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DESIGN OF A STEEL SPECIAL MOMENT FRAME SUSCEPTIBLE TO HIGH SEISMIC RISK

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ABSTRACT

This paper deals with the design of a 2-D steel special moment frame vulnerable to high seismic risk using the ASCE 7-10 Code equivalent lateral force method (ELF). The equivalent lateral force method uses an approximate procedure to find the natural period of the system to get the total base shear before the design. In this paper, the actual natural period is computed after designing a 2-D steel special moment frame and the base shear is recalculated accordingly. For this purpose, the base shear from ELF is first used to design the 2-D frame according to the drift limitations as per ASCE 7-10 Code. The direct analysis method (DAM) of AISC 360 is then used to check the strength of the steel members. The new natural period of the 2-D frame is calculated using SAP2000 finite element program. The SAP2000 natural period is used to find the new base shear. It has been found that the SAP2000 natural period increases the total base shear by 62%. Consequently, a redesign for the steel special moment frame members should be considered to account for the difference in the base shear.

Keywords: steel special moment frame, ELF, base shear, DAM, natural period, SAP2000.

1. INTRODUCTION

Seismic design requirements are given in chapter 12 of the ASCE7-10 Code [1]. For computing the base shear of a building using the equivalent lateral force method, the code gives an alternative way to compute the natural period of the system before the design. However, the validity of this procedure needs to be checked for structures prone to high seismic risks. Therefore, in this paper the actual natural period is found to get the total base shear for a 2-D steel special moment frame (SMF) in high seismic zones.

Steel special moment frames are often used as a part of the seismic-force resisting systems in buildings designed to resist earthquakes with substantial inelastic energy dissipation [2]. Design requirements for steel special moment frames can be found in a series of U.S. building codes. ASCE 7-10 sets the basic load requirements for special moment frames with associated lateral drift limits. AISC 360 (see Reference [3]) is the main AISC specification that provides design and detailing requirements for all steel buildings. In addition, AISC 341-05 (see Reference [4]) gives detailed design requirements related to materials, framing members, connections, and construction quality assurance and quality control.

It is worth mentioning that the ASCE 7-10 permits to use three types of analyses to determine member design forces and design drifts namely: equivalent lateral force, modal response spectrum, and seismic response history analysis. Equivalent lateral force analysis is the simplest procedure; however, it can lead to a conservative design [1].

According to the authors in Reference [2], "In many cases, exact analysis will determine a substantially longer building period than that determined by the approximate methods. As a result, substantial reduction in base shear forces often can be obtained by calculating building periods using the more exact". Therefore, the purpose of this work is to calculate the natural period for a steel special moment frame after the design and check whether this period increases or decreases the base shear.

2. PROBLEM DEFINITION

The SMF is a part of a six story office building located in San Jose, CA, USA. Three special moment frames (SMFs) are considered for the east-west direction of the building. Braced frames will be used in the north-south direction. For the purpose of this research, only the east-west direction is considered in the design. The middle SMF is designed here in this research. The building geometry is shown in Figure-1. The bay width in the east-west direction is 30 ft (9.15 m) while it is 40 ft (12.2 m) for the north-south direction. A typical 13 ft-6 in. (4.15 m) floor-to-floor heights is considered. The first story height is 17 ft-0 in. (5.2 m).

The occupancy type is planned to be offices with a design load of 50 psf (2.4 kN/m^2) plus a 20 psf (0.96 kN/m^2) allowance for partition walls. The design live load is 80 psf (3.83 kN/m^2) to account for corridors. Live load reductions will be considered. The roof has a mechanical penthouse with an equipment load of 120 psf (5.75 kN/m^2) and an additional structural self-weight equivalent to 40 psf (1.92 kN/m^2). The location is over the centerline on the north edge. The size is assumed to be 20 ft (6.1 m) east-west x 10 ft (3.05 m) north-south.

