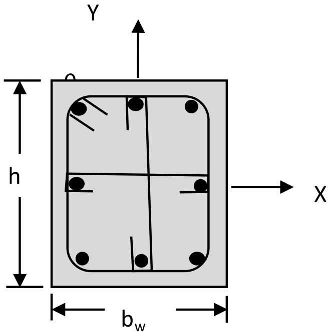
-Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees.

-No unsupported bar shall be farther than 150mm clear on each side along the tie form a laterally supported bar.

-If concrete cover exceeds 100mm, additional transverse reinforcement having cover not exceeding 100mm and spacing not exceeding 300mm shall be provided [ACI 18.7.5.7]

-If f'c exceeds 70MPa, special provisions apply [ACI 22.5.3.1]

Identification		Material Prop.		Section Properties	
ID =	6-FB	f'_c=	28 MPa	h=	500 mm
Storey =	4,5,6	E_c=	24870 MPa	b_w=	500 mm
Grid, x=	1-5	f_{yl}=	420 MPa	l_w=	3000 mm
Grid, y=	NA	f_{yt}=	420 MPa	c_c=	40 mm
		E_s=	200 GPa	A_g=	250000 mm2
0		λ=	1	I_{xx}=	5E+09 mm4
				I_{yy}=	5E+09 mm4
				d_{xx}=	450.00 mm
				d_{yy}=	450.00 mm
				h_x=	135 mm
				ϕ_{min}=	22 mm
				n_l=	12
				A_{ch}=	176400 mm2
				d_{agg}=	25 mm



Proposed Shear Reinforcement			
XX-Axis		YY-Axis	
Plastic hinge region		Plastic hinge region	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	50 mm	s=	50 mm
Ash=	213 mm2	Ash=	213 mm2
Other regions		Other regions	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	125 mm	s=	125 mm
Ash=	213 mm2	Ash=	213 mm2
Pu and Mu corresponding to Vu in x and y			
Pu=	1017 kN	Pu=	0 kN
Mu=	165 kN	Mu=	0 kN

Axial Force		Shear Force in x		Shear Force in y		Axial Force from EQ	
P_D=	652 kN	V_{Dx}=	1 kN	V_{Dy}=	0 kN	P^+_{Ex}=	129 kN
P_L=	260 kN	V_{Lx}=	1 kN	V_{Ly}=	0 kN	P^-_{Ex}=	-129 kN
P_u=	1017 kN	V_{Ex}=	99 kN	V_{Ey}=	0 kN	P^+_{Ey}=	0 kN
		V_{ux}=	101 kN	V_{uy}=	0 kN	P^-_{Ey}=	0 kN

In equilibrium!

PRESS BUTTON FOR EQUILIBRIUM

Flexural Reinforcement xx:						Flexural Reinforcement yy:					
no.	ϕ	no.	ϕ	d (mm)	Ast	no.	ϕ	no.	ϕ	d (mm)	Ast
2	22	1	22	50	###	2	22	1	22	50	###
2	22	0	0	250	774	2	22	0	0	250	774
2	22	1	22	450	###	2	22	1	22	450	###
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Moment capacity of columns and beams		
Top joint	xx-axis	yy-axis
Mprc1=	420	441
Mprc2=	420	441
Mprb1=	343	343
Mprb2=	343	343
Bot joint	xx-axis	yy-axis
Mprc1=	420	441
Mprc2=	420	441
Mprb1=	343	343
Mprb2=	343	343

Dimension and longitudinal reinforcement limits

$b_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
$h_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
b_w/h_w or h_w/b_w=		1.00	>=	0.4	OK!	[ACI 18.7.2.1 (b)]
A_{st}=	3097 mm2				OK!	[ACI 18.7.4.1]

Transverse reinforcement in plastic hinge region

l_o=	500 mm		[ACI 18.7.5.1]
$h_{x,max}$=	350 mm	OK!	[ACI 18.7.5.2, e and f]
s_{max}=	125 mm	OK!	[ACI 18.7.5.3]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
$A_{sh}/sb_{c,x}$=	0.85%	OK!	[ACI 18.7.5.4]
$A_{sh}/sb_{c,y}$=	0.85%	OK!	[ACI 18.7.5.4]

Transverse reinforcement in other regions

s_{max}=	133 mm	OK!	[ACI 18.7.5.5]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
ϕ_{min}=	9.5 mm	OK!	[ACI 25.7.2.2]

Shear force check

Vux max=	972 kN	OK!	[ACI 22.5.1.2]
Vuy max=	848 kN	OK!	[ACI 22.5.1.2]

Bi-axial moment capacity

$$\left({Mu,xx}/{Mn,xx}\right)^2+\left({Mu,yy}/{Mn,yy}\right)^2 = \quad 0.81085 \quad \text{OK!}$$

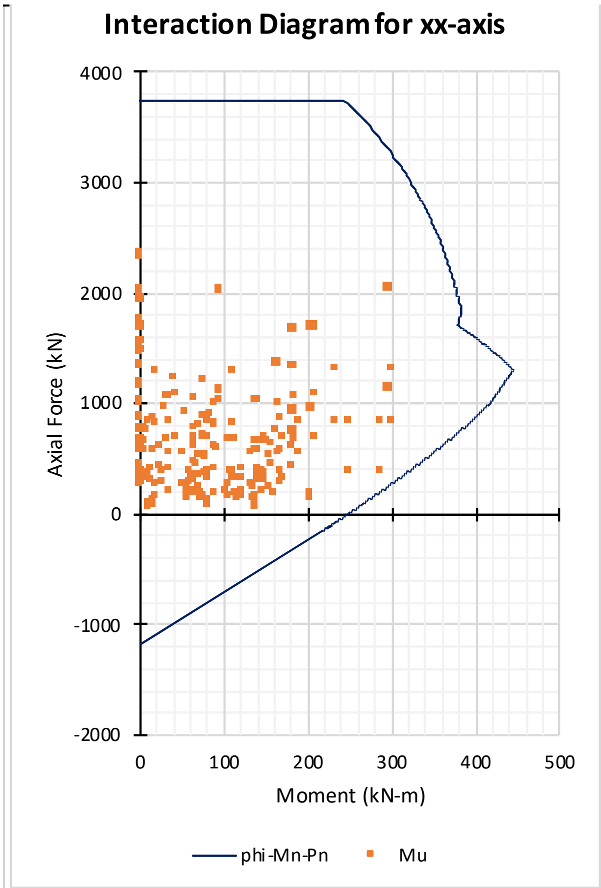
Shear capacity

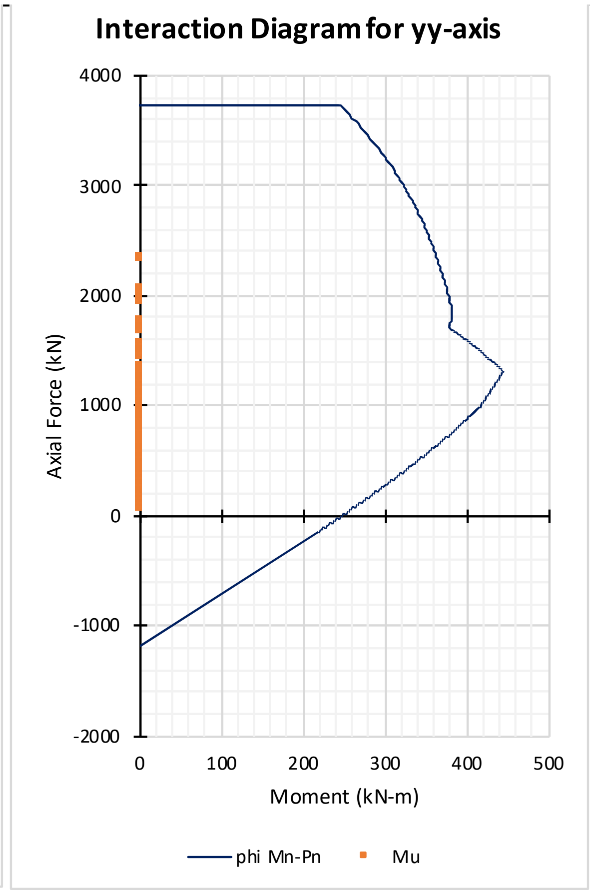
xx-axis			yy-axis		
Plastic hinge region		Other	Plastic hinge region		Other
ϕVn=	985 kN	623 kN	Vs=	862 kN	500 kN
Vu/ϕVn=	0.34	0.54	Vu/ϕVn=	0.38	0.66

Check
OK!

