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Design Application Procedures of 15 Storied 3D Reinforced Concrete Shear Wall-Frame Structure

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***Abstract*—**This paper presents the design application and reinforcement detailing of 15 storied reinforced concrete shear wall-frame structure based on linear static analysis. Databases are generated for section sizes based on automated structural optimization method utilizing Active-set Algorithm in MATLAB platform. The design constraints of allowable section sizes, capacity criteria and seismic provisions for static loads, combination of gravity and lateral loads are checked and determined based on ASCE 7-10 documents and ACI 318-14 design provision. The result of this study illustrates the efficiency of proposed method, and is expected to provide a useful reference in designing of RC shear wall-frame structures.

***Keywords*—**Structural optimization, linear static analysis, ETABS, MATLAB, RC shear wall-frame structures.

I. Introduction

EARTHQUKE causes significant damages to the reinforced concrete high-rise building structures. Design engineers in consideration of lateral loadings should design a structure in such a way that the building should withstand all the gravity and lateral loads, and should fully fulfill all the requirements that a structure is designed for. As an earthquake resistant system, shear wall-frame structures are widely used rather than any other systems in seismic region in Afghanistan. They provide considerable stiffness and strength to the structure for strong earthquake ground motions. Properly-designed shear walls can provide safety and are significantly cost effective in RC structures during seismic activity. Therefore, special consideration of designing and detailing of shear wall capable of resisting earthquake motions without undesirable loss of strength and stiffness is required.

Extensive studies and research exist on using, designing, detailing, and requirements of RC shear wall structures. For structures assigned to seismic design categories D, E or F subjected to strong ground motions shall be provided by special moment frames, structural walls or a combination of shear wall-frame or dual system as seismic force resisting systems [1], [2]. A dual system in which the moment frames itself shall be capable of resisting at least 25% of the design seismic forces and the total seismic force resistance is provided by combination of moment frames and shear walls [3]. These types of structures, however, are considered to perform well during earthquakes; some mid-rise and high-rise RC shear wall buildings have suffered damages in the past earthquakes which had been designed well to modern building codes. Therefore, evaluation and recommendations for improving shear wall design requirements in seismic design of tall building have been carried out [4].

This study follows the seismic design of reinforced concrete building code requirements of the 2012 International Building Code (2012 IBC), American Concrete Institute Building Code (ACI 318-14), American Society of Civil Engineering (ASCE 7-10) standards. Typical reinforcement requirements for structural walls, beams and column, special confinement at the wall edges and columns are discussed and presented. In addition, the structure is designed for lateral loadings based on equivalent linear static analysis. The method involves maximum values of displacements and member forces for earthquake motions. The procedures of seismic loadings are done by help of ETABS. Member forces such as beams, columns and shear walls are calculated and checked separately by each program and the results are compared. To avoid over-stressed members, a stress constraint ratio is proposed and introduced for the most critical load combinations and structural members. The placement and details of reinforcements of structural members are explained and discussed as conclusion.

II. Basic Seismic-Force-Resisting System Definition

Building is designed in such a way that lateral load resistance is provided by shear walls and frames. The structure is 15 stories with typical height of 3 m. It has 3 spans with length of 9 m, 7.5 m, and 7 m in each horizontal direction, respectively. Section sizes of members are generated based on structural optimization method using Active-Set-Algorithm optimization method based on linear static analysis [5]. The compressive strength of concrete is 27 MPa and yield strength of reinforcement is 420 MPa for all member sizes, respectively. Lateral loads are calculated based on ASCE 7-10, where the site class of the structure is D, Response modification factor R=6, and S1 and Ss are 0.51 and 1.28, for Kabul region. The assumed dead load on each frame of the building is 7.5 kN/m, and assumed dead and live loads on each slab of the building are 5 kN. Fig. 1 illustrates target model of RC shear wall-frame building structure.

Section sizes of beam and column, and the thickness of shear walls are predetermined as five sizes for every three floors. Table I shows combination of section sizes of structural element. Further details could be found on [6].

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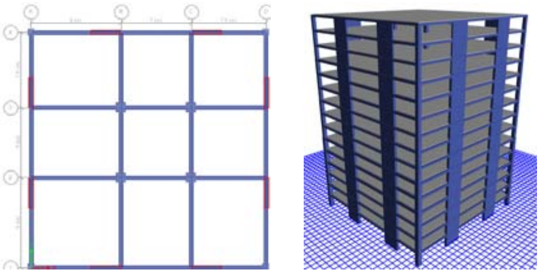


(a) Plan (b) 3D view

Fig. 1 Plan and 3D view of shear wall-frame model

TABLE I
COMBINATION OF SECTION SIZES OF STRUCTURAL ELEMENTS

Element type	Story Number	Section Sizes (bxh) cm	
	Story Number	Central	Edge
Column	1-3	100x100	87x87
	4-6	100x100	85x85
	7-9	100x100	79x79
	10-12	95x95	76x76
	13-15	59x59	47x47
		Inside	Outside
Beam	1-3	61x36	62x41
	4-6	57x38	65x43
	7-9	65x41	65x43
	10-12	62x41	65x43
	13-15	63x37	65x40
Shear wall	1-3	40	
	4-6	40	
	7-9	40	
	10-12	40	
	13-15	36	

III. EQUIVALENT LATERAL FORCE PROCEDURES

Generally, the equivalent lateral force is permitted for regular building structures up to about 20 stories [7]. Based on ASCE7-10, the structures suitable for ELF procedures include:

1. Regular or irregular buildings in seismic design class B and C.
2. All the frame buildings or regular buildings whose natural period is smaller than 3.5s, or only horizontal/vertical irregular buildings in seismic design class D, E and F.

The equivalent lateral force (ELF) procedure, as preliminary design of all structures and final design of most of the structures, provides a simple way to incorporate the effects of inelastic dynamic response into a linear static analysis [8]. In this paper, a 15 storied shear wall-frame building structure is selected as lateral-force-resisting system, and the preliminary design of the structure is done using ETABS software. For the simplicity, a combination of section sizes of beam, column and shear wall is selected for final design application; however, all section sizes are designed and checked for the most critical load combinations. Following steps can be considered for ELF procedures:

A. Determination of Design Base Shear

Design base shear is the total lateral or shear at the base of the building equal to the sum of the seismic design force at each level of the building. The seismic base shear in a given direction shall be calculated as:

$$V = C_s W \text{ (ASCE 12.8-1)} \tag{1}$$

where C_s is the seismic response coefficient for the building, and W is the effective seismic weight of the building consisting of the weight of all materials of construction incorporated into the building (based on specified mass).