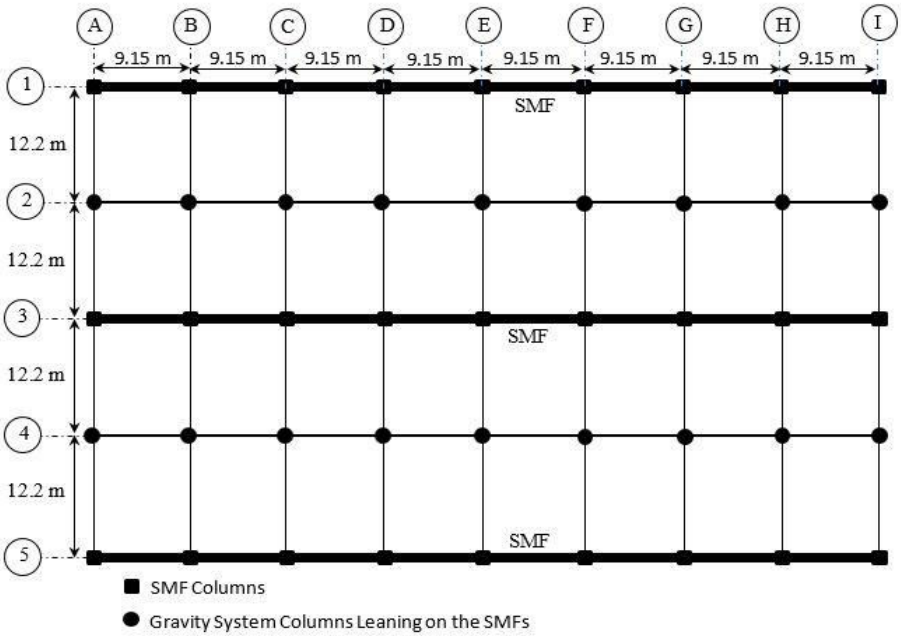

Figure-1. Special moment frames for the east-west direction.

The roof is flat with a parapet of 6 ft (1.83 m) located on the outer wall. The façade' (outer wall) is a mix of building material components and weighs 40 psf (1.92 kN/m^2). The roof and floor deck will consist of stay-in-place (SIP) metal decking on composite beams with a total deck depth of 5 in. (127 mm). The deck is a normal weight concrete with a compressive strength of 4 ksi (27.5 MPa).

The equivalent dead load for each floor and the roof is 90 psf (4.31 kN/m^2) on the floor area. This includes an approximation for the effects of the structural steel columns and flexural members, flooring and any additional dead loads.

For the seismic analysis, the frame is analyzed according to the ELF to get the base shear. The base shear calculations require finding the effective seismic weight of the building and the seismic response coefficient. Also, the soil conforms to site class D is assumed. Some other factors should be found based on the building location. The calculations are provided in detail in this paper.

The steel members are found by checking the drift limitations required by ASCE 7-10. The drift limits are achieved by applying only the lateral loads per each story. The analysis is done using the finite element program SAP2000 [5]. The plastic story drift is checked with the allowable story drifts per ASCE 7-10.

For the strength check of the members, the direct analysis method of AISC 360 is used to get the axial and flexural strengths for both the beams and columns. This method (DAM) is described in chapter (C) of the Steel Construction Manual [3]. The DAM is applied using SAP2000 and the strength of the members is checked against Chapters D, E, F, G, H, I, J, and K of the AISC specification.

Once the 2-D frame is designed, the actual natural period of this system can be calculated. The new base shear based on the new natural period can be calculated and compared with the base shear based on the approximate natural period. This is done to check the validity of the equivalent lateral force method of the ASCE 7-10 Code in designing the system considered in this paper.

