University of Nevada, Reno

# Effect of Torsional Ground Motion Component on Floor Acceleration Response in Flexible SMRF Buildings

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by

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# ABSTRACT

This preliminary study investigates the effect of torsional components of the ground motions (TGM) on the floor acceleration response of flexible special moment resisting frame (SMRF) buildings. For this purpose, computational models of three SMRF buildings were subjected to far-field ground motions with and without TGMs. Accordingly, the effect of TGM on the floor acceleration response of torsionally rigid structures with relatively short torsional period of vibration may be significant. Furthermore, TGMs may have adverse impact on rigid nonstructural components as it can be inferred from the amplifications in floor acceleration spectra, regardless of the flexibility of the parent structure.

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# 1. INTRODUCTION

Significant structural damage, economical losses and threat to life safety associated with poor performance of structures during the past earthquakes resulted in increased awareness among civil and architectural engineering communities. To better understand the response of the structures during the seismic events analytical and experimental researches are continuously going on and the results of the researches are incorporated in the structural design through different building codes. While researches on the response of structure due to translation ground motion has been extensively studied, however, few research has been done to study the effect of Translational Ground Motion (TGM) on the response of the buildings and on top of it the researches done to study the response of TGM are based on highly idealized and simplified models. Torsion in the building can be induced due to (a) unsymmetrical plan of structure and the discrepancy in the center of mass and the rigidity, (b) torsional ground motion due to seismic event, (c) different unaccounted reasons like neglecting the distribution of the live load or not including the stiffness of non-structural component. While the building can be designed for static eccentricity by calculating the eccentric distance from the distribution of the structural mass and stiffness the effect due to TGM cannot be explicitly accounted in the design process. The ASCE 7-16 requires the lateral force be shifted by 5 percent of the dimension of the structure perpendicular to the direction of the applied seismic force to account the TGM and other accidental eccentricity. In this thesis the effect of the TGM on the response of structure is studied when subjected to ground motion with and without TGM based on three full scaled analytical models.

## 1.1. Objective

The main objective of this thesis is to develop an analytical model to study the seismic floor acceleration response of building due to TGM and compare the floor acceleration amplification prescribed by the ASCE7-16. In parallel, the thesis also compares the floor acceleration amplification factor proposed by [Wieser et al, 2013] and explains the effect of the structural period on floor acceleration amplification factor when the structures are subjected to TGM.

## 1.2. Methodology

To achieve the objective, firstly three building models namely 3 Story Building (B3), 9 Story Building (B9) and 20 Story Building (B20) were chosen under the scope of post-Northridge SAC steel project designed for the Los Angeles area and modeled using the SAP2000 v 18.1.1. The boundary conditions and the section properties used on the study are based on Gupta and Krawinkler [1999] and Wieser [2011]. The twenty ground motions used were selected from the Pacific Earthquake Engineering Research Center (PEER) database with the guidelines prescribed by the ATC-63 FEMA, 2009. The ground motions were scaled according to the ASCE7-16 such that the square root of the average of the sum of the squares (SRSS) of each pair 5 percent damped horizontal response spectrum accelerations shall not be less than the design spectrum.

As the Single Station Procedure (SSP) purposed by Basu et al. [2012] requires the data of a single station only to generate the rotational ground motion from the translational ground motion, the SSP method is used in this study. The building models were subjected with the ground motion for the load cases with the TGM and without the TGM. The analyzed data were generated for the *x* and *y* direction independently and SRSS method was applied to combine the data. The maximum SPRSS responses obtained for the case with and without were compared. The study does not account for the soil-structural interaction so considers there is no slippage between the soil and structure.

## 1.3. Literature Review

While the innovation in science and technology has highly aided to validate the analytical results experimentally in earthquake engineering, however, there are knots related with the response of building due to the Torsional Ground Motions (TGM) which has not been untangled yet. Even though the effect of the TGM on the structures is dated back to mid nineteen century but a conclusive answers has not been drawn largely because of the lack of the instrument that can record the TGM. Anagnostopoulos et al. [2015] has critically reviewed the torsion induced in buildings due to seismic event and commented from the past research [Anagnostopoulos et al, 2008, 2009, 2010] that the conclusion drawn from a study is highly susceptible to be used in the other studies due to the different assumptions made during the modeling. The research shows that the result drawn from the model can be used only if the first three periods of the structure matches and element stiffness and strength of material are same.

Newmark [1969] published the very first paper to account the torsion in a structure due to the seismic event by considered a one story symmetrical building, developed torsional ground motion spectra from the theory of elasticity and gave an expression to account the eccentricity using response spectra. Tso and Hsu [1978] developed a torsional spectra and compared the spectra developed by Nathan and Makenzie [1975]. Wu and Leyendecker

[1984] studied the structure when subjected to S-H waves on geometric eccentricity building including the soil structural interaction. The result are based on different aspect ratio of the building and foundation mat and concluded that the response of the system depends greatly on the geometric eccentricity, dimension of the foundation and rotational frequency to translational frequency has to be taken into consideration when calculating the eccentricity. Vasquez and Ridell [1988] purposed an equivalent equation to calculate the eccentricity in the building developed due to torsional ground excitation. Yeh et al [1992] constructed torsional spectra by using the principle of travelling wave hypothesis for Taiwan area, studied the response of symmetrical building under torsional motion and concluded symmetrical building may undergo significant torsional motion depending on the ratio of the rotational frequency to the translational frequency. Shakib and Datta [1993] developed two way eccentric model subjected to ensemble of artificially generated non stationary random ground motion to study the ductility behavior of the system. Hahn and Liu [1994] studied the behavior of unsymmetrical buildings excited by incoherent seismic ground motions and purposed equations which are different than the eccentricity prescribed by the code. The paper shows the eccentricities developed by such ground motions are highly dependent on the ratio on the torsional to lateral frequency of vibration ($\omega_\theta/\omega_x$). De La Llera and Chopra [1994] investigated the accidental torsional in building resulting from the rotational excitation. The result was based on 30 ground motions recorded during different California earthquake and concludes that the accidental torsion has the effect of increasing the building displacement by less than 5 percentage for torsionally stiff structures or with the lateral vibration period longer than half a second. However, for short period structure with period less than half a second and torsionally flexible system the accidental torsion can be significant. Heredia-zavoni and Levya [2003] showed the effect of incoherency and wave passage effects are significant only for columns in the ground level of stiff system. Moreover the increase of column shear is higher for soft soil than for firm soil. Rigato and Medina [2007] conducted nonlinear time histories for both symmetrical and unsymmetrical building to study the influence of the angle of incident of the ground motion. Cui [2013] has numerically investigated the effect of instantaneous load eccentricities (A-Δ) on the torsional responses under bi-directional seismic force and concluded that the apparent effect of A-Δ becomes more significant when P-Δ effect is also considered. Therefore, higher torsional seismic demand has been observed due to combined A-Δ and P-Δ effect on the structural systems.

ASCE7-16 requires that the distance between the center of mass and rigidity shall be explicitly accounted by applying equivalent lateral forces at a static eccentric distance $e_s$ from the center of rigidity in addition to the shear force and overturning moment. Further, the code requires that the force shall be shifted with 5 percent of the dimension of the structure perpendicular to the direction of the applied seismic force. The additional 5 percent accidental eccentricity is introduced to account the asymmetry that might be resulted due to the uncertain distribution of live load and/or variation of physical properties of structural members, and the force induced due to the rotation of the base of the structure Chopra [1994]. Ghyamghamian [2009] developed TGM using the accelerometers at Chiba dense array and studied the accidental eccentricity levels proposed by various building codes and found the 5% accidental eccentricity is safe for building with period greater than 0.3 sec however for structures with period shorter than 0.3sec the displacement can increase by four times when torsional ground motion is considered.

## 1.4. Organization of Thesis

This thesis is presented in three chapters. Chapter 1 is an overview of the thesis which introduces the thesis, its objectives and methodologies applied to achieve the objectives. It also consist of a brief literature review. Chapter 2 is the main chapter of the thesis that describes the effect of torsional ground motion component on the response of the building. Chapter 3 concludes the thesis and provides the recommendations for future to broaden the knowledge on the effect of torsional ground motion component on the response of the buildings.