The seismic response coefficient for the building is determined as:

$$C_s = \frac{S_{DS}}{R/I} \text{ (ASCE 12.8-2)} \tag{2}$$

where S_{DS} = the design spectral response acceleration parameter in the short period range, R = the response modification factor that accounts for the reduction in seismic loads caused by inelastic action and energy dissipation, and I = the earthquake importance factor for the building and its occupancy.

The value of C_s shall not exceed the following:

$$C_s = \frac{S_{D1}}{T(R/I)} \text{ for } T \le T_L \text{ (ASCE 12.8-3)} \tag{3}$$

$$C_s = \frac{S_{D1}T_L}{T^2(R/I)} \text{ for } T > T_L \text{ (ASCE 12.8-4)} \tag{4}$$

In addition, the value of C_s shall not be less than

$$C_s = 0.044 S_{DS} I \ge 0.01 \text{ (ASCE 12.8-5)} \tag{5}$$

For structures located where S_1 is equal to or greater than $0.6g$, C_s shall not be less than

$$C_{s,\min} = \frac{0.5S_1}{(R/I)} \text{ (ASCE 12.8-6)} \tag{6}$$

S_{D1} = the design spectral response acceleration parameter, T = the fundamental period of the structure, T =12 seconds, is long-period transition period. S_1 = 0.51 g for Kabul city, is the mapped maximum considered earthquake spectral response acceleration parameter.

The design earthquake spectral response acceleration parameter at short period, S_{DS}, and at 1 second, S_{D1}, shall be determined with:

$$S_{DS} = \frac{2}{3} S_{Ms} \text{ (ASCE 11.4-3)} \tag{7}$$

$$S_{D1} = \frac{2}{3} S_{M1} \text{ (ASCE 11.4-4)} \quad (8)$$

Adjusted maximum considered earthquake response acceleration, S_{MS} , and the design spectral response acceleration parameter in the short period range, S_{M1} shall be calculated as:

$$S_{MS} = F_a S_s \text{ (ASCE 11.4-1)} \quad (9)$$

$$S_{M1} = F_V S_1 \text{ (ASCE 11.4-2)} \quad (10)$$

The approximate fundamental period value of T_a , in second, shall be obtained from

$$T_a = C_t h_n^x \text{ (ASCE 12.8-7)} \quad (11)$$

The approximate fundamental period in second for masonry or concrete shear wall structures is also permitted to be determined as follows

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n \quad \text{(ASCE 12.8-9)} \quad (12)$$

C_w is calculated as follows

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} \text{ (ASCE 12.8-10)} \quad (13)$$

where A_B = area of base of structure, ft^2 or m^2 , A_i = web area of shear wall i , m^2 or ft^2 , D_i = length of shear wall, m or ft, h_i = height of shear wall i, m or ft and x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration, h_n is the structural height, and the coefficients C_t and x can be determined from Table II.

TABLE II
VALUES OF APPROXIMATE PERIOD PARAMETERS PER ASCE 14-10

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028	0.8
Concrete moment-resisting frames	0.016	0.9
Steel eccentrically braced frames	0.03	0.75
Steel buckling-restrained braced frames	0.03	0.75
All other structural systems	0.02	0.75

Table III lists the applicable seismic design factors based on ASCE 14-10.

TABLE III
SEISMIC PARAMETERS FOR STRUCTURAL MODEL

Occupancy category		III
Site class		D
Seismic design category		D
Short period spectral response	S_S	1.28
Spectral response(1 sec)	S_1	0.51
Design short period spectral response	S_{DS}	0.8533
Design spectral response (1 sec)	S_{D1}	0.51
Importance factor	I	1
Response modification factor	C_S	0.17
Seismic response coefficient	R	6
Approximate fundamental period	T_a	0.847
Long period transition period	T_L	10
Site coefficient	F_a	1
Site coefficient	F_v	1.5

B. Vertical Distribution of Seismic Forces

According to the ASCE7-10, the lateral force at each level of the building is equal to the vertical distribution factor for each level multiplied be the seismic base shear as following:

$$F_x = C_{VX} V \text{ (ASCE 12.8-11)} \quad (14)$$

and the vertical distribution factor, C_{VX} , which is equal to the percentage of base shear that is assigned to each floor level shall be determined as:

$$C_{VX} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \text{ (ASCE 12.8-12)} \quad (15)$$

where C_{VX} = vertical distribution factor, V = total design lateral force or shear at the base of the structure, KN, w_i and w_x = the portion of the total effective seismic weight of the structure (W) located or assigned to level i or x, h_i and h_x = the height from the base to level i or x, in m. k = the exponent related to the structure period can be determined as: For structures having a period of 0.5 s or less, $k = 1$; for structures having a period of 2.5 s or more, $k = 2$; for structures having a period between 0.5 and 2.5 s, k shall be 2 or linear interpolation shall be done between 1 and 2 [3].

C. Horizontal Distribution of Forces

The seismic lateral forces produce seismic design story shear at any story, and the seismic design story shear in any story can be determined as:

$$V_x = \sum_{i=x}^{n} F_i \text{ (ASCE 12.8-13)} \quad (16)$$

where F_i = the portion of the seismic base shear, KN, shall be distributed to the different vertical elements of the lateral load-resisting-system based on the relative lateral stiffness of the vertical resisting elements and diaphragm in the story under consideration. Table IV summarizes lateral loads calculated based linear static analysis of ETABS.

TABLE IV
SEISMIC BASE SHEAR PER ASCE 7-10

Story number	h(m)	W,KN	$w_i h_i^k$ KN.m	C_{VX}	Lateral force	Story shear
15	45	9339.9	420295.7	0.117	1470.4	1470.4
14	42	9715.2	408039.9	0.114	1394.2	2864.6
13	39	9715.2	378894.2	0.106	1262.1	4126.7
12	36	9970.5	358937.7	0.100	1163.2	5289.9
11	33	10143.5	334723.9	0.094	1052.8	6342.8
10	30	10143.5	304294.4	0.085	926.4	7269.2
9	27	10186.5	275035.7	0.077	807.6	8076.8
8	24	10206.8	244963.5	0.068	690.8	8767.6
7	21	10206.8	214343.1	0.060	577.4	9345.0
6	18	10127.5	182295.9	0.051	465.7	9810.8
5	15	10156.0	152340	0.043	365.6	10176.5
4	12	10156.0	121872	0.034	270.9	10447.4
3	9	10121.9	91096.74	0.025	183.5	10630.9
2	6	10129.1	60774.72	0.017	106.5	10737.5
1	3	10129.1	30387.36	0.008	42	10779.5
Total	45	150447	3578294.8	1.000	10779.5	10779.5

D.Overturning Moment

According to ASCE section 12.8.5, the structure shall be designed to resist overturning moments produced by the lateral seismic forces, F_x, and should be calculated using:

$$M_x = \tau \sum_{i=1}^{n} F_i (h_i - h_x) \tag{17}$$

where M_x = overturning moment at level x; F_i = portion of seismic base shear, V, induced at level i, h_i, h_x = height from base to level i and x; τ = overturning moment reduction factor which, τ = 1.0 for top 10 stories, τ = 0.8 for twentieth story from the top and below, and τ = linear interpolation between 1.0 and 0.8 for stories between twentieth and tenth stories below top.

