Computers and Concrete, *Vol. 17, No. 5 (2016) 683-702*
DOI: http://dx.doi.org/10.12989/cac.2016.17.5.683

A study about determination of preliminary design & minimum reinforcement ratios

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(Received April 4, 2015, Revised January 25, 2016, Accepted February 29, 2016)

Abstract. In the standards, minimum reinforcement ratios are presented as the least reinforcement ratios that bearing elements should have in a way to include all systems and in general. However, naturally these general minimum ratios might be presented as being lower than the normally required reinforcement ratios by criteria such as system size, bearing system arrangement, section situation and distributions of the elements and earthquake effect. In this case, minimum reinforcement ratios may remain as meaningless restrictions. Then grouping the criterion that might affect reinforcement ratios according to certain parameters and creating minimum reinforcement ratios regarding preliminary design will provide ease and safety during the project designing. Moreover, it will enable fast and simple examinations in the beginning of project control and evaluation process. By means of the data which could be defined as "preliminary design & minimum reinforcement ratios", a more realistic and safe restriction compared to general minimum reinforcement ratios could be presented. As a result of numerous comprehensive studies, reinforcement ratios to include all certain systems might be obtained. Today, thanks to the development level of finite elements programs which can make reinforced concrete modelling, with the studies that are impossible to carry out beforehand, this deficiency in the minimum reinforcement ratios in the standarts may at least be partially made up with the advisory regulation of preliminary design & minimum reinforcement ratios. As the structure of the system to be examined and the diversity of the parameters range from the specific to the general, preliminary design & minimum reinforcement ratios will approximate to general minimum reinforcement ratios in real terms. By focusing on a more specific system structure and diversity of the parameters, preliminary design and even design reinforcement ratios will be approximated. In this preliminary study, a route between these two extremes was attempted to be followed. Today, it is possible to determine suggested practical ratios for project designs through carrying out numerous studies.

Keywords: finite element analysis, minimum reinforcement ratios, reinforced concrete, preliminary design

1. Introduction

A simple system was picked up as the first step and nonlinear reinforced concrete system planar analysis was performed under static equivalent seismic load via ANSYS software by 13

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http://www.techno-press.org/?journal=cac&subpage=8 ISSN: 1598-8198 (Print), 1598-818X (Online)

different reinforcement cases. Suggested reinforcement ratios were tried to be revealed for the discussed system and similar systems. The examined system was organized in a way to include the most critical and extreme values for most of the parameters to be considered; however, it was tried for the reinforcement ratios to apply not only for this system but for a general system network that have similar characteristics to this system. With reference to this study, results of further studies that will be carried out on similar and different systems may present general and inclusive preliminary design & minimum reinforcement ratios. Thereby, more extensive reinforcement ratios which may differ by system, element and seismicity instead of only minimum reinforcement ratios in the standarts or literature may be attained.

In the first application of finite element method to the reinforced concrete, reinforced concrete beams were examined by Ngo and Scordelis (1967), and concrete and reinforcement were modelled as linear elastic with 2D triangle elements. The adherence between the concrete and reinforcement was modelled and some assumptions were made regarding the formation of cracks. In latter studies, experimental results of shear-walled samples and finite element analysis were compared and positive results were obtained (Vallenas *et al.* 1979; Kotsovas *et al.* 1992). Then, in the study of Ashour and Morley (1993), crack formation in the concrete was defined based on fracture mechanics and experimental data and used in finite element method. The interaction between the concrete and reinforcement were modelled by defining new connection elements. The results of experiment beams were compared with the method and correctness of the model was verified. In the study carried out by Chan *et al.* (1994), the results of a shear-walled experiment sample was compared with the results of finite element analysis obtained by using non-linear modelling. As a result, it was reported that the mentioned method could be used to predict the bearing capacity of reinforced concrete structures safely. In the study done by Hamil *et al.* (2004), a non-linear finite element model was developed for concrete column-beam joint locations. The correctness of analysis results were presented by comparing with 16 experiment results and the success of the model was demonstrated. Today, the use of finite element models thanks to the developing computer technology enables the detailed and correct modelling of shear-walled reinforced concrete models. Micro modelling in which the continuum is particularly modelled with 3D solid or shell finite elements constantly produces realistic results by developing elements that are also able to identify concrete and reinforced concrete. Non-linear behaviour of concrete and steel is also taken into account via material identifications and analysis results approximate to experiment results can be obtained (Kwak and Kim 2004; Palermo and Vecchio 2007; Nicolas *et al.* 2008). Finite element analysis software such as ANSYS enables to identify the necessary element and material for such modelling. However, related studies are generally small-scaled and based on element. One reason for this is the fact that both modelling and especially non-linear analysis duration is rather long. Another reason is testing the attainability of the results approximate to the results of samples tested in laboratory. Thus, there are limited number of studies regarding such an analysis of a whole structure in the literature (Greeshma *et al.* 2011, Musmar 2013, Hidalgo *et al.* 2002).

Some finite element modelling studies regarding the examination of the effect of reinforcement ratio in reinforced concrete elements are as follows: In Beassason and Sigfusson’s study (2001), finite element models were developed to investigate the seismic behaviour and capacity of reinforced shear walls. Experimental data was used in these finite element models. In order to obtain load-deformation curves of shear walls having different reinforcement forms but the same geometry, ANSYS software was preferred. Finite element analysis results and results obtained from the experiments were reported to be rather compatible with each other. Erduran and Yakut’s study (2003) is about the development of damage curves for reinforced concrete column elements. Parameters pertaining to effects that create damage in reinforced concrete columns were investigated broadly using ANSYS software. Numeric load-displacement curves were obtained from the analyses under static lateral thrust loads. In order to determine the effect of concrete strength, column slenderness, magnitude of axial load, the amount of longitudinal and lateral reinforcement (stirrup) and longitudinal yield strength on the deformation capacity of the column, non-linear analyses were conducted. It was found that the most significant parameters affecting the capacity of column deformation were amount of lateral reinforcement, column slenderness and yield strength of longitudinal reinforcement. It was also observed that the amount of longitudinal reinforcement had a big impact on load-bearing capacity, yet it did not have any impact on the deflection feature of the column. In the study done by Kazaz et.al (2012) , a shear-walled model under equivalent static lateral thrust load was reinforced in accordance with ASCE/SEI 41, Eurocode 8 and Specification for Buildings to be Constructed in Earthquake Prone Areas (DBYBHY-2007). The comparison of the standarts was made through the system of which non-linear reinforced concrete analyses were conducted with ANSYS software. As related with the topic discussed in this article, in Kwak and Kim's study (2008), an algorithm regarding preliminary design and which can apply for sample structures in a typical reinforced concrete frame system developed. In the algorithm, pre-determined reinforced concrete column and beam section databases were configured and used for the optimum design on reinforced concrete frame systems. The sections in the database were configured in such a way that could satisfy the requirements of the specifications and practical restrictions. However, it was reported that sensitivity analysis and/or additional approaches based on the genetic algorithm should be adopted in order to obtain general, optimum results without considering the load and structural system. In Ersoy's study (2013), dimensioning and reinforcing suggestions regarding two to eight-storey structures to be constructed in first-degree and second-degree seismic zones were made. In the study done by Lee *et al.* (2014), web-shear capacity of prestressed hollow-core slab unit with consideration on the minimum shear reinforcement requirement was investigated. Also, in the study done by Park *et al.* (2015), minimum shear reinforcement ratio of prestressed concrete members for safe design was investigated. However, in the literature, a study suggesting reinforcement ratio regarding preliminary design by investigating the reinforcement ratio effects through finite element models was not found.

The equivalent static seismic load specified for the system was given in 2000 steps by a time-dependent rise on ANSYS software and non-linear analysis was conducted. Likewise, thanks to linear calculation methods it is possible to obtain results that cannot provide the adequate approach in a system particularly under seismic loads (Kazaz *et al.* 2006).

Apart from the studies mentioned above regarding 2D reinforced concrete system analyses done under equivalent static seismic loading, the following studies can also be exemplified: In the study carried out by Torres *et al*. (2003), non-linear behaviours of 2D reinforced concrete and prestressed concrete frames under static loads were modelled. A multi-layered model algorithm applied in general matrix formulation was created, programmed and sample analyses were done. After the model was verified by comparing with ad-hoc experiments and benchmark tests, a 2D multi-storey reinforced concrete frame sample was analysed under different sections and reinforcement ratios. Athanassiadou (2008) examined multi-storey reinforced concrete frame structures having irregularity in elevation. In this study, analyses of 6 different 2D-frames each having 10 storeys and 3 span were done via Drain-2000 and SAP2000 softwares under inelastic static pushover and dynamic time-history loadings.

