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A study on the comparison of a steel building with braced frames and with RC walls

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Abstract. In this study, two geometrically identical multi-storey steel buildings with different lateral load resisting systems are structurally analyzed under same earthquake conditions and they are compared with respect to their construction costs of their structural systems. One of the systems is a steel structure with eccentrically steel braced frames. The other one is a RC wall-steel frame system, that is a steel framed structure in combination with a reinforced concrete core and shear walls of minimum thickness that the national code allows. As earthquake resisting systems, steel braced frames and reinforced concrete shear walls, for both cases are located on identical places in either building. Floors of both buildings will be of reinforced concrete slabs of same thickness resting on composite beams. The façades are assumed to be covered identically with light-weight aluminum cladding with insulation. Purpose of use for both buildings is an office building of eight stories. When two systems are structurally analyzed by FEM (finite element method) and dimensionally compared, the dual one comes up with almost 34% less cost of construction with respect to their structural systems. This in turn means that, by using a dual system in earthquake zones such as Turkey, for multi-storey steel buildings with RC floors, more economical solutions can be achieved. In addition, slender steel columns and beams will add to that and consequently more space in rooms is achieved.

Keywords: steel building with braced frames; steel building with RC wall-steel frames; dual structural systems; earthquake resisting frames; composite beams; construction cost comparison

1. Introduction

1.1 Aim of the study

The most important characteristic of earthquake loads acting on the buildings is that damages depending from them result mainly from the behavior of the structural systems selected. Therefore, it requires special attention to select a proper structural system for a steel building which is both resistant against earthquake loads and also economical.

This paper tries to investigate the effect of different lateral load resisting systems on the above mentioned earthquake resistance and economy. This will be done by means of selecting two different lateral load resisting systems for two geometrically identical steel structures, in fact a dual system that is composed of a steel structure with reinforced concrete shear walls and the other one; a pure steel structure with braced frames.

This is going to be achieved by performing structural analysis for both of the structures under same earthquake conditions, dimensioning and comparing them economically in terms of material used, fabrication and construction costs.

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1.2 Background

1.2.1 Lateral load resisting structural systems in multi-storey steel buildings

In multi-storey steel buildings depending on the number of stories, from lowest to highest, following system are selected; rigid frames, braced frames, tube cores, frames with core and tube systems.

Steel braced frames and reinforced concrete shear walls

In order to prevent drifts and to obtain smaller beam and column sections braced frames and/or reinforced concrete walls are designed. These may be of two basic types (Fig. 1). In conventional concentrically braced steel frames, the seismic performance is primarily dependent on the behavior of the bracing elements, which are the members devoted to dissipate the input energy according the philosophy of capacity design implemented in current codes (Tenchini *et al.* 2014). A large number of research studies on seismic response and performance of concentric braced frames can be found in the literature providing recommendations for their modelling (D'Aniello *et al.* 2013, 2015). Also, since the development of significant axial forces can impair the response of link end connections and also the effectiveness of capacity design rules, another modelling suggestion regarding the boundary conditions of the links are examined in theoretical and experimental studies (Della Corte *et al.* 2007, 2013 and Mazzolani *et al.* 2009).

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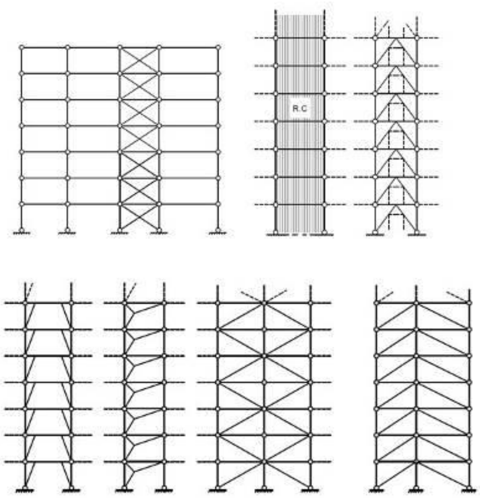