Reduction factor, τ, is permitted to be taken as 1.0 for the full height of the structure. The overturning moment shall satisfy:

$$SF = \frac{M_R}{M_x} > 1.75 \tag{18}$$

where

$$M_R = W * x_c \tag{19}$$

M_R = resisting moment, KN-m, W=weight of the building, KN, and x_c =center of mass of the building, m. Overturning moment calculated by ETABS is:

$$M_x = 348407 KN - m$$

$$M_R = 150447 * 11.83 = 1779788 KN - m$$

$$SF = \frac{1779788 KN - m}{348407 KN - m} = 5.1 > 1.75$$

E. Determination of Story Drift

The story drift, Δ, can be defined as the relative displacement between adjacent stories due to the design lateral forces, and can be computed as the difference between the deflection of the center of mass at the top and bottom of the story being considered. For structures assigned to seismic design category C, D, E or F having horizontal torsional irregularity or extreme torsional irregularity, the design story drift shall not exceed the allowable story drift given in Table V. The deflection used to determine the design story drift shall be computed as:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \text{(ASCE 12.8-15)} \tag{20}$$

where C_d = deflection amplification factor, from table 12.2-1 of ASCE, δ_{xe} = the deflection determined by elastic analysis.

TABLE V
ALLOWABLE STORY DRIFT PER ASCE 12.12-1

Structure	Occupancy category		
	I or II	III	IV
Structure, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drift.	0.025h	0.020h	0.010h
Masonry cantilever shear structures	0.010h	0.010h	0.010h
Other masonry shear wall structures	0.007h	0.007h	0.007h
All other structures	0.020h	0.015h	0.010h

Fig. 2 shows story drift of structural model by MATLAB and ETABS.

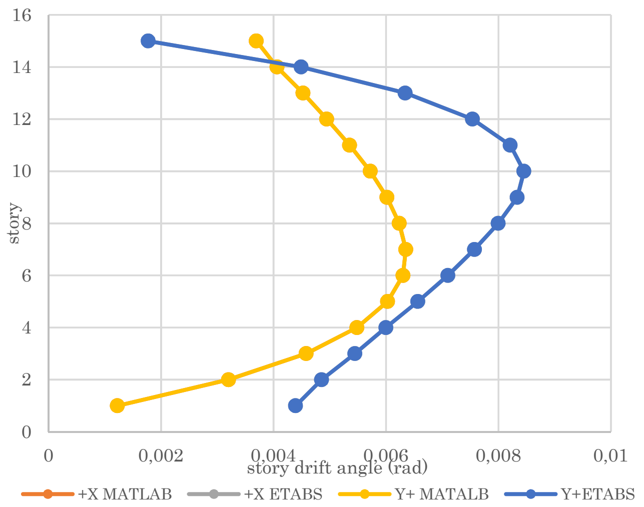


Fig. 2 Story drift

IV. COMPARISON OF STRESSES OF STRUCTURAL MEMBERS

In this section, comparison is shown for stresses in structural members through output files of ETABS and MATLAB programs. It is concerned that the feasible solution by MATLAB is judged not to be feasible by ETABS, caused by the differences of themes. In order to avoid over-stresses of elements by ETABS, a modified optimization method is proposed which reflects the differences between two analysis programs to the allowance limitation value in optimization procedures by MATLAB as:

$$g_{(X)} = \frac{R - R_{limit}}{R_{limit}} \leq \overline{r} \tag{21}$$

where, R, R_{limit} indicate response and response limit values, and $\overline{r}$ is the ratio of the response of MATLAB to ETABS for the most critical members and load combinations obtained from comparisons below.

The comparison includes dead, live and earthquake loads as well as all design load combinations. Figs. 5 and 6 show stresses in beams for dead, live and earthquake loads by both programs.

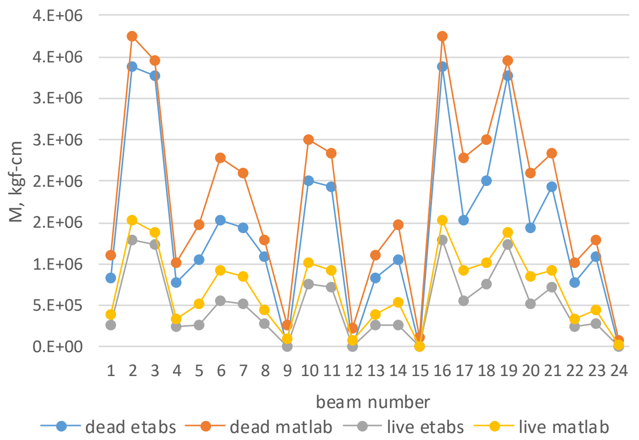


Fig. 3 M of beam (dead and live loads)

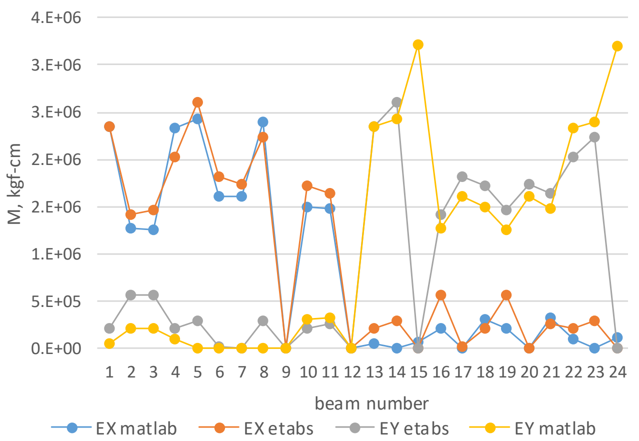


Fig. 4 M of beam (dead and live loads)

Figs. 7-12 show stresses in column and shear wall for dead, live and earthquake loads.