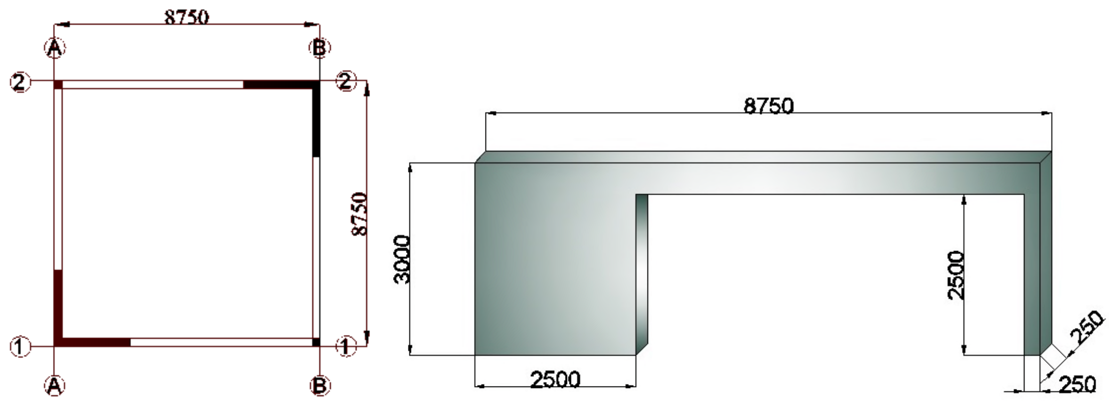


Fig. 1 Formwork plan and geometric dimensions of axis 1 for single storey (all dimensions are in mm)

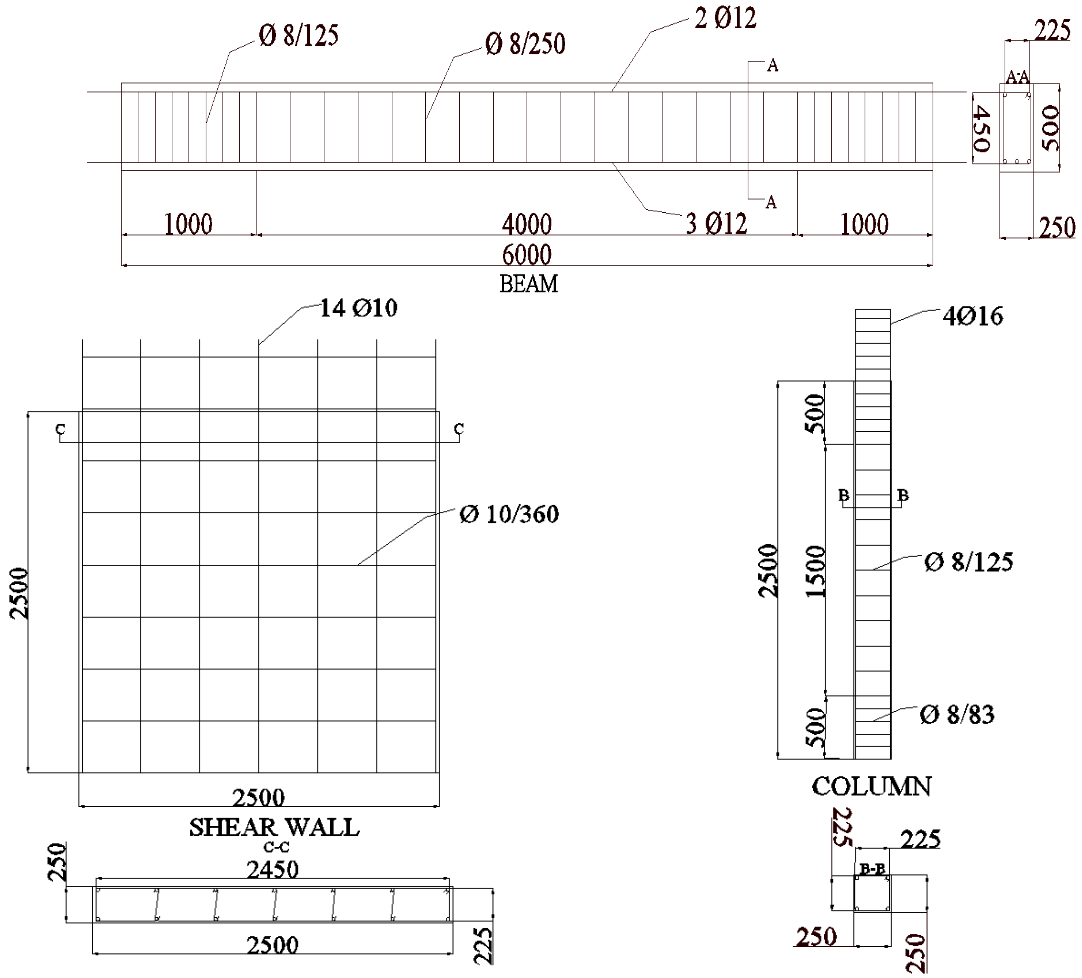


Fig. 2 Reinforcement details of beam, column and shear wall belonging to Case 1 (all dimensions are in mm)

Table 1 Reinforcement ratios belonging to each case

	Shear Wall		Column		Beam		
Case Number	Horizontal Reinforce-ment	Vertical Reinforce-ment	Longitudinal Reinforce-ment	Transversal Reinforce-ment	Tension Reinforce-ment	Compression Reinforce-ment	Transversal Reinforce-ment
1	min $\rho_{w,x,y}$	min $\rho_{w,z}$	min ρ_l	minS_s	min ρ_t	min ρ_c	minS_b
2	1.5 min $\rho_{w,x,y}$	1.5 min $\rho_{w,z}$	min ρ_l	minS_s	min ρ_t	min ρ_c	minS_b
3	min $\rho_{w,x,y}$	min $\rho_{w,z}$	1.5 min ρ_l	1.5 minS_s	min ρ_t	min ρ_c	minS_b
4	min $\rho_{w,x,y}$	min $\rho_{w,z}$	min ρ_l	minS_s	1.5 min ρ_t	1.5 min ρ_c	1.5 minS_b
5	1.5 min $\rho_{w,x,y}$	1.5 min $\rho_{w,z}$	1.5 min ρ_l	1.5 minS_s	min ρ_t	min ρ_c	minS_b
6	1.5 min $\rho_{w,x,y}$	1.5 min $\rho_{w,z}$	1.5 min ρ_l	1.5 minS_s	1.5 min ρ_t	1.5 min ρ_c	1.5 minS_b
7	1.5 min $\rho_{w,x,y}$	1.5 min $\rho_{w,z}$	1.5 min ρ_l	1.5 minS_s	1.5 min ρ_t	2.22 min ρ_c	1.5 minS_b
8	2 min $\rho_{w,x,y}$	2 min $\rho_{w,z}$	1.5 min ρ_l	1.5 minS_s	2 min ρ_t	2.96 min ρ_c	2 minS_b
9	2 min $\rho_{w,x,y}$	2 min $\rho_{w,z}$	2 min ρ_l	2 minS_s	2 min ρ_t	2.96 min ρ_c	2 minS_b
10	min $\rho_{w,x,y}$	min $\rho_{w,z}$	min ρ_l	minS_s	2 min ρ_t	2.96 min ρ_c	2 minS_b
11	1.5 min $\rho_{w,x,y}$	1.5 min $\rho_{w,z}$	1.5 min ρ_l	1.5 minS_s	2 min ρ_t	2.96 min ρ_c	2 minS_b
12	2 min $\rho_{w,x,y}$	2 min $\rho_{w,z}$	1.5 min ρ_l	1.5 minS_s	2 min ρ_t	2.96 min ρ_c	3 minS_b
13	1.5 min $\rho_{w,x,y}$	1.5 min $\rho_{w,z}$	3 min ρ_l	3 minS_s	3 min ρ_t	4.44 min ρ_c	3 minS_b

2. The examined system and its sub-cases

In this study an eight-storey, single span model structure of which bearing system consists of reinforced concrete slab, shear wall, column and beams was considered. The model consists of four similar axes in itself with regard to size and positioning of bearing system (Fig. 1). The only axis consisting of the shear wall, beam and column that symbolise the four axes of the model was divided into 13 sub-cases by changing the reinforcement ratios. After possible longitudinal and lateral loads that could affect the structure were determined on the model, non-linear analyses of the cases were done with the help of ANSYS. The data of relevant Turkish national standarts and specifications were used for minimum reinforcement ratios.The reinforcement ratios mentioned in provisions of TS500(2000) and DBYBHY-2007 were used for Case 1. These ratios were taken as min$\rho_{w,x,y}$ = 0.0025 for directions x an y and min$\rho_{w,z}$ = 0.001 for direction z for shear wall reinforcements. minρ_c = 0.00181 was taken for beam top reinforcement and minρ_t = 0.002712 for beam bottom reinforcement. Beam stirrups were taken as minS_b = ø8/25 in the middle and as ø8/12.5 in the 1/6 distances of lenght of the beam in the two ends. Column longitudinal reinforcements were taken as minρ_l = 0.0133. Column stirrups were taken as minS_s=ø8/12.5 in the middle and as ø8/8.3 in the 1/5 distances of the clear height of column in the two ends and continued in the places where it intersected with the beam. In Fig. 2, reinforcement drawings for Case 1 which was created by taking general minimum reinforcement ratios are shown. In the cases after Case 1, some of these ratios were increased. The amount of increase by minimum reinforcement ratios accepted for each case in each bearing element is shown in Table 1.