3. METHODS

3.1 Drift check

The lateral load is equally distributed among the three SMFs. The calculations are shown in the subsequent sections. The drift limits are achieved by applying only the horizontal seismic forces (Q_E). The self-weight of the members is ignored for the analysis. The moment of inertia of the beams is considered as the average of the positive and negative moment of inertias according to AISC specification. The positive moment of inertia represents the long term moment of inertia for the composite beam section, while the negative moment of inertia represents the moment of inertia of the steel section.

The analysis is done using the finite element program SAP2000 and the elastic story drift for each story is exported. The elastic drift is then multiplied by C_d/I_e to back calculate the plastic story drifts, where Cd is the deflection amplification factor and Ie is the seismic importance factor. The deflection amplification factor Cd is 5.5 for a steel special moment frame as per Table 12.2-1 of ASCE 7-10 Code. The plastic story drifts are checked with the allowable story drifts (Δa) provided by ASCE 7-10 Code, Table 12.12-1. Virtual work method (see Reference [6]) is applied using SAP2000 to find the most effective member that when changed it will give the most significant effects

on the drifts. This requires several iterations until the plastic story drift satisfy the allowable story drift of ASCE 7-10.

3.2 AISC direct analysis method (DAM)

For the strength check of the members, the direct analysis method of AISC 360 is used to get the axial and flexural strengths for both the beams and columns. The Direct Analysis Method (DAM) is described in chapter C, Design for Stability, of the Steel Construction Manual. The DAM can be used for any type of system and it is account for structural instability.

General requirements should be achieved for the DAM to be used as required by AISC 360. These requirements are:

a) Consider all deformations (flexural, shear, axial deformations, and all other parts that contribute to the displacements of the structure).

b) Consider second order effects (sway P-Δ and within member P-δ).

c) Consider geometric imperfections (initial imperfection).

d) Consider stiffness reductions due to inelasticity.

SAP2000 can take into consideration any type of deformations. In addition, P-delta effects can be modeled using SAP2000 through a nonlinear geometric static analysis case.

Initial imperfections can be taken into account through the use of notional loads as described in section 2b of Chapter C of the AISC steel construction manual. The notional loads are applied laterally and added onto any other lateral loads for B2 calculations. The notional loads are taken as a percentage of 0.2% of the gravity load (Yi) at each floor (0.002 αYi, α is a factor of 1.0 for LRFD load combinations). For a ratio of the maximum second order displacement to the maximum first order displacement ($\Delta 2^{nd}$ order/$\Delta 1^{st}$ order) that is less than 1.7 using the reduced stiffness in the analysis, the notional loads can be applied to gravity load cases only and this will be checked later.

For the stiffness reduction, AISC 360 suggests to reduce the flexural stiffness (EI) by a factor of 0.8τb, where τb = 1.0 for a ratio of Pr/Py less than 0.5, where Pr is the required compressive strength for the LRFD load combinations and Py is the axial yield strength. For the flexural stiffness in SAP2000 model, τb is used as unity and the ratio (Pr/Py) is checked after performing the analysis. In addition, the axial stiffness (EA) is reduced by a factor of 0.8 according to AISC specification.

After getting the required axial and flexural strength (Pr and Mr) for each member, Chapters D, E, F, G, H, I, J, and K of the AISC specification are applicable for the design. It is worth mentioning that the steel members should be seismically compact section according to the AISC seismic design provisions 341-05, Table I-8-1 [4].

4. SEISMIC LOAD CALCULATIONS

In this section, the lateral load (QE) is calculated using the equivalent lateral force method of ASCE 7-10 Code. As mentioned previously, the building is located in San Jose, CA, USA. Therefore, the mapped spectral response acceleration parameter at short periods (SS) and the mapped spectral response acceleration parameter at a period of 1 s (S1) are 1.5g and 0.6g, respectively, where g is the gravitational acceleration. For site class D, the short-period site coefficient Fa is 1 and the long period site coefficient Fv is 1.5 determined from Tables 11.4-1 and 11.4-2 of ASCE 7-10 Code. To determine the seismic design category, the building risk category is II as per table 1.5-1 of ASCE 7-10 Code. Making use these values along with equations 11.4-1 and 11.4-2 of ASCE 7-10 Code, the seismic design category of the building is D as per tables 11.6-1 and 11.6-2. According to ASCE 7-10, Table 12.6-1, the equivalent lateral force analysis is permitted for buildings having seismic design category D and risk category II. For this seismic design category and a building height of greater than 35 ft (10.67 m), steel special moment frame (SMF) should be used [1].