Strong column-weak beam check [ACI 18.7.3.2]

Joint	xx-axis				yy-axis			
	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check
Top	840	686	1.22	OK!	882	686	1.29	OK!
Bottom	840	686	1.22	OK!	882	686	1.29	OK!





Important notes:

-Combinations used for earthquake: 1.2D+1.0L+/-1.0E and 0.9D+/-1.0E **If different change Cap sheets accordingly

-Only symmetrical reinforcement is supported in the same axis.

-Lap splices outside lo region, the zone with lap-splice should comply with confinement reinforcement as in hinge region.

-Special provisions apply if column supports reactions from discontinued stiff members. [ACI 18.7.5.6]

-When using mechanical or welded splices check [ACI 18.7.4.3].

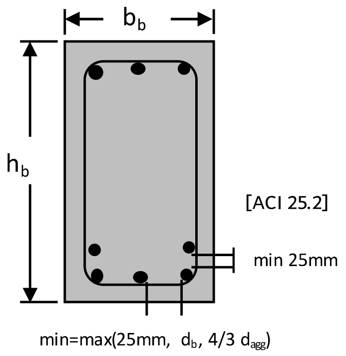
-Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees.

-No unsupported bar shall be farther than 150mm clear on each side along the tie form a laterally supported bar.

-If concrete cover exceeds 100mm, additional transverse reinforcement having cover not exceeding 100mm and spacing not exceeding 300mm shall be provided [ACI 18.7.5.7]

-If f'c exceeds 70MPa, special provisions apply [ACI 22.5.3.1]

Identification		Section Properties			Material Properties	
ID =	9-FB	h_b=	600 mm	OK!	f'_c=	28 MPa
Storey =	1,2,3,4	b_b=	250 mm	OK!	E_c=	24870 MPa
Grid, x=	1 - 5	L_b=	5500 mm		f_{yl}=	420 MPa
Grid, y=	NA	cv=	40 mm		f_{yt}=	420 MPa
		A_g=	150000 mm2		E_s=	200 GPa



0

Other input parameters	
λ=	1
min d 0	22.2 mm
min $d_{b,END2}$=	22.2 mm
ω_u=	58 kN/m

Proposed Shear Reinforcement					
END 1 (Plastic hinge 1)		CENTER (Elastic)		END 1 (Plastic hinge 1)	
ϕ =	9.5 mm	ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	2	No. legs=	2	No. legs=	2
s=	75 mm	s=	125 mm	s=	75 mm

Important notes:

-If using compression reinforcement, check requirements at the end of the spreadsheet.

-Check for requirements of max. spacing of longitudinal bars at the end of the spreadsheet.

-Lap splice locations must follow the requirements mentioned at the end of the spreadsheet.

-Check for necessary development lenghts at the end of the spreadsheet.

FORCES ACTING IN EACH SECTION								
END 1 =	1100 mm		CENTER =	3300 mm		END 2 =	1100 mm	
M_u^+=	112.00 kN-m		M_u^+=	121.00 kN-m		M_u^+=	112.00 kN-m	
M_u^-=	342.00 kN-m		M_u^-=	0.01 kN-m		M_u^-=	342.00 kN-m	
V_u=	216.00 kN	OK!	V_u=	216.00 kN	OK!	V_u=	216.00 kN	OK!
P_u=	88.00 kN	OK!	P_u=	88.00 kN	OK!	P_u=	88.00 kN	OK!
T_u=	1.00 kN-m	OK!	T_u=	1.00 kN	OK!	T_u=	1.00 kN	OK!

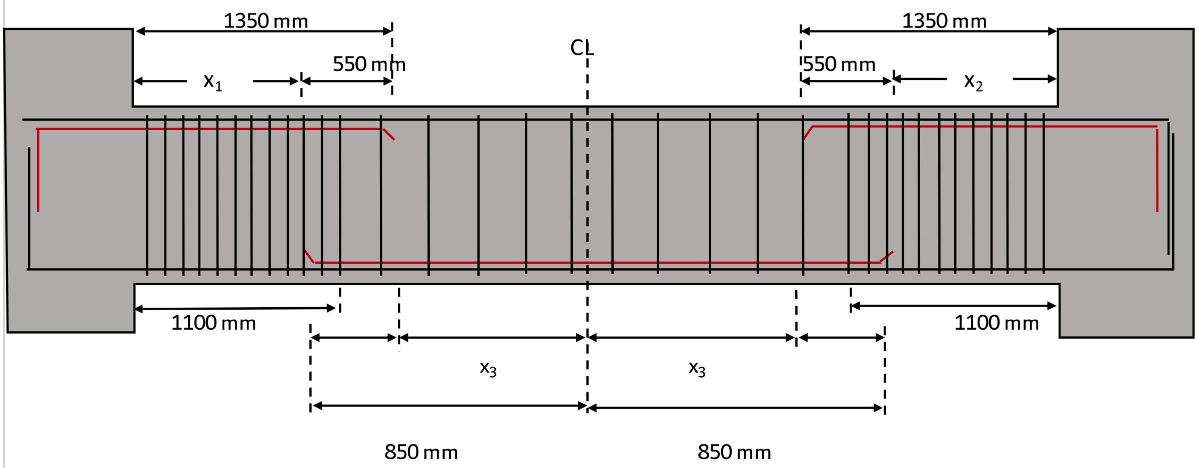
Flexural Reinforcement:					Flexural Reinforcement:					Flexural Reinforcement:				
no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)
3	19	0	0	50	3	19	0	0	50	3	19	0	0	50
2	22	0	0	94	2	22	0	0	550	2	22	0	0	94
2	22	0	0	550	0	0	0	0	0	2	22	0	0	550
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

MOMENT DESIGN								
END 1			CENTER			END 2		
Press for equilibrium						OK! All sections in equilibrium		
ϕ^+ =	0.9		ϕ^+ =	0.9		ϕ^+ =	0.9	
ϕ^- =	0.9		ϕ^- =	0.9		ϕ^- =	0.9	
ϕM_n^+ =	177 kN-m	OK!	ϕM_n^+ =	170 kN-m	OK!	ϕM_n^+ =	177 kN-m	OK!
ϕM_n^- =	333 kN-m	OK!	ϕM_n^- =	188 kN-m	OK!	ϕM_n^- =	333 kN-m	OK!
$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2	

END 1			CENTER			END 2		
$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2	
$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2	
A_s^+=	1548 mm2	OK!	A_s^+=	774 mm2	OK!	A_s^+=	1548 mm2	OK!
A_s^-=	1634 mm2	OK!	A_s^-=	860 mm2	OK!	A_s^-=	1634 mm2	OK!
$M_u^+/\phi M_n^+$ =	0.63	OK!	$M_u^+/\phi M_n^+$ =	0.71	OK!	$M_u^+/\phi M_n^+$ =	0.63	OK!
$M_u^-/\phi M_n^-$ =	1.03	OK!	$M_u^-/\phi M_n^-$ =	0.00	OK!	$M_u^-/\phi M_n^-$ =	1.03	OK!

SHEAR DESIGN								
END 1			CENTER			END 2		
ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2	ϕ = 9.5 mm	No. legs=	2
A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!	A_{sh}=	142 mm2	OK!
s=	75 mm	OK!	s=	125 mm	OK!	s=	75 mm	OK!
s_{max}=	133 mm		s_{max}=	138 mm		s_{max}=	133 mm	
λ=	1		λ=	1		λ=	1	
V_c=	0 kN		V_c=	131 kN		V_c=	0 kN	
V_s=	437 kN		V_s=	262 kN		V_s=	437 kN	
ϕV_n=	327 kN		ϕV_n=	295 kN		ϕV_n=	327 kN	
ω_u=	58 kN/m		ω_u=	58 kN/m		ω_u=	58 kN/m	
$V_{e,max}$=	310 kN		$V_{e,max}$=	246 kN		$V_{e,max}$=	310 kN	
$V_{e,min}$=	-12 kN		$V_{e,min}$=	53 kN		$V_{e,min}$=	-12 kN	
V_{dis}=	310 kN		V_{dis}=	246 kN		V_{dis}=	310 kN	
$V_{dis}/\phi V_n$=	0.95	OK!	$V_{dis}/\phi V_n$=	0.83	OK!	$V_{dis}/\phi V_n$=	0.95	OK!