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# 2. EFFECT OF TORSIONAL GROUND MOTION COMPONENTS ON FLOOR ACCELERATION RESPONSE IN FLEXIBLE SMRF BUILDINGS

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## ABSTRACT

This preliminary study investigates the effect of torsional components of the ground motions (TGM) on the floor acceleration response of flexible special moment resisting frame (SMRF) buildings. For this purpose, computational models of three SMRF buildings were subjected to far-field ground motions with and without TGMs. Accordingly, the effect of TGM on the floor acceleration response of torsionally rigid structures with relatively short torsional period of vibration may be significant. Furthermore, TGMs may have adverse impact on rigid nonstructural components as it can be inferred from the

amplifications in floor acceleration spectra, regardless of the flexibility of the parent structure.

## 2.1. INTRODUCTION

Nonstructural component (NSC) failure and consequential damage to building contents following moderate to major earthquakes are the major causes of extensive economic losses and interruption of functionality. Continued functionality is particularly of concern for essential facilities where immediate occupancy is required. Therefore, there has been an increased awareness among the engineering communities that led to numerous analytical and experimental investigations (ATC, 2017) of seismic damage to acceleration-sensitive NSCs due to amplified floor accelerations. Studies have focused on improving the understanding of acceleration demand on NSCs and developing performance based design criteria for these components (e.g. Drake and Bachman, 1996; Miranda and Taghavi, 2005; Medina et al., 2006; Wieser et al., 2013). Some of these studies were aimed to formulate the expected lateral acceleration demand on both rigid and flexible NSCs. Moreover, the effect of nonlinear behavior of parent structure and dynamic interactions on the seismic response of NSCs within structural systems have been investigated under translational and vertical components of ground motions. Subsequently, various alternative approaches were introduced to improve the current seismic design procedures for NSCs.

Another potentially contributing factor in seismic response of structural systems hence NSCs are the rotational ground motion components. A recent review by Zerva *et al*. [2018] states that the rotational ground motions may have significant effect on the seismic response of structures, however very few research studies have investigated such the effects. Kozak [2009] and Oldham [1899] presented the distortion of an obelisk in San Bruno monastery as an evidence and example rotational ground motion effects following 1783 Calabria earthquake. Newmark [1969] proposed a deterministic procedure to investigate the effect of torsion on the symmetric building arising from earthquake wave motions and concluded that even in symmetric structures; especially corner columns experience higher torsion due to rotational excitations. The analytical studies related to the dynamic response of symmetrical and asymmetrical buildings which were subjected to TGMs have been conducted by Awad [1984] and revealed that dynamic coupling between translational modes and eccentricity may amplify the apparent effect of TGMs. A similar study was conducted by Llera et al [1994] for symmetric and asymmetric single story buildings which concluded that the increase in displacements due to rotational excitation in symmetrical building could be as high as 40% compared to counterparts in asymmetric building. Moreover, Luco [1976] investigated the variation of floor velocity and acceleration at the roof level of structures due to TGMs and concluded that the stiff structures are more vulnerable to TGMs which may cause twenty times higher floor acceleration. A relatively recent compilation of analytical studies and findings have been reported by Anagnostopoulos et al. [2015]. Even though the observations associated with the TGMs date back to mid nineteenth century, most of the studies conducted are based on very simplified and highly idealized models of eccentric one-story systems. These studies concluded that overall seismic response of structures with varying dynamic properties and, with various geometric irregularities could be adversely affected by the torsional components of ground motions, which could result in increase in acceleration and displacement response. According to Basu et al. [2012a], the effects of rotational components on the seismic responses of relatively long period structures may be considerably more significant. Loghman et al [2017] have performed a series of response history analyses to investigate the effect of rotational components of ground motions on the seismic response of base isolated structures that have varying slenderness and plan aspect ratios. The results have revealed that the base shear is amplified by 33.75% whereas the roof acceleration amplification can be as high as 120% due to rotational components of ground motions for buildings that have a square plan and higher slenderness ratio. Furthermore, the effect of rotational excitations on roof acceleration increases for the buildings that have higher plan aspect ratios. Guidotti et al. [2018] have investigated the effect of the near field motions with their torsional components on the tall buildings and concluded that the interstory drift could increase up to 15% along the height of the building when TGM is considered.

The propagation characteristics of seismic waves from source to the surface results differential displacement as well as ground rotations. It is noted that appropriate sensors are not readily available; therefore, it is neither possible nor feasible to measure rotational components directly and reliably [Castellani, A. and Boffi, G 1986; Schreiber et al, 2009b; Basu et al, 2012a]. Therefore, the studies related to the effect of rotational excitations on the seismic behavior of structures are limited [Lee et al, 2012; Zerva et al, 2018]. Lee and Trifunac [1985] generated torsional components of ground motions (TGM) from their translational counterparts by using the principles of elastic wave propagation. Rutenberg and Heidebrecht [1985] developed response spectra for TGMs. Castellani et al [1986] proposed a method to generate rotational ground motions and spectra based on continuum mechanics. They also suggested that the effect of ground rotation rapidly decreases when the fundamental period of vibration of the structure increases. Furthermore, Basu et al. [2012a] proposed three different analytical methods to extract rotational components of ground motions from their translational components, and they investigated the effect of torsional and rocking components of ground motions on the behavior of different types of structures. It is suggested that TGMs may be the possible reason of the observed damage in the tall buildings in Los Angeles area during the 1971 San Fernando earthquake [Hart et al, 1975; Trifunace et al, 1996].

The present study recognizes that there may be significant uncertainty in the determination of floor acceleration demand in building structures due to TGMs. It is noted that the acceleration demand on NSCs and the torsional response of structural components are expected to elevate due to TGM. Therefore, the effect of TGM on the displacements, shear force and torsional demand of structures and on the distribution of floor accelerations are investigated. For this purpose computational models of 3-, 9- and 20-story SAC buildings are developed and subjected to various ground motions that include TGMs. The SAC buildings are relatively flexible special moment frames that have been used in related studies by other researchers. Lastly, the TGMs were generated using the Single Station method proposed by Basu et al [2012b].

## 2.2. SEISMIC CODE PROVISIONS FOR THE DESIGN OF NSCs

The seismic design provisions for NSCs are intended to ensure that these components and their attachments can withstand the accelerations and deformations induced by the design ground motions without a form of failure that may pose risk to life safety and/or required functionality. These provisions are typically based on the concepts and techniques that are similar to those used for the design of building structures. Specifically, seismic demand of NSCs is expressed as a function of ground acceleration, component's weight and its location relative to the height of the building, its ability to accommodate inelastic deformations and in-structure dynamic amplification.

In the United States, the Uniform Building Code (UBC) with its 1935 edition recommended explicitly the seismic design of NSCs. These recommendations were later extended to include components and their connections such as architectural elements, towers, tanks, contents, chimneys, penthouses, storage racks, equipment, machinery, etc. with the 1997 edition. The 1991 edition of UBC also addressed the displacement requirements in view of the equipment attachments. In the mid-1990s, Drake and Bachman [1996] discussed the seismic force and displacement requirements incorporated into the 1994 National Earthquake Hazards Reduction Program (NEHRP) provisions (NEHRP 1994) for NSCs. Accordingly, the provisions were a significant departure from 1991 NEHRP and earlier seismic provisions, such as those offered by the 1994 Uniform Building Code (UBC). It was noted that revised seismic-force and relative displacement design equations were rationally developed to account for site ground motion, dynamic characteristics of the parent structure, flexibility of the component, and importance of component function. The NEHRP provisions serves as the primary resource document for the seismic design of NSCs that is detailed in the current US building codes Bachman [2004]. Accordingly, the seismic design forces, $F_p$ are applied at the component's center of gravity and distributed relative to the component's mass distribution as follows:

$$0.3S_{DS}\,I_pW_p \leq F_p = \frac{0.4a_pS_{DS}\,W_p}{R_p/I_p}\left(1+2\,\frac{z}{H}\right) \leq 1.6S_{DS}\,I_pW_p \qquad (1)$$

in which $S_{DS}$ is the short period spectral acceleration determined based on the mapped spectral response acceleration at short periods that is adjusted for the site-soil conditions. The component amplification factor, $a_p$ represents the dynamic amplification of the component relative to the acceleration at the support point implicitly as a function of the fundamental periods of the structure ($T$) and component ($T_p$). The selection of $a_p$ depends on the flexibility of the component, which can be 1.0 (rigid) or 2.5 (flexible), and determined from the tables (ASCE 7-16) that list various common NSCs. However, the tabulation of assumed $a_p$ values is not meant to preclude more precise determination of the component amplification factor based on dynamic analyses. Furthermore, in equation (1), $I_p$ is the component importance factor that varies from 1.0 to 1.5, which represents the adjustment to design forces in view of the level of required functionality and life-safety risk; $R_p$ is component response modification factor represents the energy absorption capacity and the deformability that varies from 1.0 to 12.0 (6.0 for anchorage design) as listed in tables (ASCE 7-16) for common NSCs; $z$ is height in structure of point of attachment with respect to base; $H$ is average roof height of structure with respect to base and $W_p$ is the component's operating weight.