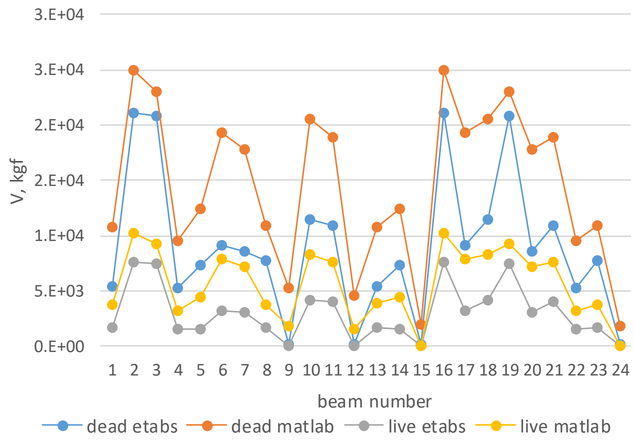


Fig. 5 V of beam (dead and live loads)

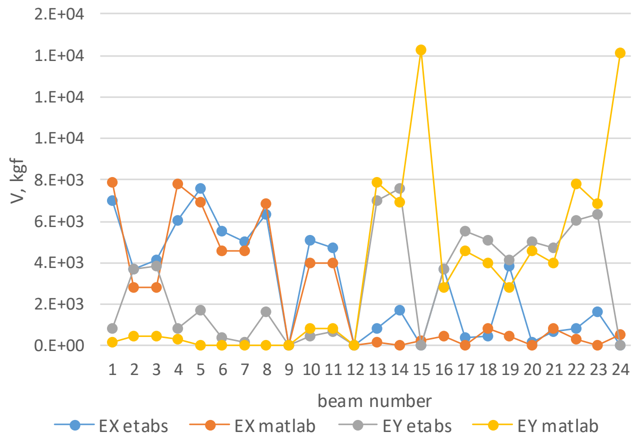


Fig. 6 V of beam (EQ loads)

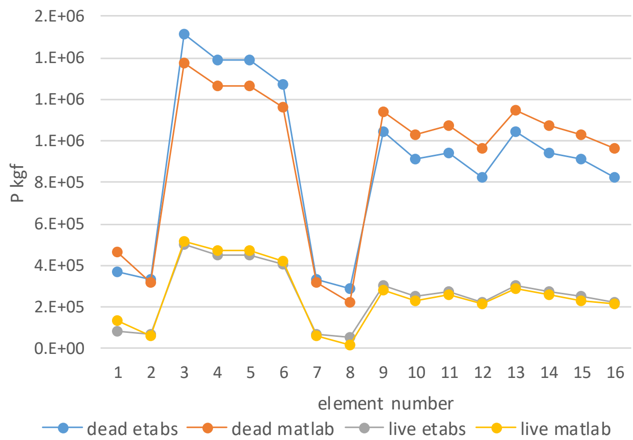


Fig. 7 P of column and wall (dead and live load)

V. Structural Design of the Building

Structural members must be designed to support all the loads acting on the structure greater than the service or actual loads in order to provide sufficient safety agonists failure. Loads can be forces for which a given structure might be proportioned such as dead, live or lateral loads. Based on ACI design code, the member is designed to resist factored loads multiplied by load factors. In addition, a strength reduction factor is considered to account the degree of accuracy and variation of materials in strength design method and can be simply explained as: Strength provided ≥ strength required to carry factored load.

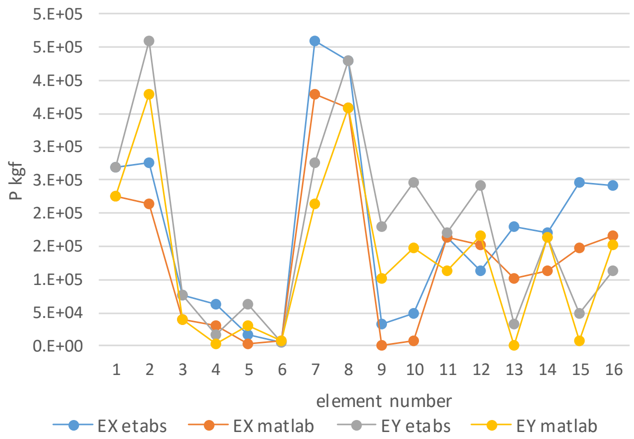


Fig. 8 P of column and wall (EQ loads)

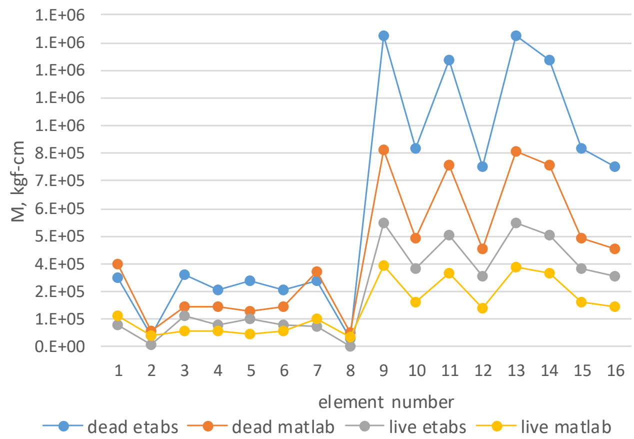


Fig. 9 M of column and wall (dead and live load)

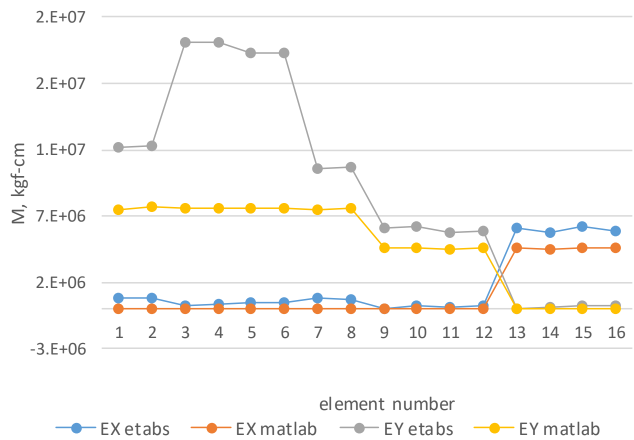


Fig. 10 M of column and wall (EQ load)

The strength required is obtained from structural analysis and applied safety factor, where the strength provided is illustrated in design code. Following structural members are selected for design, check and reinforcement detailing:

1. Beam number 17, b= 43 cm, h= 65 cm and L= 700 cm
2. Column number 4, b= 95 cm, D= 95 cm
3. Shear wall number 2, t= 40 cm