DEVELOPMENT LENGHTS AND SPLICES					
END 1		CENTER		END 2	
1. Additional reinforcement		**1. Additional reinforcement**		**1. Additional reinforcement**	
d_b=	22.2 mm	d_b=	19.1 mm	d_b=	22.2 mm
x_1=	800 mm	x_3=	300 mm	x_2=	800 mm
λ=	1	λ=	1	λ=	1
ψ_e=	1	ψ_e=	1	ψ_e=	1
ψ_s=	1	ψ_s=	0.8	ψ_s=	1
ψ_t=	1.3	ψ_t=	1	ψ_t=	1.3
A_{tr}=	142 mm2	A_{tr}=	142 mm2	A_{tr}=	142 mm2
c_b=	50 mm	c_b=	50 mm	c_b=	50 mm
s=	75 mm	s=	125 mm	s=	75 mm
n=	2	n=	1	n=	2
k_{tr}=	38	k_{tr}=	45	k_{tr}=	38
$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5
ξ=	3.25	ξ=	2.5	ξ=	3.25
l_d=	1061 mm	l_d=	702 mm	l_d=	1061 mm
l_{bast}=	1350 mm	l_{bast}=	850 mm	l_{bast}=	1350 mm
2. Development length into column		**2. Development length into column**		**2. Development length into column**	
max d_b=	22.2 mm	max d_b=	22.2 mm	max d_b=	22.2 mm
l_{dh}=	326 mm	l_{dh}=	326 mm	l_{dh}=	326 mm



MAXIMUN SPACING OF REINFORCEMENT IN TENSION FACE AND SKIN REINFORCEMENT			
Tension Face		Skin reinforcement	
c_c=	40 mm	c_c=	40 mm
f_s=	280 MPa	f_s=	280 MPa
max spacing=	279 mm	max spacing=	280 mm

LATERAL SUPPORT OF COMPRESSION REINFORCEMENT

Notes for support of compression reinforcement (this applies only when compression is taken into account):

1. Minimun size:

 No.10 if longitudinal bars are No32 or less.

 No.13 if longitudinal bars are No36 or more, and for bundles.

2. Maximun spacing:

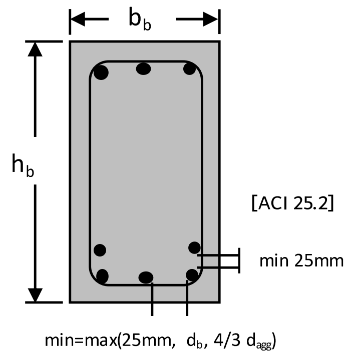
 $d_{b,long}$= 19.1 mm

 $d_{b,trans}$= 9.5 mm

 s_{max}= 250 mm

3. Longitudinal compression reinforcement shall be arranged in such a manner that every corner and alternate bar be enclosed by the corner of the transverse reinforcement with an included angle of 135 degrees or less. No bar shall be farther than 150mm clear on each side along the transverse reinforcement from such an enclosed bar.

Identification		Section Properties			Material Properties	
ID =	9-FB	h_b=	600 mm	OK!	f'_c=	28 MPa
Storey =	5,6,7	b_b=	250 mm	OK!	E_c=	24870 MPa
Grid, x=	1 - 5	L_b=	5500 mm		f_{yl}=	420 MPa
Grid, y=	NA	cv=	40 mm		f_{yt}=	420 MPa
		A_g=	150000 mm2		E_s=	200 GPa



0

Other input parameters		
λ=		1
min d	0	22.2 mm
min $d_{b,END2}$=		22.2 mm
ω_u=		58 kN/m

Proposed Shear Reinforcement					
END 1 (Plastic hinge 1)		CENTER (Elastic)		END 1 (Plastic hinge 1)	
ϕ =	9.5 mm	ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	2	No. legs=	2	No. legs=	2
s=	75 mm	s=	125 mm	s=	75 mm

Important notes:

-If using compression reinforcement, check requirements at the end of the spreadsheet.

-Check for requirements of max. spacing of longitudinal bars at the end of the spreadsheet.

-Lap splice locations must follow the requirements mentioned at the end of the spreadsheet.

-Check for necessary development lenghts at the end of the spreadsheet.

FORCES ACTING IN EACH SECTION

END 1 =	1100 mm		CENTER =	3300 mm		END 2 =	1100 mm	
M_u^+=	88.00 kN-m		M_u^+=	106.00 kN-m		M_u^+=	88.00 kN-m	
M_u^-=	316.00 kN-m		M_u^-=	0.01 kN-m		M_u^-=	316.00 kN-m	
V_u=	216.00 kN	OK!	V_u=	216.00 kN	OK!	V_u=	216.00 kN	OK!
P_u=	88.00 kN	OK!	P_u=	88.00 kN	OK!	P_u=	88.00 kN	OK!
T_u=	1.00 kN-m	OK!	T_u=	1.00 kN	OK!	T_u=	1.00 kN	OK!

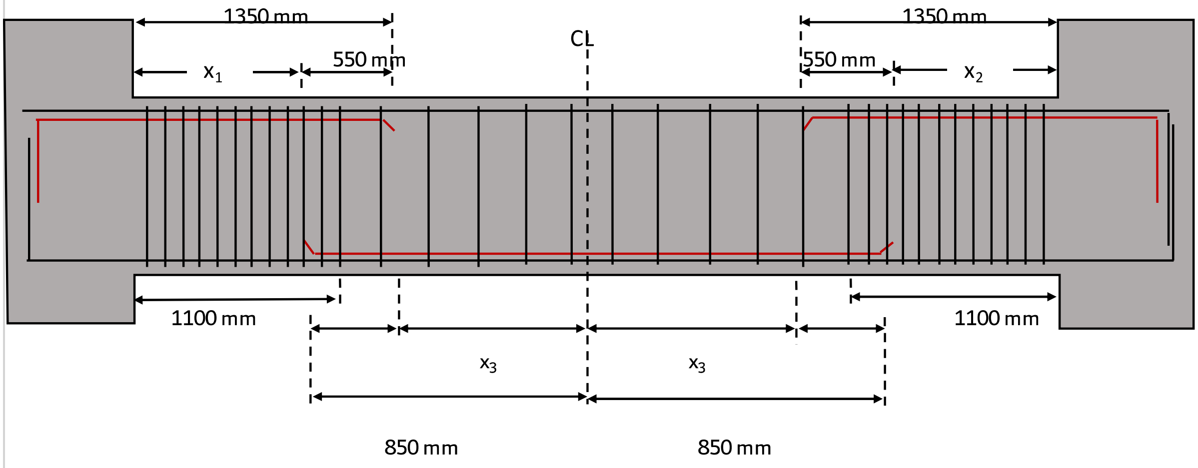
Flexural Reinforcement:					Flexural Reinforcement:					Flexural Reinforcement:				
no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)
2	22	0	0	50	2	22	0	0	50	2	22	0	0	50
2	22	0	0	94	2	22	0	0	550	2	22	0	0	94
2	22	0	0	550	0	0	0	0	0	2	22	0	0	550
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

MOMENT DESIGN								
END 1			CENTER			END 2		
Press for equilibrium						OK! All sections in equilibrium		
ϕ^+ =	0.9		ϕ^+ =	0.9		ϕ^+ =	0.9	
ϕ^- =	0.9		ϕ^- =	0.9		ϕ^- =	0.9	
ϕM_n^+ =	176 kN-m	OK!	ϕM_n^+ =	170 kN-m	OK!	ϕM_n^+ =	176 kN-m	OK!
ϕM_n^- =	316 kN-m	OK!	ϕM_n^- =	170 kN-m	OK!	ϕM_n^- =	316 kN-m	OK!
$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2		$A_s^+{}_{,min}$=	458 mm2	

END 1			CENTER			END 2		
$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2		$A_s^-{}_{,min}$=	458 mm2	
$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2		$A_{s,max}$=	3438 mm2	
A_s^+=	1548 mm2	OK!	A_s^+=	774 mm2	OK!	A_s^+=	1548 mm2	OK!
A_s^-=	1548 mm2	OK!	A_s^-=	774 mm2	OK!	A_s^-=	1548 mm2	OK!
$M_u^+/\phi M_n^+$ =	0.50	OK!	$M_u^+/\phi M_n^+$ =	0.62	OK!	$M_u^+/\phi M_n^+$ =	0.50	OK!
$M_u^-/\phi M_n^-$ =	1.00	OK!	$M_u^-/\phi M_n^-$ =	0.00	OK!	$M_u^-/\phi M_n^-$ =	1.00	OK!