Fig. 1 Braced steel frames and reinforced concrete (RC) shear walls

The disastrous effects of past earthquakes on life and properties have increased the need for a close review of the conventional lateral load resisting systems and to adopt innovative and modified lateral load resisting systems for effective and efficient mitigation of earthquake forces. Dual systems with moment resting frames and shear wall elements have gained significant popularity in the recent years as effective construction methods in high seismicity areas. The significant improvement in the seismic capacity achieved by buildings by the introduction of shear walls have led to the concept of buildings built entirely of reinforced concrete walls popularly called as RC walled buildings (Rajesh and Prasad 2014). In the steel-concrete composite structural system, partially restrained steel frame with RC infill wall (PSRCW) became a primary lateral-load resisting systems in multistory building. PSRCW consists of bare steel moment-resisting frame, RC infill wall, partially restrained connections, and shear connectors. The composite action between steel frame and infill walls is achieved by shear connectors. In the PSRCW system, the infill wall serves as the main lateral resisting element providing high lateral stiffness and strength, while the surrounding steel frame resists the gravity and most of the overturning moment due to the seismic loading (Sun *et al.* 2011). In China, concrete-filled steel tubular (CFST) frame structures are often mixed with reinforced concrete (RC) shear walls to form a high-rise building system to resist both the vertical and lateral loads efficiently (Hana *et al.* 2009). In earthquake -resistant design, the influence of steel beams developing ductile behavior with high rotation capacity has been also the subject of various researches (D'Aniello *et al.* 2012 and Güneyisi *et al.* 2013, 2014).

1.2.2 Effect of lateral loads on steel weight

Effects of lateral loads on structural steel weight are as a potential function. A diagram for steel structures given below demonstrates that steel weight increases potentially versus to storey number, as the conventional multi-storey steel buildings, either framed structures or with tubes are concerned (Fig. 2) (Arda 1978).

Fig. 2 Steel weight in multi-storey steel buildings (kg/m^2) (a) Only due to vertical loads (b) Due to vertical and lateral loads (c) Only due to lateral loads

2. Seismic design code

Turkey is located at seismic zone and many destructive earthquakes have hit the country through its history. Turkish Earthquake Code 2007 (TEC 2007) is the seismic design code which is currently in use. Eurocode 8 (EC8) is not compulsory for our country. A brief comparison of two seismic codes will be given hereby to have an idea on the design principles. As ground conditions have a huge effect on behavior of the structure under lateral loads, several spectrum types are provided by design codes for each soil type. Both codes consider the shear wave velocity to determine soil type. But, the Turkish Earthquake Code requires more specific definition of soil profile such as depth. For simplicity, generalized shear wave velocity

Table 1 Ground Types Description (EC8 and TEC 2007)

Soil Type		Definition	
EC8	TEC 2007	EC8	TEC 2007
A	Z1	Rock or other rock-like geological formation Vs>800 m/s	Very dense sediment, gravel and solid clay Vs<700 m/s
B	Z2	Deposits of very dense sand, gravel, or very stiff clay 360 m/s<Vs<800 m/s	Dense sediment gravel, very stiff clay 300 m/s<Vs<700 m/s
C	Z3	Deep deposits of dense or medium dense sand, gravel or stiff clay 180 m/s<Vs<360 m/s	Medium dense sediment and gravel, stiff clay 200 m/s<Vs<300 m/s
D	Z4	Deposits of loose-to-medium cohesionless soil Vs<180 m/s	Weak sediment, soft clay with alluvium layer High water table Vs<200 m/s
E		A surface of alluvium layer with water table a layer of Type C or D on Rock	
S1	-	A layer of at least 10 m thick soft clays/silts	-
S2	-	Sensitive clays, or any other soil profile not included in types A - E or *S*1	-

values have been shown in Table 1. Site period differences are covered by both codes but only Eurocode 8 provides soil amplification factors (S) which increases the spectral acceleration during all period range.

Also TEC 2007 requires higher ductility characteristics compare to Eurocode 8. The used material also affects the ductility behavior of the structure and TEC 2007 puts a limit to upper yield strength of the reinforcement steel. TEC 2007 aims to use ductile material and keep the reduction factor high (Table 2).

As can be seen from Table 3, soil classification is well distributed in Eurocode 8. But in Turkish Earthquake Code 2007 has limited options in terms of soil type selection. Also Eurocode 8 allows special ground type (S1 and S2) use by deriving site specific period values.

Furthermore, as it can be seen from Table 4, both codes provide different response spectrums with period differences. Also, Eurocode 8 defines the soil amplification factors at response spectrum stage. The EC8 supplied seismic zoning map is for rock conditions and soil factors (S) balance the total spectral acceleration. However, there is no such soil amplification factor in TEC2007. The TEC 2007 spectrums only vary by the change of soil period.