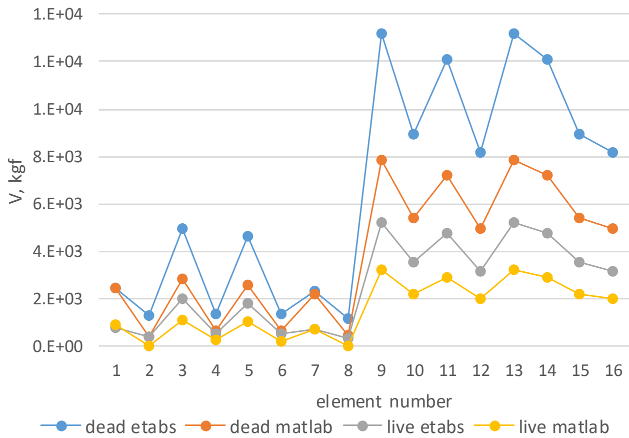


Fig. 11 V of column and wall (dead and live load)

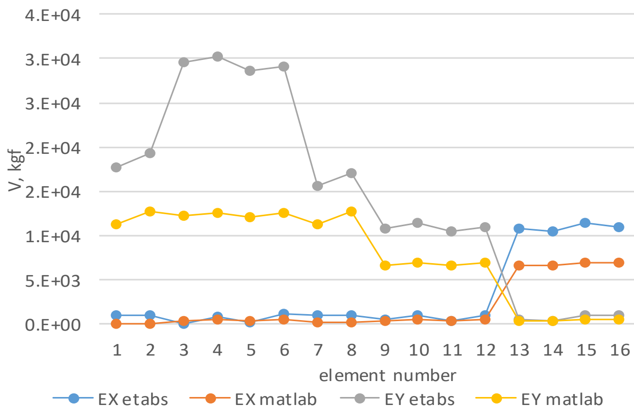


Fig. 12 V of column and wall (EQ load)

A. Beam Design

For the simplicity, the calculation is done by help of ETABS and only the design and reinforcement detailing of a combination of structural elements such as beam, column and shear wall are presented as:

- Beam width b= 42 cm
- Depth to tension reinforcement d= 61 cm
- Total beam depth h= 65 cm
- Clear cover d' =4 cm
- Concrete compressive strength $f'_c = 27$ MPa
- Reinforcing yield strength $f_y = 414$ MPa

1) Determine the Required Flexural Reinforcing of Beam

A simplified rectangular stress block approach based on flexural design procedure is considered, where the maximum depth of compression zone, $c_{\max}$, shall be calculated based on limitation of tensile steel tension not less than $\varepsilon_{s,\min}$ for tension controlled, where $\varepsilon_{s,\min} = 0.005$, and $\varepsilon_{c,\max} = 0.003$ [9].

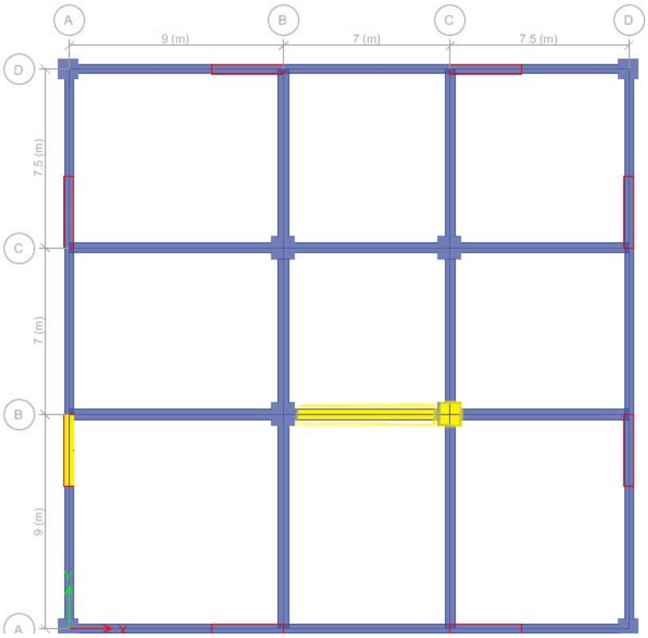


Fig. 13 Location of beam, column and shear wall

The design steps to determine the required flexure and shear reinforcements are presented based on linear static analysis. The design process begins by identifying the required positive and negative moment capacities at mid-span and each support through determining of member loads, and then required reinforcement area is obtained using static approach discussed in Chapter III. Finally, the required reinforcement and its arrangement are checked to ensure that the minimum and maximum allowed are met.

Section 1 (L=0.5m), End-I:

Determine the required positive and negative flexural capacities at supports:

$$M_{u,top} = \text{-647.12KN-m}$$
$$M_{u,bottom} = \text{323.56KN-m}$$

Determine the depth of the compression block:

$$a_1 = d - \sqrt{d^2 - \frac{2M_u}{0.85.f'_c\,\phi.b}} \tag{22}$$

$$a_1 = 61 - \sqrt{61^2 - \frac{2*64712}{0.85*2.7*0.9*41.4}} = 14.02cm$$

$$a_{11} = d - \sqrt{d^2 - \frac{2M_u}{0.85.f'_c\,\phi.b}} \tag{23}$$

$$a_{11} = 61 - \sqrt{61^2 - \frac{2*32356}{0.85*2.7*0.9*41.4}} = 5.86cm$$

The maximum depth of the compression zone can be calculated as:

$$c_{\max} = \frac{\varepsilon_{c,\max}}{\varepsilon_{c,\max} + \varepsilon_{s,\min}} d \tag{24}$$

$$c_{\max} = \frac{0.003}{0.003 + 0.005} 61 = 22.87cm$$

The maximum allowable depth of the rectangular compression block can be given by:

$$a_{\max} = \beta_1.c_{\max} \tag{25}$$
$$a_{\max} = 0.85*22.87 = 19.44cm$$

If $a \le a_{\max}$, the area of flexural steel reinforcement is given by formulas below, and the reinforcement is to be placed at the bottom for $+M_u$ or the top for $-M_u$.