SHEAR DESIGN											
END 1				CENTER				END 2			
ϕ =	9.5 mm	No. legs=	2	ϕ =	9.5 mm	No. legs=	2	ϕ =	9.5 mm	No. legs=	2
A_{sh}=	142 mm2		OK!	A_{sh}=	142 mm2		OK!	A_{sh}=	142 mm2		OK!
s=	75 mm		OK!	s=	125 mm		OK!	s=	75 mm		OK!
s_{max}=	133 mm			s_{max}=	138 mm			s_{max}=	133 mm		
λ=	1			λ=	1			λ=	1		
V_c=	0 kN			V_c=	130 kN			V_c=	0 kN		
V_s=	437 kN			V_s=	262 kN			V_s=	437 kN		
ϕV_n=	327 kN			ϕV_n=	294 kN			ϕV_n=	327 kN		
ω_u=	58 kN/m			ω_u=	58 kN/m			ω_u=	58 kN/m		
$V_{e,max}$=	302 kN			$V_{e,max}$=	238 kN			$V_{e,max}$=	302 kN		
$V_{e,min}$=	-19 kN			$V_{e,min}$=	45 kN			$V_{e,min}$=	-19 kN		
V_{dis}=	302 kN			V_{dis}=	238 kN			V_{dis}=	302 kN		
$V_{dis}/\phi V_n$=	0.92		OK!	$V_{dis}/\phi V_n$=	0.81		OK!	$V_{dis}/\phi V_n$=	0.92		OK!

DEVELOPMENT LENGHTS AND SPLICES					
END 1		CENTER		END 2	
1. Additional reinforcement		**1. Additional reinforcement**		**1. Additional reinforcement**	
d_b=	22.2 mm	d_b=	19.1 mm	d_b=	22.2 mm
x_1=	800 mm	x_3=	300 mm	x_2=	800 mm
λ=	1	λ=	1	λ=	1
ψ_e=	1	ψ_e=	1	ψ_e=	1
ψ_s=	1	ψ_s=	0.8	ψ_s=	1
ψ_t=	1.3	ψ_t=	1	ψ_t=	1.3
A_{tr}=	142 mm2	A_{tr}=	142 mm2	A_{tr}=	142 mm2
c_b=	50 mm	c_b=	50 mm	c_b=	50 mm
s=	75 mm	s=	125 mm	s=	75 mm
n=	2	n=	1	n=	2
k_{tr}=	38	k_{tr}=	45	k_{tr}=	38
$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5
ξ=	3.25	ξ=	2.5	ξ=	3.25
l_d=	1061 mm	l_d=	702 mm	l_d=	1061 mm
l_{bast}=	1350 mm	l_{bast}=	850 mm	l_{bast}=	1350 mm
2. Development length into column		**2. Development length into column**		**2. Development length into column**	
max d_b=	22.2 mm	max d_b=	22.2 mm	max d_b=	22.2 mm
l_{dh}=	326 mm	l_{dh}=	326 mm	l_{dh}=	326 mm



MAXIMUN SPACING OF REINFORCEMENT IN TENSION FACE AND SKIN REINFORCEMENT				
Tension Face		Skin reinforcement		
c_c=	39 mm	c_c=	40 mm	
f_s=	280 MPa	f_s=	280 MPa	
max spacing=	283 mm	max spacing=	280 mm	

LATERAL SUPPORT OF COMPRESSION REINFORCEMENT

Notes for support of compression reinforcement (this applies only when compression is taken into account):

1. Minimun size:

No.10 if longitudinal bars are No32 or less.

No.13 if longitudinal bars are No36 or more, and for bundles.

2. Maximun spacing:

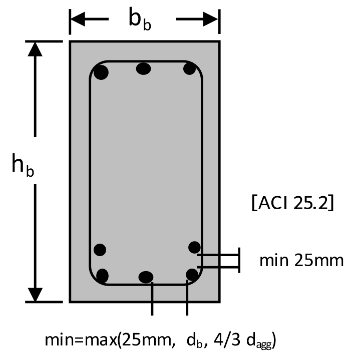
$d_{b,long}$= 19.1 mm

$d_{b,trans}$= 9.5 mm

s_{max}= 250 mm

3. Longitudinal compression reinforcement shall be arranged in such a manner that every corner and alternate bar be enclosed by the corner of the transverse reinforcement with an included angle of 135 degrees or less. No bar shall be farther than 150mm clear on each side along the transverse reinforcement from such an enclosed bar.

Identification		Section Properties			Material Properties	
ID =	9-FB	h_b=	600 mm	OK!	f'_c=	28 MPa
Storey =	8,9	b_b=	250 mm	OK!	E_c=	24870 MPa
Grid, x=	1 - 5	L_b=	5500 mm		f_{yl}=	420 MPa
Grid, y=	NA	cv=	40 mm		f_{yt}=	420 MPa
		A_g=	150000 mm2		E_s=	200 GPa



0

Other input parameters		
λ=		1
min d	0	19.1 mm
min $d_{b,END2}$=		19.1 mm
ω_u=		58 kN/m

Proposed Shear Reinforcement					
END 1 (Plastic hinge 1)		CENTER (Elastic)		END 1 (Plastic hinge 1)	
ϕ =	9.5 mm	ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	2	No. legs=	2	No. legs=	2
s=	75 mm	s=	125 mm	s=	75 mm

Important notes:

-If using compression reinforcement, check requirements at the end of the spreadsheet.

-Check for requirements of max. spacing of longitudinal bars at the end of the spreadsheet.

-Lap splice locations must follow the requirements mentioned at the end of the spreadsheet.

-Check for necessary development lenghts at the end of the spreadsheet.

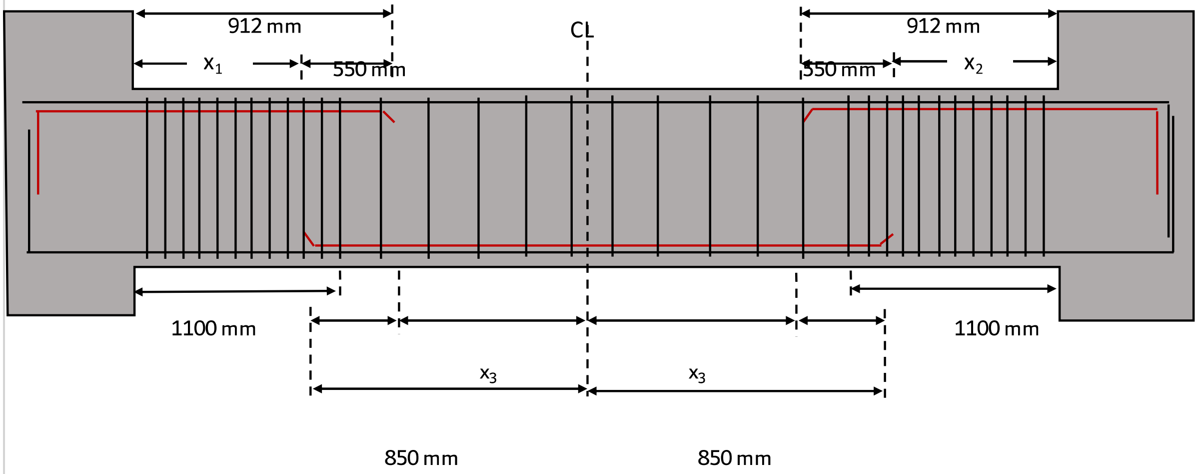
FORCES ACTING IN EACH SECTION														
END 1 =		1100 mm			CENTER =		3300 mm			END 2 =		1100 mm		
M_u^+=	50.00 kN-m				M_u^+=	101.00 kN-m				M_u^+=	50.00 kN-m			
M_u^-=	263.00 kN-m				M_u^-=	0.01 kN-m				M_u^-=	263.00 kN-m			
V_u=	201.00 kN			OK!	V_u=	201.00 kN			OK!	V_u=	201.00 kN			OK!
P_u=	13.00 kN			OK!	P_u=	13.00 kN			OK!	P_u=	13.00 kN			OK!
T_u=	1.00 kN-m			OK!	T_u=	1.00 kN			OK!	T_u=	1.00 kN			OK!
Flexural Reinforcement:					Flexural Reinforcement:					Flexural Reinforcement:				
no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)	no.	ϕ	no.	ϕ	d (mm)
2	22	0	0	50	2	22	0	0	50	2	22	0	0	50
2	19	0	0	94	2	22	0	0	550	2	19	0	0	94
2	22	0	0	550	0	0	0	0	0	2	22	0	0	550
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