Table 2 Used Material Comparison (TEC 2007, EC8, EC2, CYS EC2 NA)

	TEC 2007	EC 8	
Ductility	Medium and High	Medium	High
Characteristic strength of reinforcement	Fyk ≤ 420MPa	Fyk ≤ 600 (CYS EC2 NA)	Fyk ≤ 600 (CYS EC2 NA)
Characteristic strength of concrete	Fck ≥ C20/25	Fck ≥ C16/20	Fck ≥ C20/25
Type of the reinforcement	BÇI (220) BÇIIIa (420)	B or C type reinforcement	C type reinforcement
Minimum strain of reinforcement at maximum stress	10%	5% (EC2)	7.5% (EC2)

Table 3 Spectrum Parameters (EC8 and TEC 2007)

	Soil Factor		Beginning of Peak range (seconds)		End of Peak range (seconds)		Spectral Acceleration Coefficient	
Ground Type	EC8 (S)	TEC 2007	EC8 (TB)	TEC 2007 (TA)	EC8 (TC)	TEC 2007 (TB)	EC8	TEC 2007
Type A or Z1	1	1	0.15	0.10	0.40	0.30	2.5	2.5
Type B or Z2	1.2	1	0.15	0.15	0.50	0.40	2.5	2.5
Type C or Z3	1.15	1	0.20	0.15	0.60	0.60	2.5	2.5
Type D or Z4	1.35		0.20		0.80			
Type E or Z4	1.4	1	0.15	0.20	0.50	0.90	2.5	2.5
S1 and S2	EC8 requires special studies to provide the corresponding values of *T*B, *T*C and *T*D.							

Table 4 Spectrum Ordinates (EC8, TEC2007)

	$T \leq T_B$	$T_B \leq T \leq T_C$	$T \geq T_C$
TEC 2007	$S_e = a_{gR}[1+1.5\frac{T}{T_B}]$ $S_d = \frac{a_g}{R_a}[1+1.5\frac{T}{T_B}]$	$S_e = 2.5a_{gR}$ $S_d = \frac{2.5a_g}{R_a}$	$S_e = 2.5a_{gR}[\frac{T_C}{T}]^{0.8}$ $S_d = \frac{2.5a_g}{R_a}[\frac{T_C}{T}]^{0.8}$
EC8	$S_e = a_g S[1+\frac{T}{T_B}(\eta 2.5-1)]$ $S_d = a_g S[\frac{2}{3}+\frac{T}{T_B}(\frac{2.5}{q}-\frac{2}{3})]$	$S_e = 2.5a_g S\eta$ $S_d = \frac{2.5}{q}a_g S$	*For* $T_C \leq T \leq T_D$: $S_e = 2.5a_g S\eta[\frac{T_C}{T}]$ *For* $T_C \leq T \leq T_D$: $S_d \begin{cases} = \frac{2.5}{q}a_g S\left[\frac{T_C}{T}\right] \\ \geq \beta \cdot a_g \end{cases}$ *For* $T_D \leq T \leq 4s$: $S_e = 2.5a_g S\eta[\frac{T_C T_D}{T^2}]$ *For* $T \geq T_D$: $S_d = \frac{2.5}{q}a_g S[\frac{T_C T_D}{T^2}]$ $\geq \beta . a_g$

Table 5 Base Shear Formulas for Design Codes (EC 8, TEC 2007)

	TEC 2007	EC 8
Base Shear Formula	Vt= Sd.W/g	Fb=Sd.λ.W/g λ=0.85 for Tc<2s

The fundamental period of the structure can be found by either using dynamic analysis or either using code provided empirical formulas. However, with the latest revision of Turkish Earthquake Code, the use of empirical formulas is prohibited and only dynamic analysis is available to find fundamental period of the structure. Base shear formulas for both of the design codes are given in Table 5 (Safkan 2012).

3. Structural systems

3.1 Model-1: Steel building with braced frames

3.1.1 Description of the system

The steel structure is an eight-storey office building of 287 m^2 floor area with plan dimensions of (18,5 m×15,5 m). Each floor height is 3 m, where the total building height is 24 m. Plan and section views are depicted in (Figs. 3-4).

