Chapter

Perspective Chapter: Comprehensive and New Approximate Analysis and Design Techniques for Reinforced Concrete Structural Elements

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Abstract

In practical structural analysis and design scenarios, various software tools are commonly utilized. However, verifying the structural analysis and design can pose a significant challenge for many designers. To address this concern, the author has developed and proposed an innovative, simple, comprehensive, and reliable approximate structural analysis and design method. These methods aim to provide designers with valuable information on the final internal forces (axial/shear force, and bending moment), vertical deflections, lateral displacement/drift of buildings under lateral force, and approximate dimensions of all structural components prior to conducting software analysis and design. The preliminary estimation of beam and column dimensions may lead to an extensive trial and error process. Therefore, this study will introduce a new and reliable approximate structural analysis and design methodology using precise analytical techniques and software evaluations. This approach aims to determine approximate internal forces, establish preliminary structural dimensions, and validate the modeling, analysis, and design processes conducted through software. The methodology presented in this chapter has been applied to the analysis and design of various projects ranging from 5 to 15 stories, which were designed by the author in their capacity as the director of Alan Consulting Engineers. In addition, this chapter presents four case studies to assess the effectiveness and dependability of the proposed methods. The findings indicate discrepancies ranging from 2 to 12%.

Keywords: reinforced concrete structural systems, approximate loading, new approximate analysis techniques, and new approximate structural design, rigid frame, shear wall, coupled shear wall, foundations, spring

1. Introduction

It obvious that selecting materials, gravity and lateral bearing systems, floor type, and foundation types poses a significant challenge in real-world projects. To achieve an optimal design, building and technical facility design and analysis are typically conducted in two stages: phase I and phase II [1].

Phase 1. In the initial phase, the architectural plans are prepared and presented to the client, taking into consideration the land's characteristics, location, building usage, and existing buildings around the project. Simultaneously, in the structural sector, the construction technology, availability of materials, skilled workers, and designer's knowledge are taken into account to suggest the optimal materials, load bearing system, floor type, and foundation system to the employer.

In the structural section, the position of skeleton components is initially verified, and their dimensions are selected through an approximate analysis and design [1]. Approximate analysis pertains to structural analysis that employs simplifications in both modeling and loading conditions.

Phase 2. After obtaining approval for one of the proposed options in the initial phase, the complete details of the drawings are prepared for all four areas. In the structural division, a separate structural model of the load-bearing system, frame, shear wall, floors, and foundation system is created, analyzed, and designed based on the approved architectural drawings from the first phase. It is obvious that, the theoretical approach to the analysis of frames is evidently time-consuming, and the optimizations encounter various challenges; therefore, the analysis and design of the structural frame or frame with shear wall is carried out using ETABS/SAP. It is important to note that SAFE can be utilized for the analysis and design of the slabs and foundations. Additionally, it should be mentioned that ETABSv2016 is capable of slab design, although the design results are generally lower compared to safe. The overall project estimate is based on the phase II drawings and is provided to the client as part of the tender documents [1].

Phase 3. In the third phase, the employer conducts a tender to identify the eligible contractor, taking into account the plans and estimates developed in phase II. The chosen contractor, who may be supervised by consulting engineers from the public or private sector, or an engineer licensed to work in the private sector, is responsible for executing the construction project [1].

2. Phase 1: Conceptual design

In this phase, the initial action to be taken involves selecting an appropriate firm to carry out geotechnical investigations on behalf of the employer. The key outcomes of such investigations include determining the bearing capacity of the soil and identifying the soil type beneath the foundation. Given that the soil type impacts the lateral forces during seismic events and the bearing capacity directly influences the dimensions of the foundation, it is crucial to approach this step with utmost care and precision.

Construction projects commonly utilize steel and concrete as primary structural materials. Structural engineers are often tasked with determining the most suitable material for a project, weighing factors such as sustainability, cost, construction speed, technology, labor expertise, and material availability. Through careful consideration of these variables and discussions with stakeholders, the optimal material is ultimately chosen for the construction project.

After choosing the type of materials, for example, concrete, all structural systems suitable for sustaining the gravity and lateral loads, types of floors, and foundations must be examined by the structural engineer, and then the most optimal systems are selected. The only quantitative criterion that should be selected based on that type of load-bearing system is the height of the structure based on the relevant code or standard, while the rest of the criteria are qualitative and depend more on the engineer's judgment. **Figure 1** provides a general suggestion for choosing the type of load-bearing system for various heights. As can be seen from **Figure 1**, for low/medium rise buildings, three load-bearing systems: rigid frame, shear wall, or a combination of frame and shear wall, can be used. For tall buildings, the tube system is suitable (**Figure 2**) [3, 4].

Classification of load-bearing systems in terms of connection type:

- Rigid connection: Concrete and steel structures (**Figure 2a**)
- Pinned connection: Steel structures, precast concrete structures (**Figure 2b**)

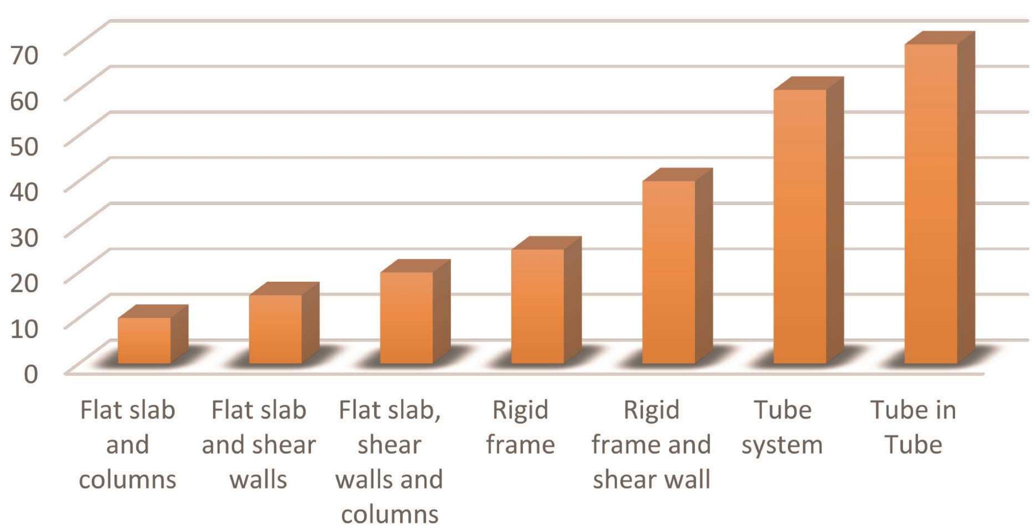


Figure 1.
Classification of structural systems based on the height of the structure.

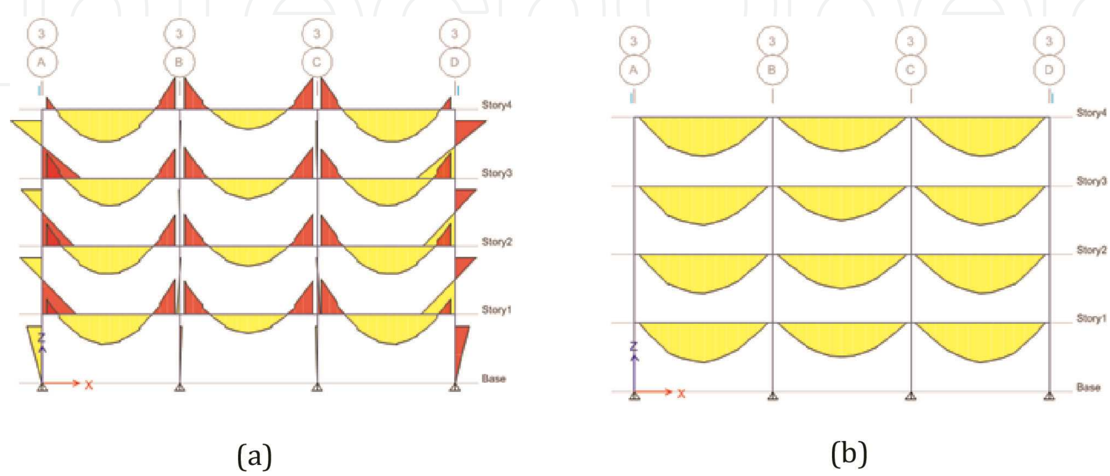


Figure 2.
Bending moment in structures with rigid and pinned connections under gravity loads using SAP 2000 [2].
(a) Rigid connections (b) Pin connections.

3. Structural analysis and design

To analyze and design a structure, the following steps need to be taken [2]:

Loading: The process of selecting and calculating the gravity and lateral loading, including wind or seismic forces, is crucial in the design of structures. It involves determining the appropriate variable, permanent, and lateral actions that act upon the structure.

Modeling (to build up computational models): To establish the geometric properties, material characteristics, cross-sectional profiles, and loading conditions and subsequently implement the determined loads onto the structural system.

Structural analysis: The impact of these actions on the structure, such as shear force, bending moment, axial forces, and deflection, will be examined through either manual or software analysis.

Verification: In order to validate a software model, it is essential to monitor the general lateral/vertical deflection, as well as the axial force/shear force/bending moment diagram under both gravity and lateral loads. These parameters must adhere to the precise specifications provided by the designer regarding deflection and internal forces, that is, axial/shear force and bending moment. To ensure the accuracy of the structural analysis results, it is necessary to employ approximate analysis techniques. If a significant disparity arises between the approximate analysis and the software-generated results, the designer must provide a clear justification; otherwise, the software results are deemed incorrect. It can be inferred that only engineers possessing extensive knowledge of manual analysis and design should utilize software for the analysis and design of structures.

Structural design and optimization: The detailed design involves conducting structural calculations to ascertain the dimensions, configuration, and specifications of structural elements and foundations, including reinforcement calculations where necessary. Adherence to the applicable Eurocodes for concrete, steel, timber, and masonry, as well as other relevant guidance documents, is essential. The design process encompasses a thorough examination of all elements to ensure accuracy and precision:

- Column, beam, and shear walls
- Beam-column connection
- Floor, that is, one-way, two-way slab, flat slab, and …
- Foundation, that is, mat, strip, pile or …
- Applying the ductility provisions for a medium or special frame

Drawing: Full details of Foundation, Columns, beams, connections, and slabs. *Preparation of tender drawings.*

Design risk assessment

Planning construction Identification of suitable forms of contract. Construction Programming and resource requirements. Outline cost analysis.

This chapter extensively covers the topics of loading and approximate analysis, which are considered powerful techniques for verifying software analysis and design.

3.1 Loading

After choosing a suitable structural system, the first step is to calculate the various loads on the structure. The minimum loads that should be considered in the design of structural systems are dead, live, snow, earthquake, and wind loads. Due to the complexity of wind and earthquake loads, in this section, only dead and live loads and a summary of wind and seismic are discussed. EC1 provides all the required details to calculate various actions, that is, dead/live (EC1-1), snow (EC1-3), and wind load (EC1-4) on the structure [5]. For seismic load, EC8 is used.

3.1.1 Dead load (permanent action)

Dead load refers to the weight of the building's fixed additions, including floors/roofs, structural weight, internal and peripheral walls, staircases, finishes, and the weight of mechanical and electrical installations. To determine the weight of each component, the subsequent steps must be followed:

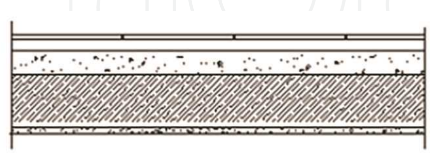
1. Providing a precise details of the elements (**Figures 3**-**6**)

2. Determining the unit mass of various materials given in codes (**Table 1**)

3. The amount of dead load is calculated as

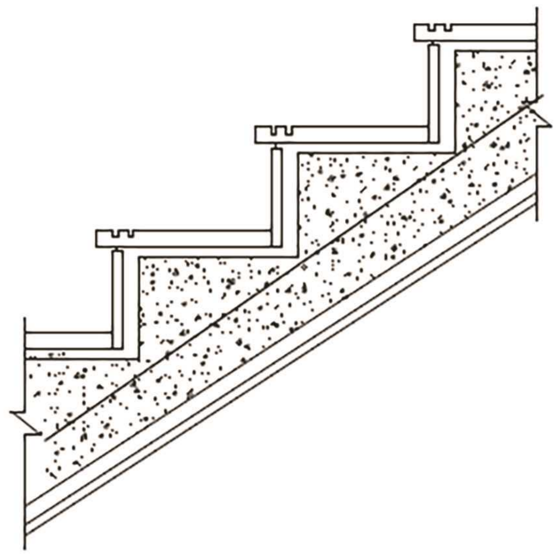
$$DL = \sum t_i \gamma_i \tag{1}$$

where.
t_i is the thickness of materials.
γ_i is the density of the materials.
Density of the materials.
The density of all the construction materials is given in Annex A. For example, the density of concert and mortar is given in **Table 1**:
Figures 3-**6** illustrate the details and the methodology for determining the dead load of the solid slab, block-joist floor, staircases, and wall.



Marble	0.025*22= 0.55	kN / m^2
Sand-Cement Mortar	0.03* 21 = 0.63	kN / m^2
Light Concrete	0.1* 12 = 1.2	kN / m^2
Slab	0.15* 25 = 3.75	kN / m^2
Plaster	0.01*13=0.13	kN / m^2
Service	0.8	kN / m^2
	DL = 7.06	kN / m2
	DL=Slab+3.5kN / m2	

Figure 3.
Dead load of solid slab [2].



$$l = 5000\ mm\ \ ;\ \ h \geq \frac{l}{20} = 25\ cm$$

Step Bearing	0.03*27= 0.81	kN / m^2
	0.02*0.2*3.3*25 = 0.33	kN / m^2
Light Concrete	0.2*0.3*3.3*22/2/ cos(33.7°) = 2.62	kN / m^2
Slab L/20=250mm	0.25*25/cos(33.7°) = 7.5	kN / m^2
Plaster	0.02* 1600/cos(33.7°) =0. 385	kN / m^2
	DL=11.65	kN / m^2

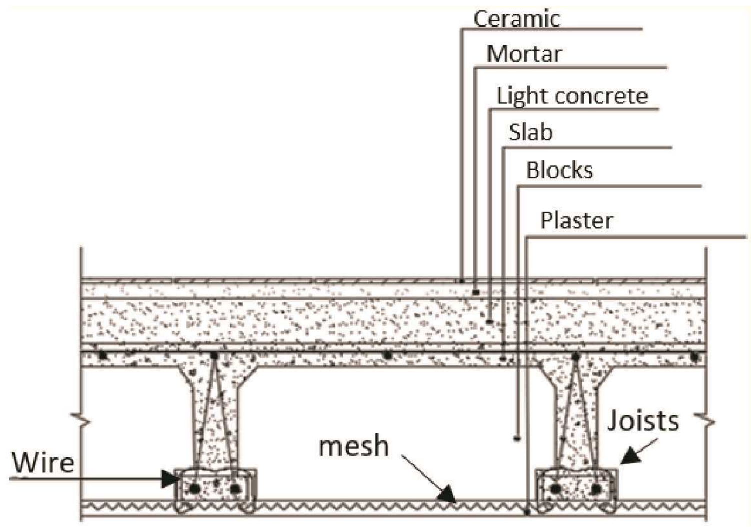
Figure 4.
Dead load of staircase [2].

3.1.2 Live (imposed) load

During the utilization of buildings or other structures, a temporary load is imposed. An illustration of this would be the weight of books in a library, students, chairs in classrooms, or equipment found in hospitals and factories. The classification of these loads is determined by the function of the spaces in EC1. The code suggests live loads in the form of UDL, concentrated load, or line load. Typically, the structural design is established based on the UDL load and is verified for point and line loads at key locations.

EC1 uses various categories to identify the variable actions (live loads). The main categories can be listed as follows:

- Residential, social, commercial, and administration: four categories (A, B, C and D)



Ceramic	$0.005(22) = 0.11$
Mortar	$0.02(21) = 0.42$
Light concrete	$0.11(13) = 1.43$
Slab	$0.05(25) = 1.25$
Joists	$[0.1(0.25)/0.6](25) = 1.04$
Plaster	$0.04(16) = 0.64$
Whitening	$0.005(13) = 0.065$
Services	0.2
DL=	**5.155** $k\ N\ m$

Figure 5.
Dead load of block and joist floor [2].

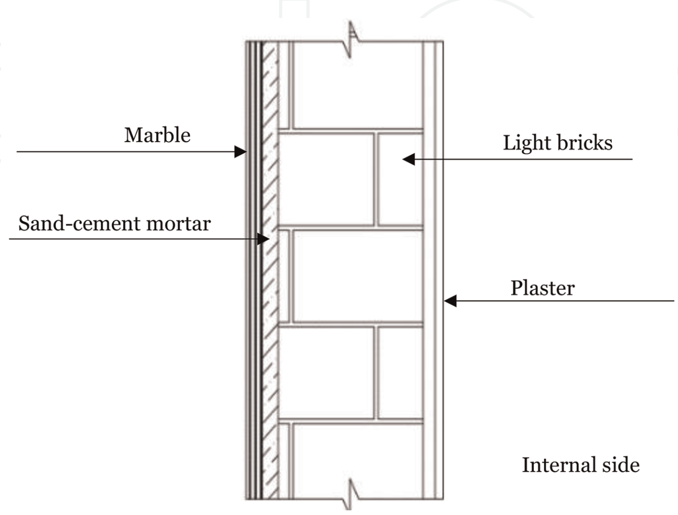


Figure 6.
Dead load of the external wall [2].

Material	Density, γ (kN/m^2)
Concrete (EN 206)	
Lightweight Class LC 1.0	9.0 to 10.0
Normal weight	24
Cement mortar	19 to 23
Granite	27 to 30
Dense limestone	20 to 29
Softwood playwood	5.0

Table 1.
Construction material's density [5].

Categories of loaded area	qk kN/m2	Qk kN
Category A • Floors • Stairs • Balconies	1.5 to 2.0 2.0 to 4.0 2.5 to 4.0	2.0 to 3.0 2.0 to 4.0 2.0 to 3.0
Category B	2.0 to 3.0	3.0–4.0
Category C • C1 • C2 • C3 • C4 • C5	2.0 to 3.0 3.0 to 4.0 3.0 to 5.0 4.5 to 5.0 5.0 to 7.5	3.0 to 4.0 2.5 to 7.0 4.0 to 7.0 3.5 to 7.0 3.5 to 4.5

Table 2.
Imposed (live) loads [5].

- Areas for storage and industrial activities: two categories (E1 and E2)
- Garages and vehicle traffic (excluding bridges): two categories (F and G)
- Roofs: three categories (H, I, and K)

An example of variable actions (live loads) is given in the **Table 2**.