MOMENT DESIGN								
END 1			CENTER			END 2		
Press for equilibrium						OK! All sections in equilibrium		
$\phi^+ =$	0.9		$\phi^+ =$	0.9		$\phi^+ =$	0.9	
$\phi^- =$	0.9		$\phi^- =$	0.9		$\phi^- =$	0.9	
$\phi M_n^+ =$	175 kN-m	OK!	$\phi M_n^+ =$	170 kN-m	OK!	$\phi M_n^+ =$	175 kN-m	OK!
$\phi M_n^- =$	278 kN-m	OK!	$\phi M_n^- =$	170 kN-m	OK!	$\phi M_n^- =$	278 kN-m	OK!
$A_s^+{}_{,min} =$	458 mm2		$A_s^+{}_{,min} =$	458 mm2		$A_s^+{}_{,min} =$	458 mm2	

END 1			CENTER			END 2		
$A_s^-{}_{,min} =$	458 mm2		$A_s^-{}_{,min} =$	458 mm2		$A_s^-{}_{,min} =$	458 mm2	
$A_{s,max} =$	3438 mm2		$A_{s,max} =$	3438 mm2		$A_{s,max} =$	3438 mm2	
$A_s^+ =$	1341 mm2	OK!	$A_s^+ =$	774 mm2	OK!	$A_s^+ =$	1341 mm2	OK!
$A_s^- =$	1341 mm2	OK!	$A_s^- =$	774 mm2	OK!	$A_s^- =$	1341 mm2	OK!
$M_u^+/\phi M_n^+ =$	0.29	OK!	$M_u^+/\phi M_n^+ =$	0.59	OK!	$M_u^+/\phi M_n^+ =$	0.29	OK!
$M_u^-/\phi M_n^- =$	0.95	OK!	$M_u^-/\phi M_n^- =$	0.00	OK!	$M_u^-/\phi M_n^- =$	0.95	OK!

SHEAR DESIGN								
END 1			CENTER			END 2		
$\phi =$ 9.5 mm	No. legs=	2	$\phi =$ 9.5 mm	No. legs=	2	$\phi =$ 9.5 mm	No. legs=	2
$A_{sh} =$	142 mm2	OK!	$A_{sh} =$	142 mm2	OK!	$A_{sh} =$	142 mm2	OK!
s=	75 mm	OK!	s=	125 mm	OK!	s=	75 mm	OK!
$s_{max} =$	115 mm		$s_{max} =$	138 mm		$s_{max} =$	115 mm	
$\lambda =$	1		$\lambda =$	1		$\lambda =$	1	
$V_c =$	0 kN		$V_c =$	130 kN		$V_c =$	0 kN	
$V_s =$	437 kN		$V_s =$	262 kN		$V_s =$	437 kN	
$\phi V_n =$	327 kN		$\phi V_n =$	294 kN		$\phi V_n =$	327 kN	
$\omega_u =$	58 kN/m		$\omega_u =$	58 kN/m		$\omega_u =$	58 kN/m	
$V_{e,max} =$	286 kN		$V_{e,max} =$	221 kN		$V_{e,max} =$	286 kN	
$V_{e,min} =$	-36 kN		$V_{e,min} =$	28 kN		$V_{e,min} =$	-36 kN	
$V_{dis} =$	286 kN		$V_{dis} =$	221 kN		$V_{dis} =$	286 kN	
$V_{dis}/\phi V_n =$	0.87	OK!	$V_{dis}/\phi V_n =$	0.75	OK!	$V_{dis}/\phi V_n =$	0.87	OK!

DEVELOPMENT LENGHTS AND SPLICES					
END 1		CENTER		END 2	
1. Additional reinforcement		**1. Additional reinforcement**		**1. Additional reinforcement**	
d_b=	19.1 mm	d_b=	19.1 mm	d_b=	19.1 mm
x_1=	300 mm	x_3=	300 mm	x_2=	300 mm
λ=	1	λ=	1	λ=	1
ψ_e=	1	ψ_e=	1	ψ_e=	1
ψ_s=	0.8	ψ_s=	0.8	ψ_s=	0.8
ψ_t=	1.3	ψ_t=	1	ψ_t=	1.3
A_{tr}=	142 mm2	A_{tr}=	142 mm2	A_{tr}=	142 mm2
c_b=	50 mm	c_b=	50 mm	c_b=	50 mm
s=	75 mm	s=	125 mm	s=	75 mm
n=	2	n=	1	n=	2
k_{tr}=	38	k_{tr}=	45	k_{tr}=	38
$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5	$(c_b+k_{tr})/d_b$=	2.5
ξ=	3.25	ξ=	2.5	ξ=	3.25
l_d=	912 mm	l_d=	702 mm	l_d=	912 mm
l_{bast}=	912 mm	l_{bast}=	850 mm	l_{bast}=	912 mm
2. Development length into column		**2. Development length into column**		**2. Development length into column**	
max d_b=	22.2 mm	max d_b=	22.2 mm	max d_b=	22.2 mm
l_{dh}=	326 mm	l_{dh}=	326 mm	l_{dh}=	326 mm



MAXIMUN SPACING OF REINFORCEMENT IN TENSION FACE AND SKIN REINFORCEMENT			
Tension Face		Skin reinforcement	
c_c=	39 mm	c_c=	40 mm
f_s=	280 MPa	f_s=	280 MPa
max spacing=	283 mm	max spacing=	280 mm

LATERAL SUPPORT OF COMPRESSION REINFORCEMENT

Notes for support of compression reinforcement (this applies only when compression is taken into account):

1. Minimun size:

    No.10 if longitudinal bars are No32 or less.

    No.13 if longitudinal bars are No36 or more, and for bundles.

2. Maximun spacing:

    $d_{b,long}$= 19.1 mm

    $d_{b,trans}$= 9.5 mm

    s_{max}= 250 mm

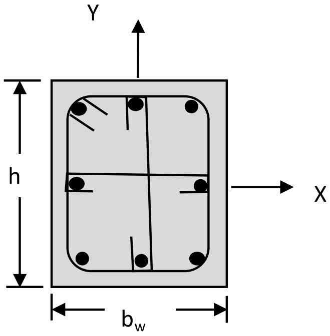
3. Longitudinal compression reinforcement shall be arranged in such a manner that every corner and alternate bar be enclosed by the corner of the transverse reinforcement with an included angle of 135 degrees or less. No bar shall be farther than 150mm clear on each side along the transverse reinforcement from such an enclosed bar.

Identification	
ID =	9-FB
Storey =	1,2,3,4
Grid, x=	1-5
Grid, y=	NA

Material Prop.	
f'_c=	28 MPa
E_c=	24870 MPa
f_{yl}=	420 MPa
f_{yt}=	420 MPa
E_s=	200 GPa
λ=	1

Section Properties	
h=	500 mm
b_w=	500 mm
l_w=	3000 mm
c_c=	40 mm
A_g=	250000 mm2
I_{xx}=	5E+09 mm4
I_{yy}=	5E+09 mm4
d_{xx}=	450.00 mm
d_{yy}=	450.00 mm
h_x=	197 mm
ϕ_{min}=	22 mm
n_l=	8
A_{ch}=	176400 mm2
d_{agg}=	25 mm

Proposed Shear Reinforcement			
XX-Axis		YY-Axis	
Plastic hinge region		Plastic hinge region	
ϕ =	12.7 mm	ϕ =	12.7 mm
No. legs=	3	No. legs=	3
s=	65 mm	s=	65 mm
Ash=	380 mm2	Ash=	380 mm2
Other regions		Other regions	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	125 mm	s=	125 mm
Ash=	213 mm2	Ash=	213 mm2
Pu and Mu corresponding to Vu in x and y			
Pu=	3051 kN	Pu=	0 kN
Mu=	316 kN	Mu=	0 kN



Axial Force		Shear Force in x		Shear Force in y		Axial Force from EQ	
P_D=	1894 kN	V_{Dx}=	20 kN	V_{Dy}=	0 kN	P^+_{Ex}=	350 kN
P_L=	778 kN	V_{Lx}=	9 kN	V_{Ly}=	0 kN	P^-_{Ex}=	-350 kN
P_u=	3051 kN	V_{Ex}=	136 kN	V_{Ey}=	0 kN	P^+_{Ey}=	0 kN
		V_{ux}=	169 kN	V_{uy}=	0 kN	P^-_{Ey}=	0 kN

In equilibrium!