For the dead load given in the problem definition section, the effective seismic weight (W) of the building is 26747 kip (118976.6 kN). The effective seismic weight is determined according to section 12.7.2 of ASCE 7-10 Code. The total base shear is calculated using the equations provided by section 12.8 of ASCE 7-10 Code as follows:

$$V = C_s W \tag{1}$$

where, C_s is the seismic response coefficient determined by:

$$C_s = \frac{S_{DS}}{(RI_e)} \le \frac{S_{D1}}{T(RI_e)} \ge 0.044 S_{DS} I_e \ge 0.01 \tag{2}$$

where, SDS and SD1 are the design earthquake spectral response acceleration parameters at short period, and at 1 s period. These parameters are calculated using equations 11.4-3 and 11.4-4 as SDS=1g and SD1=0.6g. The response modification factor is R and its value is 8 for special moment frames as per Table 12.2-1 of ASCE 7-10 Code. The seismic importance factor Ievalue is 1.0 as obtained from table 1.5-2 of ASCE 7-10 Code. T is the fundamental period of the structure (s) for the direction under consideration and is calculated using the alternative approximate procedure of the ASCE 7-10 Code as follows:

$$T_a = C_t h_n^x \tag{3}$$

in which Ta is the approximate fundamental period,hn is the structural height as defined in section 12.8.2.1 of ASCE 7-10 Code and the coefficients Ct and x are determined from Table 12.8-2 of ASCE 7-10 Code. The approximate fundamental period is calculated as 0.97

s. Making use of Equation (2), the seismic response coefficient is 0.077. This mean that the second part of Equation (2) is controlling the base shear value. However, the fundamental period is recalculated again after designing the steel members of the steel special moment frame to find which term of Equation (2) is govern.

The total base shear is calculated using Equation (1) as 2095 kip (9159 kN). This base shear is distributed among the six story of the steel special moment frame by following the procedure provided in section 12.8.3 of ASCE 7-10 Code. The steel special moment frame is designed next by checking the allowable story drifts provided by ASCE 7-10 Code, Table 12.12-1 and checking the strength of the steel members by using the direct analysis method of AISC 360.

5. RESULTS AND DISCUSSIONS

5.1 Drift check

Since the lateral load is equally distributed among the three SMFs. Each steel special moment frame of the building is taking 34 % of the base shear. For the drift calculations, the redundancy factor ρ is 1.0 as per section 12.3.4.1 of ASCE 7-10 Code. Therefore, the lateral loads applied as 0.34 QE only without considering the ρ factor. The lateral loads in each moment frame are distributed at each column in each floor level. This required dividing the horizontal component of the lateral load between the internal and external columns. For this purpose, the interior column share is considered as twice the exterior column share in the moment frame. The steel special moment frame has two exterior columns and seven interior columns. The later loads used in the drift load calculations are shown Table-1.

Table 12.12-1 of ASCE 7-10 Code gives the allowable drifts. For our building with a risk category of II, the allowable drift limit is Δa = 0.02hsx, where hsx is the story height below level x. For seismic design category D, ASCE 7-10 Code, section 12.12.1.1 requires to divide the allowable drift limit by ρ (ρ =1.3 for seismic design category D).

Table-1. Lateral loads used for the drift calculations.