3.1.3 Wind load

The force exerted on a building by the wind is known as the 'wind load'. Wind loads are influenced by factors such as wind speed and the building's shape and surface, making them challenging to accurately predict. Structures and their elements must be analyzed and designed to withstand wind loads (EC1). These loads are determined based on the average wind speed in the region, the building's height, geometry, and surrounding obstacles that affect wind flow (**Figure 7**). Similar to

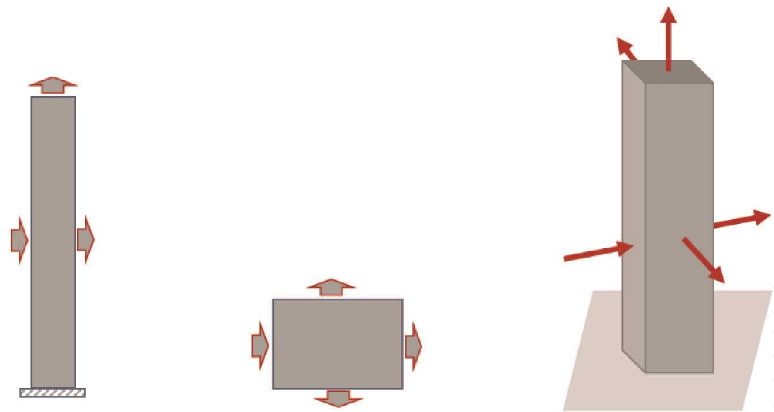


Figure 7.
Wind pressure and force direction. (a) Elevation (b) Plan (c) 3D view.

earthquake forces, wind loads are considered in two perpendicular directions and independently. It is important to note that wind and earthquake effects should not be combined in design, but rather structures should be engineered to withstand the critical impact of both loads separately. Wind pressure on a building is distributed across its exterior surface, varying based on the structure's geometry and location. Wind can cause overpressure on the windward side, potentially blowing windows in, while creating under-pressure (suction) on the leeward side, which may blow windows out. The building's shape can amplify these effects, with smooth profiles deflecting wind more effectively than textured ones, and circular buildings outperforming square shapes.

According to BS EN 1991-1-4:2005 [5], the peak pressure in Pa can be calculated using the following expression:

$$q_p(z) = C_e(z)q_b \text{ (Pa)} \tag{2}$$

$$q_b = 1/2\rho V_b^2 \text{(Pa)} \tag{3}$$

Where

$C_e(z)$ is the exposure factor (**Figure 8a**).

q_b is the basic velocity pressure (Pa).

ρ is the air density; $\rho = 1.25\, kg/m^{3.}$

V_b is the wind velocity, m/s (**Figure 8b**)

The wind pressure acting on the external surfaces, W_e (Pa), is obtained using

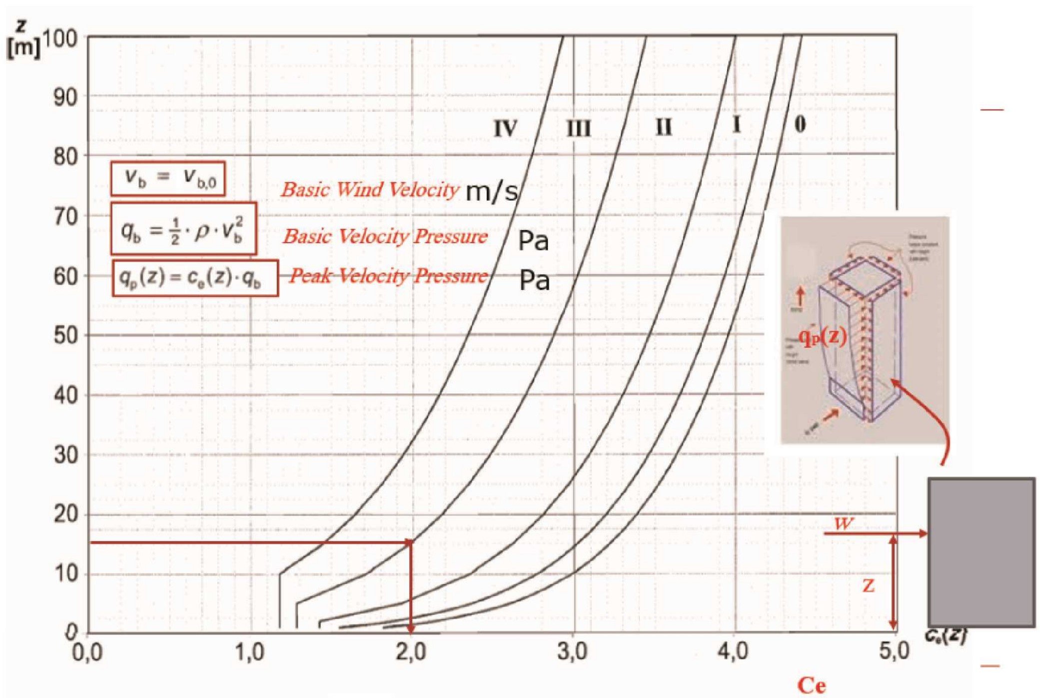
$$W_e = q_p(z)C_p(e) \tag{4}$$

where

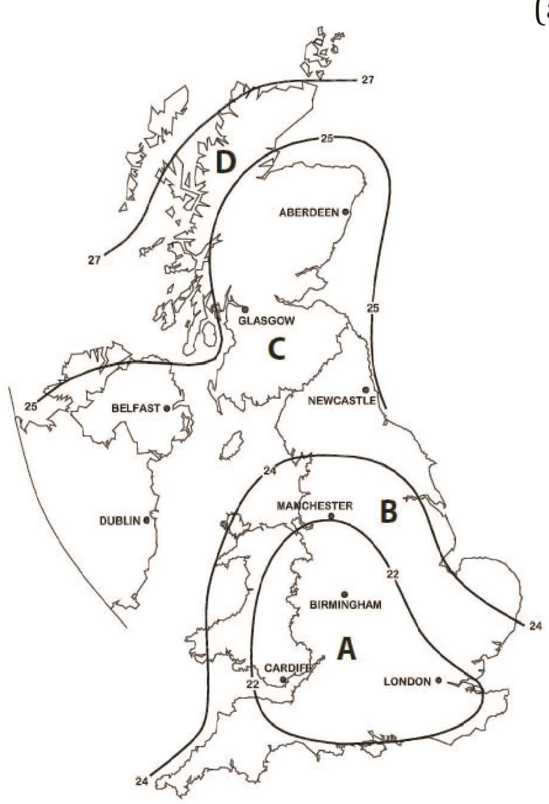
$C_p(e)$ is the pressure coefficient for external pressure (Table 7.1 BS EN 1991-1-4:2005) and **Figure 8**.

Finally, wind force on each surface, F_w, is calculated as

$$F_w = W_eA(N) \tag{5}$$



(a)



Terrain categories and terrain parameters

	Terrain category
0	Sea or coastal area exposed to the open sea
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m
NOTE: The terrain categories are illustrated in A.1.	

(b) (c)

Figure 8.
Wind peak pressure [5]. (a) Exposure factor Cez [5] (b) Wind velocity (c) Terrain category.

Where A is the reference area, m^2

To calculate wind load the subsequent steps must be followed:

1. Determine basic wind velocity, V_{bO} (**Figure 8b**)

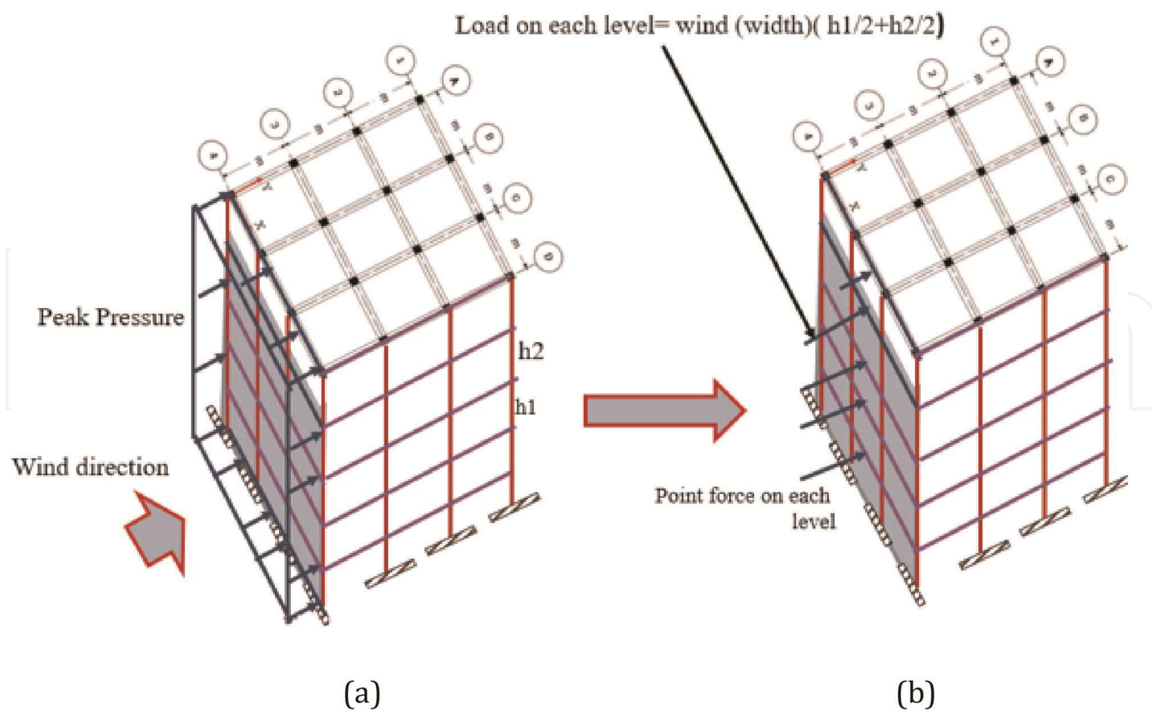


Figure 9.
Wind load on the structure. (a) Wind pressure (b) Concentrated wind force on each floor.

2. Calculate basic velocity pressure, q_b (Eq. (3))

3. Determine $C_e(z)$ (**Figure 8a**)

4. Calculate peak velocity pressure, $q_p(z)$ (Eq. (2))

5. Calculate wind load (Eq. (5) and **Figure 9**)

To analyze and design of the structures, the wind force is applied on the structure in form of point force on each floor level (**Figure 9**).

The structural configuration of the building should effectively and securely handle wind forces, transferring them to the foundations to prevent any risk of structural failure. Wind is typically identified as the primary horizontal force when assessing tall buildings through wind engineering. The structural components responsible for absorbing wind loads are typically distinct from those designed to handle dead loads and other gravity loads that originate within the building.

3.1.4 Earthquake load

Earthquakes are caused by the sudden movement of tectonic plates in the Earth's crust, which occurs along fault lines. This movement releases energy that travels through the Earth in the form of waves, causing vibrations that can be felt kilometers away from the epicenter. Areas near active fault lines are more susceptible to earthquakes.

Efforts to design structures that can withstand earthquake surface motions are continuously evolving. Alongside the guidelines provided by the EC8 Code, other codes and research teams regularly assess and update the analysis and design

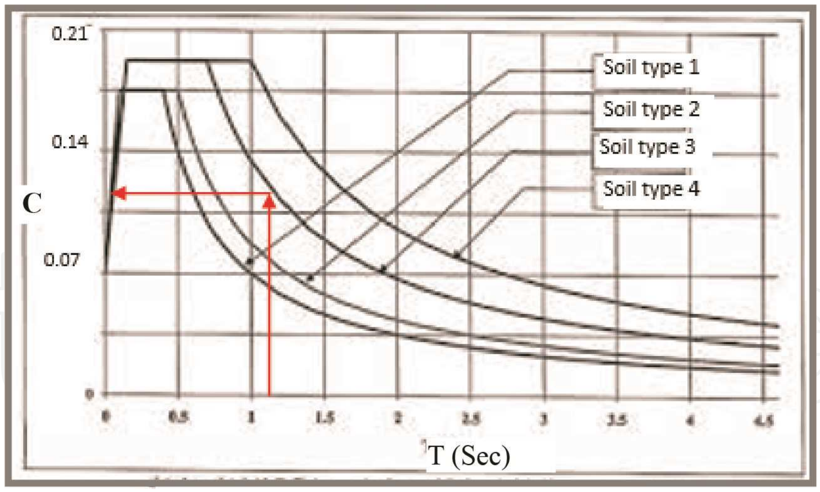


Figure 10.
Changes in the seismic response coefficient, C, with the periodicity T for various soil types.

requirements for structures against lateral earthquake loads. As a result, there are frequent changes in the building regulations regarding earthquake design.

The ACI code [6] does not specify the specific base motions for a particular site or provide detailed instructions on how the structure should be analyzed. Currently, building codes allow for the analysis of structures under the influence of various levels of earthquakes. The ASCE permits three types of structural analysis: the equivalent lateral force method, the modal spectral analysis, and the inelastic response time history analysis method.

Typically, the equivalent lateral force method is employed for buildings that are less than 50 meters in height. In some cases, it can also be used for irregular buildings, as long as all types of irregularities are carefully considered. Geotechnical studies of the soil beneath the foundation at the construction site assist designers in estimating the impact of soil type on the lateral force exerted by an earthquake.

In the equivalent lateral force method, the basic shear force can be calculated as follows:

$$V = CW \tag{6}$$

where C is the seismic response coefficient, given by code, and W is the effective seismic weight of the building. An approximate plot of the coefficient C as a function of the periodicity T is shown in **Figure 10**. For low, medium, and high-rise structures, C is approximately around 0.16, 0.06, and 0.03, respectively. For more details, refer to EC8. The approximate weight of the building can be calculated as follows:

$$W = nA(DL + \alpha LL + \Delta W_l + 3.5) + W_{walls} + a_{kh}(DL + \alpha LL) \tag{7}$$

Where

n is the number of stories

A is the floor area m^2

α is the percentage of live load participation

DL is dead load, $\mathrm{kN/m^2}$

LL is live load, $\mathrm{kN/m^2}$

ΔW_l is partitioning equivalent to overhead, kN/m^2
3.5 is the structural weight, kN/m^2
W_{walls} is the weight of all walls, kN
a_{kh} is the area of the barn, m^2

According to the author's experience, to verify the base shear force calculated by software Eq. (7) can be simplified as

$$W = 1.3nA(DL + \alpha LL + 4.5) \tag{8}$$

As the base shear force is automatically calculated by the software, Eq. (6) can provide a quick and reliable verification. For the residential buildings, the difference between software and Eq. (8) must be less than 5%.

3.2 Gravity loads distribution

The software automatically distributes the loads among the various elements. Ensuring the accuracy of this load distribution is crucial for software verification. Therefore, it is essential for all designers to have a precise understanding of gravity and lateral load distribution. After calculating the different loads, the floors must be designed before distributing the load among the frames. Once the floor/roof specifications are finalized, the gravity loads should be distributed among the frames. It is important to note that there is a fundamental difference between the distribution of gravity loads (dead and live) and lateral loads (earthquake and wind). Gravity loads, such as dead and live loads, are distributed among beams or columns based on the floor system's geometry. On the other hand, lateral loads are distributed among frames or shear walls based on their stiffness.

3.2.1 Load distribution in one-ways slabs

In one-way slabs, the load is only transferred in one direction, which means that the load on the secondary beams is directly proportional to the loading width. The calculation of each beam or joist's share will be based on its loading width. The loading width refers to the width of the slab where the loads are directly transferred to the relevant beams, which is equal to the span length between secondary beams or joist spacing. The width of the load carrier varies depending on the type of structural system of the floors and the direction of load transfer. To ensure the accuracy of the modeling and the loads inputted into the software, the load on the beams should be calculated using the presented method and compared with the values calculated by the software. The manual and software calculations should yield identical results.

The loading width of each slab, that is, a, as mentioned in the previous section, is illustrated in **Figure 11**. The load on the secondary beams of the one-way slab and the joists in the block-joist floors is a uniformly distributed load (UDL) as shown in **Figure 11** and can be determined by

$$q_D = aDL, q_L = aLL \tag{9}$$

The distribution of loads on the main beams varies between one-way and joists/ribbed systems. In the ribbed/joist system (**Figure 11b**), it is assumed that the load on the main beams is uniformly distributed.

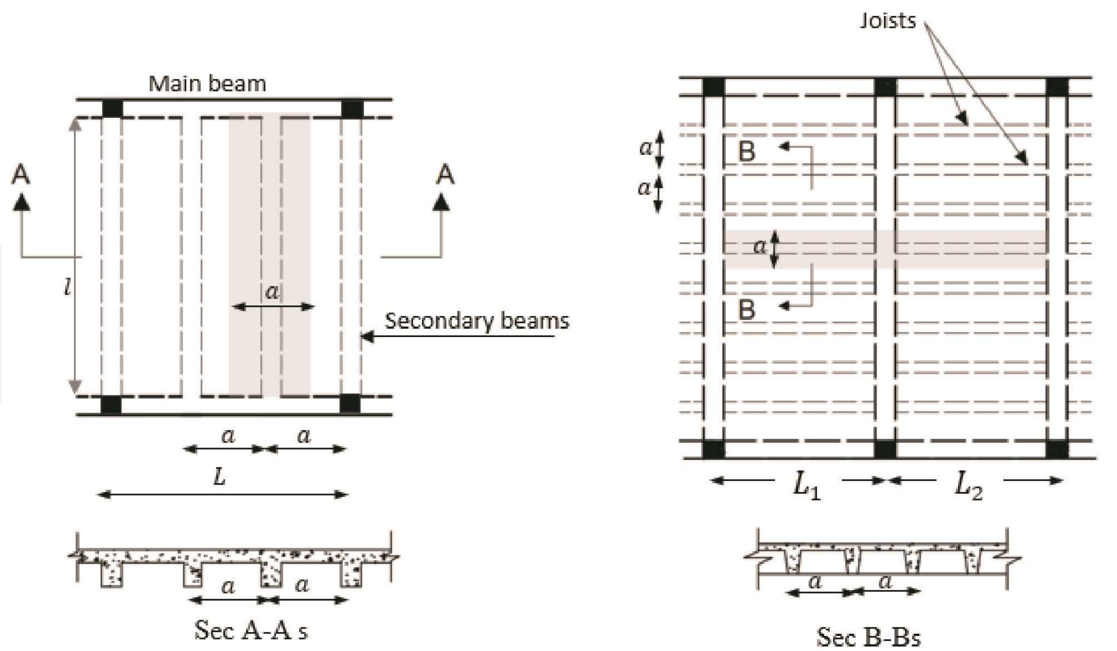


Figure 11.
Distribution of gravity loads on secondary beams and joists in one-way slab. (a) Load on the secondary beam, one-way slab; (b) Load on the joists, joist/ribbed slabs.

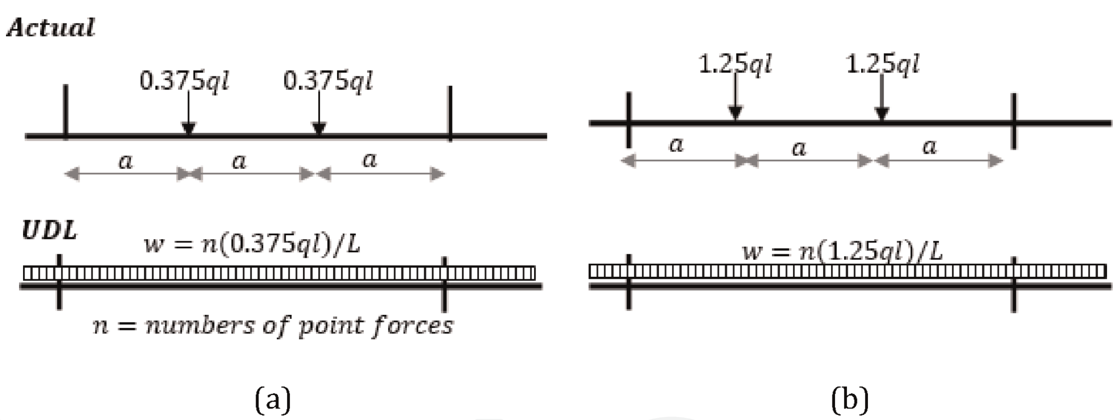


Figure 12.
Actual and approximate equivalent UDL Load on the main beam in one-way slabs (q); q is q_D or q_L (Eq. (9)). (a) Edge main beams (b) Middle main beams.

$$q_D = \left(\frac{L_1 + L_2}{2}\right)DL, q_L = \left(\frac{L_1 + L_2}{2}\right)LL \tag{10}$$

The concentrated force acting on the main beams in the one-way slabs is equivalent to the reaction supports of the secondary beams (**Figure 12**). It should be emphasized that the secondary beams are continuous, allowing for the determination of the point force on the side and middle main beams.