3. Advisory system parameters to determine preliminary design & minimum reinforcement ratios and parameter values regarding the examined system

The criterion that could affect the reinforcement ratios in structure bearing elements were divided into simple and general parameters and the values taken by these parameters for the examined system were presented. In further studies necessary thresholds for preliminary design & minimum reinforcement ratios might be determined by changing these parameters for many different cases. The proposed parameters are grouped as general structure parameters and bearing system element parameters.

3.1 General structure parameters

1) Parameter S_1 which can be defined as structure plan area (A_b) may be taken as 80 m^2 approximately for the examined system (Figs. 1 and 3).

2) Parameter S_2 which can be found as the ratio of total section area of vertical bearing elements in the plan ($\sum A_{v,i}$) to the structure plan area (A_b) can be calculated as shown in Eq. (1) and taken as 0.03 approximately for the examined system (Figs. 1 and 3).

$$S_2 = \frac{\sum A_{v,i}}{A_b} \quad (1)$$

3) Parameter S_3 which can be found as the ratio of total area of lateral bearing elements in the plan ($\sum A_{h,i}$) to the structure plan area (A_b) can be calculated as shown in Eq. (2) and taken as 0.08 approximately for the examined system (Figs. 1 and 3).

$$S_3 = \frac{\sum A_{h,i}}{A_b} \quad (2)$$

4) Parameter S_4 which can be found as the ratio of structure plan area (A_b) to total height of structure (H) can be calculated as shown in Eq. (3) and taken as 3.2 m approximately for the examined system (Figs. 1 and 3). In further studies different S_4 parameters of the same framework system could be created and examined and changes in preliminary design & reinforcement ratios could be acquired in accordance with the change of the parameter S_4.

$$S_4 = \frac{A_b}{H} \quad (3)$$

5) Total equivalent seismic load affecting the structure (V_t) can be suggested as parameter S_5 and taken as approximately 600 kN for examined system as calculated in Part 4.3. This parameter will take different values by variables such as intended purpose of the system, earthquake zone, site class and weight of structure. For this reason, it is not necessary to take this and similar variables as separate parameters. The change created by equivalent seismic load change in preliminary design & minimum reinforcement ratios might be revealed through several studies to be carried on this system and different systems.

6) Concrete class was defined as Parameter S_6 and reinforcement class was defined as Parameter S_7 and were taken as S_6= C22, S_7 = S420 respectively for the examined system. Here, “22” is characteristic compressive strength of concrete and “420” is characteristic tensile strength of reinforcement.

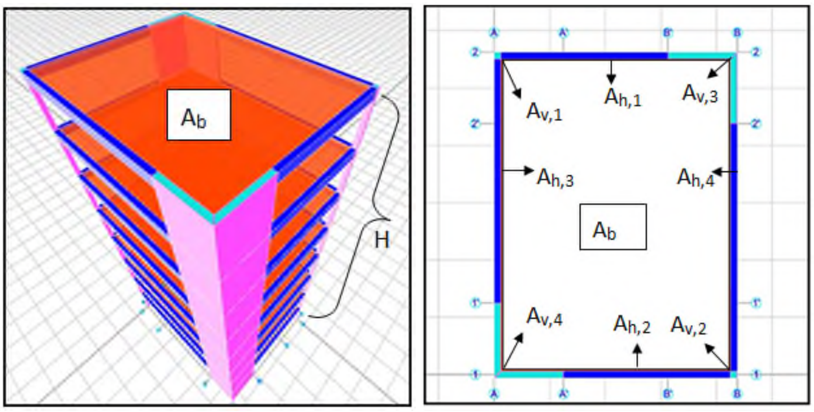


Fig. 3 Demonstration of A_b, H, $A_{v,i}$ and $A_{h,i}$ for examined system

The examined system was a symmetric system by axes x and y in the plan and does not have irregularity in elevation. Since it is expected that as the symmetry of the plan deteriorates and the irregularity in elevation rises, reinforcement ratios increase, reinforcement ratios in the symmetric systems will present the least reinforcement ratios. Therefore, along with being a broad threshold, preliminary design & minimum reinforcement ratios to be calculated for symmetric systems with regularity in elevation will satisfy asymmetric systems, too. For this reason, asymmetry parameter was ignored.

3.2 Bearing system element parameters

1) Parameter C_1 which can be found as the ratio of inertia moment ($I_{xx,s}$) of system beams by axis x-x to the length of the beam (l_b) can be calculated as shown in Eq. (4) and taken as 435 cm^3 approximately for the examined system (Fig. 4).

$$C_1 = \frac{I_{xx,b}}{l_b} \tag{4}$$

2) Parameter C_2 which can be found as the ratio of inertia moment of system columns ($I_{xx,s}$) by axis x-x to the height of the column (l_s) can be calculated as shown in Eq. (5) and taken as 110 cm^3 approximately for the examined system (Fig. 4). In further studies, preliminary design & minimum reinforcement ratios can be organised based on changes of the column section measures.

$$C_2 = \frac{I_{xx,s}}{l_s} \tag{5}$$

3) Parameter C_3 which can be found as the ratio of inertia moment of system shear walls ($I_{xx,w}$) by axis x-x to the height of the shear wall (l_w) can be calculated as shown in Eq. (6) and taken as 108500cm^3 approximately for the examined system (Fig. 4).

$$C_3 = \frac{I_{xx,w}}{l_w} \tag{6}$$

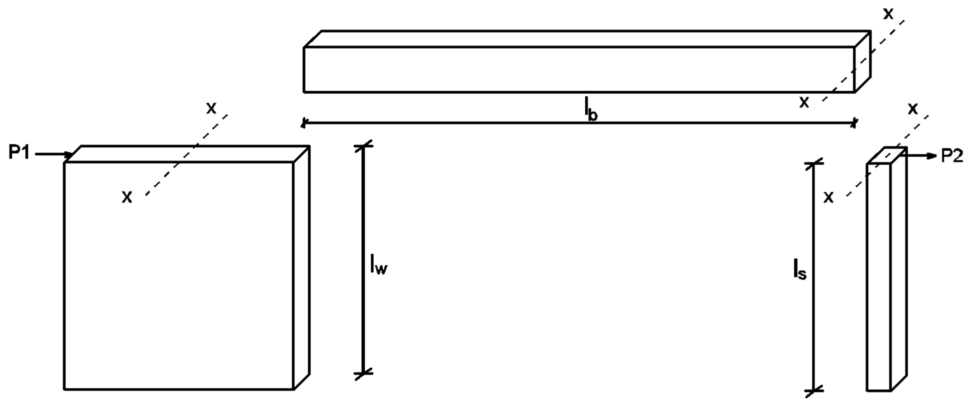


Fig. 4 Demonstration of elements' dimensions for examined system

In conclusion, systems features that can affect reinforcement ratios are represented substantially with these 10 parameters. Tabular preliminary design & minimum reinforcement ratios by different values that these parameters can take can be created and practical controls can be enabled

4. Modelling phases of the system with ANSYS software

4.1 Material properties and models

Element Solid 65 was chosen from the element library within ANSYS in order to define reinforced concrete finite elements. Definitions were made equal to C22 approximately as the concrete class and St420 as the reinforcement class. Reinforced concrete density was taken as $2.5x10^{-5}$ N/mm^3; this density was multiplied by coefficient 1.4 for the self-weight of the model to be formed and registered in the program as $3.5x10^{-5}$ N/mm^3 (TS500 2000). Reinforcement yield strength was taken as 365 MPa, elasticity module was taken as $2.1x10^5$ MPa; concrete elasticity module was taken as $2.1x10^4$ MPa, and tensile strength was taken as 1.8 MPa. Stress-strain curve of concrete was created in the study and Hognestad Model (Hognestad 1951) which was developed for unconfined concretes was cosidered and concrete was defined in Multilinear Isotropic Hardening Plasticity model (Von Mises 1913). In order to define failure surface of the concrete under triaxial stress, ANSYS uses the model developed by Willam and Warnke (1975). Reinforcement stress-strain curve of reinforcement steel was defined as linear elastic-full plastic according to studies in literature (Bilinear isotropic hardening plasticity) which bases on Von Mises yield criterion, and strength increase of the reinforcement after yield was ignored and the curve was idealised. Smeared method was used to create the reinforcements; in other words, reinforcements were defined as volumetric ratio within the Solid 65 finite element. Thus, any different element for reinforcement from ANSYS element library was not chosen. In the literature, there are a lot of studies in which smeared method has been used because it enhances the speed of modelling and analysis and provides adequate convergency.