The peak floor acceleration, hence amplification of ground acceleration at a floor height is accounted for with the (1+ 2 $z/H$) term in equation (1), which suggests that the peak floor acceleration at the roof level equals three times that at the ground level and varies linearly between $0.4S_{DS}$ to $1.2S_{DS}$. Since the vertical distribution of floor acceleration/amplification may be significantly different particularly in relatively flexible structures, an alternative equation is provided to determine the equivalent static force demand accounting for the dynamic properties of the building structures when available:

$$F_P = \frac{a_i . a_p . W_P}{(R_P/I_P)} . A_X \tag{2}$$

in which, $a_i$ is the floor acceleration at level $i$ obtained from the modal analysis and $A_x$ is the torsional amplification factor determined in accordance with ASCE 7-16 in order to account for torsional irregularities in view of story drifts, and with the limits on $F_p$ as the same for equation (1). While equation (2) may result in more accurate approximations to design forces, because of added complexity and prior information needed about the parent structure, equation (1) has been more commonly used. However, Wieser et al. [2013] proposed a modification to incorporate the effect of structural period on the floor acceleration amplification

$$\frac{PFA}{PGA} = \left(1 + \frac{T_{max} - T}{T} \frac{z}{H}\right) \tag{3}$$

where $PFA$ is the peak floor acceleration, $PGA$ is the peak ground acceleration, $T$ is the natural period of the supporting structure and $T_{max}$ is the maximum structural period for which the peak roof acceleration is equal or greater than the PGA. Nevertheless, 2.5 seconds is recommended for $T_{max}$ based on statistical evaluation of analyses data.

## 2.3. BUILDING MODELS AND GROUND MOTIONS

Three steel moment frame (SMRF) office buildings were selected for the present study. The buildings were introduced originally as part of the SAC steel project (Gupta and Krawinkler, [1999]) and consists of 3-story (B3), 9-story (B9), and 20-story (B20) tall structural systems with nominally regular configurations. These SMRF buildings represent a diverse range of low-to-midrise heights with relatively long fundamental periods of vibrations. Hence, the findings of this investigation are applicable to flexible SMRF buildings.

### 2.3.1 Description of the Buildings

Three-dimensional linear-elastic computational models of the buildings were developed using SAP 2000 as depicted in Figure 1 and Figure 2. The 3-story office building, B3, consists of a 36.6 m x 54.6 m floor plan made up of four 9.1 m bays by six 9.1 m bays, respectively, and with a total height of 11.89 m. The base of the columns in SMRFs are fixed along the strong axis and pinned about the weak axis whereas the gravity columns are pinned at the base. The gravity loads and seismic weight of the building was distributed evenly over the floor shell elements, which results in static eccentricities of $e_x$ and $e_y$ as -0.30 m and 1.10 m, in EW and NS directions respectively.

The 9-story office building, B9, also makes use of 9.1 m bays over a 45.7 m x 45.7 m floor plan. The connection between the corner column and the beams of the SMRFs are pinned to decouple the moment between the orthogonal frames. The columns are pinned at their base. The rigid behavior of the basement floors was modeled through the implementation of axially rigid x-shaped trusses acting as a solid retaining wall all around the basement levels. These braces used in the computation model provided lateral rigidity while retaining the continuity of the columns throughout the basement stories. The distribution of the gravity loads and seismic weight was uniform, hence yielding no eccentricity. The 20-story office building, B20, has a 30.5 m x 36.6 m floor plan with 6.1 m exterior SMRF bays and four central gravity columns. The overall height of the building is 80.70 m above the ground level; the height of the first floor above grade is 5.46 m with the remaining 19 stories at 3.96 m each. There are two basement floors with a total height of 7.32 m below grade. The corner columns were designed as box sections to resist the biaxial moment and all columns are pinned at their base. The floor slabs are considered one-way metal-deck concrete slabs with 140 mm thickness supported by secondary beams spaced at 3.0 m. The rigidity of the basement retaining wall was modeled by axially rigid x-shaped trusses.

In general, all of the structural elements were modeled using elastic beam-column elements. The floor systems were modeled using four-node elastic shell elements. The secondary beams supporting the floor slabs were implicitly accounted by modifying the stiffness of the shell elements. The equivalent stiffness was calculated based on the properties of the composite section of the concrete floor slab and secondary steel beams. To capture the one-way behavior of the slabs, the shell elements were attached to the main beams at the location of the intersection of the secondary beam using constraints with moment releases. Further details regarding the buildings and computational models can be found in Wieser et al. [2012].

All three buildings have relatively regular plan and stiffness distributions. Therefore, the modal periods and shapes in their respective directions are similar for all of the buildings. Table 1 summarizes the period of vibrations and corresponding mode shapes for the first six modes of the three buildings.

### 2.3.2 Ground Motion Selection

Twenty "far-field" ground motions that were developed as described in ATC-63 FEMA [2009] were selected from the Pacific Earthquake Engineering Research Center (PEER) NGA-West2 database. Even though the ATC-63 far-field record set has 22 pairs of ground motions, Cape Mendocino motion and the vertical component of Superstition Hills motion which is essential to generate TGMs, were not available on the PEER database, therefore these motions were omitted. The magnitude of the selected earthquakes ranged between

M6.5 and M7.6. The distance between the recorded station and fault rupture ranges between 11 and 26 km. Table 2 presents the Peak Ground Acceleration (*PGA*), Peak Ground Velocity (*PGV*), Primary Wave Velocity ($V_p$), Shear Wave Velocity ($V_s$), Rupture Length ($R_L$), Rupture Velocity ($R_V$) and the hypocenter depth (*HCD*) for each selected ground motion. The 5% damped pseudo acceleration and displacement response spectra for the two horizontal (x and y), and torsional (θ) components of ground motions are shown in Figure 3. Also marked in the figure are the periods that correspond to the fundamental translational (x and y) and torsional modal periods of the three buildings. The selected ground motions are scaled such that their average SRSS pseudo acceleration response spectrum matches the design spectrum (Site Class D, $S_S$=1.523g, $S_1$=0.617g). The scale factors (SF) (Table 2) were applied uniformly to all translational and torsional components of respective ground motions.