Story	Total Q_E (kN)	$0.34Q_E$(kN)
Roof (6th)	2429	825.86
5th	2353	800
4th	1813.5	616.6
3rd	1303	443
2nd	828.8	281.8
1st	431.8	146.8
SUM	9159	3114

As mentioned previously, the virtual work method is used in SAP2000 to find the members that most affect the total deflections. Many iterations have been done to get the final steel cross-sections for the beams and columns to satisfy the story drift. The procedures are well explained before in the drift check section. The effective moments of inertias for the composite beams in the negative and positive moments regions are used in the elastic analysis. Note that no constraint is assigned to account for the floor diaphragm. Also, self-weight of the members is ignored in the analysis.

The displacement at each floor is exported from SAP2000 and the elastic drifts are calculated. The plastic drifts are then calculated and checked against the allowable drifts of the ASCE 7-10 Code as shown in Table-2. The displacements presented are for the left side edge of the frame.

Table-2. Story drift check.

Story	Elastic story drift (mm)	Plastic story drift (mm)	Allowable drift Δ_a (mm)
1st	13.5	74.3	79.7
2nd	11.4	62.7	63.2
3rd	11.4	62.7	63.2
4th	10.9	59.9	63.2
5th	9.4	51.7	63.2
Roof (6th)	7.1	39.1	63.2

The members used to achieve the allowable drifts are shown in Figure-2 by using a snap shot from the SAP2000 window. The strength of these steel members are checked using the AISC direct analysis method as shown in the next subsection.

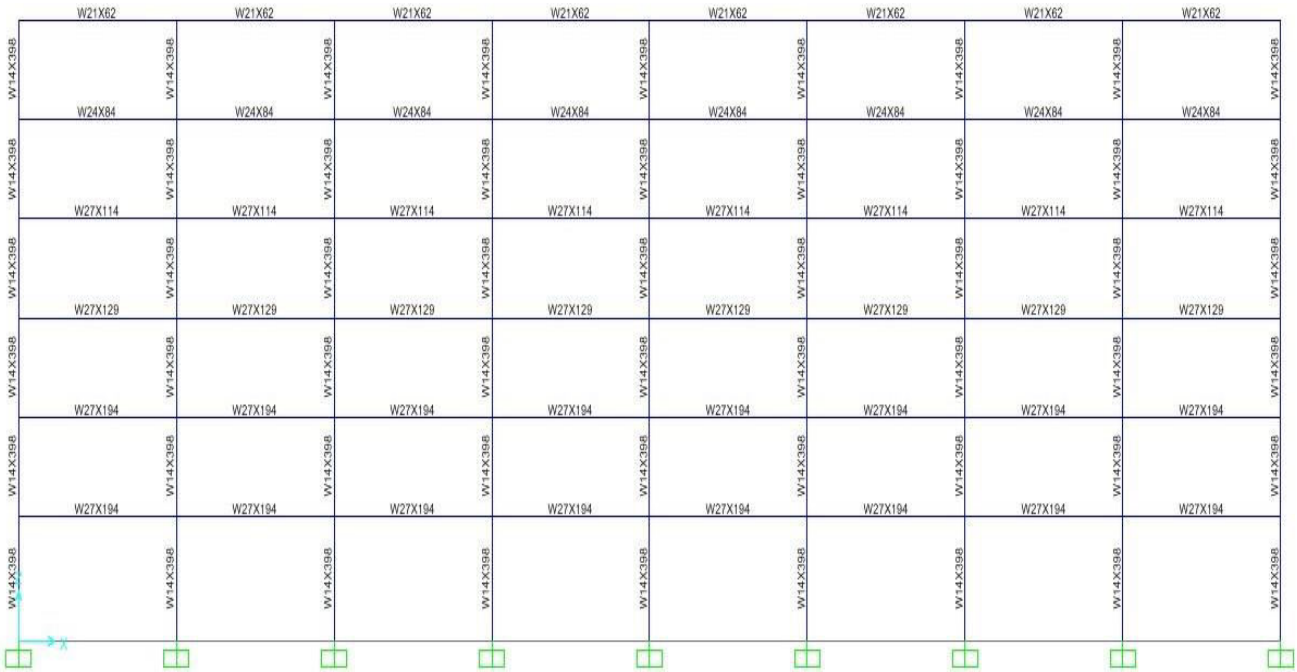

Figure-2. Steel special moment frame members used for the drift check.