3.2.2 The load on the beams in two-way slabs

The load bearing surface of each beam in two-way slabs is formed by drawing the bisector of the panel corners. In the case of two-way slabs, the load is applied to the beams in a trapezoidal shape for long-span beams or triangularly for short-span beams, as depicted in **Figure 13**. These types of loads pose challenges during the approximate analysis stage, making it necessary to convert them into an equivalent uniformly distributed load (UDL). Additionally, the load caused by the weight of the walls is uniformly distributed and should be directly added to the aforementioned UDL. These calculations are crucial in validating the software modeling, which will be utilized to validate the results of the structural analysis.

The trapezoidal load on the middle long span beams, specifically axis 2 and 3, can be represented by a uniformly distributed load. This calculation can be done using **Figure 13b**.

$$q_D = \frac{(2b-a)a}{2b}DL, q_L = \frac{(2b-a)a}{2b}LL \tag{11}$$

The load distribution on the edge beams, specifically axis 1 and 4, is equal to half of the load on the middle long span beams.

Additionally, the load acting on the middle short span beams, axis B and C, is triangular in shape as illustrated in **Figure 13c** and can be calculated accordingly.

$$q_D = (a/2)DL, q_L = (a/2)LL \tag{12}$$

The load on the edge beams, axis A and D, is half of the middle beams in short span length.

3.3 Approximate structural analysis

Upon completion of the gravity and lateral load calculations, the structures undergo analysis and design to account for the impact of these loads. The analytical methods utilized include the following [7, 8]:

- Manual analysis
    - Slope-deflection method
    - Moment distribution method
    - Matrix method
- Software analysis

For each of those methods mentioned above, it is essential to have knowledge of the dimensions of the beam and column sections, while the design of these cross sections requires information on internal forces. In practical scenarios, the dimensions of the sections may need to be assumed or determined using approximate methods. The initial assumption of beam and column dimensions can result in a lengthy trial and error process. Therefore, a robust approximate structural analysis and design approach will be presented here to address this issue. These methods enable the designer to gain a comprehensive understanding of the actual internal forces and final dimensions of columns, beams, and shear walls in various project types. It is important to note that both approximate structural analysis and design techniques are

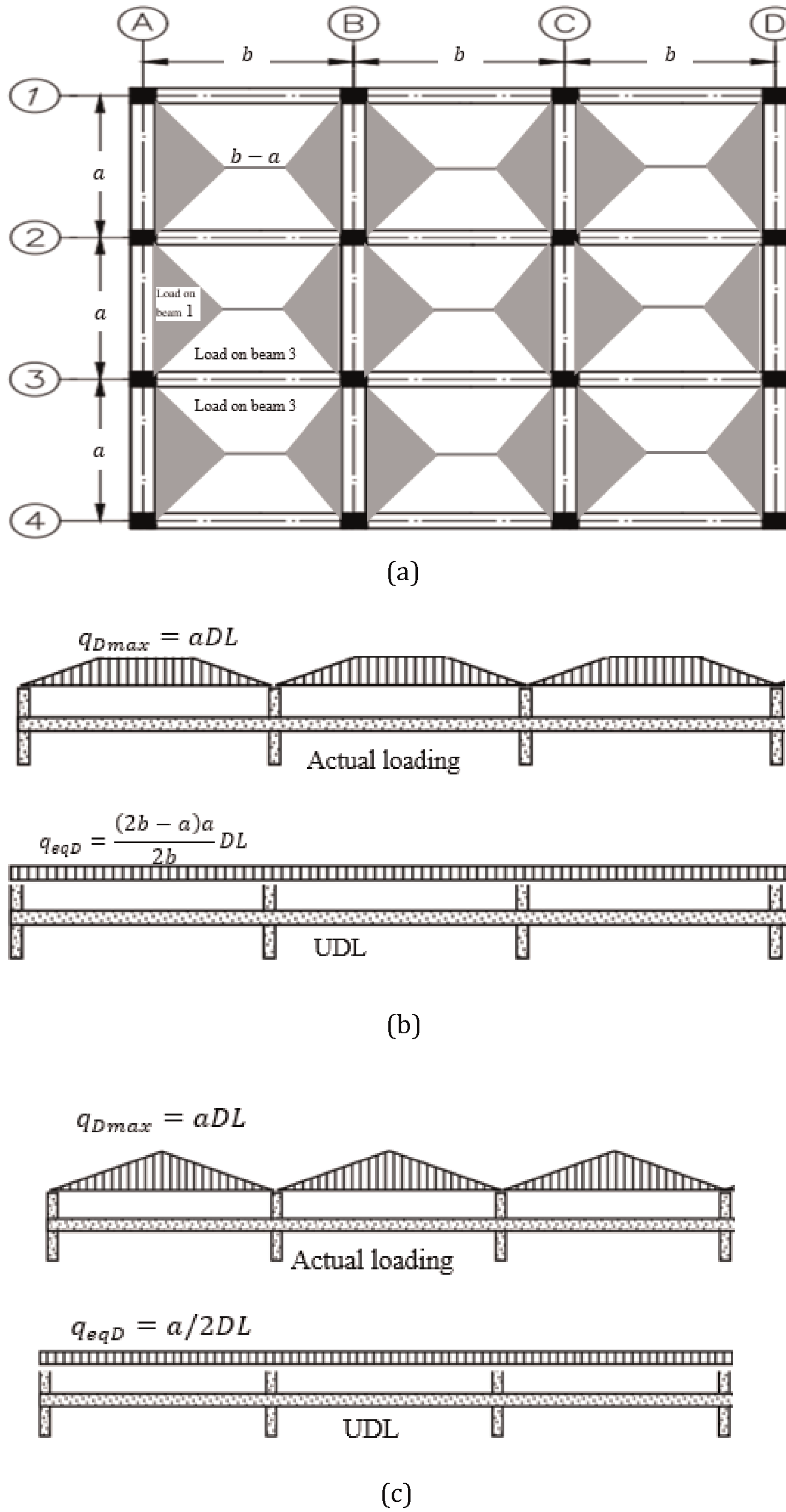


Figure 13.
Distribution of gravity loads on beams in two-way slabs. (a) Load distribution on the plan of slab. (b) Actual and equivalent UDL load on middle long span beams, that is, beam 2. (c) Actual and equivalent UDL load on middle short-span beams, that is, beam B.

straightforward, yet they serve as invaluable tools for designers in ensuring the safety of a project and the accuracy of software results.

In a pragmatic undertaking, the preliminary dimensions of the structures are initially determined through approximate analysis techniques. Subsequently, the structure undergoes a thorough analysis and design process employing precise analysis methods, either manual or computer-based. Should the sections prove unsuitable for the imposed loads, the structure is subjected to a reanalysis and redesign, incorporating new sections. This iterative process persists until the ratio of applied load to section strength falls below 1.0.

The analysis under the influence of gravity and lateral loads varies, thus necessitating a separate discussion on the specifics of each method.

It is commonly believed that only designers with a comprehensive background in structural analysis and ability to provide verifications have the authority to design structures using software. However, many analytical approaches are challenging to apply for structural verifications; therefore, the approaches presented in this chapter offer a viable alternative.

3.3.1 Approximate analysis of beam and one-way slab under the gravity loads

In the existing literature, three primary approximate analyses have been suggested for determining the initial dimensions of structural elements. These include the ACI coefficient for analyzing beams subjected to gravity loads, as well as the portal and cantilever methods for analyzing frames under lateral forces [1]. While the ACI method is recognized for its simplicity and reliability, the latter two methods tend to be more time-intensive, leading many designers to prefer alternatives.

The accuracy and reliability of the ACI coefficients method in analyzing beams and one-way slabs are evidently close to reality, making it suitable for a wide range of structures. To enhance the designer's interpretation skills and gain insight into the background of the coefficient method outlined in the ACI code [6], a continuous beam analysis with the contraflexure point assumption can provide valuable insights.

3.3.1.1 Bending moments and shear force

The position of the contraflexure point in a beam depends on its support conditions. In the both ends of a fixed beam, the contraflexure point is located at 0.2 times the length of the beam from the support. However, in a simply supported beam, the contraflexure point is located directly at the support. In a rigid reinforced concrete frame, the turning point location falls between these two values. The author proposes the position of the contraflexure point as depicted in **Figure 14**, based on the analysis of various projects. By assuming this position, the structure can be treated as a determinate system, allowing for the calculation of internal forces such as bending moments and shear forces using simple statics methods. It is important to note that the 0.1 L method, mentioned in some books, is not suitable for the approximate analysis of beams.

The bending moment analysis for the side span is shown in **Figure 15**. By applying the same method, the support bending moments at middle span length will be the same as right support of first span, that is, $\frac{wl^2}{10.8}$ and positive bending moment at the

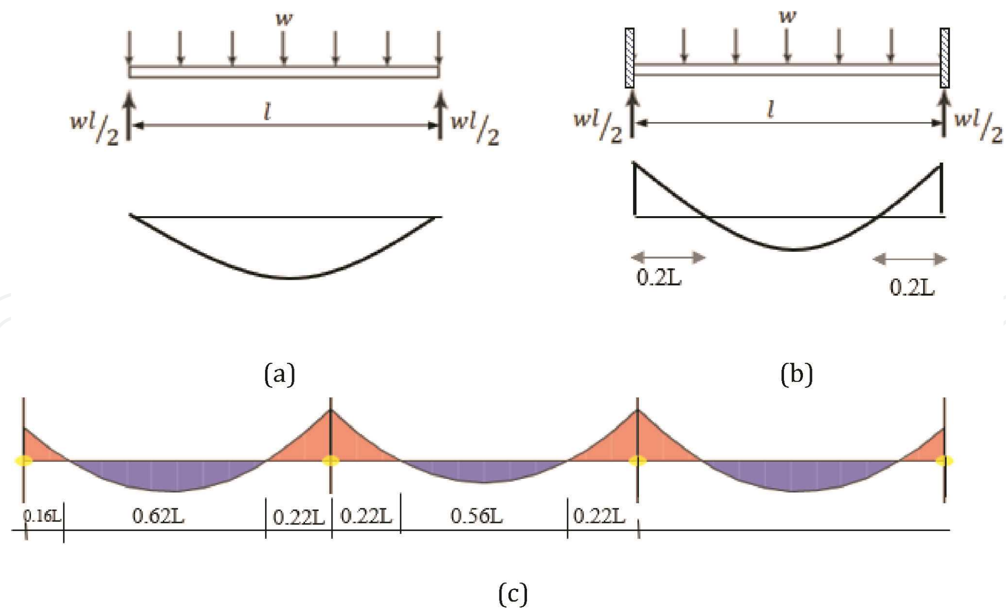


Figure 14.
Turning points in different span lengths, suggested by the author. (a) Simply supported beams. (b) Fixed beams. (c) Continuous beams.

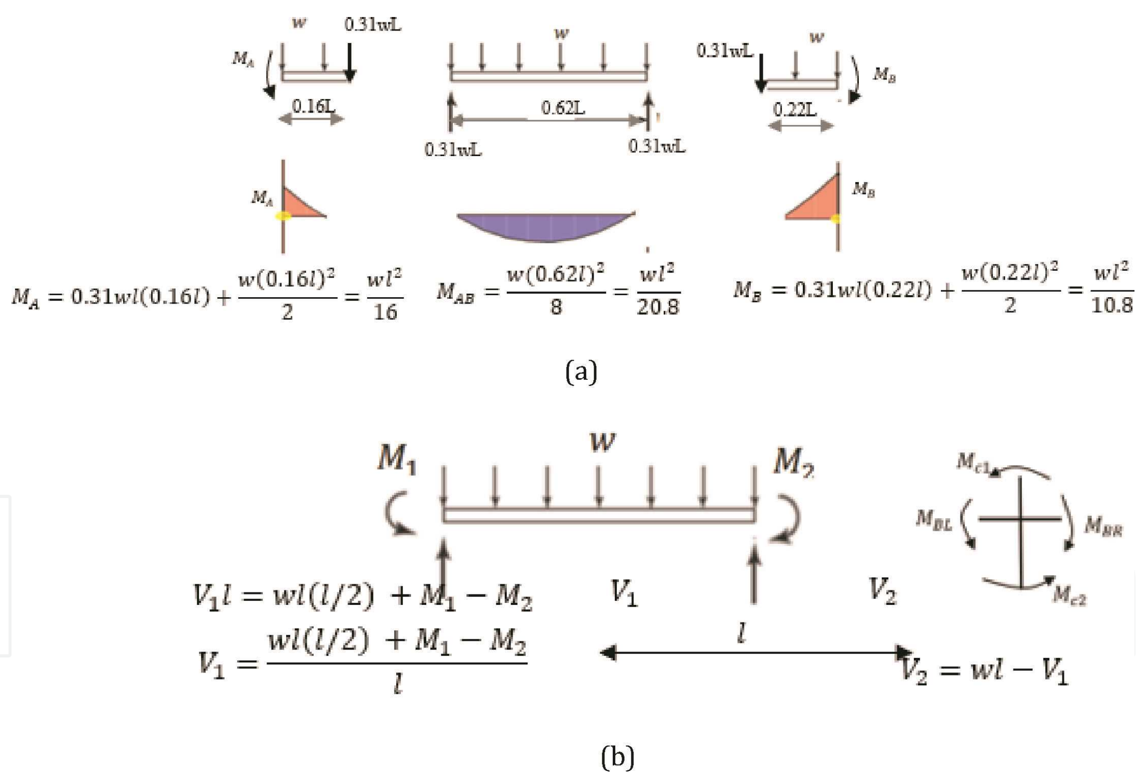


Figure 15.
Bending moment and shear force analysis in the first span length. (a) Bending moment. (b) Shear force.

middle of the beam will be $\frac{wl^2}{20.8}$. In this method, the bending moment in the columns is calculated from the equilibrium of moments at the joints

$$M_{c1} = M_{c2} = (M_{BL} - M_{BR})/2 \tag{13}$$

Eq. (13) demonstrates that when subjected to gravity load, the bending moments in the central columns with identical beam span lengths will be null. However, for beams with different span lengths, the bending moments in the middle columns will still be minimal. It is important to note that this scenario only holds true when the live load is uniformly distributed across all span lengths. In all cases, the bending moment in the side columns remains significant.

In order to obtain a more thorough and accurate estimation, the influence line method must be utilized to compute the critical bending moments at different positions under live load. The ACI coefficient method [6] has employed the influence line to determine these critical bending moments. For negative bending moments, it is assumed that the live load is distributed across both adjacent span lengths. Conversely, for positive bending moments, the live load is assumed to be applied solely on the span where the bending moment is to be determined, with the other spans being loaded in between [9].

The ACI code [6] offers a set of coefficients that can be utilized to analyze continuous beams and one-way slabs subjected to gravity loads. In the event that certain conditions are satisfied, the method of statutory coefficients can be employed.

- The number of span lengths are equal to 2 or more
- The length of the spans should be almost equal, or in two adjacent spans, the length of the longer span does not exceed 20% of the length of the shorter span length
- Loads are uniformly distributed
- The live load should not be greater than three times the dead load; LL ≤ 3DL
- The beam's moment of inertia should be constant

The limitations imposed on span lengths and maximum live load are in place to guarantee that the positioning of live load (in order to optimize the bending moment within a specific section) on the spans will not result in negative bending moments in sections typically experiencing positive bending moments (refer to **Figure 16**) [10].

ACI code allows the calculation of bending moment and shear force in continuous beams and one-way slabs using formulas that obtain the internal forces at critical points

$$M = \alpha w l_n^2 \tag{14}$$

Where

M is bending moment, kNm

α is the ACI coefficients, **Figure 17**

w is uniformly distributed dead or live load on the beam, kN/m

l_n is the clear span length, m **Figure 18**

The coefficients for continuous beam and one-way slab are the same, except for negative bending moment at the side supports (**Figure 17b**).

The comparison between the approximate analysis suggested by the author (**Figure 15**) and the ACI method reveals that the positive bending moments derived from the ACI coefficients method tend to be somewhat conservative due to the consideration of the influence line.

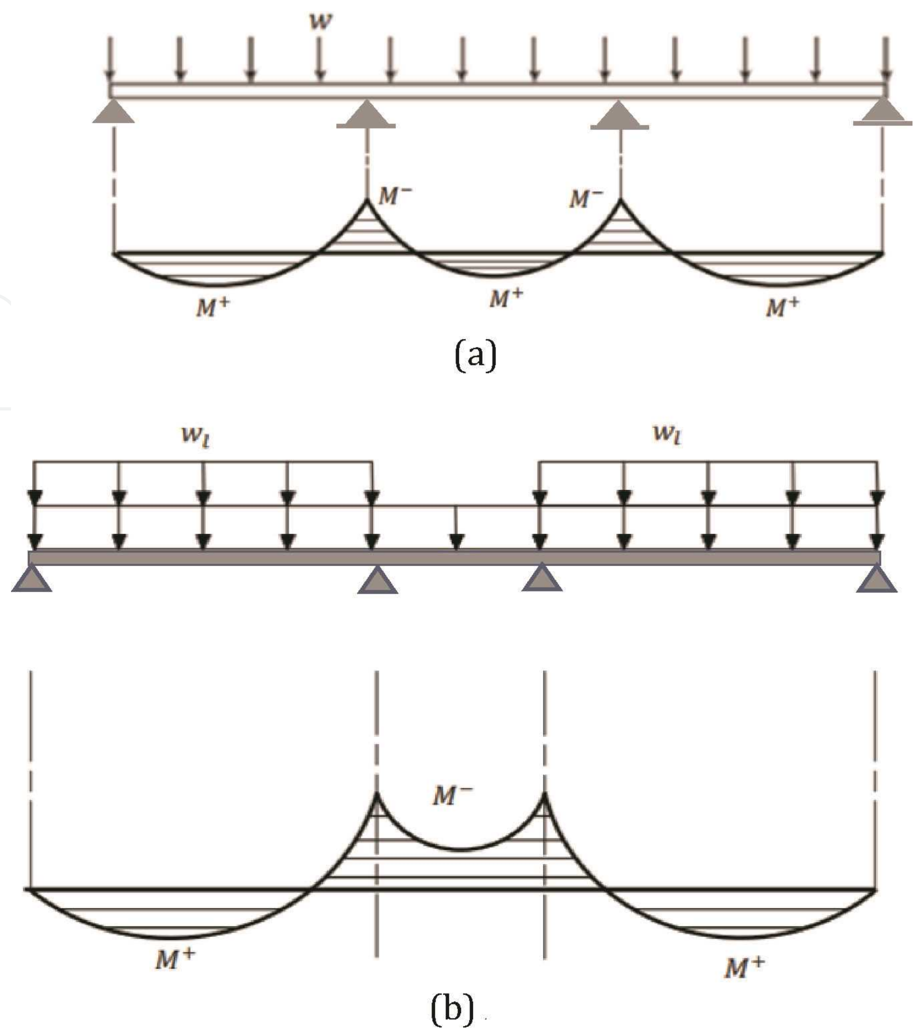


Figure 16.
The effect of span length on the shape of the bending moment diagram. (a) Approximate analysis using ACI coefficients is allowed. The UDL load applied to the span is almost identical, resulting in a positive bending moment in the middle of the spans. (b) Analysis using ACI coefficient method is not allowed, unequal spans and type of live loading caused negative bending moment throughout the middle span.