PRESS BUTTON FOR EQUILIBRIUM

Flexural Reinforcement xx:						Flexural Reinforcement yy:					
no.	ϕ	no.	ϕ	d (mm)	Ast	no.	ϕ	no.	ϕ	d (mm)	Ast
2	22	1	22	50	###	2	22	1	22	50	###
2	22	0	0	250	774	2	22	0	0	250	774
2	22	1	22	450	###	2	22	1	22	450	###
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Moment capacity of columns and beams		
Top joint	xx-axis	yy-axis
Mprc1=	520	548
Mprc2=	480	501
Mprb1=	410	410
Mprb2=	410	410
Bot joint	xx-axis	yy-axis
Mprc1=	520	548
Mprc2=	480	501
Mprb1=	390	390
Mprb2=	390	390

Dimension and longitudinal reinforcement limits

$b_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
$h_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
b_w/h_w or h_w/b_w=		1.00	>=	0.4	OK!	[ACI 18.7.2.1 (b)]
A_{st}=	3097 mm2				OK!	[ACI 18.7.4.1]

Transverse reinforcement in plastic hinge region

l_o=	500 mm		[ACI 18.7.5.1]
$h_{x,max}$=	200 mm	OK!	[ACI 18.7.5.2, e and f]
s_{max}=	125 mm	OK!	[ACI 18.7.5.3]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
$A_{sh}/sb_{c,x}$=	1.17%	OK!	[ACI 18.7.5.4]
$A_{sh}/sb_{c,y}$=	1.17%	OK!	[ACI 18.7.5.4]

Transverse reinforcement in other regions

s_{max}=	133 mm	OK!	[ACI 18.7.5.5]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
ϕ_{min}=	9.5 mm	OK!	[ACI 25.7.2.2]

Shear force check

Vux max=	1141 kN	OK!	[ACI 22.5.1.2]
Vuy max=	848 kN	OK!	[ACI 22.5.1.2]

Bi-axial moment capacity

$$\left(Mu,xx/_{Mn,xx}\right)^2 + \left(Mu,yy/_{Mn,yy}\right)^2 =$$ 0.968665 OK!

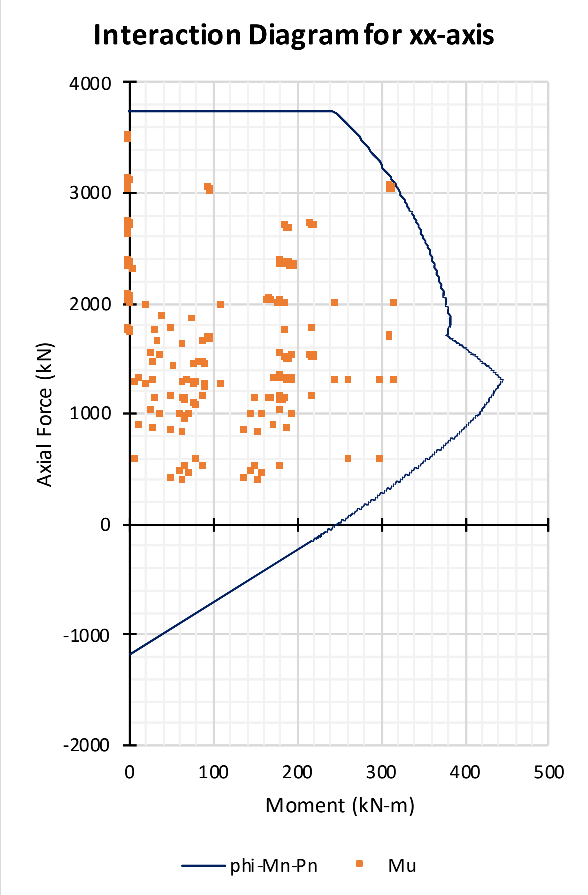
Shear capacity

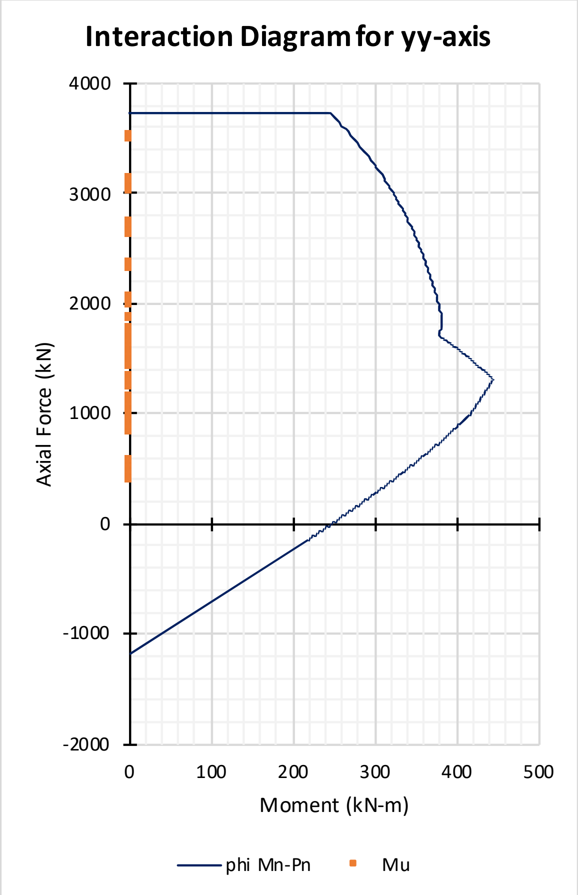
xx-axis			yy-axis		
	Plastic hinge region	Other		Plastic hinge region	Other
ϕVn=	1380 kN	793 kN	Vs=	1088 kN	500 kN
Vu/ϕVn=	0.28	0.48	Vu/ϕVn=	0.34	0.75

Check
OK!

Strong column-weak beam check [ACI 18.7.3.2]

Joint	xx-axis				yy-axis			
	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check
Top	1000	820	1.22	OK!	1049	820	1.28	OK!
Bottom	1000	780	1.28	OK!	1049	780	1.35	OK!





Important notes:

-Combinations used for earthquake: 1.2D+1.0L+/-1.0E and 0.9D+/-1.0E **If different change Cap sheets accordingly

-Only symmetrical reinforcement is supported in the same axis.

-Lap splices outside lo region, the zone with lap-splice should comply with confinement reinforcement as in hinge region.

-Special provisions apply if column supports reactions from discontinued stiff members. [ACI 18.7.5.6]

-When using mechanical or welded splices check [ACI 18.7.4.3].

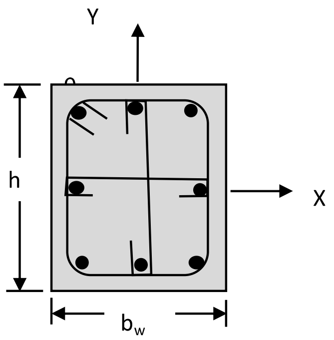
-Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees.

-No unsupported bar shall be farther than 150mm clear on each side along the tie form a laterally supported bar.

-If concrete cover exceeds 100mm, additional transverse reinforcement having cover not exceeding 100mm and spacing not exceeding 300mm shall be provided [ACI 18.7.5.7]

-If f'c exceeds 70MPa, special provisions apply [ACI 22.5.3.1]

Identification		Material Prop.		Section Properties	
ID =	9-FB	f'_c=	28 MPa	h=	500 mm
Storey =	5,6,7	E_c=	24870 MPa	b_w=	500 mm
Grid, x=	1-5	f_{yl}=	420 MPa	l_w=	3000 mm
Grid, y=	NA	f_{yt}=	420 MPa	c_c=	40 mm
		E_s=	200 GPa	A_g=	250000 mm2
0		λ=	1	I_{xx}=	5E+09 mm4
				I_{yy}=	5E+09 mm4
				d_{xx}=	450.00 mm
				d_{yy}=	450.00 mm
				h_x=	197 mm
				ϕ_{min}=	22 mm
				n_l=	8
				A_{ch}=	176400 mm2
				d_{agg}=	25 mm



Proposed Shear Reinforcement			
XX-Axis		YY-Axis	
Plastic hinge region		Plastic hinge region	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	50 mm	s=	50 mm
Ash=	213 mm2	Ash=	213 mm2
Other regions		Other regions	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	125 mm	s=	125 mm
Ash=	213 mm2	Ash=	213 mm2
Pu and Mu corresponding to Vu in x and y			
Pu=	1696 kN	Pu=	0 kN
Mu=	184 kN	Mu=	0 kN

Axial Force		Shear Force in x		Shear Force in y		Axial Force from EQ	
P_D=	1051 kN	V_{Dx}=	33 kN	V_{Dy}=	0 kN	P^+_{Ex}=	170 kN
P_L=	432 kN	V_{Lx}=	15 kN	V_{Ly}=	0 kN	P^-_{Ex}=	-133 kN
P_u=	1696 kN	V_{Ex}=	107 kN	V_{Ey}=	0 kN	P^+_{Ey}=	0 kN
		V_{ux}=	162 kN	V_{uy}=	0 kN	P^-_{Ey}=	0 kN

In equilibrium!

