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A SIMPLIFIED PRELIMINARY DESIGN METHOD FOR LOW-TO-MEDIUM RISE REINFORCED CONCRETE BUILDINGS

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Abstract

A simplified methodology is proposed in this study for the preliminary design of low-to-medium-rise reinforced concrete buildings without significant structural irregularities. The method is founded on essential principles of earthquake-resistant design; including adequate strength, stiffness, and member/system ductility. Two main classes of building-type structures have been considered for application of the method: (i) buildings consisting solely of moment resisting frames (frame-only systems), and (ii) buildings incorporating frames and structural walls (wall-frame systems). Dimensions of the vertical structural members (columns and walls) are calculated based on simple parameters such as the expected short-period spectral acceleration demand, member tributary areas, estimated gravity loads, total floor area, and the number of floors; whereas beam dimensions are related predominantly to span lengths. The inherent criteria of the method underlying dimensioning of the vertical members include: (i) to limit axial compressive stresses on columns and walls for obtaining adequate member/system ductility, (ii) to provide adequate column and wall areas for avoiding brittle shear failures, and (iii) to achieve adequate lateral stiffness for limiting the interstory drift demands. The method also incorporates a set of minimum cross-sectional dimension and minimum reinforcement (both longitudinal and transverse) requirements for beams, columns, and walls, which can be tailored to satisfy different performance levels. Application of the method does not require any structural analysis, since the method is intended to provide the engineer with a preliminary design of the structural system (in terms of not only member dimensions but also reinforcement) that is safe, yet not overly-conservative.

Both real-life and hypothetical building structures, involving 11 frame-only and 14 wall-frame systems with varying numbers of stories (4–8), were designed based on the proposed methodology. These buildings were then modeled and evaluated as per ASCE 41-13 [1] guidelines. Nonlinear static pushover analyses were conducted to assess their expected performance levels under design-level earthquake demands. At the target displacement, the damage levels of the individual beam, column, and wall members were classified in terms of plastic rotations. As well, member shear capacity checks were conducted, and interstory drift ratios were calculated. The analysis results have shown that among the entire building inventory (including both frame-only and wall-frame systems), all of the vertical members (columns and walls) satisfy the life safety performance level stipulated by ASCE 41-13, whereas the performance level of the beams can be adjusted (via adopting different sets of minimum requirements on beam dimensions and reinforcement) to comply with either life safety or collapse prevention performance levels. Shear capacities of all the members were found to be adequate, and interstory drift ratios were acceptable. Overall, the proposed methodology is believed to be a practical approach, which provides engineers with a reliable preliminary design of low-to-medium-rise reinforced concrete moment-resisting-frame or wall-frame buildings without pronounced structural irregularities.

Keywords: Reinforced concrete; preliminary design; seismic performance

1. Introduction

For the last couple of decades, significant efforts have been devoted to earthquake mitigation studies, and earthquake risk awareness of both municipal and national authorities have extensively increased with increasing population and investments in earthquake-prone areas. Comprehensive studies have been conducted and relevant codes have been updated, leading generally to more restrictive and/or conservative prescriptions. All such activities, however, do not necessarily lead to the conclusion that expected casualties and monetary losses have been decreased sufficiently to reach ‘acceptable’ levels; on the contrary, the 1992 Erzincan, 1995 Dinar, 1998 Ceyhan, 1999 Marmara, 1999 Düzce, 2002 Afyon Sultandağı, 2003 Bingöl, and 2011 Van earthquakes provide ample and most certainly undesired counter evidence. A considerable part of the structural damage has been observed in 2-8 story reinforced concrete residential and office type buildings during the post-earthquake evaluations conducted after the aforementioned earthquakes. Within the scope of the rapid seismic performance evaluation methods and post-earthquake structural damage reports, the following factors have been identified as causes of failures and deficiencies: insufficient strength, ductility, lateral stiffness, poor construction practice and improper design decisions. A strong correlation has been detected between the seismic performance of a building and the area-wise ratio of the columns, structural walls, and infills to the total floor area after the examination of buildings that experienced the Erzincan (1992) earthquake [2]. This correlation, in fact, constituted a basis for the rapid seismic evaluation methods which have been developed afterwards.

Based on fundamental principles of structural earthquake engineering and well-known engineering approximations, Ersoy [3] has derived simple expressions to be used as minimum column and structural wall areas for the preliminary design of relatively regular low-to-medium rise reinforced concrete buildings. This method is limited to wall-frame structural systems; in other words, the method would not be used in structures consisting solely of frames. The total base shear is estimated via the total floor area, expected seismic mass, and spectral acceleration picked from an appropriate design spectrum. Then, by assuming a fixed transverse reinforcement ratio, the required wall area for the estimated seismic forces is derived. The required column sections are determined by limiting the axial stress and by providing adequate shear strength for which the column tributary area, corresponding gravity loads (the seismic mass as well) and a prescribed spectral acceleration are the only parameters. Once the required cross sections are determined, certain amounts of reinforcement are suggested for the structural walls, columns, and beams.