5.2 AISC direct analysis method (DAM)

Load combinations number 5 and 6 as per section 12.4.2.3 of ASCE 7-10 Code were considered to check the strength of the steel members. This requires to calculate the vertical seismic load effect, Ev, which is determined as 22 psf (1.05 kN/m2). The dead and live loads are given in the problem definition section. Note that reduced live loads were used. In addition to the gravity loads, the lateral loads (ρ QE) is applied.

For each load combination, the notional loads and the leaning loads of the columns that do not contribute to the lateral resisting system are also calculated. For load combination 5, the total notional loads are 24 kip (106.7 kN) and the leaning loads are 5715.8 kip (25425 kN). For load combination 6, the total notional loads are 10.5 kip (46.7 kN) and the leaning loads are 2406.7 kip (10705.5 kN).

The leaning frame is modeled with a leaning column that should not buckle on its own. A high stiffness is assigned and the moments are released. Each level is coupled with a rod constraint. The leaning "column" models the effect of gravity frames on the stability. Finally, the moments in the leaning column are zero.

For the first order analysis and for both load combinations, a SAP2000 static linear analysis is created. The notional loads and the leaning loads for each load combination are considered and the first order effects are found. It is unnecessary to include the leaning loads for the first order analyses. However, including them will not affect the story drifts.

For the P-Delta effects (second-order analysis) and for both load combinations, a SAP2000 static nonlinear analysis case is created for the factored notional loads. This load will rack the frame. Next a static nonlinear SAP2000 analysis case is created with 1.2D+0.2SDSD+0.5L+0.2S or 0.9D-0.2SDSD, i.e., gravity loads. This case begins with the notional load case. A static nonlinear analysis case with the seismic lateral loads is created next and it start with the gravity analysis case, i.e., it is preloaded with gravity that has been preloaded with the notional loads. Load control is sued with 10 steps for the seismic load.

The ratio of the second order drift to the first order drift is calculated as 1.09 based on load combination 5. This value is less than 1.7. This means that the notional loads can be applied to the gravity loads only and need not to be added to the lateral load. This is required by AISC steel design manual as explained in the direct analysis method subsection, methods section. The required compressive strength (Pr) in the columns from load combinations is 1100 kip (4893 kN). The axial yield strength (Py) is calculated as 5850 kip (26022 kN). The ratio (Pr/Py) is 0.19, which is less than 0.5. Therefore, the assumption for τb = 1.0 is applicable.

AISC seismic design provisions 341-05, section 8.3 requires not to use the system over-strength factor (Ωo) for the columns if Pu/ØcPn less than 0.4, where Pu is the required axial strength of a column using LRFD load combinations. The factor Øc is 0.9 and Pn is the nominal axial strength of a column. The required axial strength is 1100 kip (4893 kN). The factored axial strength is 4470 kip (19883.6 kN) for column size of W14x398. The ratio of (Pu/ØcPn) is calculated as 1100/4470 = 0.24, which is less than 0.4. Therefore, the system over-strength factor need not to be considered in the analysis. The column is also checked for flexure and shear.

As mentioned previously, the strength check for the steel members is done according to AISC steel design manual. The required strength for each member is taken from load combination 5 or 6, whichever controls the

design. For the positive moment region in the girders, the required flexural strength (Mu+) is compared with the fully composite capacity of the steel section (ØbMnx). However, for the negative regions, the required flexural strength (Mu-) is compared with full plastic capacity of the steel section (ØbMpx). Also, the required shear strength (ØvVn) is compared with the section shear capacity (Vu). Table -3- shows the analysis and the design values for the flexure and shear. All steel members pass the design check. Hence, the drift limits of ASCE 7-10 code control the design of the steel special moment frame.