Reinforcement ratios were defined from the real constant tab within Solid 65. A total of 24 real constants were defined for different ratios and /or directions in different locations belonging to each cases.

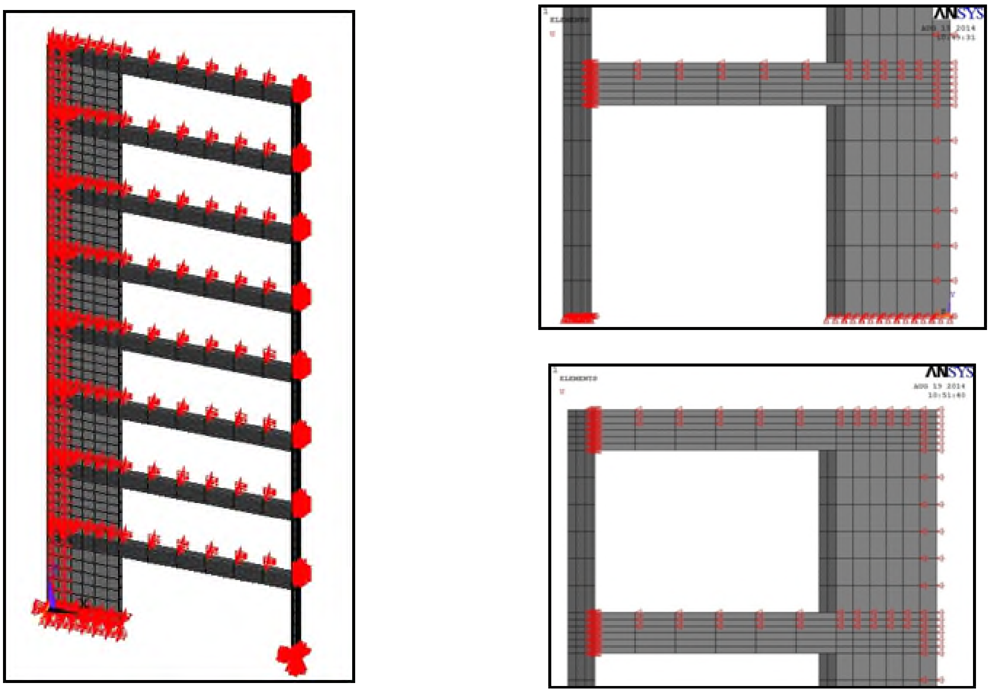


Fig. 5 Finite element networks and boundary conditions of system model

4.2 Creation of ansys finite element model geometry

After material properties and real constants were entered into the program, the process of dividing system into finite elements was carried out. As it can be seen in Fig. 5, fix supports applied to the base of the models generated as 8-storey. As intented for plane analysis, the conjuctions regions between the system components and shear walls, the beams and the slabs comes from rear side direciton were supported to be prevented displacement in z axis . Real constants were appointed to the finite elements in the related places to create reinforcements. The distributions of system real constants could be seen in Fig. 6.

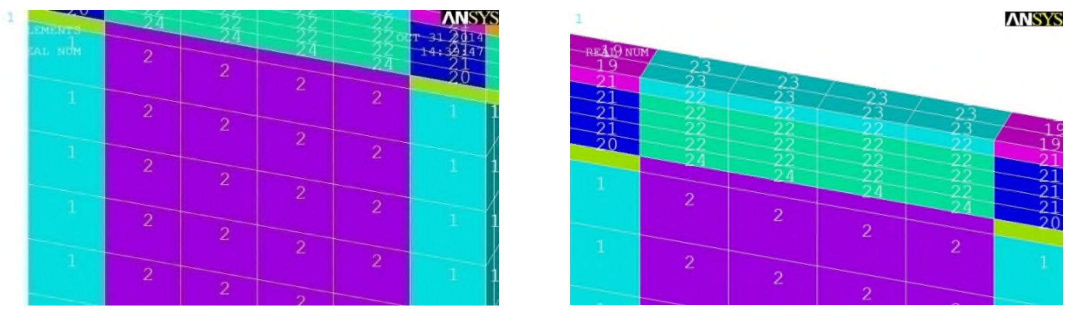


Fig. 6 The distributions of real constants

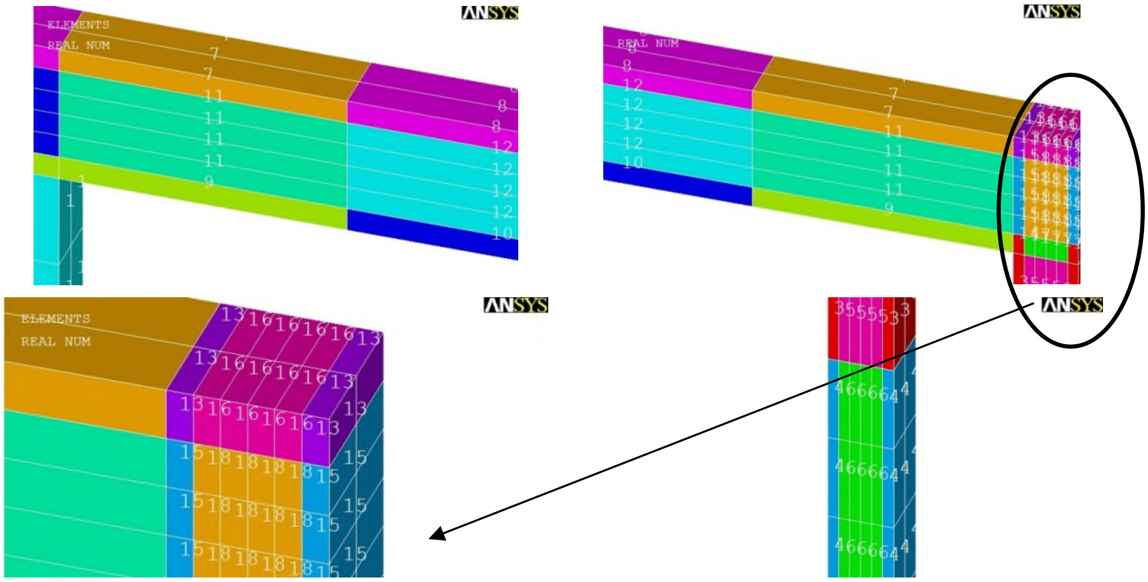


Fig. 6 Continued

4.3 Finding horizontal and vertical loads affecting to the model

Slabs thickness in the models were selected as 12 cm and system vertical loads at the top storey level, on beams where the walls sit on, for other storeys and other regions was calculated as 25.86 kN/m, 36.45 kN/m and 28.05 kN/m (TS500 2000;TS498), respectively, by considering the criterias such as live load, coating-plaster weights, wall weights and load transfer from slabs to beams. In order to determine horizontal forces, equivalent seismic load method mentioned in Specifications for Buildings to be Built in Seismic Zones (DBYBHY 2007) was used. In accordance with the direction of earthquake to be considered, total equivalent seismic load V_t (base shear force of the structure) subjecting to the entire structure was calculated with Eq. (7).

$$V_t = W \cdot A(T_1) / R_a \; ; \quad W = \sum_{i=1}^{N} W_i \tag{7}$$

Here, W, W_i, $A(T_1)$ and R_a denotes to the total weight of the building, weight of i'th storey, spectral acceleration coefficient corresponding with first natural vibration period T_1 and the seismic load reduction factor, respectively. Intended purpose of the structure is to be taken into account while finding the W_i and R_a values. While finding the R_a values, bearing system behaviour is also to be considered. The natural vibration period, effective ground acceleration coefficient, seismic zone, structure's level of importance, spectrum coefficient, site class values were taken into consideration through a set of formulations while calculating the $A(T_1)$ variable. Therefore, V_t value was used as an important parameter that included many criteria to be also considered in this study. In further studies, the preliminary design & minimum reinforcement ratios to be found by simply changing these parameters will have also symbolised many criteria included by the parameters. The first natural vibration period T_1, one of the variables to be found to obtain these parameters, was calculated with SAP2000. The period in the direction *x* for the examined structure