PRESS BUTTON FOR EQUILIBRIUM

Flexural Reinforcement xx:						Flexural Reinforcement yy:					
no.	ϕ	no.	ϕ	d (mm)	Ast	no.	ϕ	no.	ϕ	d (mm)	Ast
2	22	1	22	50	###	2	22	1	22	50	###
2	22	0	0	250	774	2	22	0	0	250	774
2	22	1	22	450	###	2	22	1	22	450	###
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Moment capacity of columns and beams		
Top joint	xx-axis	yy-axis
Mprc1=	471	484
Mprc2=	471	484
Mprb1=	390	390
Mprb2=	390	390
Bot joint	xx-axis	yy-axis
Mprc1=	471	484
Mprc2=	471	484
Mprb1=	390	390
Mprb2=	390	390

Dimension and longitudinal reinforcement limits

$b_{w,min}$=	300 mm			OK!	[ACI 18.7.2.1 (a)]
$h_{w,min}$=	300 mm			OK!	[ACI 18.7.2.1 (a)]
b_w/h_w or h_w/b_w=	1.00	>=	0.4	OK!	[ACI 18.7.2.1 (b)]
A_{st}=	3097 mm2			OK!	[ACI 18.7.4.1]

Transverse reinforcement in plastic hinge region

l_o=	500 mm		[ACI 18.7.5.1]
$h_{x,max}$=	350 mm	OK!	[ACI 18.7.5.2, e and f]
s_{max}=	125 mm	OK!	[ACI 18.7.5.3]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
$A_{sh}/sb_{c,x}$=	0.85%	OK!	[ACI 18.7.5.4]
$A_{sh}/sb_{c,y}$=	0.85%	OK!	[ACI 18.7.5.4]

Transverse reinforcement in other regions

s_{max}=	133 mm	OK!	[ACI 18.7.5.5]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
ϕ_{min}=	9.5 mm	OK!	[ACI 25.7.2.2]

Shear force check

Vux max=	1035 kN	OK!	[ACI 22.5.1.2]
Vuy max=	848 kN	OK!	[ACI 22.5.1.2]

Bi-axial moment capacity

$$\left(\frac{Mu,xx}{Mn,xx}\right)^2 + \left(\frac{Mu,yy}{Mn,yy}\right)^2 =$$ 0.285324 OK!

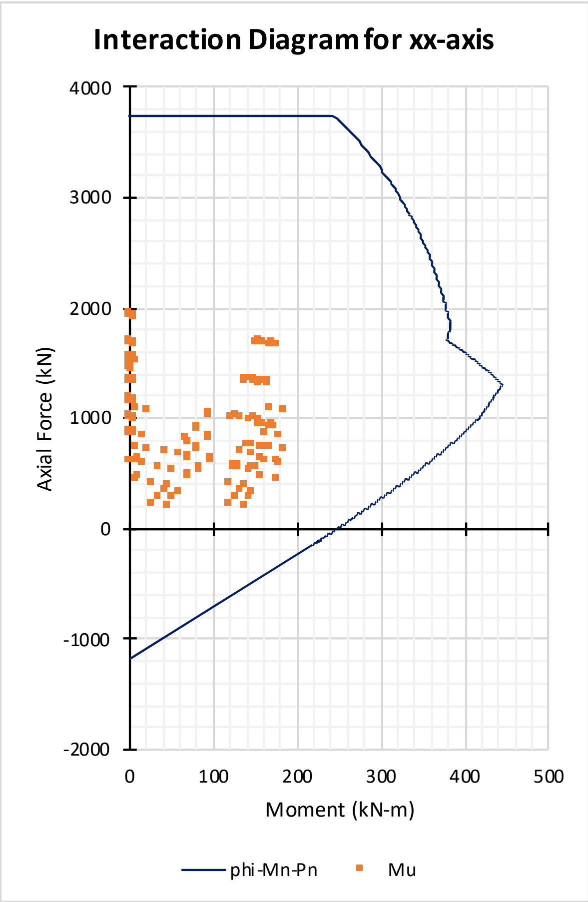
Shear capacity

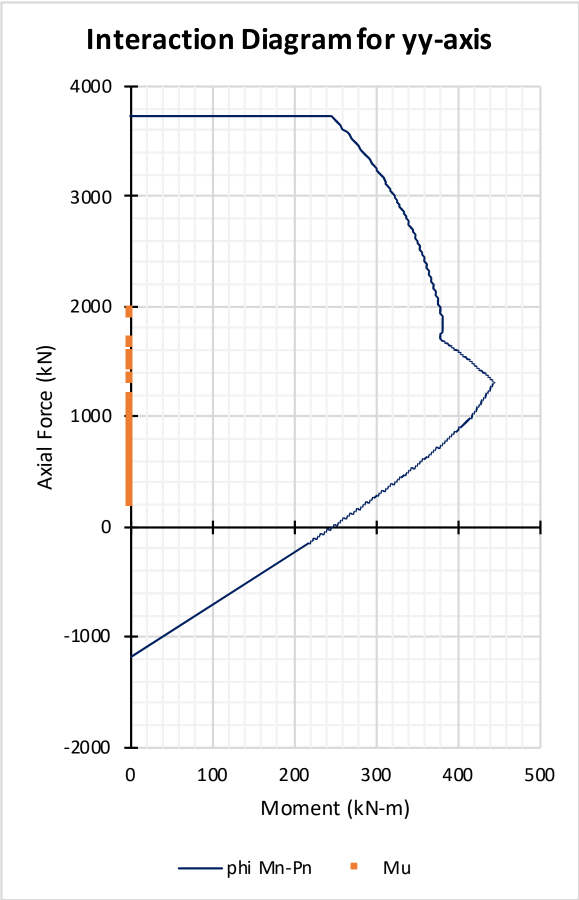
xx-axis				yy-axis			
Plastic hinge region		Other		Plastic hinge region		Other	
ϕVn=	1049 kN	687 kN		Vs=	862 kN	500 kN	
Vu/ϕVn=	0.36	0.54		Vu/ϕVn=	0.42	0.73	

Check
OK!

Strong column-weak beam check [ACI 18.7.3.2]

Joint	xx-axis				yy-axis			
	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check
Top	942	780	1.21	OK!	968	780	1.24	OK!
Bottom	942	780	1.21	OK!	968	780	1.24	OK!





Important notes:

-Combinations used for earthquake: 1.2D+1.0L+/-1.0E and 0.9D+/-1.0E **If different change Cap sheets accordingly

-Only symmetrical reinforcement is supported in the same axis.

-Lap splices outside lo region, the zone with lap-splice should comply with confinement reinforcement as in hinge region.

-Special provisions apply if column supports reactions from discontinued stiff members. [ACI 18.7.5.6]

-When using mechanical or welded splices check [ACI 18.7.4.3].

-Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees.

-No unsupported bar shall be farther than 150mm clear on each side along the tie form a laterally supported bar.