For slabs with a span of less than 3 meters and for beams where the ratio of stiffness of the column to the stiffness of the beam – at both ends – exceeds 8, the negative bending moment at the edge of the supports can be approximated as $w_u{l_n}^2/12$ instead of the values provided in **Figure 17**. By utilizing the bending moments at the ends of each span, the shear force can be determined using the approach outlined in **Figure 15**. Additionally, a conservative estimate of the shear force is presented by ACI

$$V = \beta w l_n \tag{15}$$

where V is shear force and β is ACI coefficient for shear force, **Figure 18**.

When utilizing code coefficients for beam analysis, it is necessary to calculate the bending moment of the columns, M_{CT} *and* M_{CB}, individually **Table 3**. Assuming that the larger span length next to the column sustains half of the live load in addition to the dead load, while the smaller span length only carries the dead load, the bending moment transmitted to the columns can be determined using the provided formula

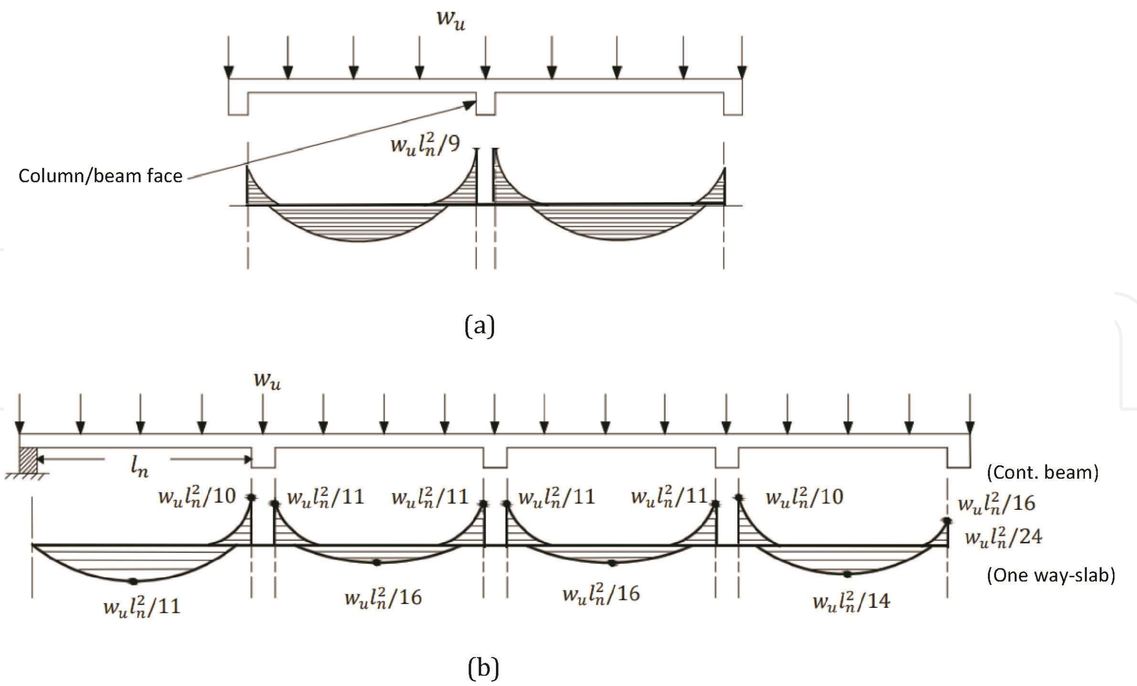


Figure 17.
Approximate analysis under gravity loads – continuous beam/one way slab – ACI coefficient for bending moments, α [2]. (a) Bending moment – two spans length (b) Bending moment – continuous beam or one-way slab.

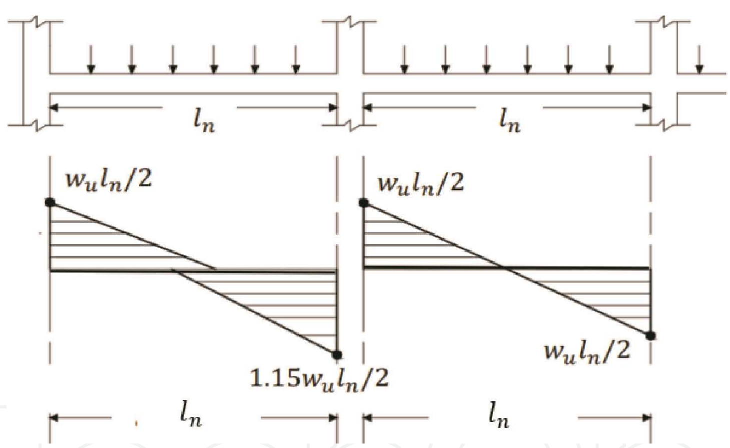


Figure 18.
Shear force values in the face of supports.

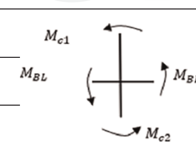
L_{n2}	Bigger span length	
L_{n1}	Smaller span length	
w_D	Uniformly distributed dead load	
w_L	Uniformly distributed live load	

Table 3.
Beam span length definitions.

$$M_{CT} = M_{CB} = 0.035\left[(w_D + 0.5w_L)L_{n2}^2 - w_D L_{n1}^2\right] \tag{16}$$

It should be emphasized that, in the majority of structures, the bending moment in middle columns under gravity load is relatively minor in comparison to the bending moment caused by lateral forces. Conversely, the axial force in columns under gravity loads is significant when compared to the axial force resulting from lateral forces.

3.3.1.2 Approximate axial force in the columns

The calculation of the axial force for each column involves multiplying the effective loading area of the column by the applied loads. The effective loading area of the column on each floor is determined by considering the center line of the panels surrounding the columns, as shown in **Figure 19**. To determine the axial force of each column on a specific story, the axial forces of the columns on the upper stories are summed up until the desired story is reached. This approach is suitable for frames with simple connections, but it may result in a significant approximation for rigid frames. To obtain more realistic axial forces for rigid frames, according to analytical approaches and the results of software analyses, the author proposes an approximate method for calculating the axial force of the columns:

$$P_D = 0.4n\left(f + \frac{a}{2}\right)\left(j + \frac{L_1}{2}\right)DL + P_{wall} + P_{Col} P_L = 0.4n\left(f + \frac{a}{2}\right)\left(j + \frac{L_1}{2}\right)LL \text{Corner columns, A1} \tag{17}$$

$$P_D = 1.25n\left(\frac{b+c}{2}\right)\left(\frac{L_1}{2}\right)DL + P_{wall} + P_{Col} P_L = 1.25n\left(\frac{b+c}{2}\right)\left(\frac{L_1}{2}\right)LL \text{Edge columns, C1} \tag{18}$$

$$P_D = 1.25n\left(\frac{a+b}{2}\right)\left(\frac{L_1+L_2}{2}\right)DL + P_{wall} + P_{Col}\ P_L = 1.25n\left(\frac{a+b}{2}\right)\left(\frac{L_1+L_2}{2}\right)LL \text{Edge columns, B2} \tag{19}$$

where n is the number of floors above the column in the target story. The weight of the walls is also obtained from the product of the UDL load of the wall by half the length of the span lengths adjacent to the column (P_{wall}). It should be noted that the

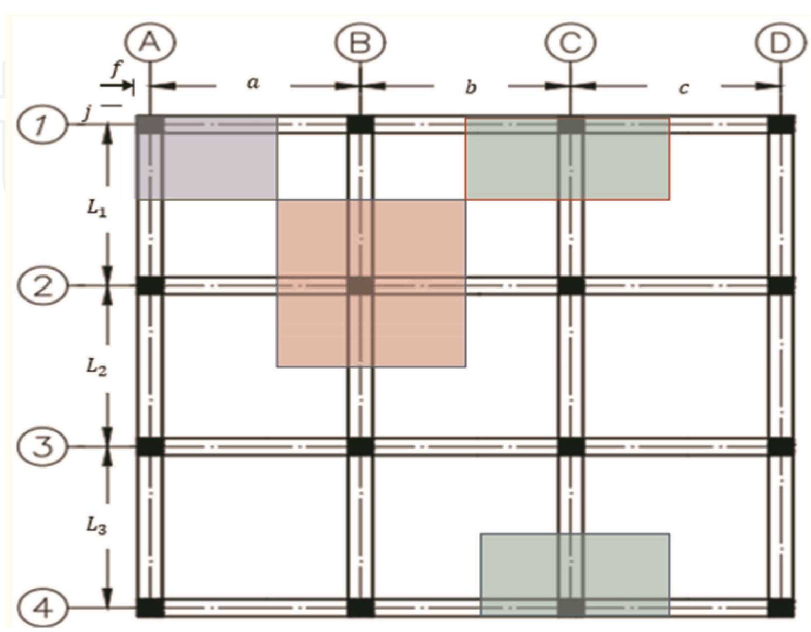


Figure 19.
Effective loading area of columns.

weight of the column itself should also be considered; product of column dimensions*concrete density*height of the column*numbers of columns

3.3.2 Approximate analysis of frames under lateral loads

Two methods are employed to examine the response of frames to the horizontal forces induced by wind or seismic activity [11]

- Portal method
- Cantilever method

Explanations regarding the methods and their advantages and disadvantages can be found in structural analysis books. However, this discussion will focus solely on the portal method, which is considered to be simpler and more practical. The principle of this method is based on assuming that the contraflexure point is located at the mid-point of the beam and column. It should be noted that this assumption leads to an underestimation of the bending moment on the ground floor by at least 30%, although the results for other floors are deemed acceptable. Despite its practicality, this method is not commonly used in practical analysis and design due to its time-consuming nature.

To comprehend the fundamentals of the portal method, a single-span frame is employed. By separating the beam and column at the turning point, the indeterminate structure is transformed into a determinate structure. The turning point is assumed to be situated in the middle of both the column and beam. It is worth mentioning that this assumption has been proven to be accurate for all floors except the ground floor (**Figure 20**).

The distribution of shear force in each floor is determined by the stiffness ratio of the columns in each story. Initially, the dimensions of the columns are unknown, so the shear force is divided based on the loading span. As a result, each column experiences a shear force of P/2. Referring to **Figure 20c**, the bending moment of the column can be expressed as $M_c = P/2(h/2)$. At point B, the sum of beam bending moments is equal to the sum of column bending moments, denoted as $\sum M_{Bi} = \sum M_{ci}$, so $M_B = M_c$ Consequently, M_B is equal to M_c. By applying equilibrium in the beam, as shown in **Figure 20c**, we can derive the necessary equations:

$$\frac{V_B L}{2} = M_B \tag{20}$$

Consequently, the beams' shear force will be equivalent to the beam bending moment divided by half of the beam span length

$$V_B = \frac{M_B}{L/2} \tag{21}$$

Based on the equilibrium of forces in the vertical direction at point B, it can be determined that the axial force of the column is equal to the shear force of the beam, denoted as $N = V_B$. Consequently, the step-by-step procedure for analyzing the frame using the portal method is outlined as follows:

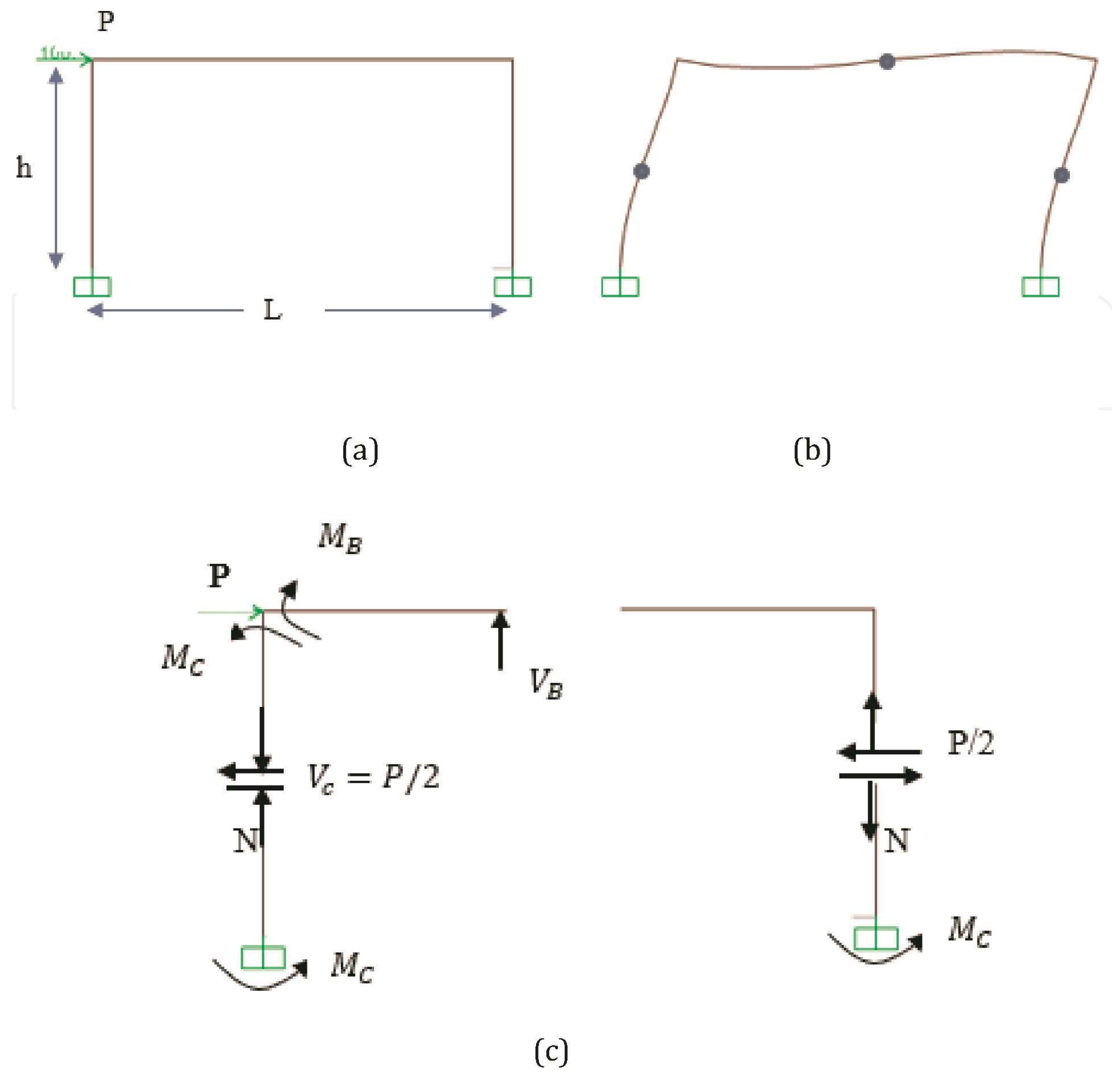


Figure 20.
The principles of the portal method. (a) One-span frame. (b) Turning points in the flexural frame. (c) Internal forces.

1. The determination of the turning points of the beam and column involves establishing their respective locations. Under an acceptable hypothesis, it is observed that the turning point on all floors, except for the ground floor, is situated in the middle of the beams and columns.

2. The shear force in each floor is computed by summing up the lateral force exerted on the floors from the roof up to the desired story.

3. The shear force of each column is equal to the loading span of the column divided by the length of the frame, multiplied by the shear force in the corresponding story.

4. The bending moment of each column is determined by multiplying the shear force of the column by half the height of the column.

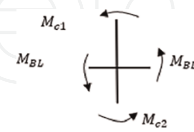
5. The bending moment of each beam is calculated by considering the equilibrium of joints

6. The calculation of the shear force for each beam involves dividing the end bending moment by half of the span length.

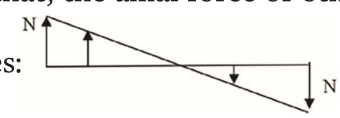
7. The axial force in each column is determined by adding the axial force of the columns in the upper floors to the variation in shear force of the adjacent beams.

8. In this approach, the shear force in the beams is uniform, resulting in only the side columns experiencing axial force while the middle columns remain unaffected by the earthquake.

$$M_{c1} + M_{c2} = M_{BL} + M_{BR}$$



It is to be noted that, author suggest that, the axial force of other columns can be calculated using the similarity of triangles:



3.3.2.1 New method suggested by the author

The portal method is characterized by its length and the numerous calculations it entails. In contrast, the author suggests a straightforward approach akin to the ACI coefficient method for determining shear force and bending moment in columns subjected to lateral loads from earthquakes. Typically, columns within each story exhibit similar dimensions in practical applications. With the assumption of a rigid floor, the stiffness of all ground floor columns will be uniform, leading to nearly equal shear forces in the columns.

$$V_c = \frac{V}{n} \tag{22}$$

In the given scenario, V represents the story shear force, V_c denotes the shear force in each column, and n signifies the number of columns in the story. On the ground floor, the turning point is positioned at a distance of 0.8 h from the support. Consequently, the bending moment in the ground floor in side and middle columns is equal to

Side columns

$$M_c = 0.8hV_c \text{ in support} \tag{23}$$

$$M_c = 0.2hV_c \text{ At the top of the column} \tag{24}$$

middle columns

$$M_c = 0.7hV_c \text{ in support} \tag{25}$$

$$M_c = 0.3hV_c \text{ At the top of the column} \tag{26}$$

In alternative narratives, as per software analyses conducted on several buildings with 5–15 stories, it is evident that the shear force exerted on the side and middle columns is not equivalent

$$V_c = 0.73\frac{V}{n} \text{ Side columns} \tag{27}$$

$$V_c = 1.19\frac{V}{n} \text{ Middle columns} \tag{28}$$

Since in other stories the turning point is in the middle of the column, the bending moment in the columns is equal to

$$M_c = \frac{V_c h}{2} \tag{29}$$

The simplified method mentioned above is employed to determine the bending moments of the upper and lower columns under the influence of earthquake or wind load. Additionally, the bending moments on the beams can be computed by utilizing joint equilibrium. Typically, calculations commence from the left side, resulting in the bending moments of the beam being equal to the bending moments in the side joints

$$M_{BR} = M_{c1} + M_{c2} \tag{30}$$

In the rest of the middle joints, the beam bending moments is equal to

$$M_{BR} = M_{BL} = (M_{c1} + M_{c2})/2 \tag{31}$$

The calculation of the shear force in the beams involves dividing the bending moment of the beam by half of its length (Eq. (21)). It is crucial to note that the calculations provided consider the scenario where the slab does not contribute to resisting lateral loads (membrane floor).

The simplified method mentioned above can be utilized as a suitable alternative to the portal method for approximate analysis during the initial design phase and verification of dimensions when inputting data into software tools.

It is important to highlight that the aforementioned method has been introduced under the premise of a symmetrical structural configuration, characterized by uniform column and beam dimensions across each level. While this assumption introduces a degree of approximation in the results, it remains a valuable approach for establishing the preliminary dimensions of structural components. Furthermore, it serves as a robust tool for validating the structural modeling, as well as the analysis and design processes performed by software.

3.4 Preliminary design of structural elements

The software utilizes two methods to select the initial dimensions as input data. The first method involves assuming initial dimensions and optimizing them through trial and error using the software. The second method is based on approximate analysis, while also considering the code limit on the beam depth.

The process of performing approximate analysis and designing is crucial. By conducting a preliminary design based on approximate analysis, the designer gains valuable insights into the final dimensions of the beams, columns, shear walls, and foundations. Additionally, approximate analyses assist the design engineer in interpreting the results of computer analyses and ultimately validating them.