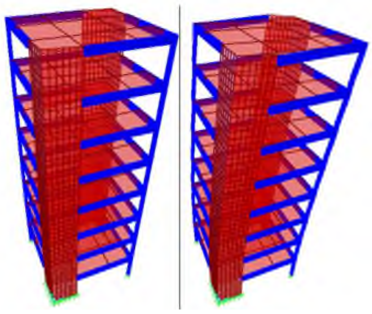


Fig. 7 SAP2000 model and modal deformation

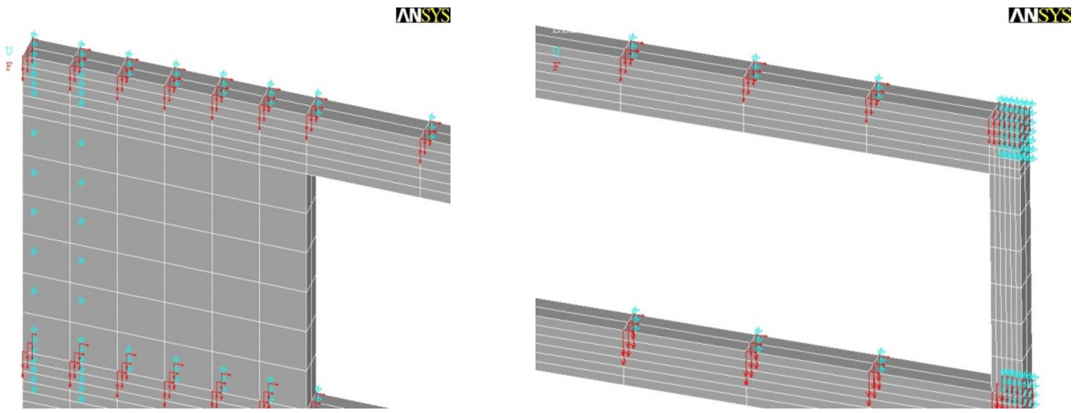


Fig. 8 Demonstration for distribution of horizontal and vertical loads

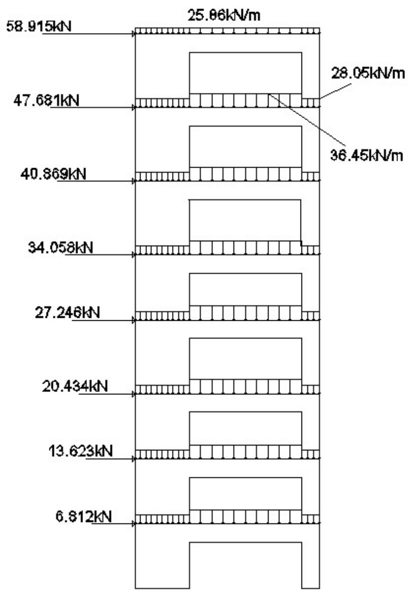


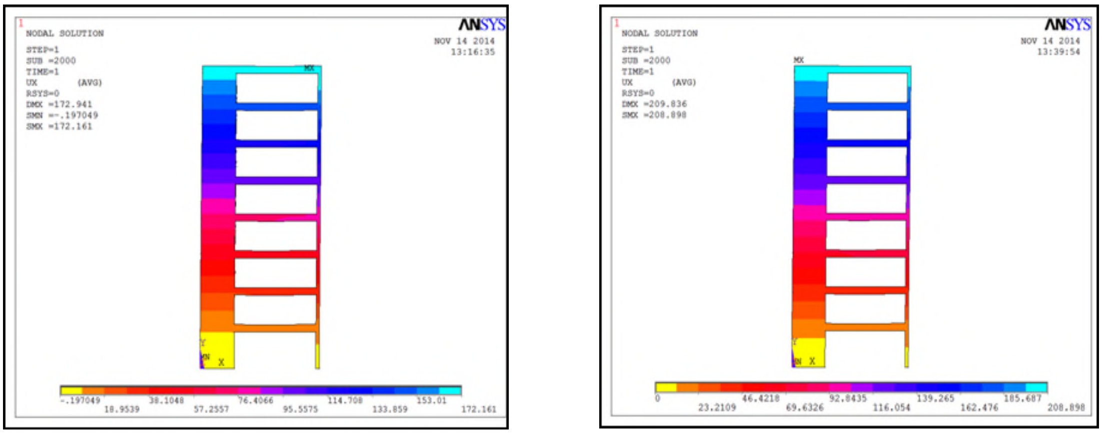
Fig. 9 Horizontal and vertical loads

was obtained as T_1=1.28 seconds. SAP2000 system modelling and exaggerated modal deflection was shown in Fig. 7.

Equivalent seismic load which was calculated by Eq. (7) was distributed on the structure storey levels as horizontal static thrusts through a set of formulations. Vertical and horizontal loads acting on the system were defined in the model (Fig. 8). The values of these loads are given in Fig. 9 collectively.

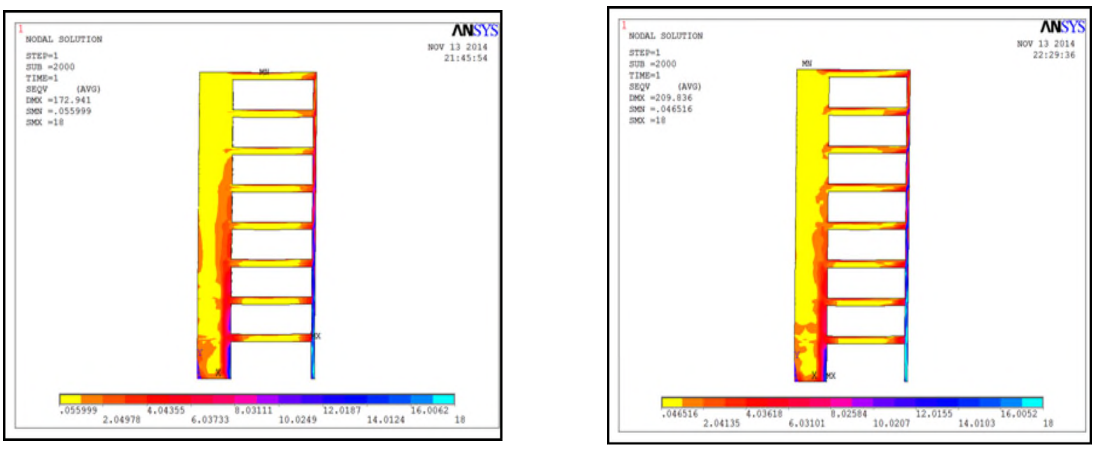
5. Analysis and results in ANSYS

Non-linear analyses were done by increasing horizontal loads without changing the vertical loads on the system. For each 13 cases, in order to find the horizontal load that the system could carry, horizontal loads were raised in the modelling on ANSYS and the cases were solved within themselves many times. Since ANSYS could not advance the analysis for horizontal loads that the



(a) Case 1 (b) Case 6

Fig. 10 Displacements in x directions



(a) Case 1 (b) Case 6

Fig. 11 Von Mises stress distributions

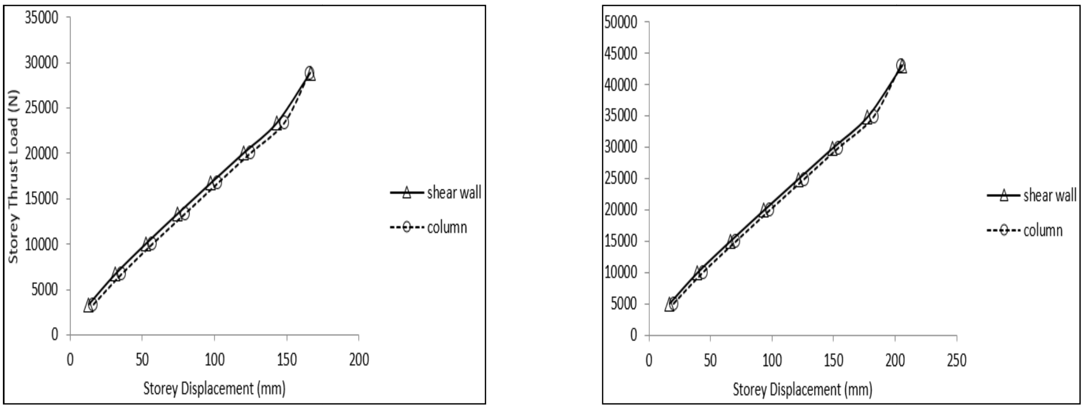
system could not bear, the last and biggest horizontal load distribution that solution could be done was determined as the horizontal load that the system could carry, and to do this, horizontal loads were changed within themselves and many analyses were done. Horizontal loads that each case could carry were discussed in Part 6. For more loads, the models were warned to be over deflected and could not be solved. Geometric non-linearity was taken into account during the analyses and the loads were loaded in 2000 steps in all analyses.