-If concrete cover exceeds 100mm, additional transverse reinforcement having cover not exceeding 100mm and spacing not exceeding 300mm shall be provided [ACI 18.7.5.7]

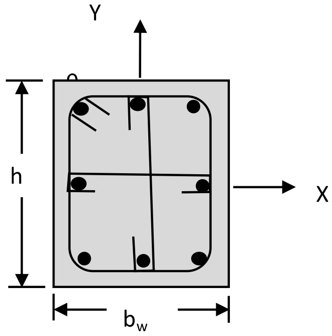
-If f'c exceeds 70MPa, special provisions apply [ACI 22.5.3.1]

Identification	
ID =	9-FB
Storey =	8,9
Grid, x=	1-5
Grid, y=	NA
0	

Material Prop.	
f'_c=	28 MPa
E_c=	24870 MPa
f_{yl}=	420 MPa
f_{yt}=	420 MPa
E_s=	200 GPa
λ=	1

Section Properties	
h=	500 mm
b_w=	500 mm
l_w=	3000 mm
c_c=	40 mm
A_g=	250000 mm2
I_{xx}=	5E+09 mm4
I_{yy}=	5E+09 mm4
d_{xx}=	450.00 mm
d_{yy}=	450.00 mm
h_x=	197 mm
ϕ_{min}=	22 mm
n_l=	8
A_{ch}=	176400 mm2
d_{agg}=	25 mm

Proposed Shear Reinforcement			
XX-Axis		YY-Axis	
Plastic hinge region		Plastic hinge region	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	50 mm	s=	50 mm
Ash=	213 mm2	Ash=	213 mm2
Other regions		Other regions	
ϕ =	9.5 mm	ϕ =	9.5 mm
No. legs=	3	No. legs=	3
s=	125 mm	s=	125 mm
Ash=	213 mm2	Ash=	213 mm2
Pu and Mu corresponding to Vu in x and y			
Pu=	678 kN	Pu=	0 kN
Mu=	106 kN	Mu=	0 kN



Axial Force		Shear Force in x		Shear Force in y		Axial Force from EQ	
P_D=	465 kN	V_{Dx}=	48 kN	V_{Dy}=	0 kN	P^+_{Ex}=	55 kN
P_L=	185 kN	V_{Lx}=	22 kN	V_{Ly}=	0 kN	P^-_{Ex}=	-15 kN
P_u=	678 kN	V_{Ex}=	77 kN	V_{Ey}=	0 kN	P^+_{Ey}=	0 kN
		V_{ux}=	157 kN	V_{uy}=	0 kN	P^-_{Ey}=	0 kN

In equilibrium!

PRESS BUTTON FOR EQUILIBRIUM

Flexural Reinforcement xx:						Flexural Reinforcement yy:					
no.	ϕ	no.	ϕ	d (mm)	Ast	no.	ϕ	no.	ϕ	d (mm)	Ast
2	22	1	22	50	###	2	22	1	22	50	###
2	22	0	0	250	774	2	22	0	0	250	774
2	22	1	22	450	###	2	22	1	22	450	###
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Moment capacity of columns and beams		
Top joint	xx-axis	yy-axis
Mprc1=	411	413
Mprc2=	413	413
Mprb1=	343	343
Mprb2=	343	343
Bot joint	xx-axis	yy-axis
Mprc1=	411	413
Mprc2=	413	413
Mprb1=	343	343
Mprb2=	343	343

Dimension and longitudinal reinforcement limits

$b_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
$h_{w,min}$=	300 mm				OK!	[ACI 18.7.2.1 (a)]
b_w/h_w or h_w/b_w=		1.00	>=	0.4	OK!	[ACI 18.7.2.1 (b)]
A_{st}=	3097 mm2				OK!	[ACI 18.7.4.1]

Transverse reinforcement in plastic hinge region

l_o=	500 mm		[ACI 18.7.5.1]
$h_{x,max}$=	350 mm	OK!	[ACI 18.7.5.2, e and f]
s_{max}=	125 mm	OK!	[ACI 18.7.5.3]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
$A_{sh}/sb_{c,x}$=	0.85%	OK!	[ACI 18.7.5.4]
$A_{sh}/sb_{c,y}$=	0.85%	OK!	[ACI 18.7.5.4]

Transverse reinforcement in other regions

s_{max}=	133 mm	OK!	[ACI 18.7.5.5]
s_{min}=	33 mm	OK!	[ACI 25.7.2.1]
ϕ_{min}=	9.5 mm	OK!	[ACI 25.7.2.2]

Shear force check

Vux max=	935 kN	OK!	[ACI 22.5.1.2]
Vuy max=	848 kN	OK!	[ACI 22.5.1.2]

Bi-axial moment capacity

$$\left(\frac{Mu,xx}{Mn,xx}\right)^2 + \left(\frac{Mu,yy}{Mn,yy}\right)^2 = \quad 0.564135 \quad \text{OK!}$$

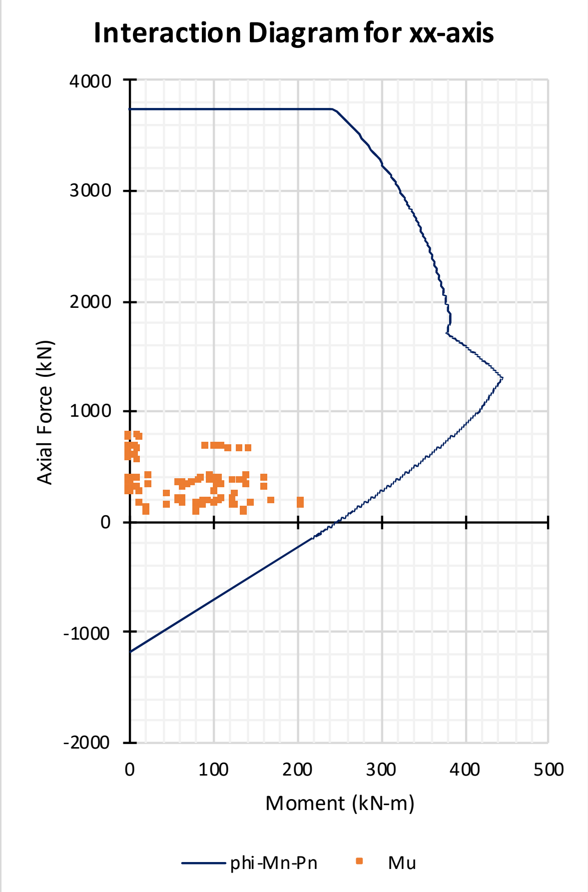
Shear capacity

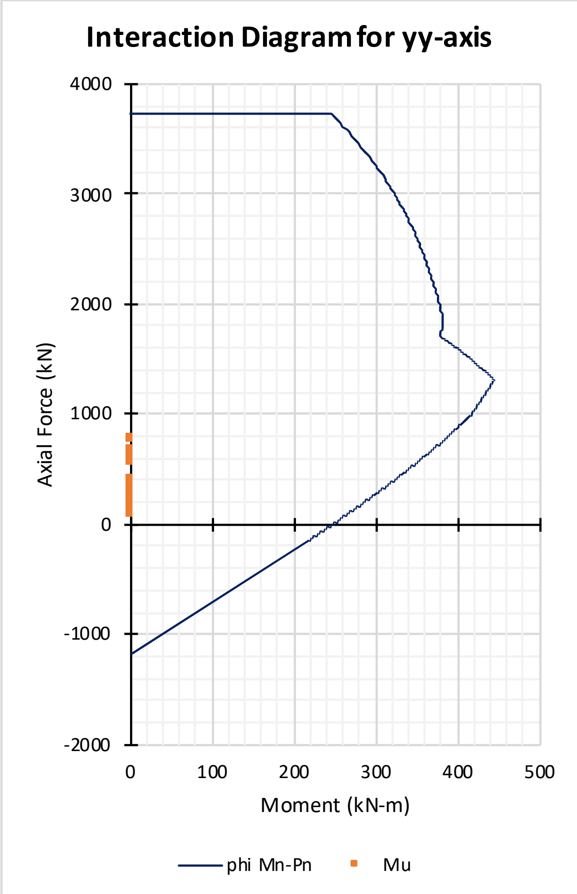
xx-axis				yy-axis			
Plastic hinge region		Other		Plastic hinge region		Other	
ϕVn=	949 kN	587 kN		Vs=	862 kN	500 kN	
Vu/ϕVn=	0.33	0.53		Vu/ϕVn=	0.36	0.62	

Check
OK!

Strong column-weak beam check [ACI 18.7.3.2]

Joint	xx-axis				yy-axis			
	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check	ΣMprc	ΣMprb	ΣMprc/ΣMprb	Check
Top	824	686	1.20	OK!	826	686	1.20	OK!
Bottom	824	686	1.20	OK!	826	686	1.20	OK!





Important notes:

-Combinations used for earthquake: 1.2D+1.0L+/-1.0E and 0.9D+/-1.0E **If different change Cap sheets accordingly

-Only symmetrical reinforcement is supported in the same axis.

-Lap splices outside lo region, the zone with lap-splice should comply with confinement reinforcement as in hinge region.

-Special provisions apply if column supports reactions from discontinued stiff members. [ACI 18.7.5.6]

-When using mechanical or welded splices check [ACI 18.7.4.3].

-Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees.

-No unsupported bar shall be farther than 150mm clear on each side along the tie form a laterally supported bar.

-If concrete cover exceeds 100mm, additional transverse reinforcement having cover not exceeding 100mm and spacing not exceeding 300mm shall be provided [ACI 18.7.5.7]

-If f'c exceeds 70MPa, special provisions apply [ACI 22.5.3.1]