In the current study, the method by Ersoy [3] is modified and also extended to address systems consisting solely of frames (frame-only systems) in addition to wall-frame systems (dual systems). The structural walls, columns, and beams of sample buildings are first designed as per stipulated by the method without any structural analysis. The buildings have then been modeled and nonlinear static analyses have been performed compatible with the provisions in ASCE 41-13 [1]. At the target displacement, the damage levels of the individual beam, column, and wall members are classified in terms of plastic rotations. Member shear capacity checks are conducted, and interstory drift ratios are calculated.

2. Scope

Although relatively irregular buildings were also designed and assessed in the context of this study to diversify the inventory and to investigate the effects of various irregularities, the method is mainly suggested for the buildings without pronounced structural irregularities. Since the safety of the method is of concern, the following restrictions should be applied to the buildings to be designed via this method: Buildings; with minimum 2 stories and maximum 8 stories, with at least 2 spans in each direction, with maximum story height of 4 m, with maximum plan length of 30 m, with the maximum ratio of plan length to plan width of 4, with maximum span length of 7.5 m and minimum span length of 3 m, with maximum cantilever (balconies, etc.) length of 2 m, with solid slab systems, and buildings without soft stories, excessive torsional irregularities, captive columns, and frame discontinuities. Besides, it is suggested to locate the vertical members as symmetrical as possible in plan, to keep the ratio of the two neighboring span lengths below 2, and not to incorporate coupled walls in the structural system. The expressions presented herein are derived for C25 (f_c=25 MPa) concrete and S420a (f_y=420 MPa) reinforcing bar, and can easily be adjusted for other material grades.

3. Preliminary Design

For a prescribed configuration of vertical structural members, tributary areas for columns are multiplied by the expected vertical loads. By limiting the axial stresses resulting from gravity loads and providing adequate shear strength for the estimated earthquake loads, the column cross sectional dimensions are determined. The dimensions for the walls are determined similarly, except that the total floor area is used instead of the total tributary area. For the frame-only systems, an expression concerning the lateral stiffness is also derived in addition to axial stress and shear force criteria. Assuming the first story as the critical one, columns and walls of the first story are dimensioned and these cross sectional dimensions are kept constant along the building height.

3.1 Buildings Consisting Solely of Moment Resisting Frames (Frame-Only Systems)

For ductility purposes, it is intended to limit the axial force on columns by $0.3A_{ci}f_{ck}$ where f_{ck} is the characteristic concrete compressive strength (assumed to be 25 MPa) and A_{ci} is the gross cross sectional area of the column. The estimated dead and live loads are multiplied by the column tributary area and set equal to $0.3A_{ci}f_{ck}$. The required minimum cross sectional area for an individual column is determined as shown in Eq. (1) where ΣA_{oi} is the total tributary area of the base columns (in m^2), g is the dead load (in kN/m^2; including the self weight of the structural members, i.e. walls, columns and beams), and q is the live load (in kN/m^2):

$$A_{ci} \geq 0.00014\left(g+q\right)\sum A_{oi} \tag{1}$$

Shear strength is the second criterion. Since the base shear force in frame-only systems is resisted only by columns, each individual column is expected to have a shear capacity greater than the shear force equal to its seismic mass (along the building height) times the spectral acceleration. The spectral acceleration is assumed to be equal to the peak value (short-period value) on the design spectrum (S_{DS}), and the force reduction factor R is taken as 4. The shear strength V_r of the column is taken as $1.5V_{cr}$ where V_{cr} is the shear cracking strength of the concrete. The suggested shear reinforcement for the mid-region of a column has been determined to ensure V_r = $1.5V_{cr}$. Thus, the shear force estimate for an individual column is set equal to its shear capacity and Eq. (2) is derived:

$$A_{ci} \geq 0.00022 S_{DS}(g+0.3q)\sum A_{oi} \tag{2}$$

In comparison with dual systems, the frame-only systems are highly susceptible to interstory drifts. The drift control in frame structures is crucial, especially for those with number of stories above a certain value. To this end, all of the base columns are considered as a single equivalent column and the seismic mass of the entire building is assumed to be lumped at the first floor level. This single degree of freedom system analogy is used for drift-restraining expressions. The total base shear is calculated in a similar way that is used in deriving Eq. (2). However, instead of the total tributary area ΣA_{oi} the total floor area ΣA_{pi} of the building is used in Eq. (3) representing the total seismic mass of a single degree of freedom system. Since the base of the first story columns is assumed fixed and the upper ends are relatively flexible, the moment inflection points for the base columns are assumed to form at a distance of $2H/3$ from the base where H is the height of the first story. This assumption yields a lateral stiffness value of $6EI/H^3$ (note that EI is the gross section rigidity) as seen in the Fig.1. If the relative interstory drift is limited with 0.010 and E is taken as 25000 MPa, the following equation is obtained:

$$\sum\left(\frac{I_i}{H_i^2}\right) \geq 6.67 \text{ x } 10^{-7} S_{DS}(g+0.3q)\sum A_{pi} \tag{3}$$

Wherein this expression, ΣA_{pi} is the total floor area (in m^2), I_i is the moment of inertia of each individual column in the direction under consideration (in m^4), and H_i is the length of the equivalent column used in the SDOF analogy (height of the i^{th} column), namely the height of the first floor (in m).