Some of the analyses results are as follows:

For example, Von Mises stress and horizontal displacement distributions for Case 1 and Case 6 were shown in Figs. 10 and 11.

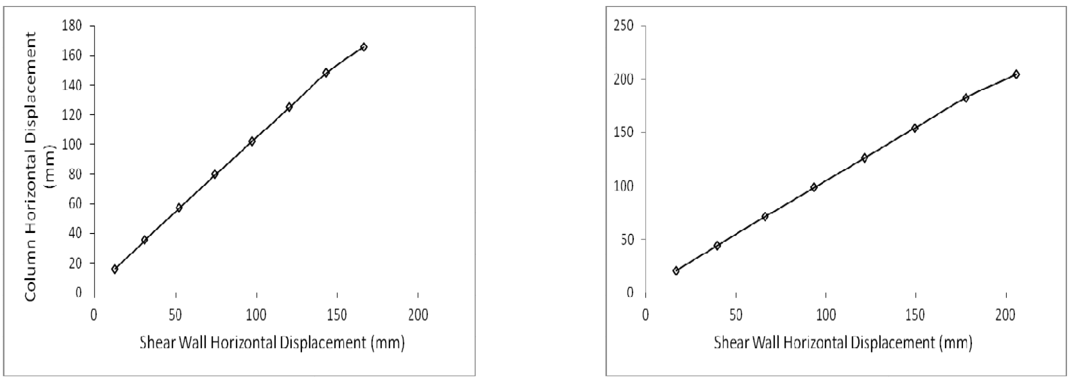
Graphical representations of storey displacements for Case 1 and Case 6 are demonstrated in Fig. 12 as an example. In Fig. 13, graphics of column-shear wall horizontal displacements for Case 1 and Case 6 were shown as an example.

Graphics of beam vertical load-maximum deflections for Case 1 and Case 6 were also presented in Fig. 14 as an example. In Fig. 15, maximum beam deflection graphics of the storeys for Cases 1 and 6 were illustrated as an example.



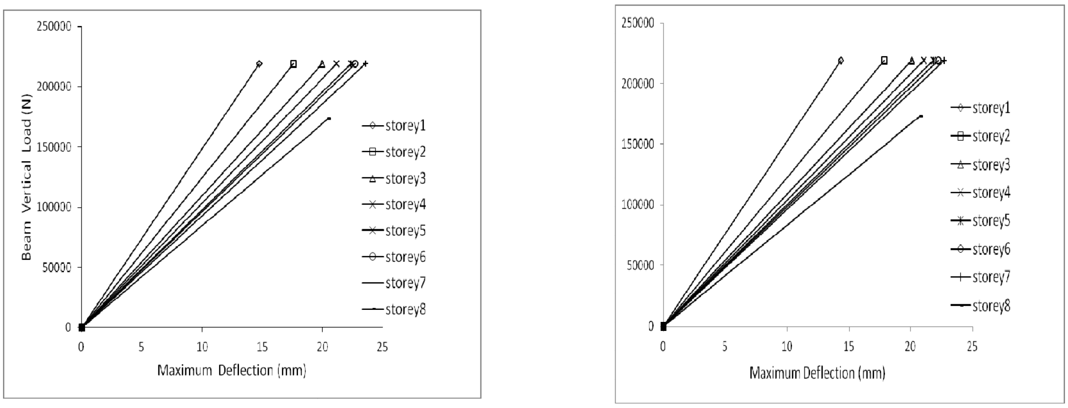
(a) Case 1 (b) Case 6

Fig. 12 Thrust load-displacement graphics



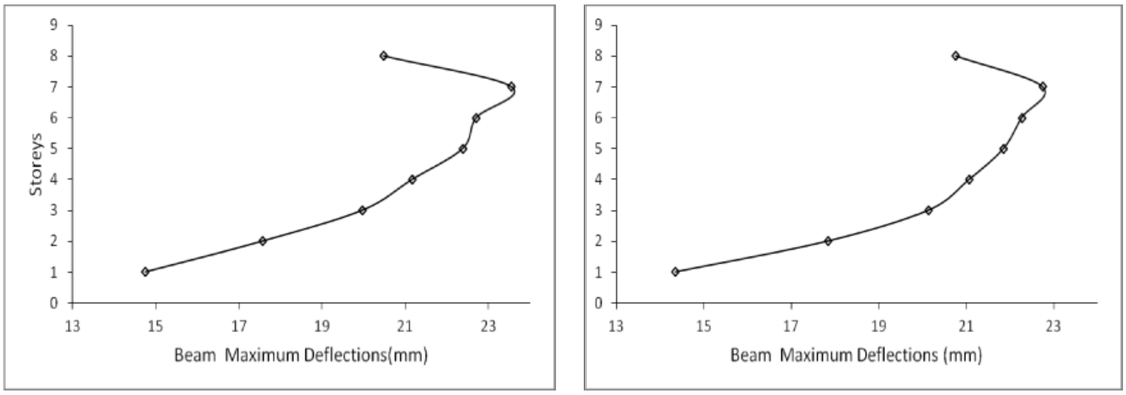
(a) Case 1 (b) Case 6

Fig. 13 Column-shear wall horizontal displacements for each storey



(a) Case 1 (b) Case 6

Fig. 14 Beam vertical load-maximum deflection graphics for each storey



(a) Case 1 (b) Case 6

Fig. 15 Maximum beam deflection graphics belonging to storeys

6. Evaluation of system analysis results

The biggest bearable horizontal thrust load belonging to each cases was determined as system horizontal load bearing capacity and presented in Table 2. What percentage of approximately 600 kN system total seismic load presented in Part 4.3 fulfilled for each case was given in Table 2. Column-shear wall ductilities obtained by taking the avarage of the areas under the storey thrust load-strorey displacement displacement curves of each element were presented in Table 2. In beam vertical load-maximum deflection graphics, the areas below the curves were also calculated,summed up and total ductility of system beams regarding the related case was obtained and presented in Table 2 for each case. For each case, system total ductility was obtained by summing up the beams' total ductilities and column-shear wall total ductilities and was also presented in Table 2.

By examining Table 1 and Table 2, the following comments could be made:

1) Reinforcement increase need of system beams is higher than system shear walls' and; therefore, beam reinforcement are more effective than shear wall reinforcement in terms of system

Table 2 Maximum horizontal load percentages that the systems could carry and ductility informations for each case

Case No	Horizontal Load %	Horizontal Load (kN)	Column-Shear Wall Ductility (kNcm)	Beam Ductility (kNcm)	System Ductility (kNcm)
1	41,00	122,3	239,8	1712,6	1952,4
2	47,00	139,8	334,6	2091,7	2426,3
3	40,00	122,3	236,5	1689,9	1926,3
4	53,00	159,8	383,9	1718,9	2102,8
5	47,00	139,8	295,2	1835,1	2130,3
6	61,00	182,2	441,5	1695,0	2136,5
7	65,00	196,1	407,0	1470,9	1877,9
8	82,00	247,1	583,7	1523,7	2107,3
9	87,00	262,1	678,0	1559,7	2237,7
10	64,00	192,2	478,0	1737,8	2215,8
11	77,00	232,2	568,2	1533,1	2101,3
12	84,00	252,1	615,2	1564,9	2180,1
13	103,00	309,6	805,1	1339,3	2144,4

behaviour. Beam reinforcements are of great importance in this system. In the event that beam compression reinforcement also increases, better improvements are attained in the system. Increasing beam compression reinforcements and stirrups alone also contribute to system behaviour significantly.

2) By increasing shear wall, beam and column reinforcement together, great improvements were attained in the system.

3) Column minimum reinforcement ratios are very inadequate for this and similar systems. Preliminary design & minimum column reinforcement ratios should at least simply change by the number of storeys in the system. Moreover, because the stiffness distribution would affect the share coming from the horizontal load, the advisory column reinforcement ratio should also be related to ratio of column stiffness to the total stiffness of vertical bearing elements of the system.