$$A_{s,top} = \frac{M_u}{\phi.f_y(d - \frac{a}{2})} \tag{26}$$

$$A_{s,top} = \frac{64712}{0.9*41.4(61 - \frac{14.02}{2})} = 32.16cm^2$$

$$A_{s,provided} = \frac{4}{3} A_{s,required} \tag{27}$$

$$A_{s,provided} = \frac{4}{3} 32.16cm^2 = 42.89cm^2$$

Applied reinforcing: 6ø2+5ø22

$$A_{s,bottom} = \frac{M_u}{\phi.f_y(d - \frac{a}{2})}$$

$$A_{s,bottom} = \frac{32356}{0.85*41.4(61 - \frac{5.86}{2})} = 15.83cm^2$$

$$A_{s,provided} = \frac{4}{3} 15.83cm^2 = 21.11cm^2$$

Applied reinforcing: 5ø22

Calculate minimum and maximum area of flexural reinforcement:

$$A_{s,\min 1} = \frac{0.25\sqrt{f_c}}{f_y} bd \tag{28}$$

$$A_{s,\min} = \frac{0.25\sqrt{27}}{414} 42*61 = 8.03cm^2$$

$$A_{s,\min} = \frac{1.4}{f_y} bd \tag{29}$$

$$A_{s,\min 2} = \frac{1.4}{414} 42*61 = 8.66cm^2$$

Section 2 (L=4.7):

$$M_{u,top} = \text{-161.78KN-m}$$
$$M_{bottom} = \text{263.24KN-m}$$

The maximum depth of the compression zone

$$a_1 = d - \sqrt{d^2 - \frac{2M_u}{0.85.f'_c\,\phi.b}} \tag{30}$$

$$a_1 = 61 - \sqrt{61^2 - \frac{2*16178}{0.85*2.7*0.9*41.4}} = 3.18cm$$

$$a_{11} = d - \sqrt{d^2 - \frac{2M_u}{0.85.f'_c\,\phi.b}} \tag{31}$$

$$a_{11} = 61 - \sqrt{61^2 - \frac{2*26324}{0.85*2.7*0.9*41.4}} = 5.27cm$$

Required flexural reinforcement:

$$A_{s,top} = \frac{M_u}{\phi.f_y(d - \frac{a}{2})} \tag{32}$$

$$A_{s,top} = \frac{12431}{0.9*41.4(61 - \frac{3.18}{2})} = 5.61cm^2$$

$$A_{s,provided} = \frac{4}{3}5.61cm^2 = 7.48cm^2$$

Applied reinforcing: 2ø22

$$A_{s,bottom} = \frac{M_u}{\phi.f_y(d - \frac{a}{2})} \tag{33}$$

$$A_{s,bottom} = \frac{26324}{0.9*41.4(61 - \frac{5.27}{2})} = 12.1cm^2$$

$$A_{s,provided} = \frac{4}{3}12.1cm^2 = 16.13cm^2$$

Applied reinforcing: 4ø22
Section 3 (L=6.5m):

$$M_{u,top} = \text{-649.64KN-m}$$
$$M_{bottom} = \text{324.82KN-m}$$

The maximum depth of the compression zone

$$a_1 = d - \sqrt{d^2 - \frac{2M_u}{0.85.f'_c\,\phi.b}} \tag{34}$$

$$a_1 = 61 - \sqrt{61^2 - \frac{2*64964}{0.85*2.7*0.9*41.4}} = 14.07cm$$

$$a_{11} = d - \sqrt{d^2 - \frac{2M_u}{0.85.f'_c\,\phi.b}} \tag{35}$$

$$a_{11} = 61 - \sqrt{61^2 - \frac{2*32482}{0.85*2.7*0.9*41.4}} = 6.58cm$$

Required flexural reinforcement:

$$A_{s,top} = \frac{M_u}{\phi.f_y(d - \frac{a}{2})} \tag{36}$$

$$A_{s,top} = \frac{64964}{0.9*41.4(61 - \frac{14.07}{2})} = 32.3cm^2$$

$$A_{s,provided} = \frac{4}{3}32.3cm^2 = 43.06cm^2$$

Applied reinforcing: 6ø22+5ø22

$$A_{s,bottom} = \frac{M_u}{\phi.f_y(d - \frac{a}{2})} \tag{37}$$

$$A_{s,bottom} = \frac{32482}{0.9*41.4(61 - \frac{6.58}{2})} = 15.1cm^2$$

$$A_{s,provided} = \frac{4}{3}15.1cm^2 = 20.13cm^2$$

Applied reinforcing: 5ø22

The minimum flexural reinforcement required in a beam section is the minimum of the following:

$$A_{s,\min 1} = \frac{0.25\sqrt{f_c}}{f_y}bd \tag{38}$$

$$A_{s,\min} = \frac{0.25\sqrt{27}}{414}42*61 = 8.03cm^2$$

$$A_{s,\min 2} = \frac{1.4}{f_y}bd \tag{39}$$

$$A_{s,\min 2} = \frac{1.4}{414}42*61 = 8.66cm^2$$

The beam flexural reinforcement is limited to a maximum given by:

$$A_s \le 0.025bd \tag{40}$$

$$A_s \le 0.025*42*61 = 64.05cm^2$$

The balanced reinforcement ratio is given by:

$$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\left(\frac{600}{600+f_y}\right) \tag{41}$$

$$\rho_b = \frac{0.85*0.85*27}{414}\left(\frac{600}{600+414}\right) = 0.027$$

The maximum allowable ratio of reinforcement is calculated as:

$$\rho_{\max} = \frac{0.85\, f_c'}{f_y}\left(\frac{3}{7}\beta_1\right) \tag{42}$$

$$\rho_{\max} = \frac{0.85*27}{414}\left(\frac{3}{7}0.85\right) = 0.020$$

The maximum allowable area of reinforcement is:

$$A_{s,\max} = \rho_{\max} bd \tag{43}$$

$$A_{s,\max} = 0.02*42*61 = 51.7cm^2$$

The upper limit of 0.04 times the grass area of the tension reinforcement:

$$A_s \le 0.04bd \tag{44}$$

$$A_s \le 0.04*42*61 = 102.48cm^2$$

$$\rho_{mim} \le \rho \le \rho_{\max} \tag{45}$$

2) Determine the Required Shear Reinforcing of Beam

The required force to be carried by shear reinforcing is given by:

$$V_s = \frac{V_u}{\phi} - V_c \tag{46}$$

$V_u = 392.4KN$ by computer analysis

$$V_c = 0.17\lambda\sqrt{f_c}bd \tag{47}$$

$$V_c = 0.17\sqrt{2.7}*42*61 = 435.54KN$$

$$V_s = \frac{A_v f_y d}{s};\ \frac{A_v}{s} = \frac{V_s}{f_y d} = \frac{V_u/\phi}{f_y d} \tag{48}$$

$$\frac{A_v}{s} = \frac{392.4}{0.414*0.61*0.65} = 2390\frac{mm^2}{m}$$

Applied shear reinforcement: 104ø10@67mm c-c

The spacing of shear reinforcement shall be:

$$s \le \left\{\frac{d}{4}, 8d_b, 24d_{bs}, 300mm\right\} \tag{49}$$

$$s \le \left\{\frac{61}{4}, 8*22, 24*10, 300mm\right\} <=> \{152, 176, 240, 300mm\}$$

$$s <=> 67mm < 152mm$$

Table VI shows flexural and shear reinforcements of beam under consideration:

TABLE VI
FLEXURAL AND SHEAR REINFORCEMENT OF BEAM

Beam length (cm)	Beam depth h (cm)	Beam width b (cm)	Flexural reinforcement			Shear reinforcement
			top	middle	bottom	
700	65	42	11ø22	5 ø22	5ø22	104ø10@6.7cm

B. Column Design

1) Determine the Required Longitudinal Reinforcing of Column

The required amount of reinforcement was obtained by the help of etabs based on design internal forces as follows:

$$P_u = 2856.2KN$$
$$M_{u2} = 492.12KN - m$$
$$M_{u3} = -125.68KN - m$$
$$D/C\ Ratio = 0.281$$

b=D=95cm

The minimum and maximum longitudinal reinforcement is limited:

$$A_{s,\min} = 0.01A_g \tag{50}$$

$$A_{s,\min} = 0.01*950*950 = 9025mm^2$$

$$A_{s,\max} = 0.06A_g \tag{51}$$

$$A_{s,\max} = 0.06*902500 = 54150mm^2$$

$$A_{s,provided} = 9025mm^2 = 0.01A_g;\ A_{s,provided} = 9025mm^2$$

Applied longitudinal reinforcement: 24ø22@150mm c-c

Provided longitudinal reinforcement falls between minimum and maximum allowable limit. Fig. 14 shows interaction diagrams of concrete member subjected to combined flexure and axial loads. The diagram shows the relationship between axial load and bending moments at failure.

The interaction surface points which represent the internal forces in each combination are inside the volume by critical curvature.

2) Determine the Required Shear Reinforcing of Column

The shear reinforcement is designed for each design combination in major and minor directions of the column. The nominal shear force shall not exceed the shear strength.

$$V_u \le \phi V_n,\ \text{where}\ V_n = V_c + V_s \tag{52}$$

The design shear force is obtained based on computer analysis by etabs as following:

Major shear V_{u2}:

$$V_u = 446.57KN$$

$$P_u = 2856.27KN$$
$$M_u = -25.25KN - m$$

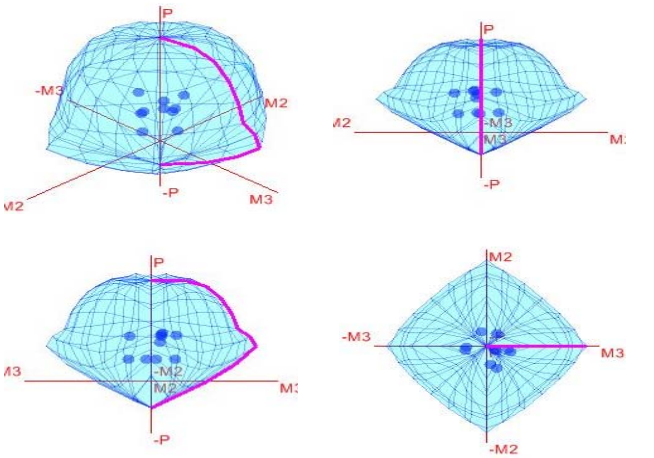


Fig. 14 Interaction surface for column

For special moment resisting frame design, if the factored axial compressive force, P_u, including earthquake effects is small of $(P_u \le f_c' A_g / 20)$; if the shear force contribution from earthquake, V_E, is greater than half of the total factored maximum shear force, $V_E \ge 0.5V_u$, then the concrete capacity is taken zero, $V_c = 0$.

$$(P_u \le 2.7 * 9025 / 20) = 1218.4KN < P_u$$

and $V_E = 399.5 > 0.5 * 446.57 = 223.3KN$, then $V_c = 0$

$$\frac{A_v}{s} = \frac{V_u - \phi V_c}{\phi f_{ys} d} \quad (53)$$
$$\frac{A_v}{s} = \frac{V_u - 0}{\phi f_{ys} d}$$
$$\frac{A_v}{s} = \frac{(446.57 - 0) * 10^4}{0.65 * 41.4 * 91} = 1823 \frac{mm^2}{m}$$

Provided shear reinforcement: 17ø10@170mm c-c

Minor shear V_3:

$$V_u = 498.3KN$$
$$P_u = 2856.27KN$$
$$M_u = 489.16KN - m$$

Shear force carried by concrete:

$$(P_u \le f_c' A_g / 20) \quad (54)$$
$$(P_u \le 2.7 * 2856.27 / 20) = 3855.9KN < P_u$$

and $V_E \ge 0.5V_u$; $V_E = 374.7 > 0.5 * 498.3 = 249.15KN$, then

$$V_c = 0$$
$$\frac{A_v}{s} = \frac{V_u - \phi V_c}{\phi f_{ys} d} \quad (55)$$
$$\frac{A_v}{s} = \frac{V_u - 0}{\phi f_{ys} d}; \ \frac{A_v}{s} = \frac{(498.3 - 0) * 10^4}{0.65 * 41.4 * 91} = 2035 \frac{mm^2}{m}$$

Provided shear reinforcement: 20ø10@150mm c-c

Table VII shows reinforcement provided in column under consideration.

TABLE VII
FLEXURAL AND SHEAR/TRANSVERS REINFORCEMENT OF COLUMN

Station Location	Required Rebar Area (mm^2)	Required Reinf Ratio	Current Reinf Ratio	Wall A_g mm^2
Top	5686	0.0047	0.0026	1200000
Bottom	3000	0.0025	0.0026	1200000

C. Shear Wall Design

1) Determine the Required Longitudinal Reinforcing of Shear Wall

Design and check of shear wall is done based on stresses on shear wall by help of etabs, then the required reinforcement is checked manually. Table VIII shows the design load combination associated with the specified required reinforcing area.

TABLE VIII
FLEXURAL DESIGN FOR PU, MU2 AND MU3

Station Location	Flexural Combo	P_u kN	M_{u2} kN-m	M_{u3} kN-m	Length mm	Thickness mm
Top	DWal4	4342.6	643.6	4195.3	3000	400
Bottom	DWal10	4186.16	-524.5	-2526.3	3000	400

Table IX shows the amount of reinforcement corresponding to the above load combination.