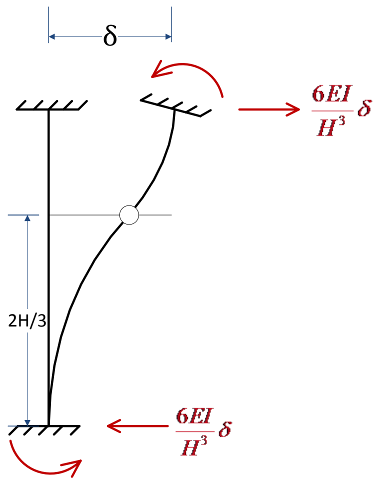


Fig. 1 – Lateral stiffness of a column with *2H/3* inflection point assumption

3.2 Buildings Incorporating Frames and Structural Walls (Wall-Frame Systems)

In wall-frame systems, the axial load acting on a column is limited to $0.35A_{ci}f_{ck}$, and Eq. (1) turns into Eq (4):

$$A_{ci} \geq 0.00012\left(g+q\right)\sum A_{oi} \tag{4}$$

In wall frame systems, the columns are assumed to resist 1/3 of the total base shear, and the force reduction factor R is taken as 3. Hence, Eq. (2) turns into Eq. (5):

$$A_{ci} \geq 0.0001S_{DS}(g+0.3q)\sum A_{oi} \tag{5}$$

The force reduction factor R for wall-frame structures is assumed to be 3. Moreover, the walls are assumed to resist 100% of the base shear. Taking both vertical and horizontal reinforcement in the wall web as 0.0025 and equating the shear capacity of the wall to the total base shear, the total wall area required for each orthogonal direction is obtained as in Eq. (6) where ΣA_{wi} is the required wall area per direction, and ΣA_{pi} is the total floor area of the entire building:

$$\sum A_{wi} \geq 0.0002S_{DS}\left(g+0.3q\right)\sum A_{pi} \tag{6}$$

In order to provide adequate wall area for low rise buildings and/or buildings with small floor areas, the total wall area for both directions should also satisfy Eq. (7):

$$\sum A_{wi} \geq 0.0007S_{DS}\left(g+0.3q\right)A_{pt} \tag{7}$$

Lastly, total column and wall area should satisfy Eq. (8) for each orthogonal direction:

$$\left(\sum A_{ci}+\sum A_{wi}\right) \geq 0.0003S_{DS}\left(g+0.3q\right)\sum A_{pi} \tag{8}$$

4. Reinforcement and Detailing

For confinement, the amount of minimum transverse reinforcement at column end zones are given in Eq. (9), as specified in the 2007 Turkish Seismic Code (TSC) [4]:

$$A_{sh} \geq 0.3 s b_k \left[\left(A_c / A_{ck} \right) - 1 \right] \left(f_{ck} / f_{ywk} \right) \; \rightarrow \; A_{sh} \geq 0.075 s b_k \left(f_{ck} / f_{ywk} \right) \tag{9}$$

The minimum dimension for a column cross section is 300 mm, and the longitudinal column reinforcement ratio is 0.01. For a column section in both wall-frame structures and frame-only structures, the ratio of the longer side to the shorter side (*h/b*) should not exceed 2. The theoretical premises behind this condition are: To satisfy the strong column principle in both directions, to prevent higher shear demands in strong directions of the columns, and to ensure that the moment inflection point forms in reasonable heights, i.e. the characteristic of the moment diagram is different from those of the shear walls. In order to prevent weak column behavior, the longitudinal reinforcement is increased with increasing *h/b*: When *h/b=2*, the longitudinal reinforcement requirement is modified to 0.015, and for intermediate values of *h/b*, linear interpolation is suggested. The transverse reinforcement ratio in each direction along the intermediate height of a column is greater than 0.00165 (to ensure V_r=1.5V_{cr}) for square column sections. For *h/b*=2, this ratio increases to 0.0025. The maximum vertical spacing between two consecutive ties and cross ties is 200 mm, and the maximum horizontal spacing between tie or cross tie legs is 25 times the tie cross sectional diameter.

For confinement at beam end regions, the maximum tie spacing at beam ends are the minimum of 8 times the diameter of the smallest longitudinal reinforcement, 150 mm, and 1/4 of the effective depth. The flexural reinforcement ratios at beam ends are 0.006 and 0.003 at top and bottom, respectively. The transverse reinforcement ratio for the beam mid-region is at least 0.0025, and the spacing between ties and cross ties is less than half the effective depth of the beam.

The minimum structural wall width is 250 mm, the transverse reinforcement in confinement zones (boundary regions) is as stipulated by TSC, the vertical and horizontal web reinforcement is 0.0025 times the gross wall cross sectional area and the corresponding maximum spacing between two adjacent reinforcing bars is 250 mm. The vertical reinforcement ratio for boundary regions are 0.002 and 0.001 of the gross wall area along the wall critical height (1/6th of the wall height) and outside of the critical height, respectively. In each orthogonal direction at least 2 structural walls or wall wings are located, as symmetrically in plan as possible.

After numerous iterative trial-error cycles of structural analyses, the minimum beam depth for wall-frame structures and for frame-only structures are decided to be 1/12 and 1/10 of the span length (*L*), respectively. The smallest dimensions for a beam are 300 mm x 500 mm.