3.4.1 Beams

The ACI coefficients method is employed to calculate the bending moment on the beams at various points (typically on the first middle column) under the dead and live

	h_{min}			
	Simple support	**One end cont.**	**Both end cont.**	**Cantilever**
One-way slab	$l/20$	$l/24$	$l/28$	$l/10$
Beam	$l/12$	$l/15$	$l/15$	$l/6$

Table 4.
Minimum beam depth [6].

load, for the preliminary design of the dimensions (b, h). On the other hand, the author's simplified method is utilized to determine the bending moment in the columns.

By considering the loading combination of 1.2D + 1.0 L + 1.0E, the factored bending moment of the beam is computed at the first middle support (**Table 4**). Consequently, the minimum cross section dimensions of a beam with a singly reinforced section are determined to be

$$bd^2 = \frac{M_u}{0.2f_c}, b = 0.65d, h = d + 55mm \quad (32)$$

As stated in chapter 8 [2], the regulatory approach offers a more straightforward solution for determining the initial dimensions (**Table 3**). The constraints presented in **Table 3** are primarily intended to regulate the deflection caused by gravitational loads. However, given the significant influence of earthquake forces on beam design, it is recommended by the author to establish the fundamental dimensions in order to effectively manage lateral deformation and accommodate sufficient space for reinforcement bars

Braced frame

$$\text{Depth} \geq \text{h}_{\text{min}}\ b = 0.65h\ \text{Width} \quad (33)$$

Unbraced frame

$$\text{Depth} \geq 1.5\text{h}_{\text{min}}\ b = 0.65h\ \text{width} \quad (34)$$

The final dimension would be the maximum of those two approaches.

3.4.2 Columns

The bending moment caused by dead and live loads is considered insignificant, with the exception of the side columns. For the middle columns, the axial force resulting from dead and live loads, as well as the bending moment in the columns due to earthquake, are calculated using the author's proposed method of 1.2D + 1.0 L + 1.0E.

In practical design, it is advisable to incorporate earthquake/wind force when calculating the initial dimensions of columns. To achieve this, the author proposes a straightforward approach outlined by author [2], which involves utilizing the code limitations on lateral displacement. According to chapter 8 [2], the minimum moment of inertia of each column is as follows

Unbraced frame

$$I > 12\frac{VC_d h_s^2}{nE} \text{ Less than five floors} \tag{35}$$

$$I > 14.5\frac{VC_d h_s^2}{nE} \text{ More than five floors} \tag{36}$$

Braced frames

Given a similar approach and assuming that the frames can withstand a maximum of 30% lateral load, the calculation for determining the minimum moment of inertia of each column is as follows:

$$I > 3.6\frac{VC_d h_s^2}{nE} \text{ Less than five floors} \tag{37}$$

$$I > 4.5\frac{VC_d h_s^2}{nE} \text{ More than five floors} \tag{38}$$

where

$$I = \frac{bh^3}{12}$$

V= shear force in the story
C_d= displacement magnification factor, for example, $C_d = 5.5$
h_s= story height
E= elastic module
n= numbers of column in the story
b=column dimension parallel to the bending moment axis
h= column dimension normal to the bending moment axis

The effectiveness of the proposed method has been confirmed through a range of real-world projects involving 5- to 15-story reinforced concrete structures over the past two decades, all of which were designed by the author.

The dimensions of b and h are carefully selected by the designer in order to provide the required moment of inertia, $I = \frac{bh^3}{12}$. To simplify matters, it is advisable to initially consider the columns as being square in shape. In the initial stages of a project, the columns on each story can be treated as identical. However, during the optimization stage, their dimensions can be adjusted according to the internal forces using specialized software.

The following simplified method can also be used to calculate the initial dimensions of the columns

Column under axial force and bending moment

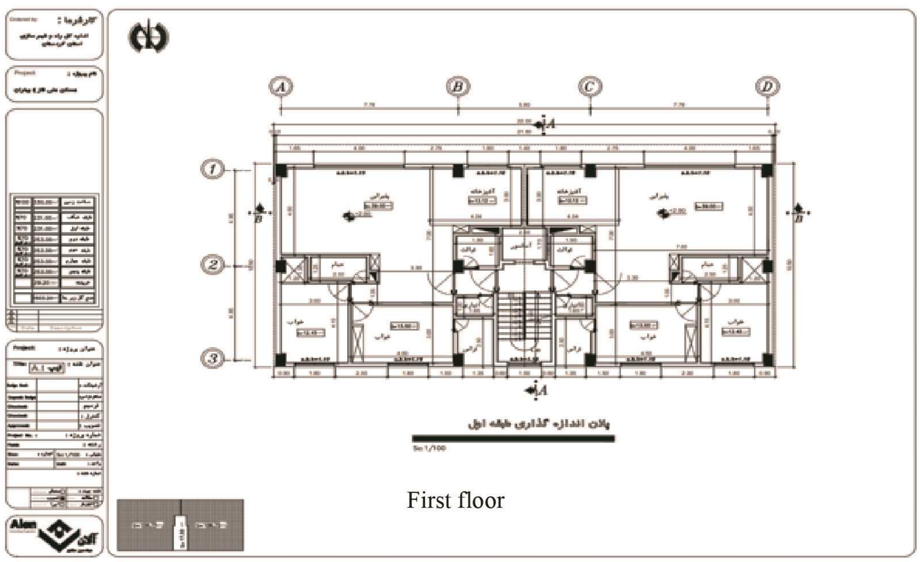
$$bh \geq \frac{P_u}{0.3f_c} \tag{39}$$

Column under axial force only

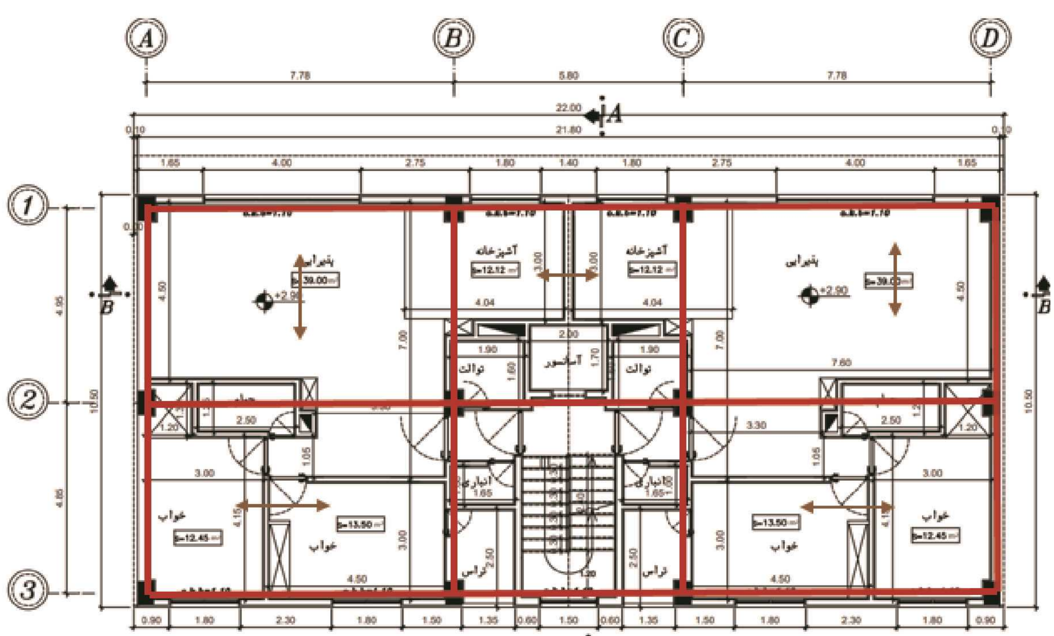
$$bh \geq \frac{P_u}{0.7f_c} \tag{40}$$

3.5 Case study 1: Structural analysis and design; National Housing: Kurdistan - Bahran Sanandaj 2020–2022

The National Housing project represents a residential development situated in the outskirts of Sanandaj, within the Kurdistan province. This project comprises 35 blocks, categorized into 15 distinct types, with each block containing between 5 to 10 units. A standard architectural layout of the floors is illustrated in **Figure 21**.



(a)



(b)

Figure 21.
The National Housing project. (a) Typical floor plan – first floor (b) Column/beam layout, and floor joist direction.

3.5.1 Summary of loading

(KN/m^2)	Location	Dead	Live	Partition	Snow	
	Roof	4.99	1.5	-	Distributed	1.26
	Story2-6	5.01	2.0	1.0	-	
	balcony	5.01	3.0	-	-	

Wall Load	Location	height	Wall Type		KN/m^2	KN /m
	Roof	0.85	External with Cladding		-	3.15
			External without Cladding		-	2.7
	Story	2.9	External with Cladding	With Opening	2.31	7.6
				Without Opening	3.3	10.89
			External without Cladding		2.8	7.8

3.5.2 Approximate structural analysis

Performing approximate analyses and creating designs according to these findings is crucial. Approximate analysis aids the designer in understanding the outcomes of the computer analysis and ultimately confirming their accuracy. The approximate analyses for both gravity and lateral loads are conducted under various scenarios.

3.5.2.1 Gravity load

As the span lengths of beams are approximately the same, ACI method provides acceptable results. In the beam 2 (**Figure 21b**), maximum bending moment at the first middle column assuming

$$M_{DL} = \frac{w_D l_n^2}{9} = \left(\frac{20(4.75)^2}{9}\right) = 50.13\ kNm$$

$$M_{LL} = \frac{w_L l_n^2}{9} = \left(\frac{8.0(4.75)^2}{9}\right) = 20.06\ kNm$$

where

$$w_D = 8.0/2[(5.0)] = 20\ kN/m$$

$$w_L = 8/2[(2.0)] = 8.0\ kN/m \text{ and } l_u = 4.75$$

3.5.2.2 Lateral forces

The seismic load calculation involves categorizing the country of Iran into four distinct zones. The specific project is situated in Sanandaj, Kurdistan, an area characterized by a significant earthquake risk, with a seismic coefficient of A = 0.35 g. An RC frame exhibiting intermediate ductility has been selected for this project. The base shear force is determined accordingly.

$$V = CW \geq 0.12\text{AIW}$$

$$C = \frac{ABI}{R} = 0.157$$

$$W = 1.3nA(DL + \alpha LL + 4.5)$$

$$W = 17{,}335.89\ kN$$

$$V = 0.157\ (15{,}554) = 2{,}722\ kN$$

For simplicity, to analyze structure under lateral loads, the method proposed by authors is used.

According to the proposed method and considering 12 columns in each floor (**Figure 21**), shear force in each ground column is

$$V_{c,s} = 0.9\frac{V}{n} = 0.9\ \frac{2722}{12} = 204.15\ kN\ Edge\ column$$

$$V_{c,m} = 1.20\frac{V}{n} = 1.20\ \frac{2592.3}{12} = 272.20\ kN\ Middle\ columns$$

Assuming the contraflexure point at 0.8 h from the support, the bending moment in the ground floor is equal to

Edge column:

$$M_{c,bot} = 0.8hV_c = 0.8(3.1)204.15 = 506.29\ kNm\ at\ the\ support$$

$$M_{c,top} = 0.2hV_c = 0.2\ (3.1)204.15 = 126.57\ kNm\ at\ the\ top\ of\ column$$

Middle column:

$$M_{c,bot} = 0.7hV_c = 0.7(3.1)272.20 = 590.67\ kNm\ at\ the\ support$$

$$M_{c,top} = 0.3hV_c = 0.3\ (3.1)272.20 = 253.14\ kNm\ at\ the\ top\ of\ column$$

In the first floor (V = 2592.3 kN), also the shear forces in the side and middle columns are not equal

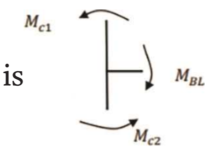
$$V_{c,s} = 0.70\frac{V}{n} = 0.70\ \frac{2592.3}{12} = 151.21\ kN\ side\ column$$

$$V_{c,m} = 1.2\frac{V}{n} = 1.2\ \frac{2592.3}{12} = 259.23\ kN\ middle\ columns$$

Since in other floors, the contraflexure point is in the middle of the column, the bending moment at the first floor and in the side columns are equal to

$$M_c = \frac{V_c h}{2} = \frac{151.21(3.0)}{2} = 226.81\ kNm$$

Using equilibrium in Joint A2, the bending moment of the beam is

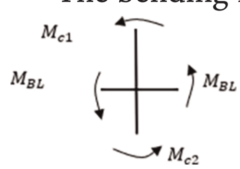


$$M_{BL} = M_{c1} + M_{c2} = 126.57 + 226.81 = 353.38\ kNm$$

Bending moment in the middle column of the first floor is

$$M_c = \frac{V_c h}{2} = \frac{259.23(3.0)}{2} = 388.85\ kNm$$

The bending moment of the beams in the left and right hand of joint B2 is equal to



$$M_{BR} = M_{BL} = \frac{(M_{c1} + M_{c2})}{2} = \frac{253.14 + 388.85}{2} = 321.00\ kNm$$

The summary of the results is shown in **Figure 22**.

3.5.3 Preliminary design of structural elements

3.5.3.1 Beams

According to Eq. (32), the minimum dimensions of the beam B (**Figure 22**) assuming singly reinforced section are equal to

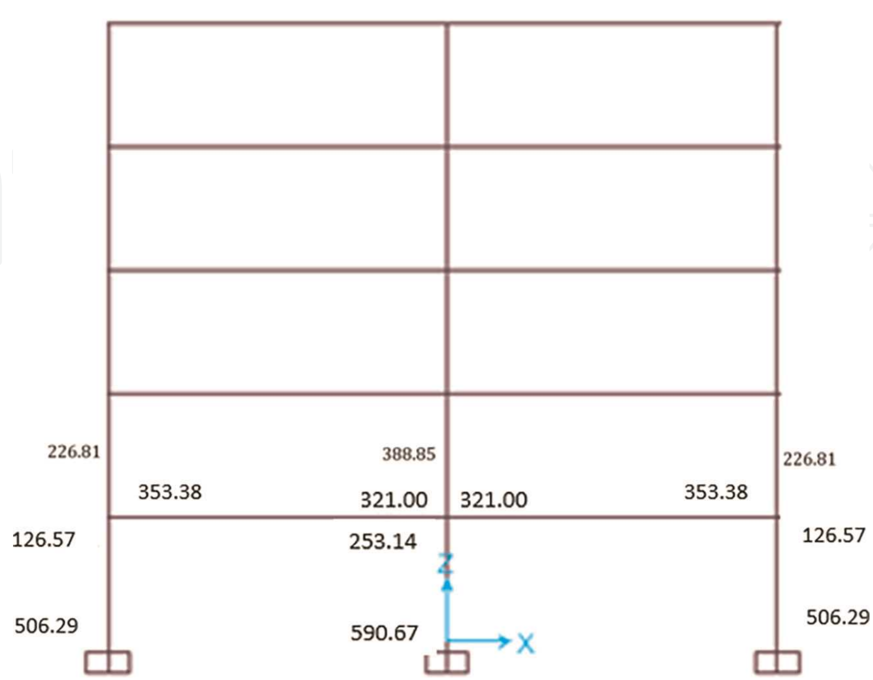


Figure 22.
Proposed method; approximate analysis – bending moment under lateral force kNm – Fame B.

	Story	G, 1	2,3	4,5
Columns	550(550)	450(450)	400(400)	mm
Beam	350(550)	350(450)	300(400)	mm

Table 5.
Summary of initial design.

$$bd^2 = \frac{M_{Ed}}{0.2f_c}, b = 0.65d, h = d + 55mm$$

Load combo : $1.2D + L + E$ (Iranian code)

$$M_{Ed} = 1.2(50.13) + 20.06 + 353.38 = 433.60kNm$$

$$bd^2 = \frac{433.60E6}{0.2(30)}, b = 0.65d, d = 480.55\ mm$$

$$b = 0.65d = 0.65(480.55) = 312.35\ mm$$

$$h = 480.55 + 55 = 535.55\ mm,$$

$$use\ h = 550\ mm$$

$$b = 350\ mm$$

Furthermore, according to **Table 3**, minimum depth of the beam in unbraced fame system in seismic zone is L/18.5 for continuous beam (ACI/Iranian code). To consider the effect of seismic for on the dimensions author propose Eq. (33):

$$\text{Effective deptd} \geq 1.5\text{d}_{\min} = 1.5\left(\frac{4.55}{18.5}\right) = 368.91\ mm$$

$$h = 368.91 + 55 = 423.91\ mm, h = 450\ mm$$

$$b = 0.65h = 0.65(450) = 292.5\ mm, b = 300\ mm$$

Final dimension based on design: 350(550) mm

3.5.3.2 Columns

In a practical design, it is better to calculate the initial dimensions of the columns considering the earthquake force **Table 5**. For this purpose, the author suggests a simple method.

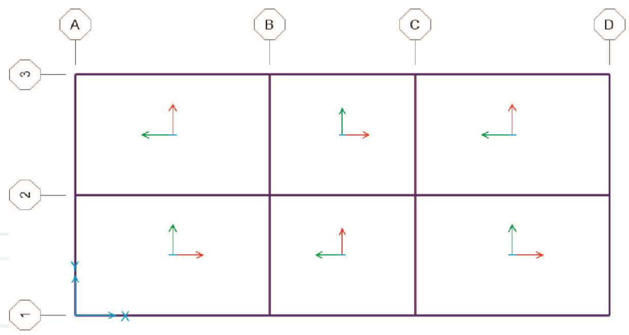
In an Unbraced six story RC frame, the initial dimension can be calculated as follow

$$I > 14.5\frac{VC_d h_s^2}{nE} = 14.5\frac{2722(1000)4.5(3200)^2}{12(25000)} = 60624384400\ mm^4$$

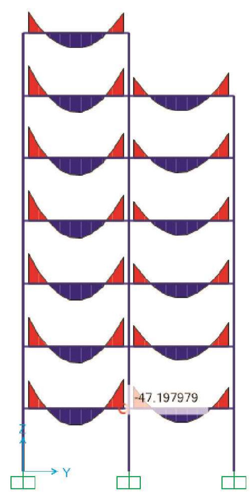
$$\frac{bh^3}{12} = 60624384400, b = h = 519.35\ mm\ use\ b = h = 550\ mm$$

3.5.4 Verifications

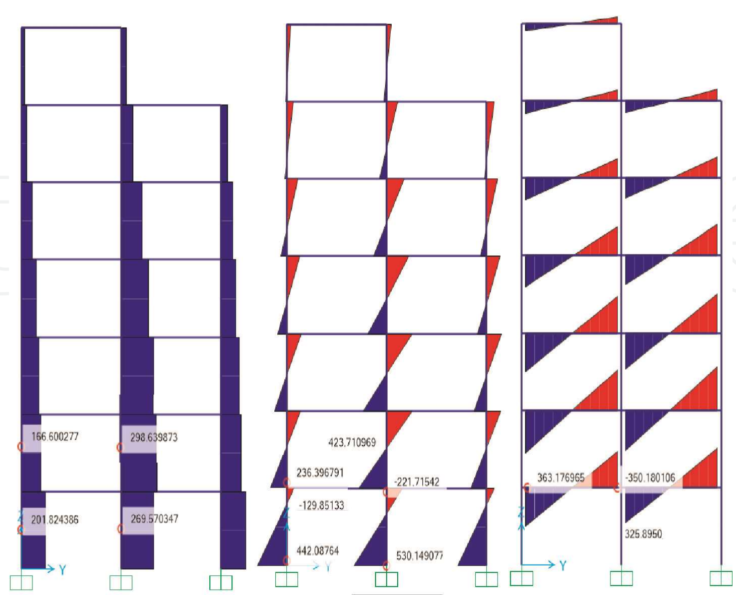
Manual analysis skills are considered essential for a designer. In the absence of dependable manual analysis to validate software analyses and modeling, software



(a)



(b)



(a) (b) (c)

Figure 23.
Software analysis under seismic load. (a) Plan and joists direction (red arrows). (b) Bending moment under dead load (permeant action): Frame B. (c) Proposed method: $M_{DL} = 50.13$ *kNm Shear force in columns (Kn) (d) Bending moment in columns, kNm (e) Bending moment in beams, kNm.*

analysis and design may result in a catastrophe. In order to validate the loading, structural modeling, and reliability of the analyses, the software analysis (**Figure 23**) was compared with approximate analyses proposed by the author (**Figure 22**). The results show that the difference is between 1 and 12%. Thus, it can be inferred that the modeling and results obtained by software are deemed acceptable.