Table-3. Analysis and design values for flexure and shear.

Story	Steel section	M_u^+(kN.m)	$Ø_b M_{nx}$ (kN.m)	M_u^- (kN.m)	$Ø_b M_{px}$(kN.m)	V_u (kN)	$Ø_v V_n$ (kN)
1st	W27x194	1100	4176	1873	3213	707	2812
2nd	W27x194						
3rd	W27x129	674	3019	1524	2006	610	2246
4th	W27x114	506	2717	1303	1749	569	2076
5th	W24x84	374	1888	1041	1139	503	1511
Roof (6th)	W21x62	299	1303	713	732	396	1121

It is worth mentioning that all the steel members are checked against the seismic compactness according to the AISC seismic design provisions 341-05, Table I-8-1. In addition to the above, the design checks for the reduced beam section (RBS) is required. All the theory related to the reduced section design can be found in the AISC: Steel construction manual [3], AISC 341: Seismic provisions for structural steel buildings [4], and AISC 358: Prequalified connections for special and intermediate steel moment frames for seismic applications [7]. To cover the scope of this research only, the design checks for the reduced beam section design are not presented. The actual fundamental period of the special moment frame is calculated next to evaluate the base shear calculated before.

5.3 Fundamental period and base shear evaluation

The special moment frame is designed and all the steel members are known. Now, the fundamental period can be easily determined using modal analysis [8]. This is done by running a SAP2000 modal case. The self-weight of the steel members is used to define the source of the mass for the frame. The fundamental period is determined from the SAP2000 analysis as 0.294 s. The approximate natural period using Equation (3) is 0.97 s as calculated before. Hence, the total base shear needs to be recalculated considering the new natural period. For this purpose, Equations (1) and (2) are used. The seismic response coefficient is now Cs = 0.125 instead of Cs = 0.077. Hence, the new base shear is calculated as 3343 kip (14870 kN) instead of 2095 kip (9159 kN). There is an increase of 62 % in the base shear for the east-west direction of the building. Since three special moment frames were used, the percentage increase for one steel special moment frame is 20.67 %. It is worth mentioning that the (SDS/RIe) term of Equation (2) is control the seismic response coefficient Cs instead of the (S1/TRIe) term because of the change happened in the natural period of the system.

6. CONCLUSIONS

A 2-D steel special moment frame that is an element of a building consisting of six-story susceptible to high seismic risk which is designed in this paper. The building is an office building located in San Jose, CA, USA. Regarding the seismic load calculations, the equivalent lateral force method of the ASCE 7-10 Code is used. This method uses an approximate procedure to calculate the natural period of the system before the design. Therefore, the fundamental period of the system is recalculated after designing the steel special moment frame to evaluate the base shear. The steel special moment frame is designed by satisfying the drift limitations provided by ASCE 7-10 Code and checking the strength of the steel members using the AISC 360 direct design method. It has been found that the drift checks control the design of the steel members for the special moment frame than the strength checks.

The natural period of the system is recalculated by performing a modal analysis using SAP2000. It has been found that the new fundamental period is less than the approximate period from ASCE 7-10 Code. This led to a significant increase in the total base shear. The base shear is increased by 62 %. Since three steel special moment frames for the east-west direction of the building were used, an increase in the base shear of 20.67 % occurred for one steel special moment frame. Hence, a redesign for the steel special moment frame using the design procedure explained in this research should be considered. The designer can choose other option by increasing the number of steel special moment frames in the east-west direction of the building to make the natural period longer. In this case, the base shear is lowered; thus, lighter steel sections can be obtained.

In addition, the designer can be cautioned by ASCE 7-10 Code to recalculate the natural period after designing a steel special moment frame if the equivalent lateral load method is used for the seismic load calculations.

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