5. Modeling and Structural Analysis

Various hypothetical and real life buildings for a total of 14 wall-frame structures (thirteen 8-story, one 4-story), and 11 frame-only (nine 8-story, two 4-story) structures are first designed (in terms of cross sectional areas and preliminary reinforcement amounts) according to the proposed method without any structural analysis, and then modeled and analyzed as per the nonlinear static procedure (NSP) in ASCE 41-13 guidelines. As stated previously, C25 concrete and S420a steel material grades were used for all buildings. The buildings were modeled as a 3D composition of discretized line (frame) elements in SAP2000 v15.1 [5], representing the beams and columns. The structural walls are also modeled using line elements. The seismic mass is discretized and distributed to the nodes where beams and columns/walls are connected. The base is assumed as fixed and no soil-structure interaction is considered. The beam-column joints are considered as rigid joints and rigid diaphragm constraints are assigned to each floor level, since only buildings with solid slab systems with regular geometry are considered. Nonlinear hinges are assigned to member ends as zero-length lumped plasticity hinges. Only unidirectional flexural hinges (M3) are assigned to the beam members, whereas interacting axial force and bi-directional bending hinges (P-M2-M3) are assigned to the walls and columns. The force displacement relationships and acceptance criteria of the hinges are implemented as per stipulated in ASCE 41-13.

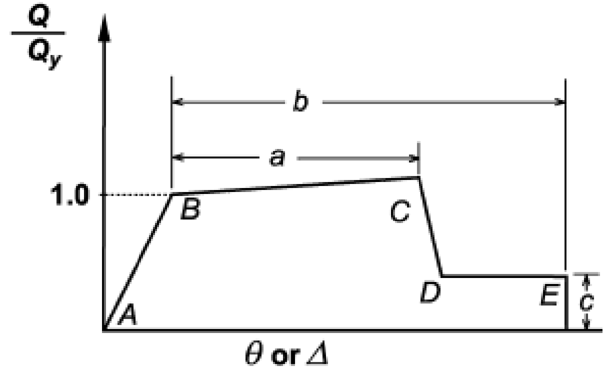


Fig. 2 – Typical load-deformation backbone from ASCE 41-13 (2014).

Strain hardening characteristics are not implemented in the plastic hinge models, and an elastic-perfectly plastic moment-rotation backbone is adopted. However, in the post-processing stage, the rotations of the members are substantiated to remain within the specified acceptance criteria. In other words, the final plastic rotations are checked to ensure that the plastic rotation value at a hinge was not beyond the rotation limits specified in ASCE 41-13. Another issue is the flexural rigidities of the members with cracked sections. There is an apparent scatter among various studies on the cracked section rigidities. Moreover, during the analyses, it was observed that cracked section rigidities can significantly change the modal responses, leading to significant changes in corresponding lateral load patterns and the interstory drifts. The cracked section multipliers listed in Table 10.5 of ASCE 41-13 are adopted. The applicability of single mode pushover is restricted with certain limitations in ASCE 41-13. In order to decide whether the higher mode effects are significant for a specific building or not, two linear response spectrum analyses are performed, one with single mode and the other with a sufficient number of modes to satisfy 90% modal mass participation. If the ratio of story shears obtained from these two analyses are less than 1.3, it is concluded that the higher mode effects are not very significant and single mode pushover is considered applicable. This criterion has been checked for all the buildings.

The seismic hazard data used to construct the design spectrum is adopted from the one prescribed in the Turkish Seismic Code with 10% probability of exceedance in 50 years, i.e. 475-year return period, corresponding to a short-period spectral acceleration coefficient S_{DS} of 1.0.

The target displacement is obtained by the coefficient method in ASCE 41-13 and pushover analyses are performed for the two orthogonal directions in both negative and positive senses (+x, -x, +y, -y). The coefficient method requires a pushover curve and the curve must be continued to at least 1.5 times the target displacement since there is a great scatter in the seismic hazard analysis and the building must be able to resist displacement demands beyond the target displacement. The target displacement is then multiplied by the coefficient η which represents torsional irregularity. For a specified lateral load pattern exerted on the building, η is the ratio of the maximum floor displacement to the average floor displacement. η is calculated for all stories and the maximum value of which is multiplied by the target displacement. At this new target displacement, the plastic rotations and shear forces are checked against the acceptance criteria and member capacities.

6. Building Models and Performance Levels

Illustrative plan views of the buildings used in the analyses are depicted in Figures 3 and 4.