Eccentrically steel braced frames are placed along (x) and (y) directions as lateral load resisting frames in accordance with the Turkish Earthquake Code 2007's regulations. In fact, they are designed basically against earthquake loads which are far more governing wind loads in this study. Floors of both buildings shall be of reinforced concrete slabs of same thickness (10 cm) which rest on composite beams. Some floor finishing and insulation shall be foreseen both for office rooms and also wet rooms (Celep 2007). Studs are used on top of steel beams, so as to achieve economy in steel and also stability on decks. Steel trapezoidal sheets are used on top of steel beams, which are serving as formwork while pouring floor slabs (Arda and Yardimci 1991). Suspended ceilings, necessary electrical

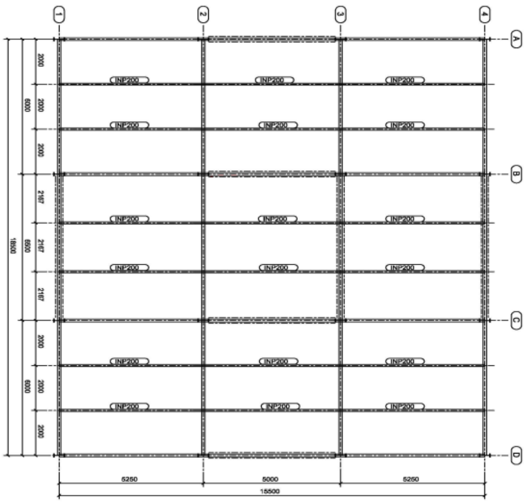

Fig. 3 Typical floor plan

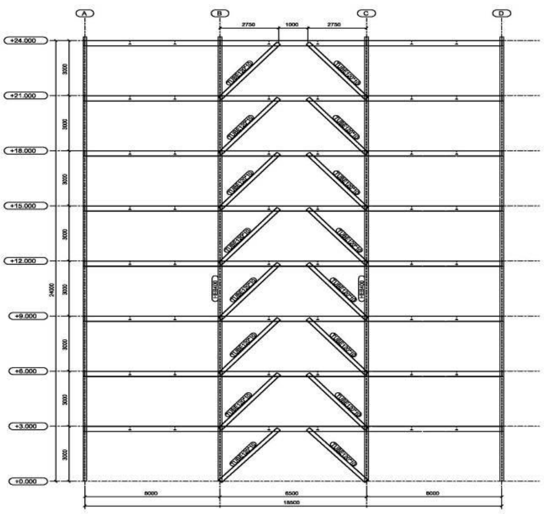

Fig. 4 Eccentric braced frames (Axis 1, 3 and 4) & (A, B, C and D)

and mechanical installations shall all be furnished for everywhere applicable whose superloads are going to be taken into account as dead loads as well. The façades are assumed to be covered with classical prefabricated materials of light-weight aluminum cladding with insulation. The roof shall be a flat roof with a RC slab and sloped from center to sides and corners to down stops.

3.1.2 Analysis of the loads

Earthquake loads are evaluated according to Turkish Earthquake Code where some special values are taken as;

Earthquake zone (most severe zone): 1

Importance factor (office building): 1,0

Live load contribution factor (n): 0,3

Peak ground acceleration (A_o): 0.40 g

Behavior coefficient (R): 7

Soil class (Z_n): Z_2 (Dense sediment gravel, very stiff clay, 300 m/s<V_s<700 m/s)

Table 6 Dead, snow and live loads

Floor Name	dead load (kN/m^2)	snow load (kN/m^2)	live load (kN/m^2)
roof (24,00 m elevation)	2,943	0,736	1,962
standard floor (+3,00 m,…+21,00 m elevation)	4,561	-	1,962

Table 7 Total mass distribution

ΣW_{EQ} (kN)
11881,800

Table 8 Total earthquake loads in *x* and *y* directions

ΣF_{ix}(kN)	ΣF_{iy}(kN)
943,754	930,180

Spectral period (T_A): 0.15 s, Spectral period (T_B): 0.40 s

Behavior coefficient is selected as R=7, which is applicable for “eccentrically braced frames” where energy dissipation is increased. So, in both (*x*) and (*y*) directions of the building, earthquake forces are resisted for these “braced frames”.

Dead loads, snow loads, live loads are calculated and summarized in Table 6.