TABLE IX
FLEXURAL REINFORCEMENT RATIO OF SHEAR WALL

Column depth D(cm)	Column width b(cm)	Flexural reinforcement	Shear/transvers reinforcement
95	95	24 ø22	20 10@15cm

Based on ACI design code, the minimum longitudinal area of reinforcement shall be $A_{s,\min} = 0.0025A_g$

$$A_{s,\min} = 0.0025 * 400 * 3000 = 3000mm^2$$

Based on computer analysis, the required longitudinal reinforcement ratio is less than current reinforcement ratio, so we consider the amount of reinforcement ratio required by analysis:

$$A_{s,\min} = 0.0025 * 400 * 3000 = 3000mm^2$$
$$A_{s,required} = 5686mm^2 > A_{s,\min} = 3000mm^2$$

Applied reinforcement: 12ø 22 in two layers

2) Boundary Element Check

There are two approaches to check the requirements for boundary element, a) if the maximum extreme fiber compressive strength, $\sigma \geq 0.2 f'c$, or b) $c \geq \dfrac{l_w}{600(1.5\delta_u / h_w)}$, where $\delta_u / h_w \geq 0.005$, then boundary element is required.

$$\delta_u = \delta_{u,elastic\ \text{analysis}} (\frac{C_d}{I}) \tag{56}$$

$$\delta_u = 19(\frac{5}{1}) = 95mm$$

$$c \geq \frac{l_w}{600(1.5\delta_u / h_w)} \tag{57}$$

$c = 852mm$ by etabs $\dfrac{3000}{600(1.5*95/3000)} = 105mm < c$; the boundary element is required.

The boundary element shall extent vertically above and below the critical section at least:

$$\begin{Bmatrix} \dfrac{M_u}{4V_u} \\ l_w = 3000mm \end{Bmatrix} \tag{58}$$

$$\frac{M_u}{4V_u} = \frac{3018.6KN-m}{4*802.56KN} = 940mm,\ h_b = 1300mm$$

The minimum required length of boundary zone at each end of the wall shall be:

$$l_b = \max\{c/2,\ c - 0.1 l_w\} \tag{59}$$

$$l_b = \max\{852/2,\ 852 - 0.1*3000\}\,mm = \{426,\ 552\}\,mm$$

$$l_b = 552mm$$

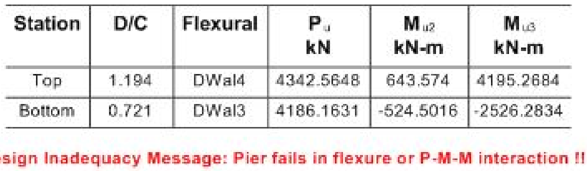
Programs introduced boundary length of 550 mm so we consider maximum length of 550 mm in both legs of shear wall accordingly.

TABLE X
BOUNDARY ELEMENT CHECK

Station Location	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa	Stress Limit MPa	C Depth mm	C Limit mm
Top–Left	488.2	DWal3	4076.2	-3018.6	8.4	5.4	788.2	112.2
Top–Right	538.7	DWal3	4541.6	588.2	4.8	5.4	838.7	666.7
Bottom–Left	552.5	DWal6	4651.8	-271.3	4.3	5.4	852.5	666.7
Bottom–Right	528.8	DWal6	4452.9	1787.5	6.7	5.4	828.8	112.2

After checking of applied flexural reinforcement, the amount of provided flexural reinforcement was not sufficient to resist corresponding load combinations. Therefore, the shear wall was designed using section designer tool, and the applied reinforcement for wall and boundary zones are as following:

- Total wall length: 3000 mm
- Clear cover: 25 mm
- Reinforcement of wall: 8ø18@221 mm c-c each layer
- Boundary length: 550 mm
- Boundary reinforcement: 4ø20@168 c-c each layer



Station	D/C	Flexural	P_u kN	M_{u2} kN-m	M_{u3} kN-m
Top	1.194	DWal4	4342.5648	643.574	4195.2684
Bottom	0.721	DWal3	4186.1631	-524.5016	-2526.2834

Design Inadequacy Message: Pier fails in flexure or P-M-M interaction !!

Fig. 15 Flexural design for P, M_3 and M_2

Fig. 16 shows reinforcement detailing of shear wall by section designer.

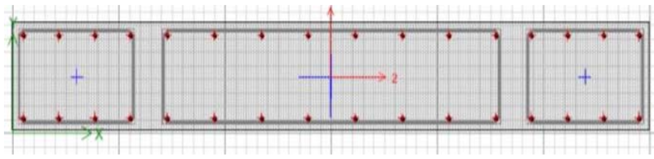
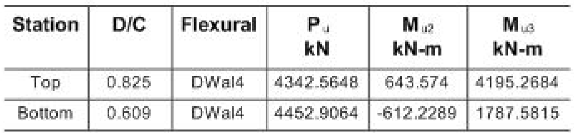


Fig. 16 Shear wall-detailing of reinforcement arrangement

Fig. 17 shows design to capacity ratio of shear wall. The amount of flexural reinforcement seems to be reasonable and sufficient.



Station	D/C	Flexural	P_u kN	M_{u2} kN-m	M_{u3} kN-m
Top	0.825	DWal4	4342.5648	643.574	4195.2684
Bottom	0.609	DWal4	4452.9064	-612.2289	1787.5815

Fig. 17 Flexural design for P, M_3 and M_2

3) Determine the Required Shear Reinforcing of Shear Wall

The design load combination associated with the specified shear reinforcing based on computer analysis is shown in Table XI.

TABLE XI
SHEAR DESIGN OF SHEAR WALL

Station Location	ID	Shear Combo	P_u kN	M_u kN-m	V_u kN	ΦV_c kN	ΦV_n kN
Top	Leg 1	DWal4	4342.6	4195.3	802.6	918.4	1663
Bottom	Leg 1	DWal4	4452.7	1787.6	802.6	932	1676.6

The required shear reinforcement can be obtained as:

$$\frac{A_v}{s} = \frac{(\phi V_n - \phi V_c)}{\phi f_y d} \tag{60}$$

$$d = 0.8 l_w \tag{61}$$

$$\frac{A_v}{s} = \frac{1676.6 - 931.9}{0.75*41.4*0.8*300} = 1676 \frac{mm^2}{m}$$

$$A_v = 1676 / 4*79 = 5.3$$

Total height of wall: $5.3*3=16$
Provided shear reinforcement: 16ø10@187mm c-c

Table XII shows provided reinforcement of shear wall under consideration.

TABLE XII
FLEXURAL AND SHEAR/TRANSVERS REINFORCEMENT OF SHEAR WALL

Total shear wall length (cm)	Flexural reinforcement	Transvers/shear reinforcement	Boundary length	Flexural reinforcement	Shear/transvers reinforcement
300	8ø18@22cm	16ø10@18cm	55	4ø20@16.8cm	30ø10@10cm

VI. CONCLUSION

In this study, the seismic design of structure was done following International Building Code (IBC 2012), American Society of Civil Engineering (ASCE 7-10) standards, and American Concrete Institute Building Code (ACI 318-14). Typical reinforcement requirements for structural wall, beam and column were discussed and presented using ETABS structural analysis software. The placement and detailing of reinforcement of structural members were explained and discussed as conclusion.

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