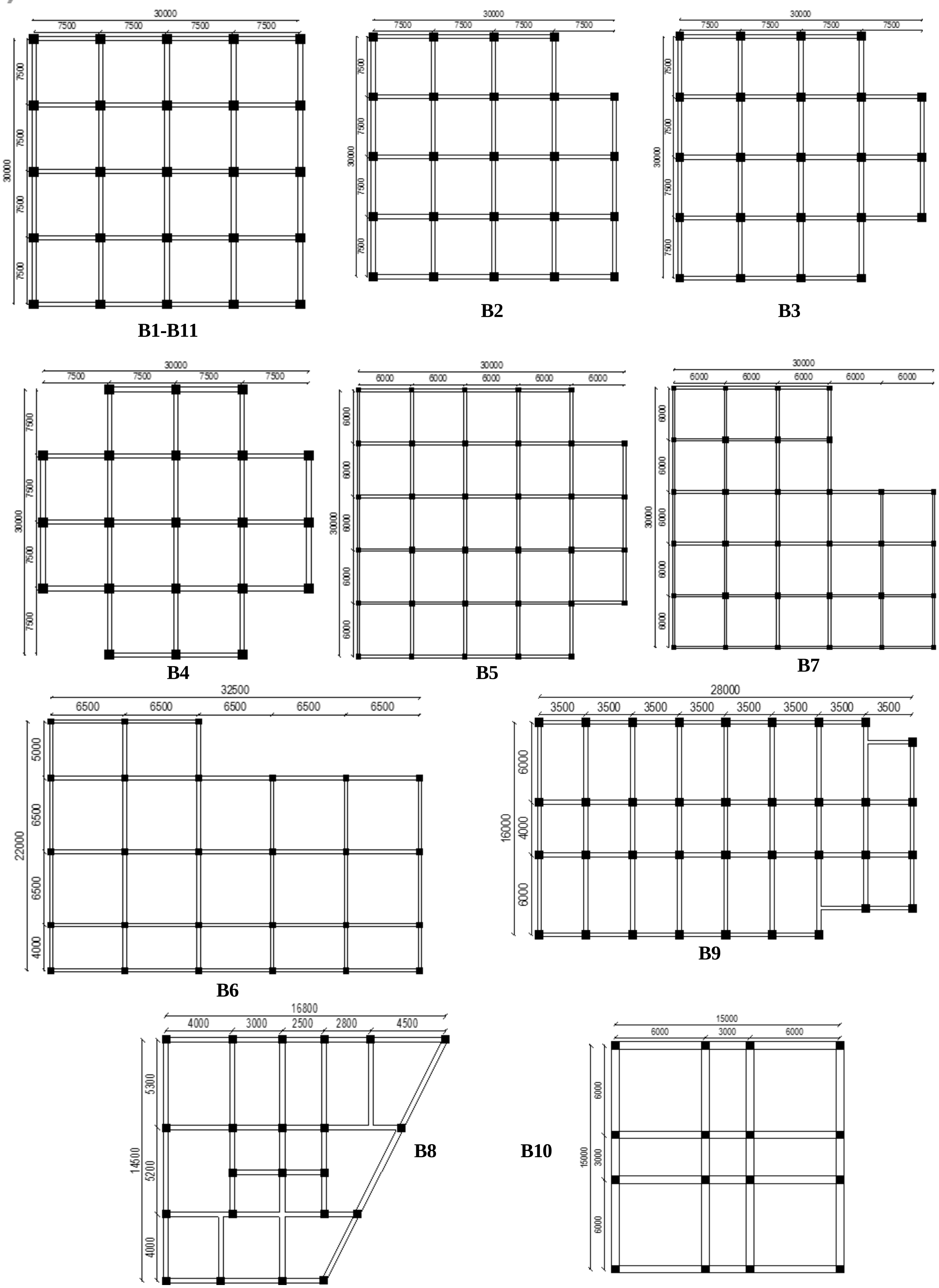


Fig. 3 – Frame-only buildings

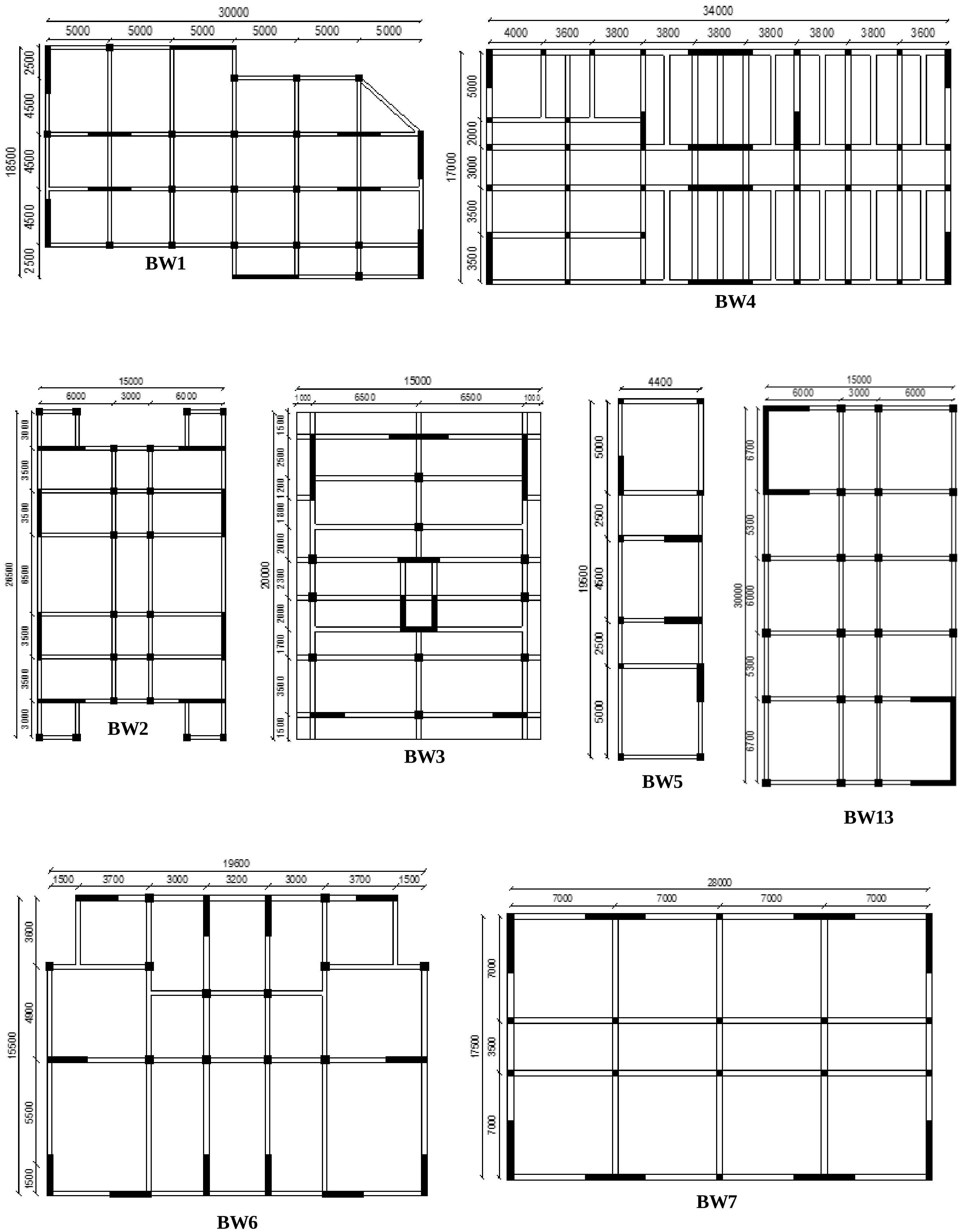