Total mass distribution value during earthquake is given in Table 7.

Total earthquake loads in *x* and *y* directions acting on the structure are given in Table 8. As earthquake loads govern wind loads as far as the overall strength of the structure is concerned, structural analysis will be performed based on earthquake loads only.

3.1.3 Structural analysis of the system

65 load combinations are incorporated in the structural analysis as indicated in Turkish Earthquake Code.

__Structural model__

All analysis performed so has been incorporated a

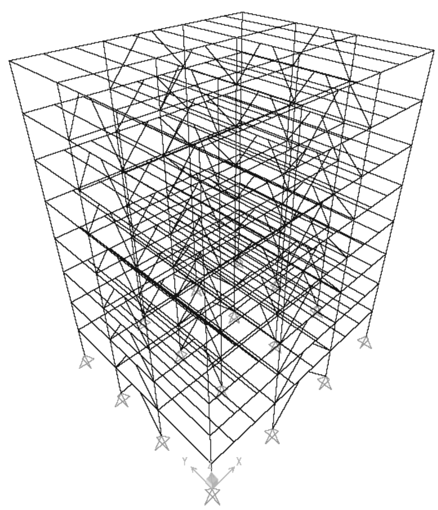

Fig. 5 Sap2000 3D static system model

Table 9 Maximum relative displacements

δ_{ix}/h_i	δ_{iy}/h_i
0,0098	0,0100

Table 10 Steel members' sections

Member	Profile
Floor Beams (interior)	INP200
1,2,3,4 Axis Main Beams	HEB280
A-1,2 Axis btw. A-3,4 Axis Beams	HEB160
D-1,2 Axis btw. D-3,4 Axis Beams	HEB160
A,B,C,D-2,3 Axis Beams	HEB280
B-1,2 Axis btw. B-3,4 Axis Beams	HEB200
C-1,2 Axis btw C-3,4 Axis beams	HEB200
1 and 4 Axis Columns	HEB300
A and D Axis Columns	HEB300
2,3 and B,C Axis Columns	HEB400
Members of Braced Frames (diagonals)	Box 120×120×10

modern FEM (finite element method) software called SAP2000, where all structural members are modelled into a structural system (Fig. 5).

Relative column displacements in x and y direction are evaluated and the maximum relative displacements are given in Table 9. It is seen that these values are below the limit $(\delta_i/h_i) < 0{,}02$ according to the Turkish Earthquake Code.

3.1.4 Dimensioning of main members

All members of the structure have been dimensioned according to the TS 648 Turkish Design Code for Steel Structures and Turkish Earthquake Code. As a result, all members have been collected into the Table 10.

3.2 Model-2: Steel building with reinforced concrete wall-steel frames

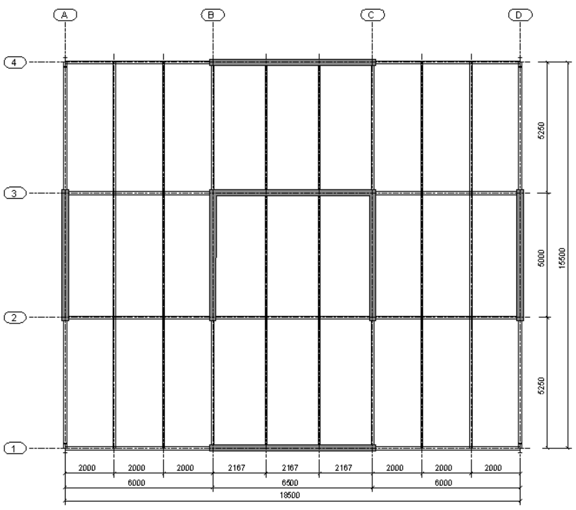

Fig. 6 Typical Floor Plan

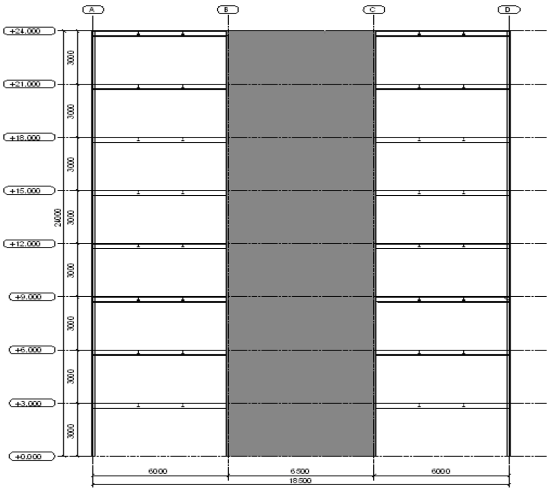

Fig. 7 RC Shear Walls (Axis 1, 3 and 4) & (A, B, C and D)