4) The minimum reinforcement ratios were also inadequate for long beams.Increasing beam reinforcements by 50% with regard to the minimum ratio did not suffice to provide the expected improvement in the system. Similarly, increasing beam shear - compression reinforcements by 50% with regard to the minimum ratio was not sufficient in this system. In case of increasing beam reinforcements by 100 % regard to the minimum ratio and taking beam compression reinforcements as being equal to tensile reinforcements, expected improvements were obtained in the system. In this case, for preliminary design minimum conditions, beam lateral-longitudinal reinforcement ratios should simply be presented at least as changing by beam section and lengths.

5) Minimum shear wall reinforcement ratios were seen to be below the level to satisfy this and similar systems. It can be said that preliminary design minimum shear wall reinforcement ratios should at least be simply presented as dependent on both number of system storey and shear wall latitude and thickness. Furthermore, preliminary design minimum shear wall reinforcement ratios should depend on the ratio of shear wall stiffness to the ratio of the system’s vertical bearing elements’ total rigidity because the distribution affects the share coming from the horizontal load.

7. Preliminary design & minimum reinforcement ratios for suggested for the analysed and similar systems

The examined system was designed so as to take the most critical and extreme values for most of the parameters mentioned above. Thus, for the suggested reinforcement ratios to include a broader system discrepancy the following method was used, so these ratios will be acceptable as preliminary design & minimum reinforcement ratios not for only this system but for a general system network with similar features. It will be useful for further studies to clarify the limit of this similarity and to be inclusive of similar systems with different parameter values. But, of course, from specific to general ratios to be found as preliminary design & minimum reinforcement ratios will come closer to define general minimum reinforcement ratios. For more specific systems, preliminary design and even design reinforcement ratios will be approximated. In this introductory study a method between the two extremes was tried to be followed. Therefore, the reinforcement ratios in Case 8, Case 9, Case 12 and Case 13 in which the systems satisfied the horizontal seismic loads were taken into account (Table 2), and the smallest of these ratios for each element was suggested as preliminary design & minimum reinforcement ratios.

Shear wall reinforcements increased by 100 % in Case 8, 100 % in Case 9, 100 % in Case 12 and 50 % in Case13 by general minimum reinforcement ratios. Since increasing shear wall reinforcements as 50 % by general minimum ratio is a solution in which the system bears the loads, it will be sufficient to take preliminary design & minimum reinforcement ratio for shear wall reinforcement as 50 % more of the general minimum reinforcement ratio for this and similar systems. It can be said that solutions to be obtained by dropping below these ratios would require increasing reinforcement in elements except for the shear wall or would cause over system deformation. In this case, advisory shear wall preliminary design & minimum reinforcement ratios for this and similar systems may be given as $\rho'_{w,x,y}$ = 1.5x0.0025 = 0.00375 for x and y directions, and $\rho'_{w,z}$=1.5x0.001=0.0015 for z direction.

Column reinforcements increased by 50 % in Case 8, 100 % in Case 9, 50 % in Case 12 and 200 % in Case 13 by general minimum reinforcement ratios. Since increasing column reinforcements as 50 % by general minimum ratio is a solution in which the system bears the loads, it will be sufficient to take preliminary design & minimum reinforcement ratio for column reinforcement as 50 % more of the general minimum reinforcement ratio for this and similar systems. It can be said that solutions to be obtained by dropping below these ratios would require increasing reinforcement in elements except for the column or would cause over system deformation. In this case, advisory column preliminary design & minimum reinforcement ratios for this and similar systems may be given as ρ'_l = 1.5x0.0133 = 0.01995 for column longitudinal reinforcements. Column stirrups (S'_s) can be given as ø8/8.3 in the middle and as ø8/5.5 in column-beam intersection location and 1/5 distances of column length on the two ends.

Beam bottom reinforcements and stirrups increased by 100 % in Case 8, 100 % in Case 9 and 200 % in Case 13 by general minimum reinforcement ratios. In Case 12, the rise was 100% for bottom reinforcements and 200 % for stirrups. (Table1). In all these cases, beam top reinforcements were taken as equal to increased bottom reinforcements. Since increasing beam bottom reinforcements and stirrups as 100% by general minimum ratio and taking top reinforcement as equal to increased bottom reinforcement is a solution in which the system bears the loads, it will be sufficient to take preliminary design & minimum reinforcement ratio in beam bottom reinforcement and stirrups as 100% more of the general minimum reinforcement ratio and as 196.36 % more in beam top reinforcements for this and similar systems. It can be said that solutions to be obtained by dropping below these ratios would require increasing reinforcement in elements except for the beam or would cause over system deformation. In this case, advisory beam preliminary design & minimum reinforcement ratios for this and similar systems may be given as $\rho'_c = 2.96x0.00181 = 0.0054$ for beam top reinforcements and $\rho'_t = 2x0.002712 = 0.0054$ for beam bottom reinforcements. Beam stirrups (S_b) can be given as ø8/12.5 in the middle and as ø8/6.3 in 1/6 distances of beam length on the two ends.

As a result, when Case 11 is examined (Table 1) it is seen that 77 % of horizontal seismic loads were carried (Table 2). It can also be said that this seismic load bearing capacity obtained for preliminary design & minimum reinforcement ratios suffice.

8. Conclusions

1) The criterion that might affect the reinforcement ratios in structure bearing elements as simple and general parameters are as follows: As general structure parameters, parameter S1defined as structure plan area, parameter S2 that could be found as the ratio of total section area of vertical bearing elements to the ratio of structure plan area, parameter S3 that could be found as the ratio of the total area of horizontal bearing elements of the structure in the plan to the ratio of structure plan area, parameter S4 that could be found as the ratio of the plan area of the structure to the total height of structure, parameter S5 as the total equivalent seismic load that acts on the structure, parameter S6 as the concrete class and parameter S7 as reinforcement class were suggested.

As bearing system element parameters, Parameter C_1 that could be found as the ratio of inertia moment of system beams with respect to x-x axis to the length of beam, parameter C_2 that could be found as the ratio of inertia moment of system columns with respect to x-x axis to the height of column, and parameter C_3 that could be found as the ratio of inertia moment of system shear walls with respect to x-x axis to the height of shear wall were suggested.

2) For the examined system**,** S1 was found to be 80 m^2, parameter S2 was found to be 0.03, parameter S3 was found to be 0.08, parameter S4 was found to be 3.2 m, parameter S5 was found to be 600 kN, parameter S6 was found to be C22, parameter S7 was found to be St420, parameter C1 was found to be 435 cm^3 , parameter C2 was found to be 110 cm^3 and parameter C3 was found to be108500 cm^3. The preliminary design & minimum reinforcement ratios below were suggested as being applicable for these and approximate parameters in the most general sense. In further studies, with the changes of these parameters the necessary threshold for the preliminary design & minimum reinforcement ratios can be determined for numerous different cases. Tabular preliminary design & minimum reinforcement ratios may be prepared in accordance with different values of the parameters to provide practical controls.

3) Suggested preliminary design & minimum reinforcement ratios of shear wall for this and similar systems may be presented as $\rho'_{w,x,y} = 0.00375$ in directions x and y, and $\rho'_{w,z} = 0.0015$ in direction z. Column preliminary design & minimum reinforcement ratios may be presented as $\rho'_l = 0.01995$ for longitudinal reinforcements. Column stirrups (S'_s) may be presented as ø 8/8.3 in the middle and as ø8/5.5 in 1/5 distances of the height of the column in two ends and on condition that they are maintained likewise in places where they intersect with the beam. Beam preliminary design & minimum reinforcement ratios may be presented as $\rho'_c = 0.0054$ for beam top reinforcements and $\rho'_t = 0.0054$ for beam bottom reinforcements. Beam stirrups (S_b) may be presented as ø8/12.5 in the middle and as ø8/6.3 in 1/6 distances of the length of the beam in two ends.

4) Preliminary design & minimum reinforcement ratios for this and similar systems presented in article 3 above are approximate values to minimum reinforcement ratios generalisation. For more specific cases**,** in order to accept values that may apply for this and similar systems and that are approximate to preliminary design, it would be useful to refer to Table 1 and Table 2.

References

Ashour, A.F. and Morley, C.T. (1993), "Three dimensional nonlineer finite element modelling of reinforced concrete structures", *Finite Elem. Anal. Des.*, **15**(1), 43-55.

Athanassiadou, C.J. (2008), "Seismic performance of R/C plane frames irregular in elevation", *Eng. Struct.*, **30**(5) , 1250-1261.