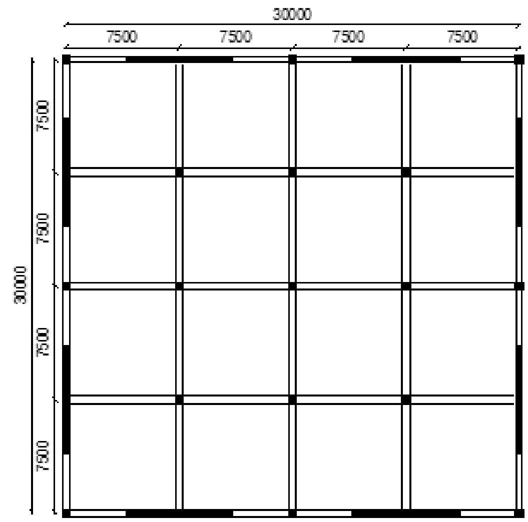
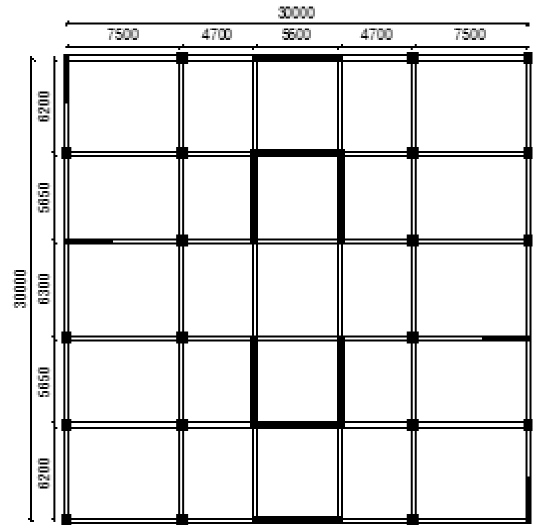