Table 11 Dead, snow and live loads

Floor name	dead load (kN/m^2)	snow load (kN/m^2)	live load (kN/m^2)
roof (24,00 m elevation)	3,757	0,736	1,962
standard floor (+3,00 m,…+21,00 m elevation)	6,190	-	1,962

Table 12 Total mass distribution

ΣW_{EQ} (kN)
15218,180

Table 13 Total earthquake loads in x and y directions

$\sum F_{ix}$(kN)	$\sum F_{iy}$(kN)
2229,460	2024,020

3.2.1 Description of the system

The system is identical to the Model-1, except for replacement of steel bracings with reinforced concrete shear walls at the same locations (Figs. 6-7).

3.2.2 Analysis of the loads

Dead loads, snow loads, live loads are calculated and summarized in the following Table 11.

Total mass distribution value during earthquake is given in Table 12.

Total earthquake loads in x and y directions acting on the structure are given in Table 13. As earthquake loads govern wind loads as far as the overall strength of the structure is concerned, structural analysis will be performed based on earthquake loads only.

3.2.3 Structural analysis of the system

Structural model

Load cases, the main ones are same as in Model-1. Load combinations are same as in Model-1.

As in Model-1, all analysis for Model-2 is performed by FEM software SAP2000, where all structural members are modelled into a structural system (Fig. 8).

Table 14 Maximum relative displacements

δ_{ix}/h_i	δ_{iy}/h_i
0,0053	0,0083

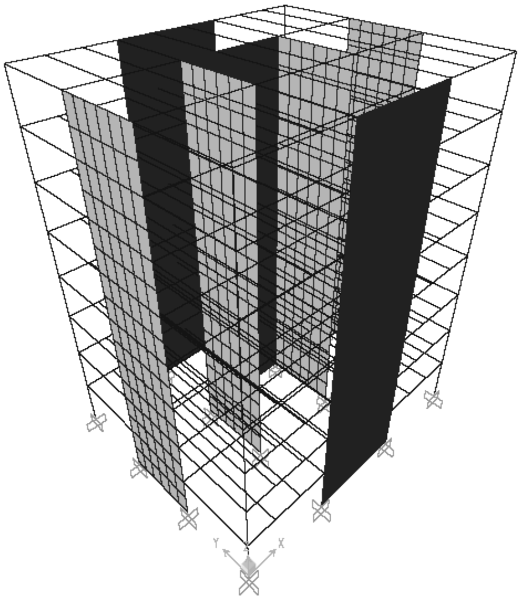

Fig. 8 Sap2000 3D static system model

Table 15 Steel members' sections

Member	Profile
Floor Beams (interior)	INP200
1,2,3,4 Axis Main Beams	HEB280
A, B, C, D Axis Main Beams	HEB280
2 Axis btw. B, C Axis Beams	HEB300
Columns	HEB300

Relative column displacements in x and y direction are evaluated and the maximum relative displacements are given in Table 14. It is also seen that these values are below the limit $(\delta_i/h_i)<0{,}02$ according to the Turkish Earthquake Code.

3.2.4 Dimensioning of Main Members

All members of the structure have been dimensioned according to the TS648 Turkish Design Code for Steel Structures and Turkish Earthquake Code. As result all members have been collected into Table 15.

3.2.5 Dimensioning of reinforced concrete shear walls

Reinforced concrete shear walls cast-in-situ are of quality of C25 with high ductility level. Design rules for earthquake resistance highly ductile shear walls were adopted from Turkish Earthquake Code.

Cross-section requirements for RC shear wall

-Structural walls are the vertical elements of the structural system where the ratio of length (l_w) to thickness (b_w) in plan is equal to at least seven.

b_w=20 cm, $l_w = 650\,\text{cm} \geq 7\ b_w = 7\times 20 = 140\,\text{cm}$

-Shear wall thickness shall not be less than 1/20 the highest storey height and 150 mm.