### 2.3.3 **Generation of TGM components**

Three different methods have been proposed by Basu et al. [2012b] to generate torsional components of ground motion namely Single Station Procedure (SSP), Multi Station Procedure (MSP) and Surface Distribution Methods (SDM). SSP requires relevant data of a single station only whereas MSP and SDM require data from multiple stations. Therefore, SSP were adopted under the scope of this study to generate TGMs. It is noted that in their original work, Basu et al. [2012b] introduced SSP based on both point and line source representations of fault rupture. In the present study, SSP with line source representation has been adopted to generate torsional components of ground motions (*TGM*). This procedure requires the three translational acceleration records, event and recording station-specific data; therefore, it can be conveniently used with earthquake records available in most of the ground motion databases. One of the underlying assumptions in SSP is that the soil medium is a homogenous elastic half space. Furthermore, SSP assumes the existence of a principal plane, the vertical plane containing the recording station and the epicenter, and the propagation of P and S waves along this principal plane. The vertical plane passing through the hypocenter and the recording station is defined as the principal plane. The torsional acceleration is computed from the SH component (horizontally polarized S wave) of the horizontal acceleration, which is obtained by taking the resultant of the components of the measured accelerations along the direction normal to the principal plane. The SSP accounts for the finite length of fault rupture, symmetrical and unsymmetrical bilateral rupture, and an arbitrary location for the epicenter, however, it was demonstrated that the latter did not influence the torsional spectrum [Basu et al., 2012b]. The overall procedure can be summarized as follow. First, the waves associated with the three translational acceleration histories are decomposed in to their body waves; namely, P (compressional), SV (shear wave as vertical particle motion) and SH (shear wave as horizontal particle motion). The reflections of the incident body waves are considered as the reflections of P and SV waves produce both P and SV waves, while the reflections of SH produce SH waves only. Wave propagation and particle movement associated with these body waves, as described by Achenbach [1973], were used to determine the displacement fields in three orthogonal directions with respect to the principal plane. It was noted that the resultant displacement fields satisfy plane-stress condition and hence the rotational components of the ground acceleration were derived. The method uses station-specific shear wave and primary wave velocities in addition to the parameters listed in Table 2 for the determination of the resultant displacement fields and the associated stresses. In summary, the distance between the epicenter of the earthquake and recording station, hypocenter depth, rupture length ($R_L$), rupture velocity ($R_V$), station specific shear wave velocity ($V_s$) and primary wave velocity ($V_p$) are required to decompose the translation ground motions into body waves. Interested readers are referred to Basu et al. [2012b] for further details on generation of rotational components of ground motions. For the present study, the required data were obtained from PEER database and the primary wave velocity ($V_p$) was determined as recommended by Suwal and Kuwano [2013].

## 2.4. EVALUATION OF SEISMIC RESPONSE

A series of linear response history analyses were performed for all considered buildings subjected to ground motions with and without the TGMs. The ratios of average maximum response quantities as well as individual response histories from cases with and without TGMs were compared. The response quantities considered are resultant absolute horizontal floor accelerations, intestory drifts, story shear forces, overturning moments and torsion.

Figure 4 compares the roof absolute acceleration response histories along the two horizontal axes at one of the corners of the buildings subjected to Canyon County ground motion records (ID#2). As can be seen in the figure, the maximum absolute accelerations (max($a_x$, $a_y$)) are 1.06g, 1.02g, and 0.96g for B3, B9, and B20, respectively, when the TGM is not considered. On the other hand, corresponding maximum floor accelerations are 1.46g, 1.33g and 1.18g for B3, B9 and B20 when torsional ground motion is considered. The comparison of cases with and without TGM indicates 37%, 30% and 22% higher floor acceleration for B3, B9, and B20, respectively.

Two categories of NSCs can be considered in view of their acceleration response: rigid components and flexible components. Rigid components have short fundamental periods of vibration (≤ 0.06 sec) thus have a comparable total acceleration response to that of the floor, while the acceleration response of flexible components which have longer natural periods (> 0.06 sec) may be amplified. For this reason, the absolute maximum of the resultant of horizontal floor acceleration (PFA) was used to describe the response of rigid NSCs while the resultant floor spectral acceleration (FSA) was used to describe the response of flexible counterparts. For rigid nonstructural components, PFA was normalized by the resultant PGA of the corresponding ground motion. This parameter is a measure of the dynamic amplification of the ground motion by the structure. For flexible nonstructural components, the ratio FSA/PGA is considered, which can be viewed as the total acceleration amplification since it accounts for the amplification provided by both the supporting structure and the NSC.

Some of the dynamic characteristics and features of the buildings pertaining to the potential effect of TGMs and torsional response must be stated. It is noted that all of the three buildings possess nominally regular and symmetric configurations. Table 3 summarizes plan aspect ratios (W/B), floor radii of gyration squared ($r^2$), the longest fundamental periods and corresponding circular frequencies of vibration for the translational ($T_H$, $\omega_H$) and torsional modes ($T_R$, $\omega_R$), and the ratios of these frequencies ($\omega_R/\omega_H$). In general, the frequency ratios greater than 1.00 implies torsionally rigid structural systems for which the overall seismic response is expected to be governed by translational response. Despite the obvious differences in three buildings, the frequency ratios are approximately the same (~1.6). Furthermore, all of the buildings have zero static eccentricity ($e_x$ and $e_y$) between the center of rigidity (CR) and center of mass (CM), except for B3 with an *e*/r less than 5%. Therefore, B3 can also be presumed to be a symmetric structural system. Consequently, coupling of translational and torsional modes is not expected (as was verified from the mass participation factors), and torsional response will primarily be induced due to TGMs. On the other hand, the periods of vibration of the fundamental torsional modes are 0.55, 1.21, and 2.00 sec for B3, B9 and B20, respectively. As can be seen in Figure 3(a)-(c), the acceleration demand on these structures corresponding to their fundamental periods vary significantly. In particular, torsional acceleration demand essentially diminishes for B20, and it is the largest for B3.

### 2.4.1 Absolute Floor Acceleration

The variation of floor absolute acceleration amplification (PFA/PGA) along the height of each building is presented for the cases with and without TGM in Figure 5. The average PFA/PGA was determined as the ratio of peak resultant absolute floor acceleration to corresponding peak resultant ground acceleration (PFA/PGA). The comparisons are shown at the corner as well as at the CM of B3, B9 and B20, and with amplification factors implied by Equations (1) – (3).

The effect of TGM on the amplification of peak floor acceleration is evident in Figure 5. Accordingly, the increase in amplification factor due to TGM is approximately 7% at the CM of the buildings, whereas the corresponding increase are 33%, 23% and 12% when measured at the corner of the roof floor levels of B3, B9 and B20, respectively. Also listed in Table 4 are average amplification and corresponding standard deviations for the cases considered. It is apparent that the PFA at each floor decreases and the effect of TGM on

PFA diminishes as the fundamental period of the supporting structure increases. Furthermore, it is noted that all of the three buildings possess nominally regular and symmetric configurations, and the overall response is governed by the translational response. The acceleration amplification at the center and corner of a building essentially remain the same when TGM is not considered. On the other hand, peak floor acceleration response increases from CM to the edges of a given floor level due to TGM, and this effect reduces for relatively flexible buildings. Therefore, floor acceleration-induced force demand on NSCs located near the outside boundaries of a floor is expected to be higher due to TGMs even when the supporting building structure is regular and symmetric. Furthermore, the observations confirm earlier studies (e.g. Wieser et al. 2012) that the horizontal floor accelerations are dependent on the period, relative height, as well as modal characteristics of the structures. Clearly, Equation (1) (ASCE 7-16) leads to highly conservative NSC design forces housed in relatively long period structures ($T \geq 0.50$ sec) Pekcan [2003] even when the TGMs are considered explicitly.

### 2.4.2 Effect of Period of Supporting Structures

In order to show the influence of the dynamic characteristics of the supporting structure on the acceleration response of flexible nonstructural components, the normalized total floor acceleration spectra (FSA/PGA) of each building responding elastically are presented in Figure 6. The resultant absolute floor acceleration responses for each case (with and without TGM) was used to generate the 5%-damped average response spectra. The fundamental translational modal periods (Mode 1 & 2, Mode 5) and the fundamental torsional modal periods (Mode 3) of the supporting structure are highlighted in these plots, and $T_p$ refers to the fundamental period of NSC.

As can be seen in the figure, the peaks in the total acceleration amplification spectra coincide generally with the translational modal periods of the supporting structure. The slight deviation of the peak from the modal periods of the supporting structure is attributed to the effect of the bidirectional nature of the total acceleration amplification spectra and to the slight variation in modal frequencies in each orthogonal direction. Furthermore, the total acceleration amplification at the fundamental period of the supporting structure is reduced as the period of the structure increases. Table 5 summarizes the peak floor acceleration amplification for the first five modes for the buildings along with the ratio of amplifications calculated for with and without TGM cases. The ratios suggest that the acceleration amplifications due to TGM at periods corresponding to fundamental translational periods can be up to 82% higher than those when TGM is not considered. This effect appears to reduce drastically for flexible symmetric structures. However, it is important to note from Figure 6 that the effect of TGM persists in the shorter period range ($T_p \leq 0.5$ sec; hence for relatively rigid NSCs) of all of the buildings, as the amplifications due to TGM remain significantly higher than those without TGM.