The comparison of the bending moments under gravity load derived from the proposed method, which yields $M_{DL} = 50.13\ kNm$, and the software analysis, which results in $M_{DL} = 47.19\ kNm$ (as illustrated in **Figure 23a**), indicates a discrepancy of 5.8%.

Additionally, a comparison between the approximate analysis (**Figure 22**) and the software analysis (**Figure 23**) conducted under lateral seismic load reveals discrepancy in columns ranging from (129.85–126.57)/129.85 = 2.5% to (590.67–530.14)/590.14 = 10.03%. Additionally, for the beams, the variation is observed to be between 2.7 and 8.22%.

3.6 Approximate analysis and design of shear wall

When selecting the dimensions of the shear wall section, it is imperative to ensure that it meets the necessary criteria:

a. The section must possess adequate strength to withstand the applied factored axial force, bending moment, and shearing force.

b. The section should also exhibit adequate lateral stiffness to restrict lateral displacement within the acceptable limits stipulated by the relevant building code.

There is no universally accepted approach for selecting the dimensions of a shear wall (**Figure 24**). To establish a suitable criterion for the initial design of these dimensions, the author suggests an approximate method [[2], chapter 11]. This method estimates the minimum required stiffness of the wall to restrict the lateral displacement within an acceptable value. The assumption is made that the wall

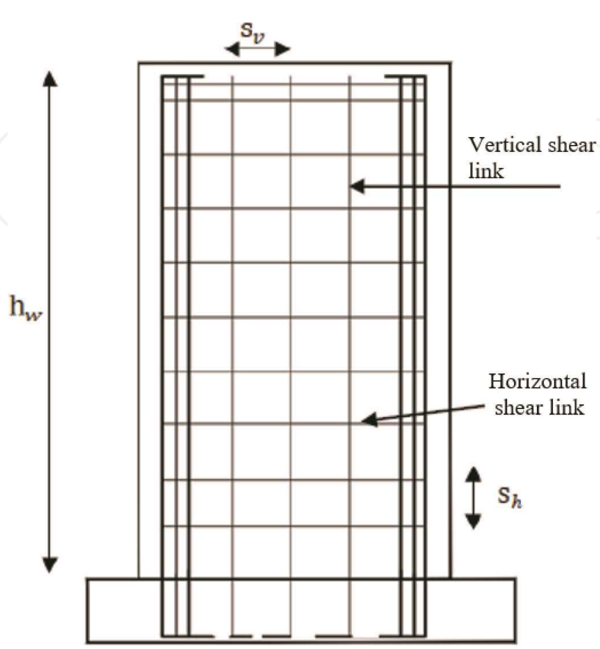


Figure 24.
Details of a typical shear wall.

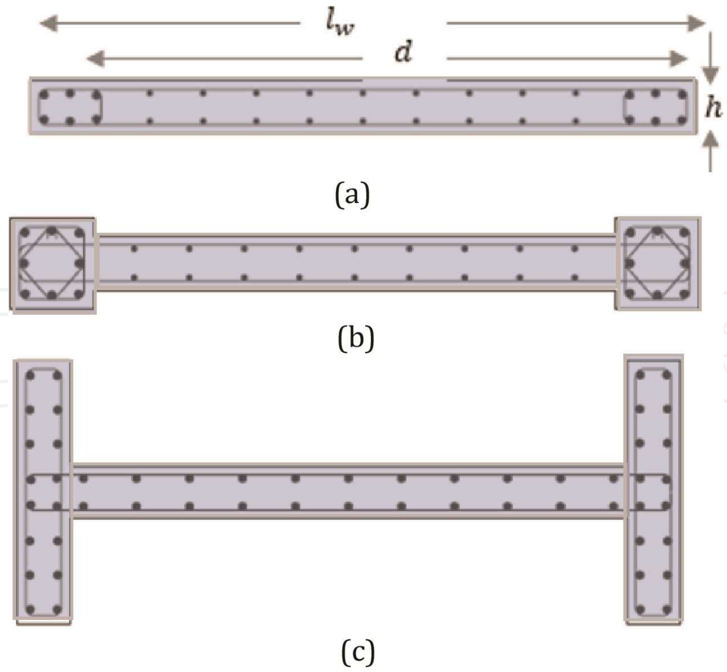


Figure 25.
Shear wall with boundary elements. (a) Boundary element inside the wall. (b) Shear wall with enlarged boundary element. (c) Shear wall with concentrated reinforcement in wings.

behaves like a cantilever beam subjected to a uniformly distributed load, with a constant rigidity of EI.

3.6.1 Wind load

To calculate the minimum uncracked moment of inertia for all walls in the direction of the wind load, considering a maximum drift of 1/500 of the story height, the following formula can be used [12]:

$$\sum I_g = \frac{500Vh_w^2}{2.8E_c} \tag{41}$$

where h_w is the wall height (**Figure 24**).

3.6.2 Seismic load

By employing a comparable approach, considering the code limit on the displacement, the minimum moment of inertia of the uncracked walls can be determined in the desired earthquake force direction [[2], Chapter 11]

$$\sum I_g = \frac{C_d Vh_w^2}{0.07E_c} \tag{42}$$

Once the minimum $\sum I_g$ is determined, the walls can be chosen in a manner that ensures the minimum moment of inertia, as specified in Eq. (42), is adequately provided.

$$I_{gi} = \sum I_g/n = \frac{hl_w^3}{12} \tag{43}$$

where h is the thickness and l_w is the length of the shear wall (**Figure 25**).

In the desired direction, the designer will assume the number of considered walls as 'n', and I_{gi} represents the second moment inertia of each wall. In accordance with the national regulations [13], the minimum thickness of the wall can be determined based on the shear resistance criterion. This determination of thickness of shear wall is determined assuming the maximum shear force that the wall can withstand, V_u, equal to maximum shear resistance of shear wall, along with a 50% safety factor

$$h \geq \frac{V_u}{0.5n\sqrt{f_c}d} \; where \; h \geq \frac{h_w}{15} \tag{44}$$

For the initial design, d is assumed to be $0.8l_w$ ($d = 0.8l_w$) Replacing h in Eq. (43) by Eq. (44), the length of the each wall can be calculated as follows:

$$l_w = \sqrt{\frac{4.8\,n\sqrt{f_c}}{V_u} I_{gi}} \tag{45}$$

Usually, initially, I_{gi} of each wall is determined from Eq. (43), and then the initial length of the wall is calculated from Eq. (45), and finally, the initial thickness of each wall is obtained from Eq. (44). During the optimisation procedure, the dimensions of the wall can be changed.

Although it is deliberately simplified in the above calculations, it provides a close approximation to reality. It is clear that the final decision regarding the specifications of the walls will be made based on computer analyses and taking into account the effect of the frames and regulations. In addition, this method is also a strong and reliable tool for verifying the results of computer analysis.

Considering the size and height of a building, various shear walls are used (**Figure 25**). According to the ACI regulations [6], if the compressive stress caused by the joint effect of axial force and bending moment at the end of the compressive zone, under factored loads, is more than $0.2f_c$, the boundary elements (**Figure 25**) should be used. The relevant details are described in Chapter 14 [2]. The stress is calculated assuming that the walls are not cracked:

$$\sigma = \frac{P}{A} + \frac{M}{W} \geq 0.2f_c \tag{46}$$

Irrespective of the wall type, the procedure for determining the equivalent axial force remains consistent. The equivalent axial force acting on the boundary elements can be computed in the following manner (**Figure 26**).

$$\mathrm{P}_{ueq1} = \frac{P_u}{2} - \frac{M_u}{(0.8l_w)} \text{ Tensile force} \tag{47}$$

$$\mathrm{P}_{ueq2} = \frac{P_u}{2} + \frac{M_u}{(0.8l_w)} \text{ Compressive force}$$

The initial dimensions of boundary elements can be determined using the above equivalent axial force on the boundary elements

$$h_1 b_1 \text{ or } h_2 b_2 \geq \frac{\mathrm{P}_{ueq2}}{0.7f_c} \tag{48}$$

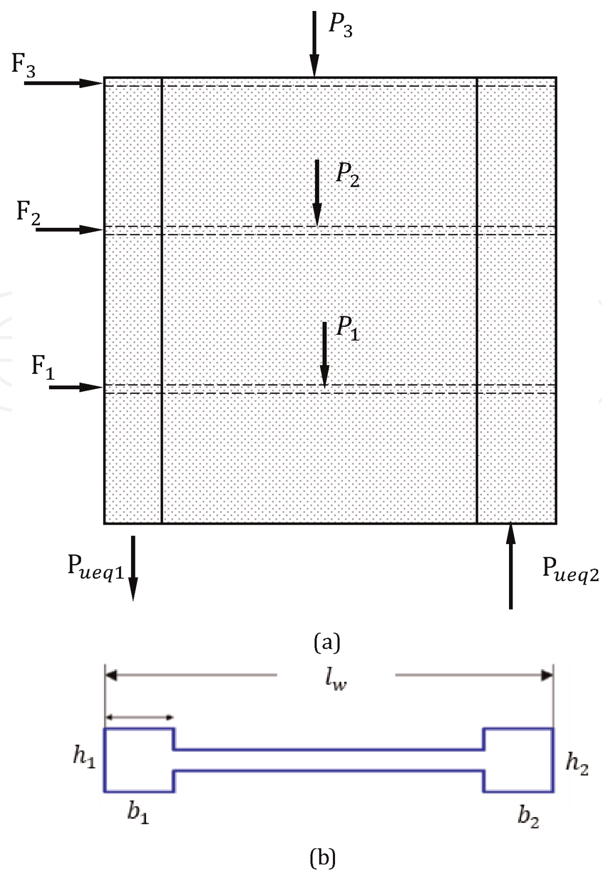


Figure 26.
Geometric characteristics of common shear walls for the simplified C and T method. (a) Equivalent axial force on boundary elements. (b) Cross section.

$$A_{st} \geq \frac{P_{ueq1}}{0.9 f_y} \tag{49}$$

A_{st} is the area of longitudinal rebars in the columns or in boundary elements.

3.7 Case study 3: Shear Wall subjected to lateral loads from earthquakes

In a five-story concrete structure featuring a flat-slab design, in axis X, three symmetrical shear walls are employed to resist lateral forces. As illustrated in **Figure 27**, the roof experiences a dead load of 6.5 kN/m^2 and a live load of 2.0 kN/m^2. The task involves designing a shear wall that incorporates a boundary element. At the base of the wall, the dead load is determined to be 220 kN, while the reduced live load is 50 kN. The concrete utilized has a compressive strength of $f_c = 25 MPa$, and the reinforcing bars exhibit a tensile strength of f_y =400 MPa.

1. Estimation of the basic shear force. Using Eqs. (6) and (8) and assuming C = 0.165, the basic shear force would be

$$V = CW$$

$$W = nA(DL + 0.2LL + 4.5)1.3$$

$$W = 5(228)(6.5 + 0.2(2) + 4.5)1.3 = 16{,}894.8\ kN$$

$$V = 0.1586[16{,}894.8\ ] = 2680.44\ kN$$

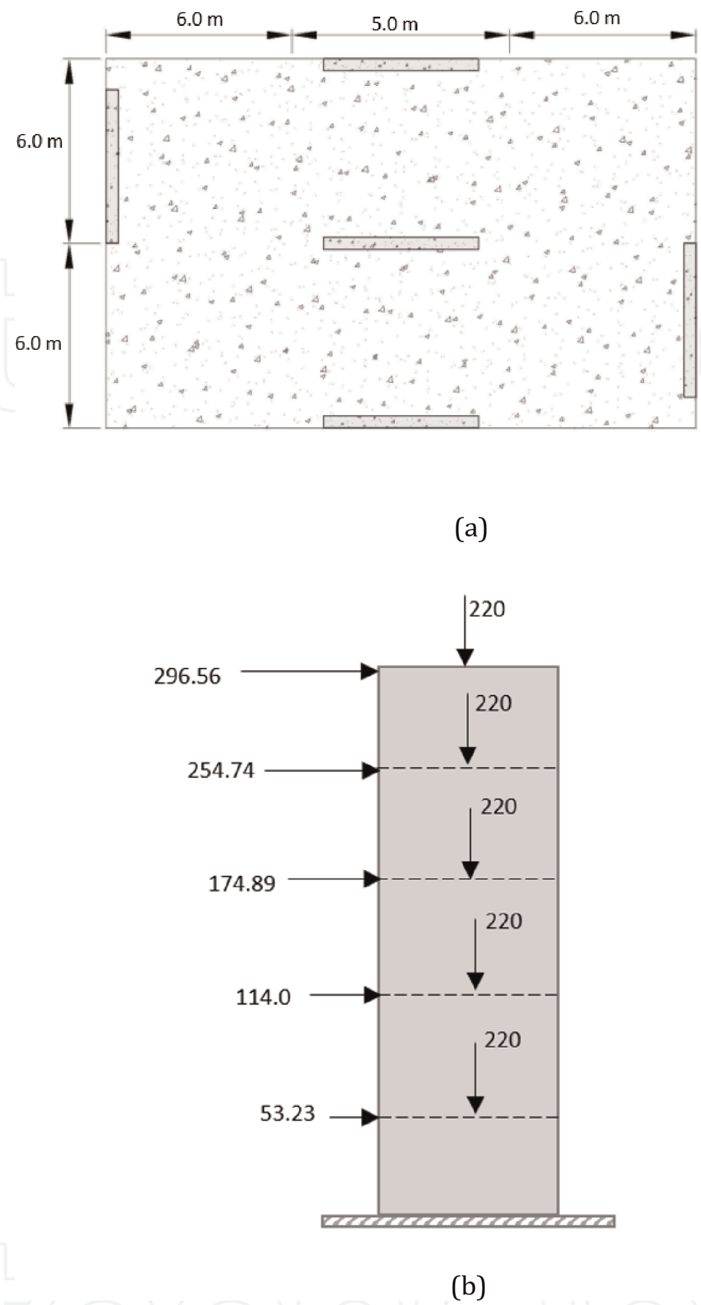


Figure 27.
Shear wall. (a) Plan. (b) Lateral load on each wall (kN): the height of ground floor is 2.8 m and the rest is 3.2 m.

2. Initial dimension

$$\sum I_g = \frac{C_d V h_w^2}{0.07E_c} = \frac{5.5\ (2680.44)1000(15600)^2}{0.07(25000)} = 2.05E12$$

where
C_d = displacement magnification factor, e.g., $C_d = 5.5$
E = 25,000 MPa
Consequently, the moment of inertia for each wall is determined to be $I_{gi} = 2/05\text{E}12/3 = 6.83E11$. The wall length is derived from Eq. (45)

$$l_w = \sqrt{\frac{4.8\, n\sqrt{f_c}}{V_u} I_{gi}} = \sqrt{\frac{4.8(3)\sqrt{25}}{2680.44(1000)} 6.83E11} = 4283.25, l_w = 5000\ mm$$

To calculate the thickness Eq. (44) is used

$$h \geq \frac{V_u}{0.5n\sqrt{f_c}d} = \frac{2680.44(1000)}{0.5(3)\sqrt{25}(0.8)5000} = 89.35\ where\ h \geq \frac{h_w}{15} = \frac{2800}{15}$$
$$h = 186.67\ mm$$

The minimum thickness must be 300 mm, as the thickness of the boumadry elements is equivalent to that of the shear wall, so $h = 300\ mm$ and $l_w = 5000\ mm$.

According to Iranian code [13], to design shear walls, two load combinations, $1.2D + 1.0L + 1.0E$ and $0.9D + 1.0E$, are taken into account to calculate critical bending momnet and axial force on each wall (**Figure 24**).

$$\mathrm{M}_u = 1.0 \sum P_i h_i = 1.0[296.56(15.6) + 254.74(12.4) + 174.89(9.2) + 114.06(6.0) + 53.23(2.8)] = 10227.5\ kNm$$

$$N_{u1} = 1.2[(220)5 + 0.3(5)15.6\ (25)] + 1.0(5)50 = 2272.0\ kN$$

$$N_{u2} = 0.9[(220)5 + 0.3(5)15.6\ (25)] = 1516.50\ kN$$

The maximum stress on the edge of shaer wall is

$$\sigma_c = \frac{N_{u2}}{hl_w} + \frac{6M_u}{hl_w^2} = \frac{2272.0\ (1000)}{300(5000)} + \frac{6(10227.5)10^6}{300(5000^2)} = 9.69\ MPa$$

As $\sigma_c = 9.69MPa$ is more than $0.2f_c = 5\ MPa$, the boundary elements needs to be provided. In this category of shear walls, the thickness of boundary elements is equivalent to the shear wall itself; therefore, the length of the boundary can be determined using Eq. (47)

$$\mathrm{P}_{ueq2} = \frac{N_{u2}}{2} + \frac{M_u}{(0.8l_w)} = \frac{2272.0}{2} + \frac{10227.5}{[0.8(5.0)]} = 3692.87kN$$

$$h_1 b_1 \text{ or } h_2 b_2 \geq \frac{\mathrm{P}_{ueq2}}{0.7f_c} = \frac{3692.87(1000)}{0.7(25)} = 211021.14 \text{ mm}$$

$$b_1 = b_2 = \frac{211021.14}{300} = 703.41\ mm, b_1 = 750\ mm$$

The area of longitudinal reinforcement in boundary elements is determined using Eq. (49)

$$A_{st} \geq \frac{\mathrm{P}_{ueq1}}{0.9f_y}$$

$$\mathrm{P}_{ueq1} = \frac{N_{u1}}{2} - \frac{M_u}{(0.8l_w)} = \frac{1516.50}{2} - \frac{10227.5}{[0.8(5.0)]} = -1798.63kN$$

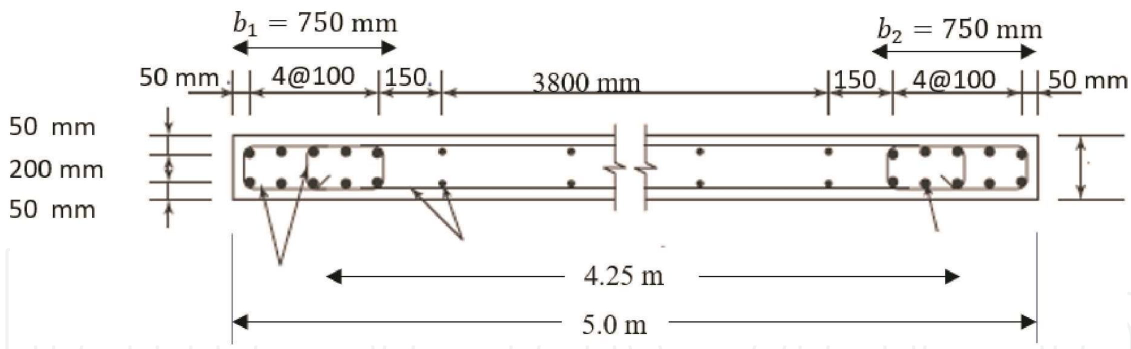
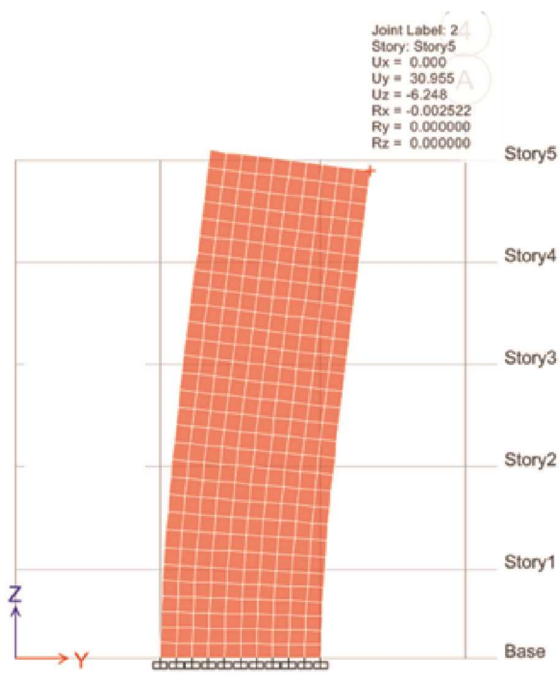


Figure 28.
Shear details.