$b_w = 20\,\text{cm} \geq H_k/20 = 300/20 = 15\,\text{cm}$ and $b_w = 20\,\text{cm} \geq 15\,\text{cm}$

-Wall end zones shall be developed on both ends of walls where H_w/l_w>2.0. Wall end zones may be developed within the wall itself or within an adjoining wall or in an enlarged section at the edge of the wall.

-The critical wall height (H_{cr}) measured from the foundation level shall be determined as to satisfy the unfavorable one of the following conditions given in Eqs. (1)-(2) provided that it does not exceed $2l_w$. Here H_w, is the wall height measured from level that reduce more than 20% of length of the wall in plan or from the top of the ground

$$2l_w \geq H_{cr} \geq l_w \tag{1}$$

$$H_{cr} \geq H_w/6 \tag{2}$$

H_{cr} = 650 cm is chosen.

$2\times l_w = 1300\,\text{cm} \geq 650\,\text{cm} \geq l_w = 650\,\text{cm}$ and

$650 \geq 2400/6 = 400\text{cm}$

Reinforcement requirements

- Total cross section area of each of the vertical and horizontal web reinforcement on both faces of structural wall shall not be less than 0.0025 of the gross section area of the wall web remaining in between the wall end zones. The spacing of longitudinal and transverse reinforcement in wall web shall not be more than 250 mm.
- Excluding the wall end zones, reinforcement mesh on both faces of the wall web shall be tied each other by at least 4 special seismic crossties per unit square meter of the wall surface. However, excluding the wall end zones, at least 10 special seismic crossties per unit square meter of the wall surface shall be used along the critical wall height. Crosstie diameter shall be at least equal to that of the horizontal reinforcement.
- Horizontal web rebars of the shear walls shall be bent 90 degrees at the outer edge of the wall end zone and tied to the vertical corner reinforcement at the other face by a 135 degrees hook or in the case where horizontal web rebars are terminated at the wall end without 90 degrees bent, ⊃ shaped horizontal bars with the same diameter of web reinforcement shall be placed at both ends of the wall. Those bars shall be extended inside the wall web by at least the development length measured from the inner boundary of the wall end zone.
- The ratio of the total area of vertical reinforcement at each wall end zone to the cross wall cross section area shall not be less than 0.001. However, this ratio shall be increased to 0.002 along the critical wall height. Amount of vertical reinforcement at each wall end zone shall not be less than 4∅14.
- Vertical reinforcement at wall end zones shall be confined as similar to columns, by transverse reinforcement made of hoops and crossties, in accordance with the below given rules.
  • Diameter of transverse reinforcement to be used at wall end zones shall not be less than 8 mm. Horizontal distance between the legs of stirrups and / or crossties,

denoted as a, shall not be more than 25 times the diameter of hoops or crossties.

- At least 2/3 of the transverse reinforcement determined by Eq. (3) for the confinement zones of columns shall be provided at wall end zones along the critical wall height. Vertical spacing of hoops and / or crossties shall not be more than half the wall thickness and 100 mm, nor shall it be less than 50 mm. Such reinforcement shall be extended into the foundation by at least a height equal to twice the wall thickness.

$$A_{sh} \geq 0.30\, s\, b_k [(A_c / A_{ck}) - 1](f_{ck} / f_{ywk}) \qquad (3)$$
$$A_{sh} \geq 0.075\, s\, b_k (f_{ck} / f_{ywk})$$

Where,

s is spacing of transverse reinforcement

b_k is core diameter of the wall end zone (distance between the centers or outermost rebars)

A_{sh} is along the height corresponding to transverse reinforcement spacing s, sum of projections of cross section areas of all legs of hoops and crossties of columns or wall end zones in the direction perpendicular to b_k considered.

A_c is the gross section area of wall end zone

A_{ck} is the concrete core area within outer edges of confinement reinforcement

f_{ck} is the characteristic compressive cylinder strength of concrete

f_{ywk} is the characteristic yield strength of transverse reinforcement

- Vertical spacing of hoops and / or crossties at wall end zones outside the critical wall height shall not be more than the wall thickness and 200 mm. However transverse reinforcement diameter and spacing at wall end zones shall never be less than the web rebars.