### 2.4.3 Story Displacements and Forces

Figure 7 shows ratio of various response quantities for the cases with and without TGMs. Accordingly, the drift demand is amplified by 10% due to TGM in the relatively stiff building and it reduces up to 1% when the structure flexibility increases.

As can be seen in Figures 7(b) and 7(c), story shear forces and overturning moments amplify approximately 10% to 15% for the relatively rigid 3-story SMRF. The apparent effect of TGM on story shear force and overturning moment demand reduces gradually for flexible structures. As it was noted earlier, all three buildings are nominally symmetric and consideration of 5% accidental eccentricity would certainly account for the observed increase in deformation and force demands, however only marginally for B3. It is anticipated that the effect of TGM in terms of these response quantities may become more evident for stiff structural systems such as braced frames. Finally, Figure 7(d) suggest that the most significant effect of TGM is on the torsional moment demand on columns. While the torsional moments due to horizontal seismic loading are insignificant for symmetric structures, TGMs impose explicit demand particularly in the lower frequency range for the so-called torsionally riding buildings with relatively short-period torsional modes.

## 2.5. CONCLUDING REMARKS

The effect of TGM on the overall seismic response of special moment resisting frame buildings with varying heights were investigated under the scope of this study. The particular focus was on the preliminary assessment of additional floor acceleration demand placed on nonstructural components (NSC) due to the torsional components of ground motions (TGM). For this purpose, computational models of three- (B3), nine- (B9) and twenty-story (B20) SAC buildings were developed. It is noted that the SAC buildings are nominally symmetric with regular plan configuration. The buildings were subjected to two horizontal components of ground motions with and without TGMs. The torsional ground motions were generated using the Single Station Procedure (SSP), which was introduced recently by Basu et. al [2012a].

The results indicate that the effect of TGM on the floor acceleration may be significant for torsionally rigid structures with relatively short torsional period of vibration ($\leq$ 0.50 sec).

The increase in acceleration demand may be up to 34% for B3. It was also observed that the amplification of acceleration is higher at the corner than the center of mass. In general, amplification factor depends largely on the fundamental period of the structure and smaller floor acceleration has been observed for the relatively flexible B6 and B20 buildings. However, TGMs may still have a significant impact on rigid NSCs as it can be inferred from the floor acceleration spectra. In other words, while the floor acceleration spectra evaluated for B9 and B20 suggest insignificant amplification for flexible NSCs, corresponding amplifications for relatively rigid NSCs ($T_p \leq 0.5$ sec) can be in excess of 50% higher due to TGMs.

The amplification of story displacement, shear force and overturning moment demand due to TGMs is insignificant. However, the torsional moment demand increases significantly beyond the levels that lateral seismic forces would imply when only the translational components of ground motions are considered.

Finally, it is noted that the observation and conclusion on this preliminary investigation are based on only three symmetric SMRF building models, and on linear elastic response analyses. Further analyses that consider nonlinearity, irregular plan configurations, range of fundamental period of vibrations and, different seismic force resisting systems are necessary for a more comprehensive understanding about the effect of TGM on the structural response.

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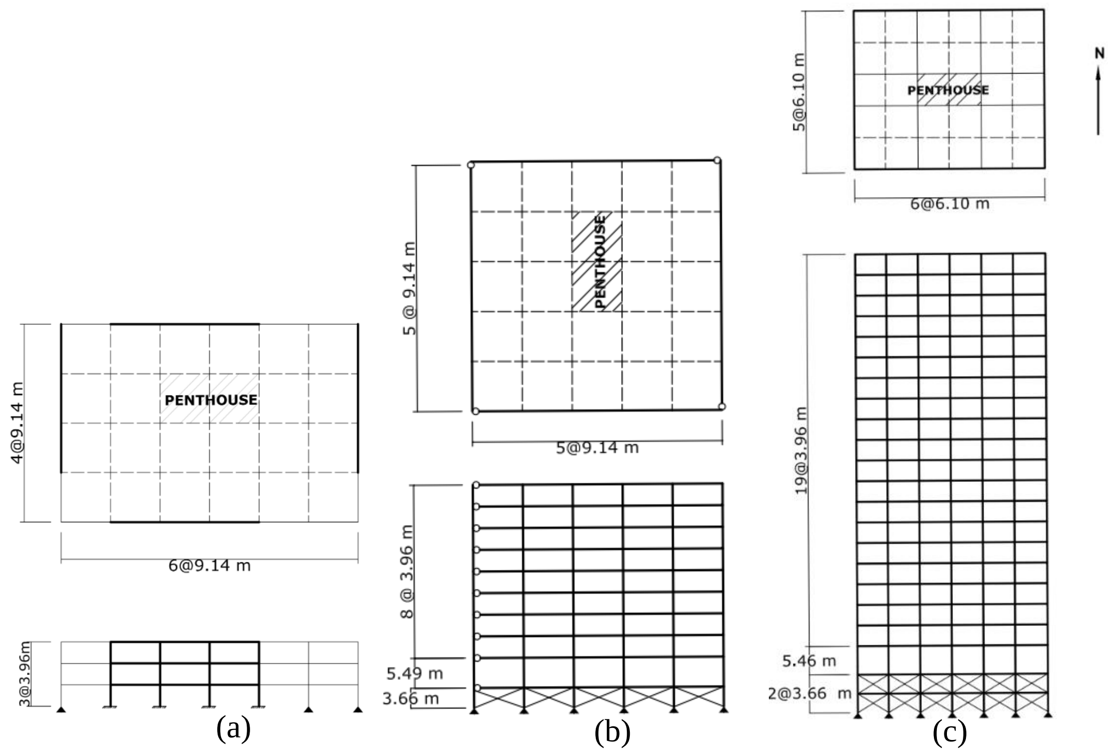