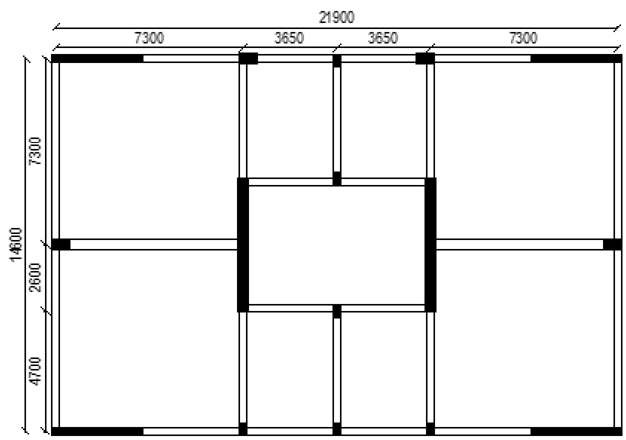
Fig. 4 – Wall-frame structures



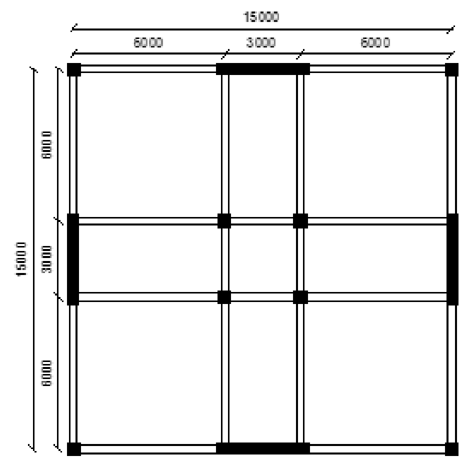
BW9



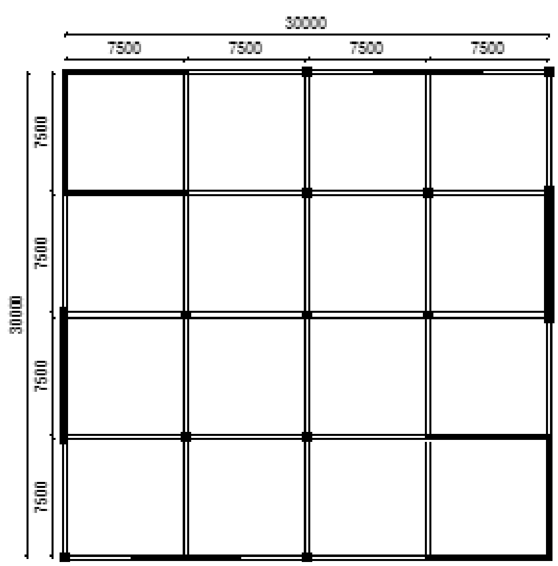
BW12



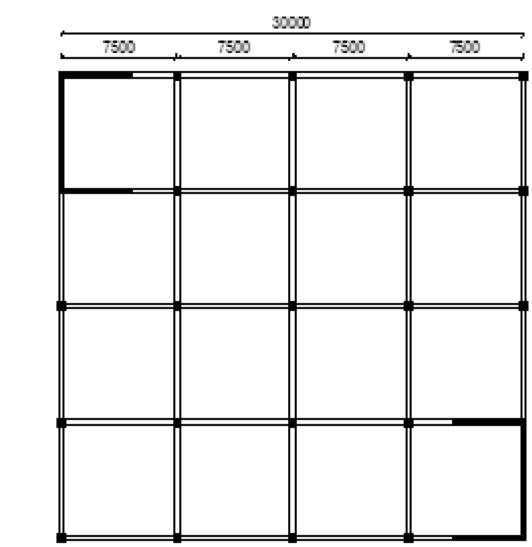
BW8



BW11



BW10



BW14

Fig. 4 – Wall-frame structures (*cont'd.*)

Tables 1 and 2 provide information on building characteristics, as well as the interstory drift ratios and the minimum shear capacity/demand ratios obtained for each type of structural member in a building. ASCE 41-13 does not define any acceptance criteria for drift ratios obtained from the linear or nonlinear static procedures. However, when the cracked section rigidities are defined according to the upcoming 2016 version of the Turkish Seismic Code (0.35*EI* for beams, 0.7*EI* for columns, and 0.5*EI* for walls) and linear response spectrum analyses are conducted, the drift ratios presented in the tables are obtained. The drift ratios obtained for both types of structural systems are satisfactory. The shear capacities of the cross sections are also computed using design material strengths and compared to the earthquake demands in the tables, and it is observed that shear capacity/demand ratios larger than 1 are attained.

Table 1 ─ General properties, interstory drift ratios, and shear checks of frame-only buildings

Building	H (m)	# of Storeys	η_x	η_y	$(\delta/H)_{max}$	Column Shear C/D	Beam Shear C/D
B1	4	8	1.00	1.00	0.0156	1.55	1.12
B2	4	8	1.04	1.01	0.0159	1.54	1.12
B3	4	8	1.00	1.07	0.0160	1.55	1.12
B4	4	8	1.00	1.00	0.0154	1.57	1.12
B5	4	8	1.00	1.06	0.0160	2.14	1.22
B6	4	8	1.10	1.19	0.0152	1.96	1.24
B7	4	8	1.00	1.03	0.0157	2.08	1.22
B8	4	8	1.30	1.18	0.0136	1.63	1.21
B9	4	8	1.02	1.41	0.0139	2.07	1.37
B10	4	4	1.00	1.00	0.0104	1.89	1.00
B11	4	4	1.00	1.00	0.0135	1.43	1.14

Table 2 ─ General properties, interstory drift ratios, and shear checks of wall-frame buildings

Building	H (m)	# of Storeys	η_x	η_y	$(\delta/H)_{max}$	Wall Shear C/D	Col. Shear C/D	Beam Shear C/D
BW1	3.5	8	1.14	1.11	0.0127	1.54	1.62	1.75
BW2	3.5	8	1.00	1.00	0.0144	2.16	1.62	1.53
BW3	3.5	8	1.23	1.00	0.0133	1.47	1.27	1.00
BW4	4	8	1.02	1.07	0.0127	2.46	1.41	1.13
BW5	4	8	1.16	1.03	0.0135	2.84	1.66	2.25
BW6	4	8	1.41	1.00	0.0151	2.94	2.1	1.40
BW7	4	8	1.00	1.00	0.0116	3.63	1.64	2.53
BW8	4	8	1.00	1.00	0.0108	3.3	2.06	2.48
BW9	4	8	1.00	1.00	0.0076	2.96	1.53	2.38
BW10	4	8	1.01	1.03	0.0064	1.01	1.92	2.36
BW11	4	8	1.00	1.00	0.0141	4.99	1.96	3.52
BW12	4	8	1.00	1.32	0.0087	1.11	2.23	1.92
BW13	4	8	1.00	1.00	0.0097	1.18	1.42	3.38
BW14	4	4	1.00	1.00	0.0045	1.1	1.61	1.92