(a)

Flexural Design for P and M_3 —Tension Reinforcement

Station Location	Edge Length mm	Rebar Area mm²	Tension Combo	P_u kN	M_{u3} kN-m
Left Top	600	3122	DWal7	1264	7725.76
Right Top	600	3122	DWal8	1264	-7725.76
Left Bot	750	4812	DWal7	1348	10227.504
Right Bot	750	4812	DWal8	1348	-10227.504

(b)

Figure 29.
ETABS model. (a) Softwar model. (b) Results.

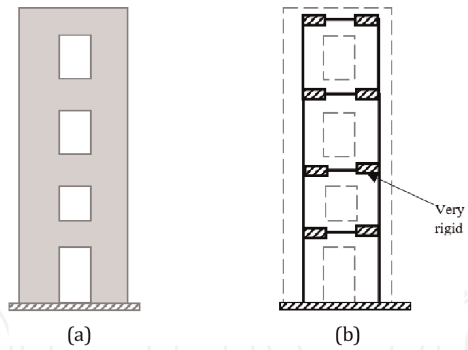


Figure 30.
Idealized coupled shear walls model. (a) Coupled shear wall (b) Idealized structural model.

$$A_{st} \geq \frac{1798.63(1000)}{0.9(400)} = 4996.19mm2\ use\ 10H25$$

To demonstrate the effectiveness of the proposed method, the identical project was simulated using ETABS. The findings indicate a strong correlation of the proposed method (**Figure 28**) with ETABS results (**Figure 29**) regarding the dimensions of the boundaries and the area of the longitudinal reinforcement bars.

3.8 Approximate analysis of coupled shear walls

Shear walls can be simplified by using an equivalent frame, as shown in **Figure 24**. In this frame, the column's moment of inertia matches that of the wall on each side of the opening. Similarly, the beam's moment of inertia matches the wall between the upper and lower openings. The moment of inertia of the beam in the rigid section, outside the openings, can be assumed as $100I_b$ (**Figure 30**).

Finite element method is commonly employed in the analysis and design of coupled walls in practice, thanks to the availability of reliable commercial software such as SAP2000 and ETABS. This eliminates the necessity for certain simplifications. Nevertheless, in order to comprehend the behavior and lay down a foundation for verifying the outcomes of computer analysis, an approximate analysis method is suggested here. This method is based on the results of software analysis carried out on buildings with 8, 12, and 16 stories by the author. Further information regarding these specific types of walls can be found in Chapter 14 [2].

The bending moment, M_o, and shear force, V_{si}, on each story can be determined as follows:

$$M_o = \sum F_i h_i \quad (50)$$

and

$$V_{si} = \sum F_i \quad (51)$$

The absence of a turning point at the same location as the story height in the coupling wall prevents manual calculation of the shear force, unlike in frames. The software analysis conducted by the author on the coupled walls [[2], chapter 11] reveals that, in a minimum of 50% of the stories, the turning point in the columns is situated very near the top of the story. As we progress upward, the turning point gradually shifts toward the middle of the story in the upper stories. Nevertheless, by assuming the turning point to be in the middle of the coupled beam, the shear forces up to the initial 50% of the stories can be determined conservatively

$$V_{bi} = V_{si}(0.8)h_w/(l_b) \tag{52}$$

Where V_{bi} is the beam's shear force, V_{si} is the shear force in the story, h_w is the story height, and l_b is the length of coupled beam.

For upper stories, the shear force of the coupling beam is calculated as follows:

$$V_{bi} = [(V_{si}h_{wi}/2 + V_{si+1}h_{wi+1}/2)]/(l_b) \tag{53}$$

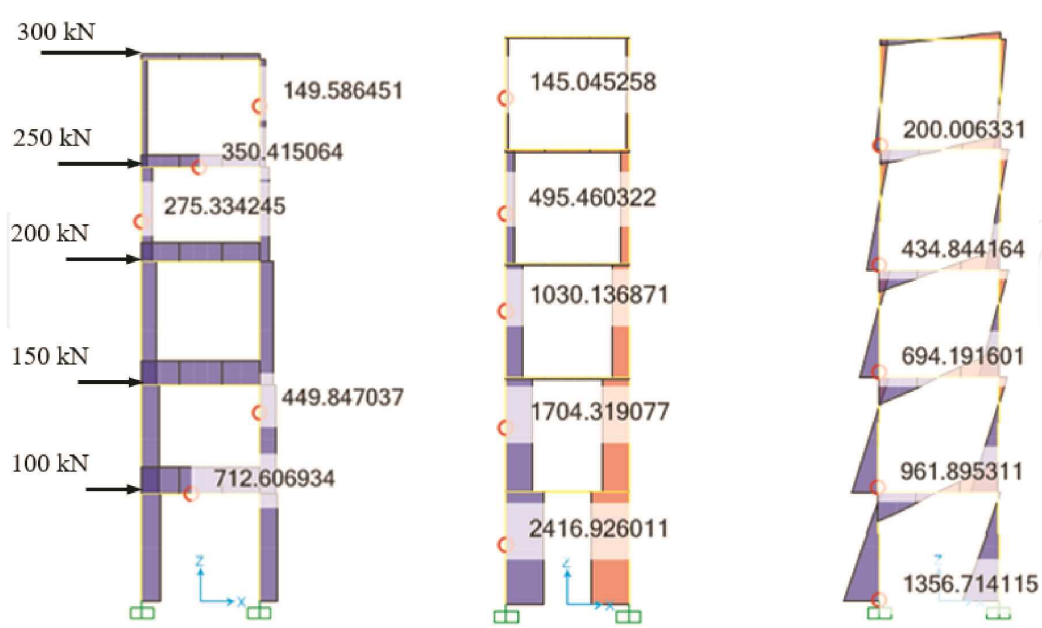
where V_{si+1} is the shear force of the story and h_{wi+1} is the height of the column above the coupling beam.

The axial force – tensile or compressive – acting on the walls is obtained from the sum of the shearing force acting on the coupling beams in the floors above the desired column:

$$T_o = \sum_{i=1}^{n} V_{bi} = C_o \tag{54}$$

After calculating V_{bi}, the bending that is tolerated by the walls, $T_o l$, can be determined. Therefore, the contribution of the walls from the bending moment of M_o can be calculated [2]

$$M_{w1} + M_{w2} = M_o - T_o l_b \tag{55}$$



(a) (b) (c)

Figure 31.
*Analysis of the coupled wall assuming the beam- column model similar to **Figure 29b** using SAP 2000, span 3.2 m, floor height 3 m. (a) Lateral force and shear force diagram (b) Axial force diagram (c) Bending moment diagram.*

3.9 Case study 3: Coupled shear wall

To confirm the aforementioned relationships (Eqs. (52)-(54)), a coupled shear wall was simulated in SAP2000, as illustrated in **Figure 31b**. Based on Eq. (52), the shear force acting on the link beam of the first floor is equivalent to

$$V_{b1} = \frac{V_{s1}(0.8)h_{s1}}{l_b} = \frac{1000(0.8)3.0}{3.2} = 750\ kN$$

Also, the shear force acting on the link beam of the fourth floor is equivalent to

$$V_{b4} = [(V_{s4}h_{s4}/2 + V_{s5}h_{s5}/2)]/(l_b)$$

$$V_{b4} = \frac{[550(3)/2 + 300(3)/2]}{3.2} = 398.44\ kN$$

The results presented indicate that the shear forces experienced by the beams on the first and fourth floors closely align with the findings from SAP 2000, as illustrated in **Figure 31a**.

Using **Figure 31a**, we have

$$M_o = \sum F_i h_i = 10500\ kNm$$

$$T_o = \sum_{i=1}^{n} V_{bi} = C_o = 2416.93\ kN$$

The axial force of shear walls, as illustrated in **Figure 31a** and expressed in Eq. (54), aligns closely with the results obtained from the software, as shown in **Figure 31b**.

In this instance, the ratio $T_o l_b / M_o$ is determined to be 0.74, suggesting that the structure can be classified as a coupled beam [13]. Given that the moments M_o and $T_o l_b$ are specified in Eq. (55), it is possible to compute the moment contribution arising from the shear walls.

$$M_{w1} + M_{w2} = 10500 - 2416.93(3.2) = 2765.82\ kNm$$

As the stiffness of both walls is identical

$$M_{w1} = M_{w2} = 1381.91\ kNm$$

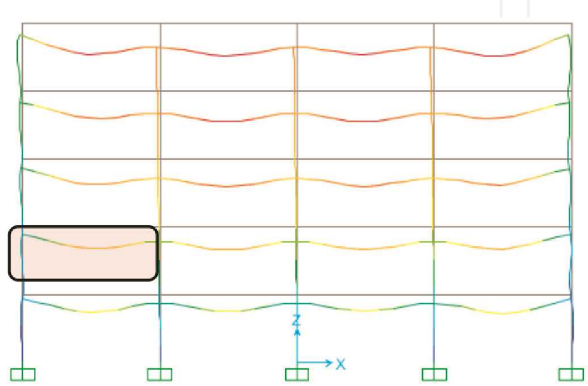


Figure 32.
Deflection due to gravity loads.

which agrees well with software result (**Figure 31c**)

The analysis of the proposed analytical method in conjunction with the outcomes from SAP2000 indicates that the method in question possesses adequate precision for the validation of the projects.

3.10 Approximate deflection

According to the developed method by author [2], in a continuous beam of a rigid frame (**Figure 32**), the deflection in each span can be determined using the following expression

$$y_{max} = \frac{5l^2}{48EI_e}[M_s - 0.1(M_2 + M_1)] \tag{56}$$

where

M_s is the positive bending moment in the middle of span length and M_1 *and* M_2 are the end bending moment. I_e is effective moment of inertia; $I_e = 0.5I_g$, and E is elastic module; $E = 5000\sqrt{f_c}.f_c$ is the compressive strength of concrete.

Considering the method of ACI coefficients for the analysis of beams in rigid frames, the maximum deformation in the side and middle spans is calculated as follows:

Side span lengths

$$M_1 = \frac{ql^2}{16} M_2 = \frac{ql^2}{10} \tag{57}$$

By substituting the above values into Eq. (56), the maximum deflection in the side spans is equal to

$$y_{max} = \frac{ql^4}{174EI_e} \tag{58}$$

Middle span lengths

$$M_1 = \frac{ql^2}{11} \qquad M_2 = \frac{ql^2}{11} \tag{59}$$

By substituting the above bending moments into Eq. (56), the maximum deflection in the middle spans is determined to be equal to

$$y_{max} = \frac{ql^4}{217EI_e} \tag{60}$$

Eqs. (58) and (60) can serve as effective expressions for validating computer modeling as well. If the computer-generated results for the vertical deflection of the beams or slabs closely match the aforementioned values or exhibit an acceptable deviation, along with the approximate structural analyses, it can be inferred that the loading data and computer modeling have been verified.

3.11 Lateral displacement

The previous section covers the analysis of deflection in continuous beams. Various deformations must be considered when designing reinforced concrete frames.

One of the most significant deformations is the lateral displacement of the frames. It is crucial to control this displacement to avoid causing discomfort to residents and damage to partitions, facades, and windows. As lateral displacement can result in an increase in the P-Δ effect, which is inversely proportional to the lateral stiffness, controlling it becomes crucial. Typically, software is used to calculate the amount of lateral displacements but manual calculations is crucial for software structural and analysis verifications.

The horizontal displacement of each floor level can be determined based on the shear force and stiffness of the structure. If we consider the floors to be rigid, the lateral stiffness of the frames on each story can be computed in the following manner [8]:

$$k = n\frac{12EI}{h^3} \tag{61}$$

Based on the research conducted by the author [2], it has been found that the stiffness mentioned above is significantly greater than the true value. Consequently, according to analytical calculation and software modeling, the author suggests a modified stiffness that takes into account the beams when calculating the stiffness of the frames.

$$k = n\frac{5EI}{h^3}\ \text{Ground floor} \tag{62}$$

$$k = n\frac{3EI}{h^3}\quad \text{Other floors} \tag{63}$$

Upon determining the shear force and story stiffness, the relative displacement of each story, denoted as Δ, can be calculated in the subsequent manner.

Where

Δ_{rei} is relative displacements

V_i is shear force in the story

K_i is story stiffness

$$\Delta_{rei} = \frac{V_i}{k} \tag{64}$$

The procedure to calculate lateral displacement in a five-story building is described in the **Table 6**. The method can be applied to buildings with various stories.

Story	Lateral force (kN)	Shear force (kN) V_i	Story stiffness, N/mm	Relative displacement $\Delta_{rei} = \frac{V}{k}$ (mm)	Lateral displacement (mm) $\Delta_i = \sum \Delta_{rei}$
4	F4	F4	K4	V4/K4	$\Delta_{re0} + \Delta_{re1} + \Delta_{re2} + \Delta_{re3} + \Delta_{re4}$
3	F3	F3 + F4	K3	V3/K3	$\Delta_{re0} + \Delta_{re1} + \Delta_{re2} + \Delta_{re3}$
2	F2	F2 + F3 + F4	K2	V3/K2	$\Delta_{re0} + \Delta_{re1} + \Delta_{re2}$
1	F1	F4 + F3 + F2 + F1	K1	V1/K1	$\Delta_{re0} + \Delta_{re1}$
Ground	F0	F0 + F1 + F2 + F3 + F4	K0	V0/K0	Δ_{re0}

Table 6.
Lateral deflection (Example for five stories).

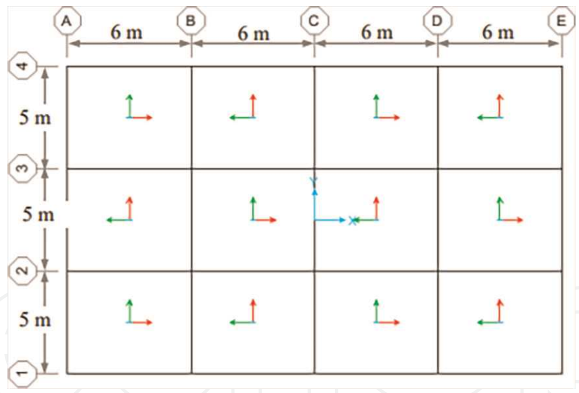


Figure 33.
Floor plan.

3.12 Case study 4: Vertical and lateral deflection

In a five-story educational building located in Sanadaj, Kurdistan, a joist system has been utilized for the roofing framework (see **Figure 33**). The following is a summary of the loading conditions:

$$\mathrm{DL} = 4.95\, kN/m^2$$

$$\mathrm{LL} = 3.5\, kN/m^2$$

It is estimated that 20% of the live load may be regarded as a permanent load. In addition, to avoid damaging the partitions, a gap of about 35 mm between the beams and the partitions is filled with foam.

3.12.1 Approximate analysis and design

3.12.1.1 Initial design

In accordance with the methodology outlined in case study 1, the preliminary demotions of the beam and column are shown in **Table 7**.

3.12.1.2 Vertical deflection in the beams

According to the proposed approximate method, deflection resulting from the permanent load, $\delta_{i,sus}$ in beam 2 (**Figure 33**), can be calculated. Given that the lengths of the spans are identical, the maximum deflection will take place in the side spans

$$\delta_{i,sus} = \frac{q_{sus} l^4}{174 E I_e} \text{ Side spans}$$

Story	4	2,3	G, 1	
Columns	450(450)	400(400)	350(350)	mm
Beam	350(500)	350(450)	300(400)	mm

Table 7.
Summary of initial design.

Where
The loading width for beam 2 is 2.5 m, then

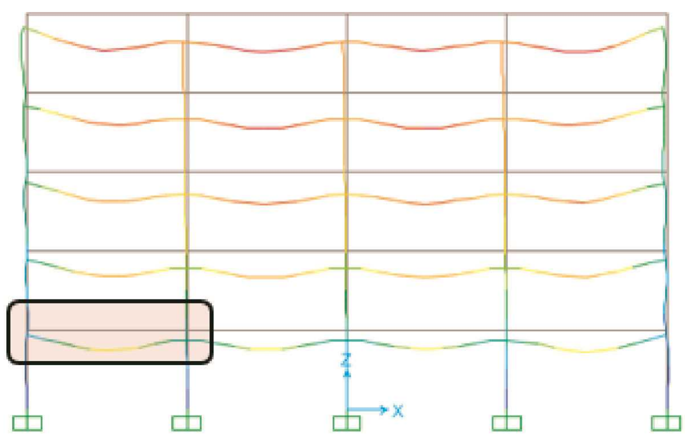
$$q_{sus} = 2.5(4.95 + 0.2(3.5)) = 14.13\,kN/m$$

$$I_e = 0.5I_g = 0.5\frac{bh^3}{12} = 0.5\frac{350(500)^3}{12} = 1.82(10^9)\;mm^4$$

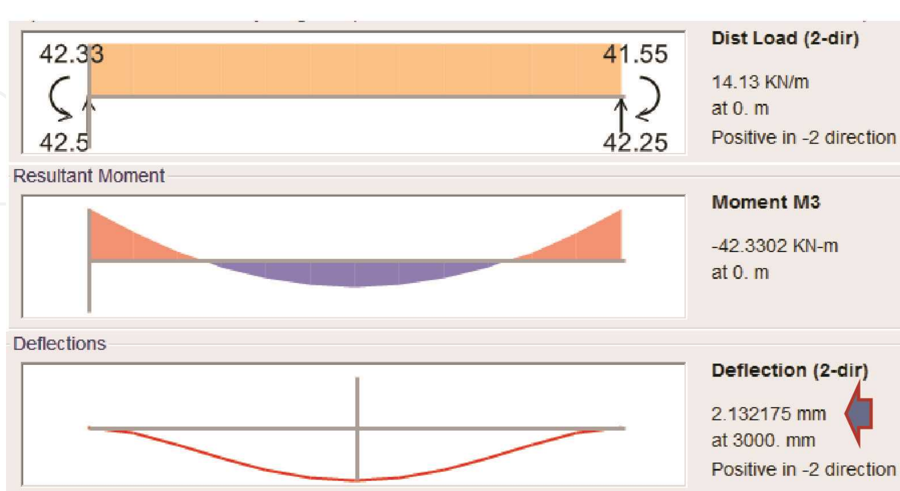
$$E_c = 5000\sqrt{f_c} = 5000\sqrt{25} = 25000\;MPa$$

$$\delta_{i,sus} = \frac{14.13(6000)^4}{174(25000)1.82(10^9)} = 2.31\;mm$$

The same structure is modeled by SAP 2000. The comparison between the outcomes of manual calculations and software analyses demonstrates the effectiveness of the proposed approximate method. As illustrated in **Figure 34**, the immediate



(a)



(b)

Figure 34.
Immediate deflection – software analysis. (a) Deflection under dead load – beam 2. (b) Bending moment diagram and deflection under dead load.

deflection of beam2 derived from software analysis closely aligns with the results obtained from the approximate approach, denoted as $\delta_{i,sus}$.