**Figure 1**. (a) 3-story (B3) (b) 9-story (B9) (c) 20-story (B20) SAC buildings

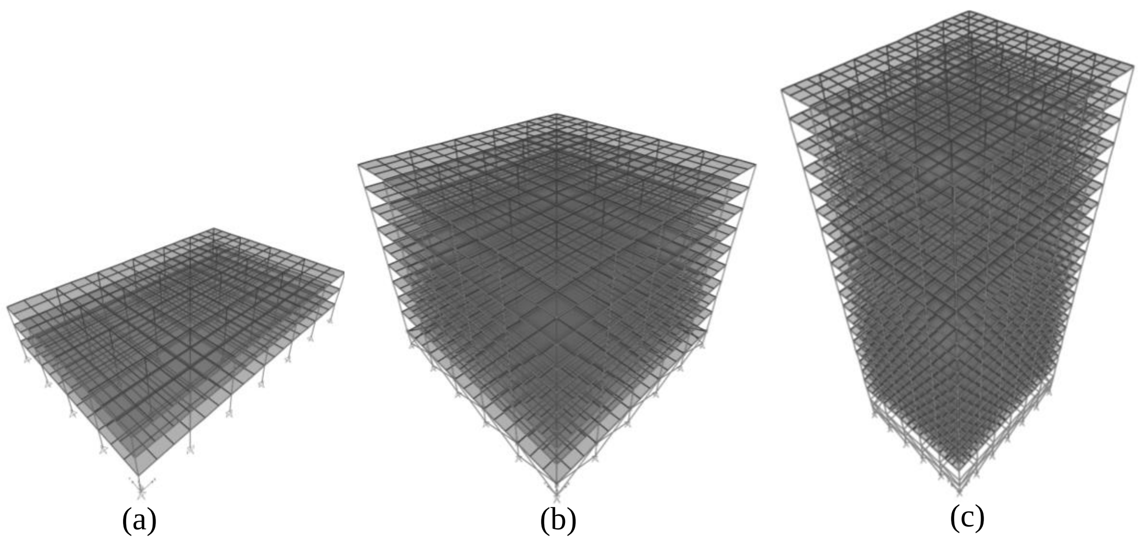


**Figure 2.** Three-dimensional view of (a) 3-story (B3) (b) 9-story (B9) (c) 20-story (B20) SAC buildings models

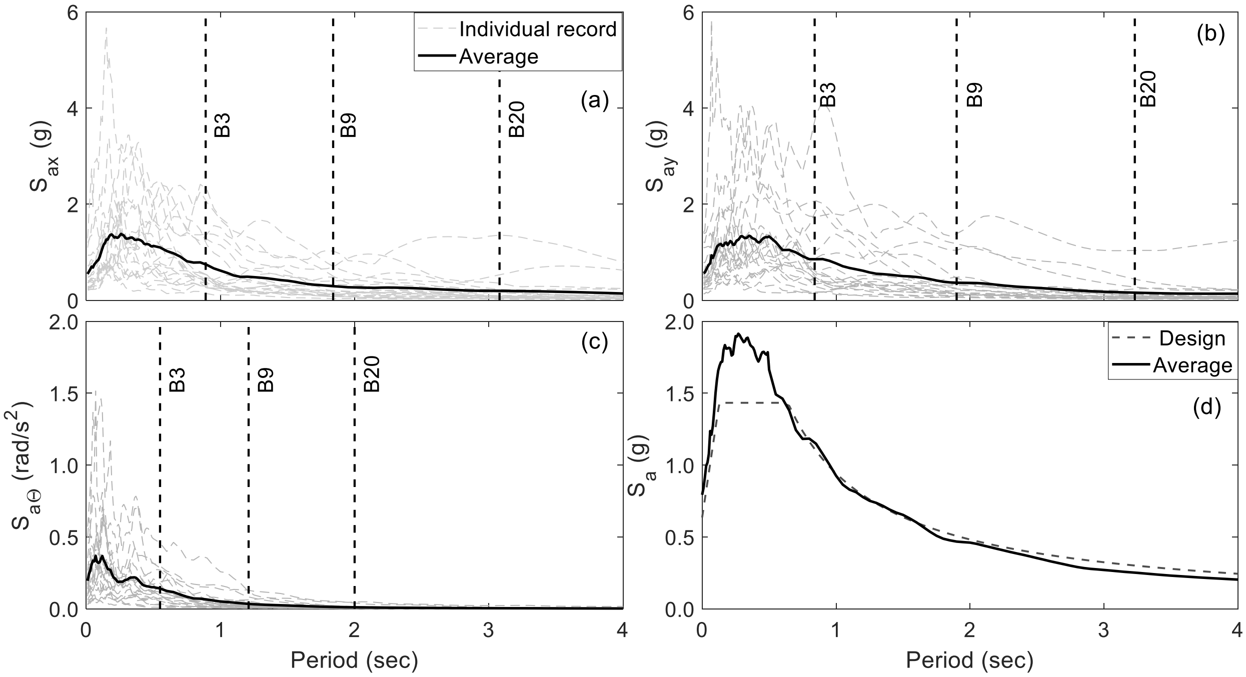


**Figure 3.** 5% damped response spectra (a) *x*, (b) *y*, (c) torsional (d) matched 5% damped design and average SRSS spectra