At the target displacement level, plastic rotations at member ends and corresponding ASCE 41-13 performance levels are shown in Table 3 and Table 4. In these tables, E corresponds to "*elastic*" state, IO stands for "*immediate occupancy*", LS stands for "*life safety*", and CP represents "*collapse prevention*" performance levels. The wall and beam percentages in the tables are defined only for the walls and beams in the direction under consideration. As can be seen from the tables, all of the vertical members satisfy the life safety performance level, which is essentially the intended purpose of this method, via proper dimensioning and adequate reinforcing of the vertical members. Furthermore, with a beam depth of *L/10* in frame-only structures, it can be seen that almost all of the beams satisfy the life safety performance level as well. However, in irregular buildings such as B8 and B9, the overall performance level of beams can remain somewhere in between life safety and collapse prevention. In wall-frame structures all vertical members satisfy the life safety performance level, and in some buildings, the structural walls satisfy the immediate occupancy performance level. In a significant portion of the frame-only buildings, especially those that are free of frame discontinuities and those with symmetrical structural configurations, most of the columns satisfy the immediate occupancy performance level.

Table 3 ─ Performance level of structural elements of frame-only buildings as per ASCE 41-13

Building	Columns %				Beams %			
	E	IO	LS	CP	E	IO	LS	CP
B1	81.50	18.50	0.00	0	25.00	15.00	60.00	0
B2	78.65	16.15	5.21	0	25.00	14.47	60.53	0
B3	77.72	13.59	8.70	0	25.00	13.89	61.11	0
B4	77.98	16.67	5.36	0	25.00	18.75	56.25	0
B5	88.24	11.76	0.00	0	25.00	12.95	62.05	0
B6	85.65	12.96	1.39	0	25.00	15.34	59.66	0
B7	87.50	12.50	0.00	0	25.00	12.50	62.50	0
B8	84.09	10.80	5.11	0	30.63	22.50	35.63	11
B9	64.58	20.83	14.58	0	28.70	25.46	25.93	20
B10	50.00	12.50	37.50	0	33.33	41.67	25.00	0
B11	69.00	6.00	25.00	0	25.00	25.00	50.00	0

Table 4 ─ Performance level of structural elements of wall-frame buildings as per ASCE 41-13

Building	Walls %				Columns %				Beams %			
	E	IO	LS	CP	E	IO	LS	CP	E	IO	LS	CP
BW1	0.00	0.0	100.00	0	52.08	28.47	19.44	0	5.50	55.00	38.00	2
BW2	0.00	0.0	100.00	0	38.75	26.25	35.00	0	10.42	60.42	29.17	0
BW3	0.00	0.0	100.00	0	72.50	10.00	17.50	0	5.88	16.18	77.94	0
BW4	0.00	0.0	100.00	0	55.09	25.00	19.91	0	9.72	42.78	45.83	2
BW5	0.00	0.0	100.00	0	29.69	40.63	29.69	0	4.17	8.33	79.17	8
BW6	0.00	0.0	100.00	0	50.00	10.42	39.58	0	3.75	8.75	62.50	25
BW7	0.00	100.0	0.00	0	95.83	0.00	4.17	0	0.00	6.25	93.75	0
BW8	0.00	0.0	100.00	0	90.00	0.00	10.00	0	3.57	46.43	50.00	0
BW9	0.00	100.0	0.00	0	95.59	2.94	1.47	0	0.00	55.00	45.00	0
BW10	0.00	100.0	0.00	0	95.45	2.27	2.27	0	0.00	60.16	39.84	0
BW11	0.00	0.0	100.00	0	87.50	0.00	12.50	0	0.00	10.00	90.00	0
BW12	0.00	0.0	100.00	0	88.75	9.38	1.88	0	1.92	41.35	51.92	5
BW13	0.00	0.0	100.00	0	86.25	8.75	5.00	0	1.39	36.11	45.83	17
BW14	0.00	0.0	100.00	0	50.00	35.00	15.00	0	25.00	52.50	22.50	0

7. Conclusion

A simplified methodology is proposed in this study for the preliminary design of low-to-medium-rise reinforced concrete buildings without significant structural irregularities. The method is founded on essential principles of earthquake-resistant design; including adequate strength, stiffness, and member/system ductility.

A set of real-life and hypothetical building structures, involving 11 frame-only and 14 wall-frame systems with varying numbers of stories (4–8), were designed based on the proposed methodology. These buildings were then modeled and evaluated as per ASCE 41-13 guidelines. Nonlinear static pushover analyses were conducted to assess their expected performance levels under design-level earthquake demands. At the target displacement, the damage levels of the individual beam, column, and wall members were classified in terms of plastic rotations. As well, member shear capacity checks were conducted, and interstory drift ratios were calculated. Results indicated that the performances of the buildings designed according to the methodology proposed were favorable, in terms of both the plastic rotations developing at member ends and the shear capacity/demand ratios obtained for the members. Overall, the proposed methodology is believed to be a practical approach, which provides engineers with a reliable preliminary design of low-to-medium-rise reinforced concrete moment-resisting-frame or wall-frame buildings without pronounced structural irregularities.

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