3.12.1.3 Lateral displacements

To determine the relative displacement of various stories, as indicated in Eq. (64), it is essential to consider the shear force along with the corresponding stiffness values. The lateral forces exerted on the structure are depicted in **Figure 35**, while the stiffness of the individual stories is defined by Eqs. (62) and (63). The method outlined in **Table 6** is employed to estimate the lateral displacement (**Table 8**).

Story stiffens

$$k = n\frac{5EI}{h^3} = 20\frac{5(23500)450^4/12}{3000^3} = 297{,}421\, N/mm \text{ Ground floor}$$

$$k = n\frac{3EI}{h^3} = 20\frac{3(23500)450^4/12}{3000^3} = 178{,}453\, N/mm \text{ Other stories}$$

Additionally, an examination of the lateral displacement presented in **Table 6**, when juxtaposed with the software analyses depicted in **Figures 36** and **37**, reveals that the discrepancy is minimal, measuring less than 7%.

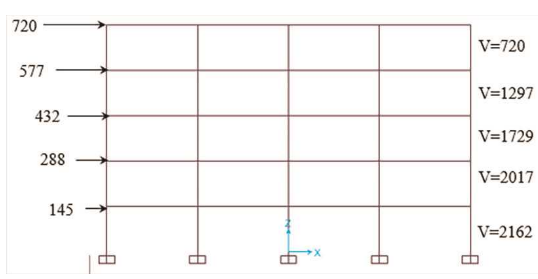


Figure 35.
Lateral load on the building and story shear force.

Story	Lateral force (kN)	Shear force (kN)	Story stifness (N/mm)	Relative displacement $\Delta = \frac{V}{k}$ (mm)	Lateral displacement (mm)
4	720	720	178,453	4.04	39.55
3	577	1297	178,453	7.27	35.51
2	432	1729	178,453	9.67	28.24
1	288	2017	178,453	11.30	18.57
G	145	2162	297,421	7.27	7.27

Table 8.
Lateral displacements- proposed method.

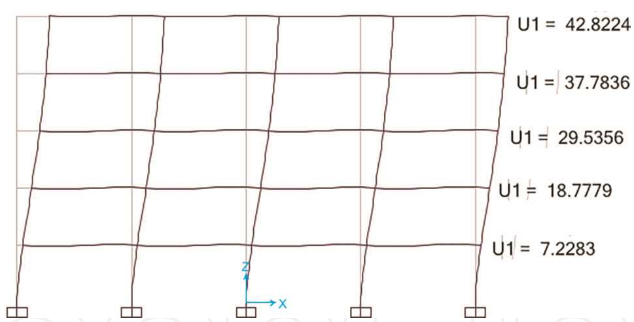


Figure 36.
Lateral displacement – software analysis.

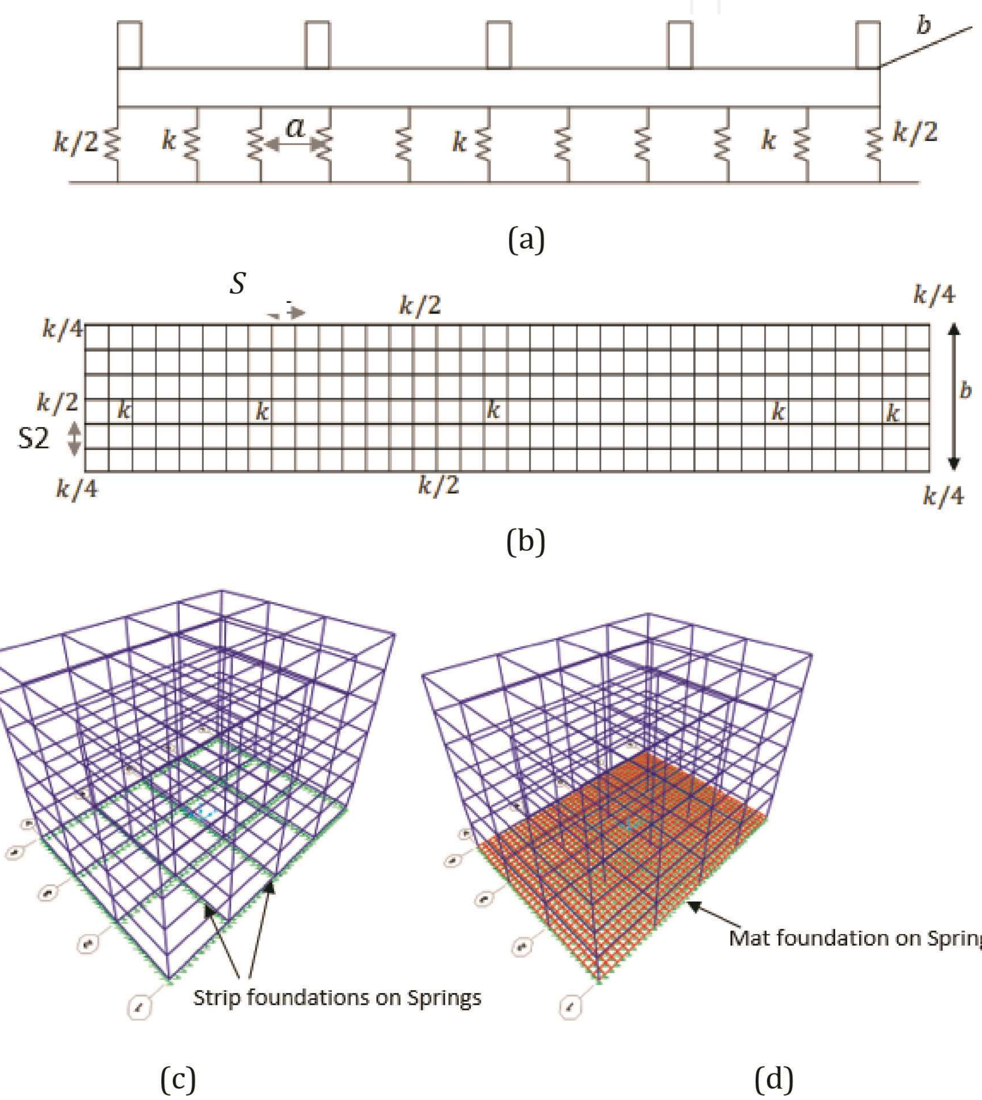


Figure 37.
Approximate analysis of foundations – modeling soil by springs. (a) Beam model; $k = baK_s$ *(b) Shell model;* $k = S_1S_2K_s$ *(c) Using beam element and spring to model strip foundation (d) Using shell element and spring to model mat foundation.*

3.13 Approximate analysis of foundations

SAFE software is utilized for the analysis and design of foundations. This software employs finite element (FE) methods and springs to accurately represent the behavior of the soil. However, due to the complex two-way interaction between the foundation and the soil, manually verifying the analysis of foundations would pose considerable challenges. In order to address this issue and offer a practical verification approach, two methods can be employed depending on the type of foundations being considered.

3.13.1 Two-way strip foundations

To validate the analyses and design conducted using SAFE, it is essential to follow the outlined procedures.

- The primary software utilized for modeling the frame is employed.
- Supports at the base of the columns are eliminated.
- Beam elements, matching the dimensions of the foundations, are used to connect the columns (**Figure 37c**).
- The foundations are segmented, and springs are allocated to the joints (**Figure 37c**). The properties of these springs can be determined based on soil stiffness (K_s) as indicated in the soil report.
- Both the structure and foundation are analyzed within a single model.
- A comparison is made between the bending moment results obtained from this method and those from SAFE, thereby confirming the accuracy of the SAFE results.

3.13.2 Mat foundations

1. *Software*: The previously discussed method can also be utilized in this context. Specifically, for modeling the foundation, a combination of a shell element and a spring is employed (see **Figure 37d**).
2. *Analytical*: Based on the author's experience, employing approximate analysis for flat slabs subjected to gravity loads can achieve an adequate level of accuracy. Each strip (**Figure 38**) may be evaluated utilizing a uniformly distributed load (UDL) of Q_u, along with the coefficient method as outlined in **Tables 9** and **10**.

$$Q_u = \frac{\sum R_{ui}}{bl}$$

Where
R_u – factored reaction supports
b and l are the foundation's dimensions.
It is important to acknowledge that this method is applicable solely to gravity load.

To calculate the thickness of pad, strip, combined, and mat foundation, two methods can be utilized:

1. Thickness of foundation is two times of column dimension

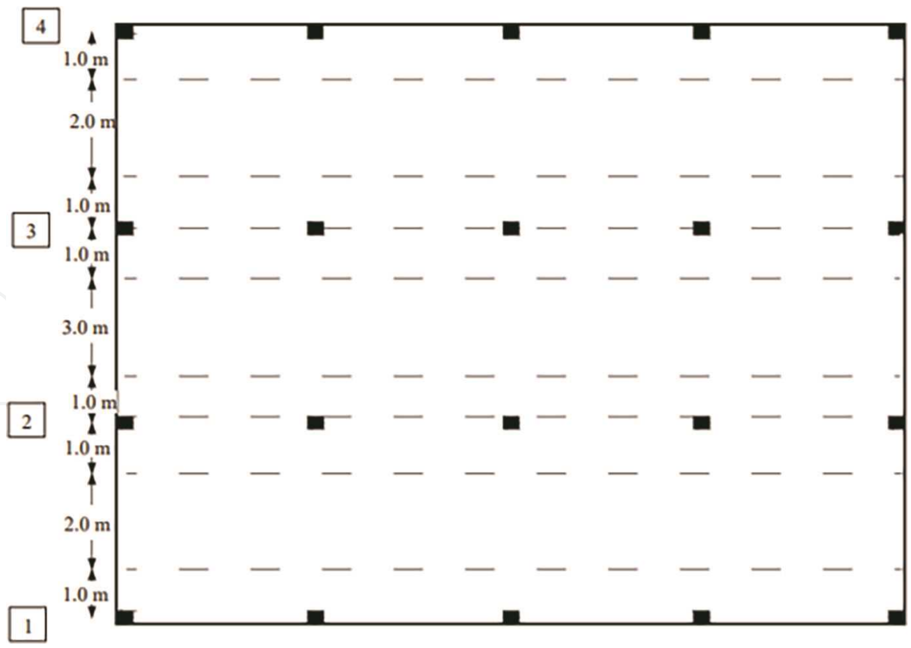


Figure 38.
Mat foundation.

		at the first internal column		*at the middle of span length*	*at the middle columns*	
M_u	α	−0.086		0.063	−0.063	
V_u	β	0.6	0.6	—	0.5	0.5

$M_u = \alpha q_u l_n^2$.
$V_u = \beta q_u l_n$.
$q_u = (l_x\ R\ l_y)Q_u$.

Table 9.
Approximate analysis of mat foundations under gravity load [4].

	Column strip	Middle strip
$M_{uc} = k_{ci}M_u$	$k_c^- = 60–80\%(70\%)$	$k_c^+ = 20–40\%(30\%)$
$M_{um} = k_{mi}M_u$	$k_m^- = 50–65\%(55\%)$	$k_m^+ = 30–50\%(45\%)$

Table 10.
Dividing the bending moment between the middle and column strips [2].

2. Simplified analytical method proposed by author [[2], Chapter 12]

$$(2.6v_c)d^2 + [1.3(c_1 + c_2)v_c]d - P_u = 0 \tag{65}$$

Where
v_c is the two-way shear strength of the concrete
c_1 *and* c_2 is column dimension
P_u maximum factored axial force in the columns
d is effective depth of foundation, so foundation depth is $h = d + 75\,mm$

4. Conclusion

The advanced software available for structural analysis and design is widely utilized by consulting engineers and professionals within the engineering field; however, students typically do not engage with such software during their academic training. It is crucial to emphasize that the application of this software is reserved for engineers who possess adequate knowledge and skills in manual analysis and design. Consequently, a computer cannot supplant the role of an engineer under any circumstances. The primary function of computer software in this context is to enhance efficiency and streamline processes. The relationship between engineers and computers can be likened to that of a surgeon and a scalpel; the effectiveness of the tool is contingent upon the expertise of the individual wielding it. Ultimately, while both the computer and the scalpel serve as valuable instruments, it is the trained professional who is capable of achieving meaningful outcomes. This chapter book addresses and rectifies the aforementioned limitations through a scientific approach.

The newly developed approximate analysis and design methodology for reinforced concrete (RC) structural elements, validated through four case studies, offers a robust foundation for structural designers to verify software modeling, analysis, and design processes for complex projects. Additionally, this approach will assist students in estimating internal forces and dimensions of various elements within multi-story RC structures, particularly in scenarios where traditional analytical methods may fall short.

5. Questions

1. Explain how the material for a specific building is chosen?
2. For a concrete building, what factors are effective in choosing the gravity and lateral bearing system?
3. What is the main difference between tall and low-rise buildings?
4. Deformation of a 12-story concrete building is more than the permissible limit. If only the bending frame is used, increasing the dimensions of the column or beam provides a more economical result, why?
5. In a composite system, the flexural frame alone must sustain 30% of the lateral load alone. Meanwhile, shear walls should also be designed for 100% lateral load. It seems that 130% is considered in the design. Explain the relevant concept?
6. Why does the dual system’s efficiency decrease as the building height increases?
7. Tips to enhance the performance of a flexural frame against lateral load?
8. What is the maximum allowable height for a rigid frame with medium ductility?
9. Define tall building?
10. Illustrate the tubular system by sketching the diagram.
11. Explain the strengths and weaknesses of tubular systems.
12. Discuss the basic philosophy of using tubular systems.
13. What is the economical height for tubular systems?
14. What is the main philosophy of using bundled tubes and tube-in-tube system?
15. Find the best position for outrigger and belt trusses along the height of an 80-story building. Consider the use of one, two, three, or four outrigger and belt trusses.
16. What criteria are used to select gravity load-bearing systems for floors/roofs?
17. Consider three concrete buildings with 5 m, 10 m, and 15 m center-to-center columns. Suggest the most economical roofing system for the buildings. Discuss your choices.
18. The width of a 15-story concrete building is 10 meters, while the width of a 30-story concrete building is 42 meters, which one can be considered a tall building, why?

6. Problems

1. The plan of a seven-story concrete building is shown in **Figure 39**. The building is located in the city of Sanandaj. The building utilizes a medium gravity and lateral load bearing system within its rigid frame. By adjusting the dimensions of columns and beams every two stories, along with selecting the suitable floor system, the building can be effectively designed using the methods outlined in this chapter. The designer has the flexibility to make other necessary assumptions.

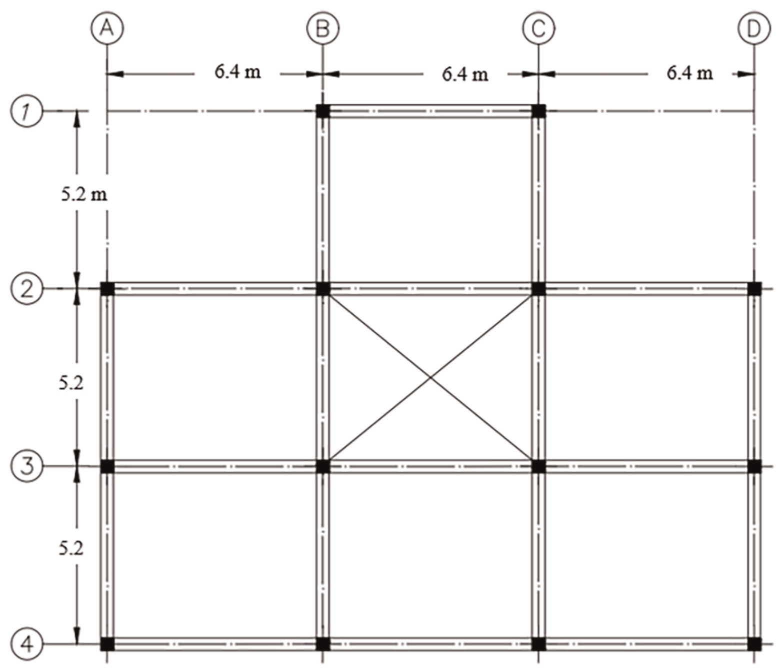


Figure 39.
Floor plan – medium bending frame.

2. The design of a 19-story concrete building in Mahabad is illustrated in **Figure 40**. The structure features a medium bending frame for gravity and lateral support, along with a special shear wall. Utilize the methods outlined in this chapter for approximate analysis and design, ensuring all assumptions are carefully selected by the designer.

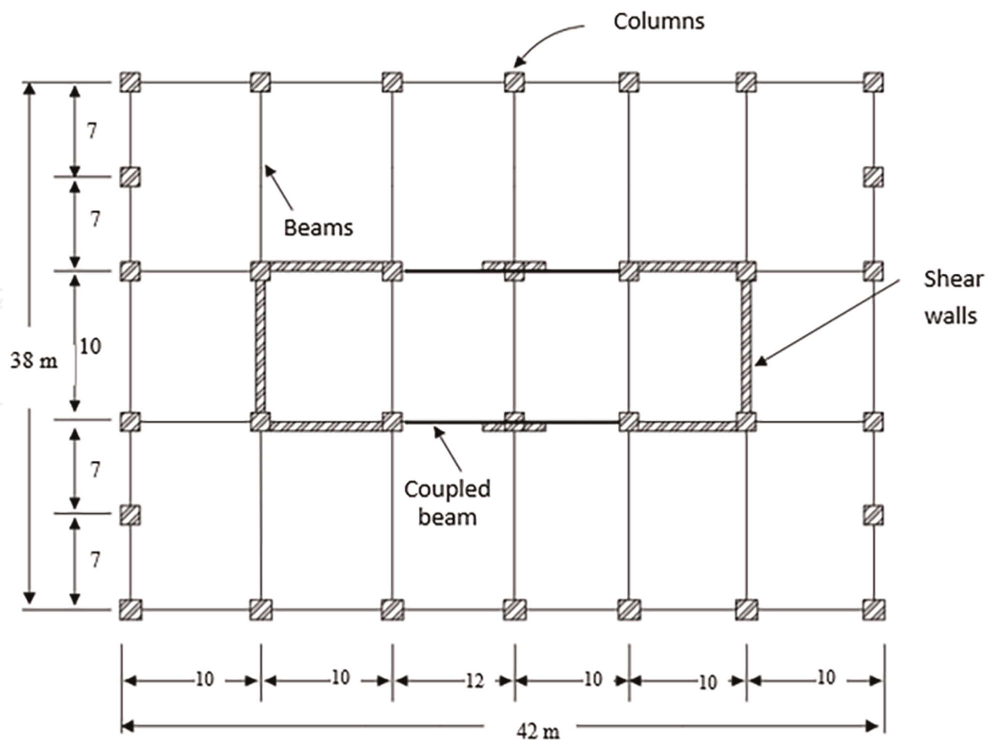


Figure 40.
Floor plan – dual load bearing system.

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