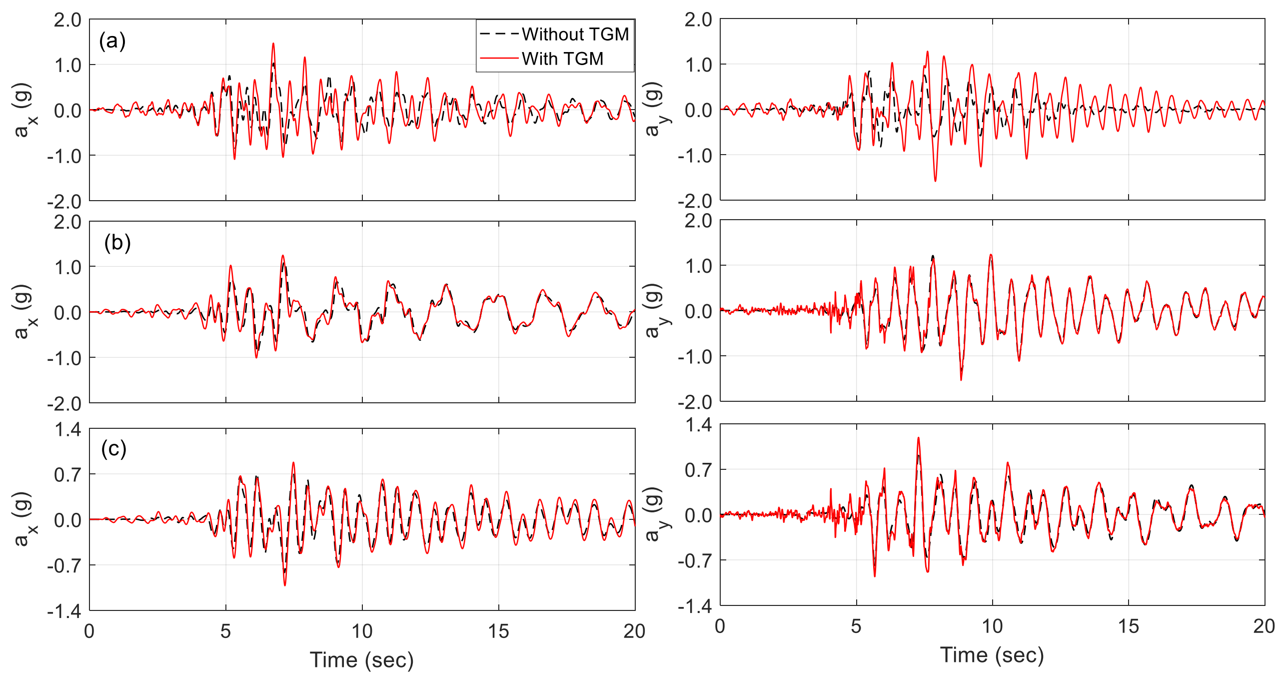


**Figure 4**. Roof absolute acceleration response histories with and without TGM (a) B3, (b) B9, (c) B20

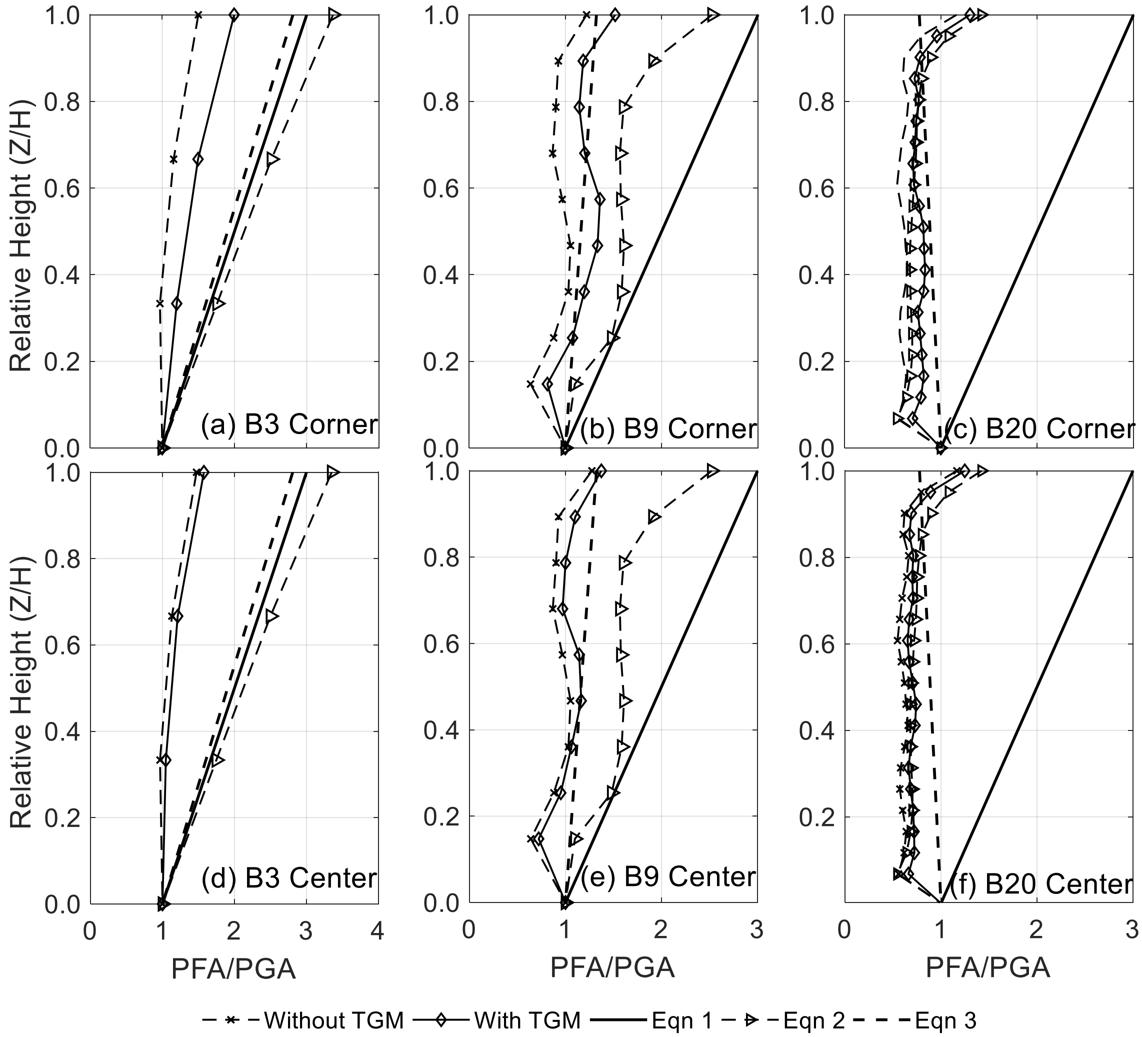


**Figure 5**. PFA/PGA for B3, B9 and B20 at the center of mass and corner of floor diaphragm

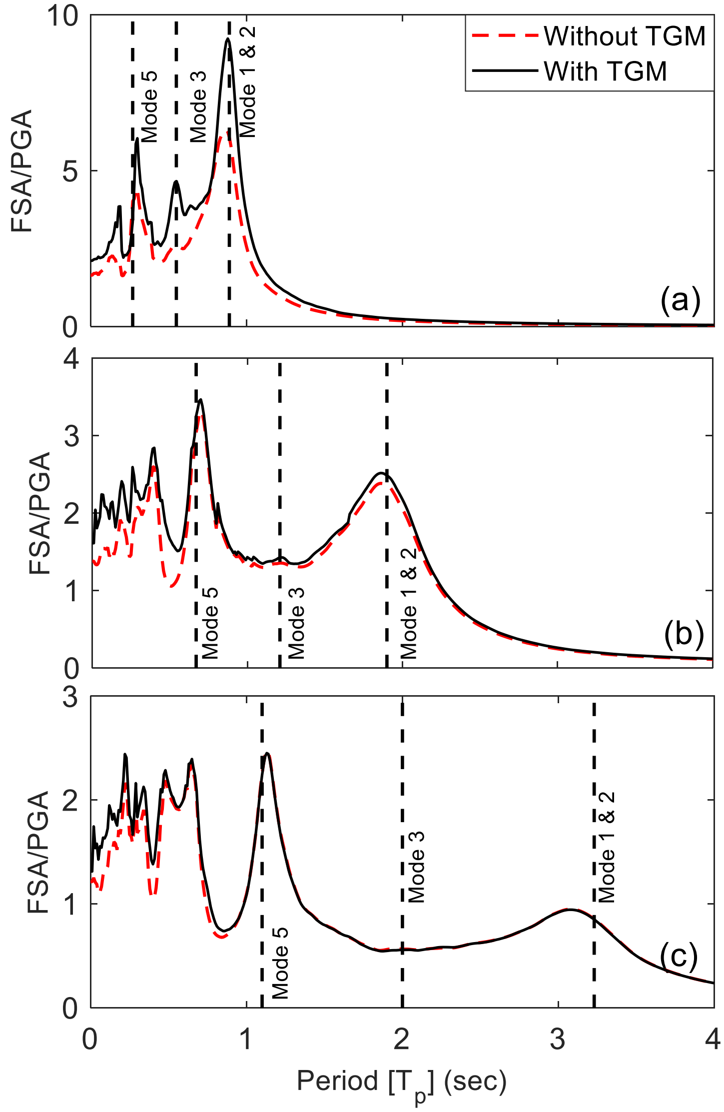


**Figure 6**. Peak floor acceleration amplification factor (PFA/PGA) (a) B3, (b) B9 and (c) B20

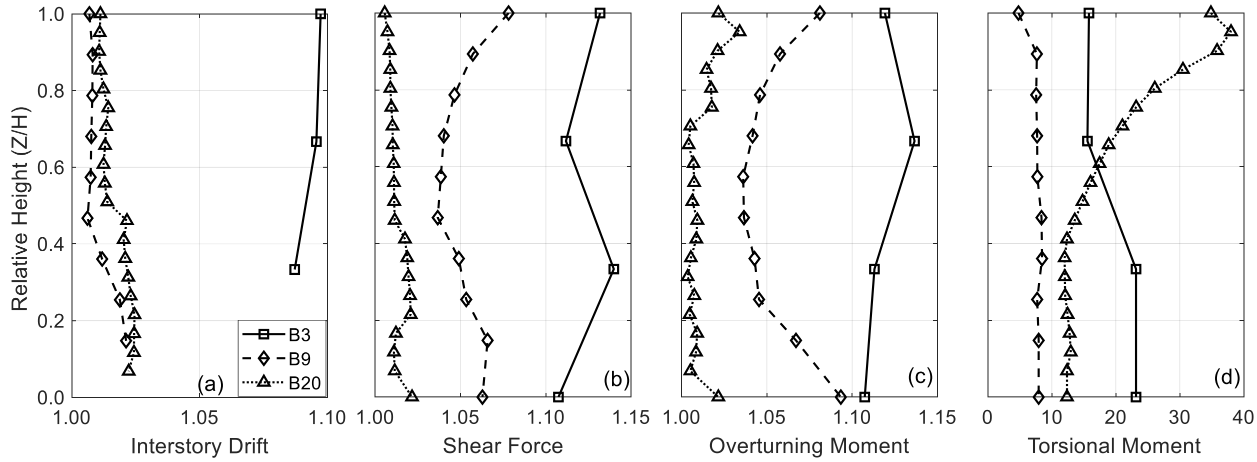


**Figure 7**. Ratio of Maximum Average (a) Interstory Drift (b) Story Shear (c) Overturning Moment and (d) Torsional Moment with and without TGM

**Table 1**. Fundamental periods and mode shapes

| Mode | B3 | | B9 | | B20 | |
|---|---|---|---|---|---|---|
| | Period (sec) | Description | Period (sec) | Description | Period (sec) | Description |
| 1 | 0.89 | X | 1.90 | Y | 3.23 | Y |
| 2 | 0.84 | Y | 1.84 | X | 3.08 | X |
| 3 | 0.55 | RZ | 1.21 | RZ | 2.00 | RZ |
| 4 | 0.29 | X | 0.70 | Y | 1.14 | Y |
| 5 | 0.27 | Y | 0.67 | X | 1.10 | X |
| 6 | 0.22 | X | 0.45 | RZ | 0.73 | RZ |