Reinforcement calculations

At the shear wall in the ground section, internal forces for (G+Q+E) loading case are:

N=3601 kN, M=5252,05 kNm. According to TS500 Design Code for Reinforced Concrete Structures

$$n = \frac{N}{b \times h \times f_{cd}} \qquad (4)$$

$$m = \frac{M}{b \times h^2 \times f_{cd}} \qquad (5)$$

$$n = \frac{3601}{200 \times 6500 \times 17} = 0{,}16 \quad \text{and} \quad m = \frac{5252{,}05 \times 10^6}{200 \times 6500^2 \times 17} = 0{,}037$$

As it cannot be read any values from the chart, minimum reinforcement will be used.

Then, shear wall end zones and web reinforcement can be calculated as following:

$$A_{s,endzones} = 0{,}0025 \times 200 \times 6500 = 3250 mm^2 \Rightarrow 12\phi 20 chosen$$

$$A_{web} = A_s / 6 = 3250 / 6 = 542 mm^2 \Rightarrow \phi 10 / 200 mm$$

$$(393 mm^2 / m \times 3{,}9 m = 1533 mm^2)$$

$$A_{\min,end} = 0{,}002 \times 200 \times 6500 = 2600 mm^2$$

Shear wall reinforcements are detailed in Fig. 9.

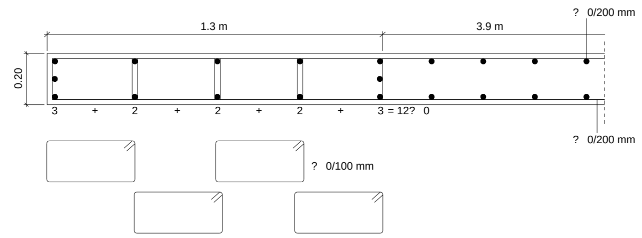

Fig. 9 Shear wall reinforcements

Result

Total concrete used for shear walls

: 199 m^3

Total reinforcement bar used for shear walls

: 147,0 kN

4. Comparison between two structural systems with respect to their construction cost

In order to evaluate the influence of the design parameters on the constructional costs, the material consumption has been calculated for all the examined cases. In particular, the steel consumption has been computed in terms of total weight, while concrete consumption in terms of total cast volume (Tenchini *et al*. 2014). Conclusions that are obtained from the comparison of two structural systems can be summarized as following:

The steel self-weight at the system with steel bracings being 1769 kN is now 762 kN for the system with reinforced concrete shear walls.

In fact, the self-weight at the system with reinforced concrete shear walls is increased by 3433,5 kN in overall. This is because of reinforced concrete shear walls having 519 kN as self-weight. This also means the production of reinforced concrete addition to steel construction. Subsequently, this comparison of their constructions cost will be performed as following:

Construction cost of Model-1: Steel building with braced frames

Steel self-weight=1769 kN=180 tons

Unit cost of steel construction*: 1600 €/ton

Total cost of steel construction : 180×1600=€288,000

Cost of foundation work:=€34,700

Total construction cost for Model-1:=**€322,700**

Construction cost of Model-2: Steel building with reinforced concrete wall-steel frames

Steel self-weight=762 kN=78 tons;

Unit cost of steel construction: 1600 €/ton

Total cost of steel construction: 78×1600=€124,800

Unit cost of shear wall concrete construction**: 270 €/m^3

Total cost of shear wall concrete construction: 199×€27=€53,730

Cost of foundation work:=€34,700
Total construction cost for Model-2:=**€213,230**

**Includes steel material, delivery, manufacturing and erection of steelwork;*
i.e., Engineering+Procurement+Construction
***Includes formwork material and workmanship, concrete material and workmanship, delivery and placement of concrete;*
i.e., Engineering+Procurement+Construction

5. Conclusions

Two different structural systems are structurally analyzed under the same factors as given below:

-Geometry
-Local earthquake conditions
-Load conditions
-Soil conditions
-Same material for their façades and floors
-Same place on floor for earthquake resisting systems

And they were compared with respect to their construction costs of their structural systems.

The Model-2's (steel building with RC wall-steel frames) construction cost of its structural system, is found to be by approximately almost 34 % more economical in comparison to the Model-1 (steel building with braced frames). Additionally, Model-2 provides more space in the interior of the building hence there are slenderer columns and no vertical bracings at all, which would be another cost advantage by renting of the building.

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