Beassason, B. and Sigfusson, T. (2001), "Capacity and earthquake response analysis of RC-shear walls", *Nord. Concrete Res.*, **27**, 1-14.

Chan, H.C., Cheung Y.K. and Huang, Y.P. (1994), "Nonlinear modelling of reinforced concrete structures", *Comput. Struct.*, **53**(5), 1099-1107.

CSI. SAP2000 ultimate v16 (2013), *Structural analysis program*, Berkeley, California, USA.

DBYBHY-07 (2007), "Specification for buildings to be constructed in earthquake prone areas", Ministry of Public Works and Resettlement, Ankara, Turkey.

Erduran, E. and Yakut, A. (2003), "Drift based damage functions for reinforced concrete columns", *Comput. Struct.*, **82**, 121-130.

Ersoy, U. (2013), "A simple approach for preliminary design of reinforced concrete structures to be built in seismic regions", *Technical J. Turkish Chamber of Civil Eng.*, **24**, 6559-6574.

Greeshma, S., Jaya K.P. and Annilet, S.L. (2011), "Analysis of flanged shear wall using ansys concrete model", *Int. J. Civil Struct. Eng.*, **2**(2), 454-465.

Hamil, S.J., Baglin, P.S. and Scott, R.H. (2004), "Finite element modeling of reinforce concrete beam-column connections", University of Durham.

Hidalgo, P.A., Jordan, R.M. and Martinez, M.P. (2002), "An analytical model to predict the inelastic seismic behavior of shear-wall, reinforced concrete structures", *Eng. Struct.*, **24**(1), 85-98.

Hognestad, E. (1951), "A study on combined bending and axial load in reinforced concrete members", University of Illionis Engineering Experiment Station, Urbana- Champaign, IL, 43- 46.

Kazaz, İ, Yakut, A. and Gülkan, P. (2006), "Numerical simulation of dynamic shear wall tests: a benchmark study", *Comput. Struct.*, **84**(8), 549-562.

Kazaz, İ., Gülkan, P. and Yakut, A. (2012), "Performance limits for structural walls: an analytical perspective", *Eng. Struct.*, **43**, 105-119.

Kotsovas, M.D., Pavlovic, M.N. and Lefas, I.D. (1992), "Two-and three-dimensional nonlinear finiteelement analysis of structural walls", Published in "Nonlinear Seismic Analysis and Design of Reinforced Concrete Buildings", edited by Fajfar P. and Krawinkler H., Elsevier Applied Science, pp. 215-227.

Kwak, H.G and Kim, J. (2008), "Optimum design of reinforced concrete plane frames based on predetermined section database", *Comput.Aid. Des.*, **40**(3), 396-408.

Kwak, H.G. and Kim, D.Y. (2004), "FE analysis of RC shear walls subject to monotonic loading", *Mag. Concrete Res.*, **56**(7), 387-403.

Lee, D.H., Park, M.K., Oh, J.Y., Kim, K.S., Im, J.H. and Seo, S.Y. (2014), "Web-shear capacity of prestressed hollow-core slab unit with consideration on the minimum shear reinforcement requirement", *Comput. Concrete*, **14**(3), 211-231.

Musmar, M.A. (2013), "Analysis of shear wall with openings using solid65 element", *Jordan J. Civil Eng.*, **7**(2), 164-173.

Ngo, D. and Scordelis, A.C. (1967), "Finite element analysis of reinforced concrete beams", *J. ACI*, **64**(3), 152-163.

Nicolas, I., Nguyen, X.H., Kotronis, P., Mazars, J. and Reynouard, J.M. (2008), "Shaking table tests of lightly RC walls: numerical simulations", *J. Earthq. Eng.*, **12**(6), 849-878.
Palermo, D. and Vecchio, F.J. (2007), "Simulation of cyclically loaded concrete structures based on the finite-element method", *ASCE J. Struct. Eng.*, **133**(5), 728-738.
Park, M.K., Lee, D.H., Ju, H., Hwang, J.H., Choi, S.H. and Kim, K.S. (2015), "Minimum shear reinforcement ratio of prestressed concrete members for safe design", *Struct. Eng. Mech.*, **56**(2), 317-340.
Structural Analysis Guide of Ansys (2013), *ANSYS Inc. Release 14.0*, Canonsburg, PA USA.
Torres, L1., López-Almansa, F., Cahís, X. and Bozzo, L.M. (2003), "A numerical model for sequential construction, repairing and strengthening of 2-D concrete frames", *Eng. Struct.*, **25**(3), 323-336.
TS 498 (1997), "Design loads for buildings", Turkish Standarts Institue, Ankara, Turkey.
TS500 (2000), "Requirements for design and construction of reinforced concrete structures", Turkish Standarts Institue, Ankara, Turkey.
Vallenas, J.M., Bertero, V.V. and Popov, E.P. (1979), "Hysteretic behavior of reinforced concrete structural walls", Report No. UCB/EERC-79/20, Berkeley, University of California, USA.
Von Mises, R. (1913), "Mechanik der festen Köper im plastisch deformablen Zustand", Nachrichten von der Gesellschaft der Wissenchaften zu Göttingen, Mathematisch-Physikalische Klasse, **1**, 582- 592.
Willam, K.J. and Warnke, E.D. (1975), "Constitutive model for the triaxial behaviour of concrete", *Proceedings of the International Association for Bridge and Structural Engineering*, ISMES, Bergama, Italy, **19**, 174-203.

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Notations

$\sum A_{v,i}$ **:** Total section area of vertical bearing elements of the structure in the plan

$\sum A_{h,i}$ **:** Total area of lateral bearing elements of the structure in the plan

S_1: Structure plan area (A_b)
S_2: Ratio of total section area of vertical bearing elements in the plan to the structure plan area
S_3: Ratio of total area of lateral bearing elements in the plan to the structure plan area
S_4: Ratio of structure plan area to total height of structure
S_5: Total equivalent seismic load affecting the structure
S_6: Concrete class
S_7: Reinforcement class
$A(T_1)$: Spectral acceleration coefficient corresponding to the first natural vibration period(T_1)
Ø: The angle of i'th reinforcement of finite element (i=1,2,3) makes with x-y plane in ANSYS
H_i : Height of i'th storey measured from the top foundation level
h_i : Height of i'th storey
H: Total height of building
C_1: Ratio of inertia moment of system beams by axix x-x to the length of the beam
C_2: Ratio of inertia moment of system columns by axix x-x to the height of the column
C_3: Ratio of inertia moment of system shear walls by axix x-x to the height of the shear wall
$I_{xx,b}$: Inertia moment of system beams with respect to x-x axis
$I_{xx,s}$: Inertia moment of system columns with respect to x-x axis
$I_{xx,w}$: Inertia moment of system shear walls with respect to x-x axis
l_b : Length of Beam
l_s : Height of Column

l_w : Height of Shear wall
min$\rho_{w,x,y}$: Minimum reinforcement ratios in the standarts in accordance with x and y directions in the shear wall
$\rho'_{w,x,y}$: Preliminary design & minimum reinforcement ratios suggested in accordance with x and y directions in the shear wall
min$\rho_{w,z}$: Minimum reinforcement ratios in the standarts in accordance with z direction in the shear wall
$\rho'_{w,z}$: Preliminary design & minimum reinforcement ratios suggested in accordance with z direction in the shear wall
min ρ_c : Minimum reinforcement ratios in the standarts for beam top reinforcements
ρ'_c : Preliminary design & minimum reinforcement ratios suggested for beam top reinforcements
min ρ_t : Minimum reinforcement ratios in the standarts for beam bottom reinforcements
ρ'_t : Preliminary design & minimum reinforcement ratios suggested for beam bottom reinforcements
minS_b : Minimum reinforcement ratios in the standarts for beam stirrups
S'_b : Preliminary design & minimum reinforcement provisions suggested for beam stirrups
min ρ_l : Minimum reinforcement ratios in the standarts for column longitudinal reinforcements
ρ'_l : Preliminary design & minimum reinforcement ratios suggested for column longitudinal reinforcements
minS_s : Minimum reinforcement provisions in the standarts for column stirrups
S'_s : Preliminary design & minimum reinforcement provisions suggested for column stirrups
R_a : Seismic load reduction factor
T_1 : First natural vibration period of building
θ: The angle of i’th reinforcement of finite element (i=1,2,3) makes with x plane in ANSYS
W : Total weight of building calculated by considering live load participation factor
W_i : Weight of i’th storey of building by considering live load participation factor
V_t : In the equivalent seismic load method, total equivalent seismic load acting
on the building (base shear) in the earthquake direction considered