X: Translational in EW; Y: Translational in NS; RZ: Torsional

**Table 2**. Details of the selected ground motion records

| ID | Earthquake | Recording Station | PGA (g) | PGV (cm/s) | $V_s$ (m/s) | $V_P$ (m/s) | $R_L$ (Km) | $R_V$ (Km/s) | HCD (km) | SF |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Northridge | Beverly Hills-Mulholland | 0.52 | 62.8 | 356 | 617 | 18.00 | 2.90 | 17.50 | 1.6 |
| 2 | Northridge | Canyon Country - WLC | 0.48 | 44.9 | 309 | 535 | 18.00 | 2.90 | 17.50 | 1.0 |
| 3 | Duzce, Turkey | Bolu | 0.82 | 62.1 | 326 | 565 | 46.80 | 2.80 | 14.00 | 1.5 |
| 4 | Hector Mine | Hector | 0.34 | 41.7 | 685 | 1186 | 44.00 | 2.20 | 14.80 | 1.0 |
| 5 | Imperial Valley | Delta | 0.35 | 33.0 | 275 | 476 | 50.00 | 2.70 | 9.96 | 1.0 |
| 6 | Imperial Valley | El Centro Array #11 | 0.38 | 42.1 | 196 | 339 | 50.00 | 2.70 | 9.96 | 1.0 |
| 7 | Kobe, Japan | Nishi-Akashi | 0.51 | 37.3 | 609 | 1055 | 60.00 | 2.70 | 17.90 | 1.0 |
| 8 | Kobe, Japan | Shin-Osaka | 0.24 | 37.8 | 256 | 443 | 60.00 | 2.70 | 17.90 | 1.0 |
| 9 | Kocaeli, Turkey | Duzce | 0.36 | 58.9 | 276 | 478 | 137.5 | 3.00 | 16.00 | 3.0 |
| 10 | Kocaeli, Turkey | Arcelik | 0.22 | 39.6 | 523 | 906 | 137.5 | 3.00 | 16.00 | 1.0 |
| 11 | Landers | Yermo Fire Station | 0.25 | 51.4 | 354 | 613 | 71.80 | 2.80 | 7.00 | 3.0 |
| 12 | Landers | Coolwater | 0.42 | 42.3 | 271 | 469 | 71.80 | 2.80 | 7.00 | 1.0 |
| 13 | Loma Prieta | Capitola | 0.53 | 35.0 | 289 | 501 | 40.00 | 2.80 | 17.48 | 1.0 |
| 14 | Loma Prieta | Gilroy Array #3 | 0.56 | 44.7 | 350 | 606 | 40.00 | 2.80 | 17.48 | 1.0 |
| 15 | Manjil | Abbar | 0.52 | 52.1 | 724 | 1254 | 71.70 | - | 16.00 | 2.5 |
| 16 | Superstition Hills | El Centro Imp. Co | 0.36 | 46.4 | 192 | 333 | 20.00 | 2.50 | 9.00 | 1.0 |
| 17 | Chi-Chi, Taiwan | CHY101 | 0.44 | 115 | 259 | 449 | 88.00 | 2.80 | 8.00 | 3.5 |
| 18 | Chi-Chi, Taiwan | TCU045 | 0.51 | 39.1 | 705 | 1221 | 88.00 | 2.80 | 8.00 | 1.0 |
| 19 | San Fernando | LA-Hollywood | 0.21 | 18.9 | 316 | 547 | 20.00 | 2.50 | 13.00 | 1.0 |
| 20 | Friuli | Tolmezzo | 0.35 | 30.8 | 425 | 736 | 13.00 | - | 5.10 | 1.0 |

**Table 3**. Peak floor acceleration amplification with and without TGM

| | Plan aspect ratio W/B | Radius of gyration ($m^2$) $r^2$ | Translational Period (sec) Frequency (rad/sec) $T_H$ ($\omega_H$) | Torsional Period (sec) Frequency (rad/sec) $T_R$ ($\omega_R$) | Frequency ratio $\omega_R/\omega_H$ |
|---|---|---|---|---|---|
| B3 | 1.50 | 362 | 0.89 (7.06) | 0.55 (11.4) | 1.62 |
| B9 | 1.00 | 349 | 1.90 (3.31) | 1.21 (5.19) | 1.57 |
| B20 | 1.20 | 189 | 3.23 (1.95) | 2.00 (3.14) | 1.62 |

**Table 4**. Selected PFA/PGA at the corner and corresponding standard deviations

| Z/H | B3 | | | B9 | | | B20 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Without TGM | With TGM | With/ Without | Without TGM | With TGM | With/ Without | Without TGM | With TGM | With/ Without |
| 1.00 | 1.50 (0.53) | 2.00(0.78) | 1.33 | 1.22(0.38) | 1.52(0.44) | 1.24 | 1.16(0.19) | 1.3(0.24) | 1.12 |
| 0.67 | 1.16(0.35) | 1.50(0.51) | 1.29 | 0.88( 0.17) | 1.22(0.24) | 1.39 | 0.58(0.17) | 0.72(0.22) | 1.25 |
| 0.33 | 0.97(0.18) | 1.20(0.42) | 1.24 | 0.99 (0.26) | 1.16(0.33) | 1.18 | 0.59(0.14) | 0.78(0.17) | 1.32 |

**Table 5**. Peak floor acceleration amplifications with and without TGM

| Mode | B3 | | | B9 | | | B20 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Without TGM | With TGM | With/ Without | Without TGM | With TGM | With/ Without | Without TGM | With TGM | With/ Without |
| 1 | 6.07 | 9.12 | 1.50 | 2.36 | 2.48 | 1.05 | 0.85 | 0.86 | 1.01 |
| 2 | 6.14 | 7.98 | 1.30 | 2.37 | 2.50 | 1.06 | 0.94 | 0.95 | 1.00 |
| 3 | 2.56 | 4.66 | 1.82 | 1.35 | 1.42 | 1.05 | 0.56 | 0.57 | 1.01 |
| 4 | 4.36 | 5.86 | 1.34 | 3.32 | 3.46 | 1.04 | 2.44 | 2.46 | 1.01 |
| 5 | 1.73 | 2.29 | 1.32 | 3.04 | 3.25 | 1.07 | 1.15 | 1.16 | 1.01 |

# 3. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

This chapter includes a brief summary of the thesis, conclusion drawn and the recommendations for the future work to enhance the knowledge on the effect of torsional ground motion on the response of the structures.

## 3.1. Summary

There is a knowledge gap to clearly understand the effect of the torsional ground motion on the response of the structures. Past studies are based on highly simplified and idealized uniform elastic bars, shear beam model or single story models. There is a large uncertainty in the floor acceleration demand of the buildings due to the TGM and the effect is not explicitly considered in the design code. In an effort to bridge the gap, the three full scaled building models were developed based on the SAC steel building project to investigate the effect of TGM on the flexible steel buildings.

The present study helps to understand the real effect of the TGM on the structures based on full scaled models. It was observed the floor acceleration demand of building can increase by as much as 33% for relatively stiff structures and the amplification varies with a floor. Higher amplification was observed at the corner of the floor in comparison to the center. The increment in the floor acceleration significantly depends on the flexibility of the structure and can have high impact on the force experienced by the nonstructural components. The study also shows that the peak floor acceleration amplification increases noticeably when TGM is considered in the analysis.

### 3.2. Conclusion

The effect of the overall seismic response of the moment resisting frame building that have different periods were investigated. For that, three dimensional analytical models of 3, 9 and 20 story building designed for the Los Angeles area under the scope of post-Northridge SAC project was chosen and analyzed using SAP2000 v18.1.1. Twenty far field ground motions were selected for the analysis purpose and based on the guidelines of ATC-63 FEMA and series of response history analyses were performed. Single Station Procedure (SSP) which was proposed by Basu et. al [2012a] was used to generate the rotational component of the ground motions. Following conclusion were drawn from the study:

- The effect of TGM may be significant for torsionally rigid structures with relatively shorter torsional period of vibration ($\leq 0.50$ sec).

- The effect of the TGM on the floor acceleration is significant and it can increase the acceleration demand as high as 34%.
- The acceleration amplification due to TGM varies within a floor and the amplification factor is higher at the corner in comparison to the center of the floor diaphragm.
- In general, amplification factor largely depends upon the fundamental period of the structure and smaller floor acceleration has been observed for the relatively flexible B6 and B20 buildings.
- The peak of acceleration amplification spectra coincides with the modal periods of the supporting structure and the acceleration amplification reduces for the fundamental period as the natural period of the structure increase.
- While the floor acceleration spectra evaluated for B9 and B20 suggest insignificant amplification for flexible NSCs, corresponding amplifications for relatively rigid NSCs ($T_p \leq 0.5$ sec) can be in excess of 50% higher due to TGMs

## 3.3. Recommendations for future work

While it is clear that the effect of torsional ground motion significantly depends upon the period of the structures, present study is based on only three analytical models so following recommendations can be drawn from the study:

- To have a clear understanding on the effect of TGM further analysis that considers nonlinearity, irregular plan configurations, range of fundamental period of vibrations and, different seismic force resisting systems are necessary.

- The response of structure under TGM can depend upon the structural framing system and the building materials. The analysis can be done for different seismic resisting system with same period and the effect can be compared.