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THE BENEFITS OF LINEAR TIME-HISTORY ANALYSIS IN CONSULTING PRACTICE AND THE PRECISION OF THE CODE CALCULATED PERIOD

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Abstract

Seismic design of structural systems has long been recognized as the component of the design posing the highest levels of risks, challenges and uncertainties to the design. A common code-based seismic design approach for determining the seismic loads for regular structures is the equivalent static force procedure (ESFP). Like all simplified methods, the ESFP has limitations. Some of the limitations of the ESFP include the estimation of the fundamental period of the structure, excluding the contribution of higher modes, deficiency in providing means for calculating response quantities other than force and displacements, and failure to account for the multi-directionality nature of seismic events.

Linear dynamic methods of analysis such as linear modal response spectrum analysis (MRSA) and linear time history analysis (LTHA) have been used to reduce the uncertainties and reduce the limitations associated with the ESFP. Although practicing time history analysis has been commonly incorporated in research, it is not fully recognized as an effective analysis method by structural engineers. The increased processing power of computing hardware and the availability of progressive computer aided design tools has diminished the computational effort involved in performing time history analysis for the design and retrofit of structures. Therefore, time history analysis can now be incorporated in the industry as one of the primary methods of analysis, leading to more economical and efficient designs.

This paper outlines the benefits of incorporating LTHA, as the primary method of analysis, in seismic design. The emphasis is on regular structures, for which the codes of practice do not require a dynamic analysis for. First a large building database has been studied. The empirical periods, as suggested by the codes of practice, and the actual analytical periods are compared and evaluated. Next, available guidelines for adopting a linear time history analysis as the method of analysis and the selection and scaling of ground motions, provided by the codes of practice, are reviewed. The results of 4 case study buildings analyzed and retrofitted using LTHA is presented. The buildings vary in sizes and lateral load resisting systems (LLRS) and were designed according to the provisions of the 2010 National Building Code of Canada (NBCC 2010) [1]. The seismic base shears obtained by doing a LTHA are compared to the corresponding values determined from code-based ESFP and MRSA. Finally, the advantages and limitations of doing a LTHA are evaluated.

Keywords: Seismic Design; Linear Time-History Analysis; Period of Vibration; Ground Motion Scaling; Case Studies

1. Introduction

The common approach to the seismic design of regular structures is the ESFP as described by design standards such as the NBCC 2015 [2], ASCE 7-10 [3], etc. The complex nature of earthquake excitations, uncertainty in determining the uniform hazard spectra (UHS) as well as the complex inelastic behavior of structural systems under extreme loads has made the task of providing a set of general, but effective, guidelines for the ESFP a challenge. The codes of practice often provide conservative guidelines to overcome these challenges and uncertainties. In many areas of low to moderate seismic hazard, a conservative approach to seismic design is justified. However, in areas of high seismic hazard and for existing or sensitive structures these assumptions can be overly restrictive. For instance, as shown in subsequent sections, the empirical expressions for predicting the structures fundamental period is, in many cases, conservative. MRSA has been widely used as an alternative method for the seismic design of structural systems, considered to provide more realistic results. However, previous studies have shown that using a MRSA has limitations.

With increasing computational power over the past decades, advanced methods of analysis can be used more widely for the seismic design of structures in practice. Reducing the uncertainties and some of the conservatism associated with the code-based ESFP, will reduce the design earthquake loads and will lead to a more economical design. For this purpose, time-history analysis can provide a useful alternative for the seismic design of structures. For the design of regular structures, where the inelastic behavior is somewhat predictable, nonlinear time-history analysis (NTHA) may be unnecessary and computationally expensive. For such structures, LTHA can provide significant improvement to the design within the framework of the codes of practice, with limited effort.

This paper investigates the benefits of incorporating LTHA in the design of structures in engineering practice. The study outlines the limitations associated with using the code-based ESFP and MRSA methods. In the first section of the study, the precision of the code-calculated fundamental period is evaluated, using a large building database. The NBCC 2010 [1], NBCC 2015 [2] and ASCE 7-10 [3] guidelines for conducting a LTHA, including the selection and scaling of ground motion time histories, the number of selected records, and limits on the response quantities are reviewed and presented. The analysis results of 4 case study buildings are compared by performing ESFP, MRSA, and LTHA. Throughout the study, the emphasis is on regular structures for which the codes of practice do not require a dynamic analysis. Finally, the advantages of using a LTHA as the primary method of analysis are presented.

2. Precision of the Code Calculated Fundamental Period

Predicting the design loads imposed on structures by earthquake excitations is directly related to the dynamic characteristics of the structure. The code-based ESFP uses the first mode of vibration and assumes that the total response is within the first mode. For determining the fundamental period of the structure, empirical expressions are provided by the standards.

NBCC 2015 [2] provides the following equations:

$$T_a = 0.085(h_n)^{3/4} \quad \text{for steel moment frames} \tag{1}$$

$$T_a = 0.075(h_n)^{3/4} \quad \text{for concrete moment frames} \tag{2}$$

$$T_a = 0.1N \quad \text{for other moment frames} \tag{3}$$

$$T_a = 0.025(h_n) \quad \text{for braced frames} \tag{4}$$

$$T_a = 0.05(h_n)^{3/4} \quad \text{for shear walls and other structures} \tag{5}$$

where T_a is the structures fundamental period, h_n is the height of the structure in meters, and N is the number of stories.

NBCC 2015 [2] allows the use of fundamental period from an analytical model, however, limits the period to $1.5T_a$, for moment resisting frames, to $2.0T_a$, for braced frames and concrete shear walls, and to T_a, for other structures in Eq. (5).

The empirical expressions for determining the fundamental period of vibration, given by ASCE 7-10 [3], are as follows:

$$T_a = 0.0724(h_n)^{0.8} \quad \text{for steel moment frames} \tag{6}$$

$$T_a = 0.0466(h_n)^{0.9} \quad \text{for concrete moment frames} \tag{7}$$

$$T_a = 0.0731(h_n)^{0.75} \quad \text{for steel braced frames} \tag{8}$$

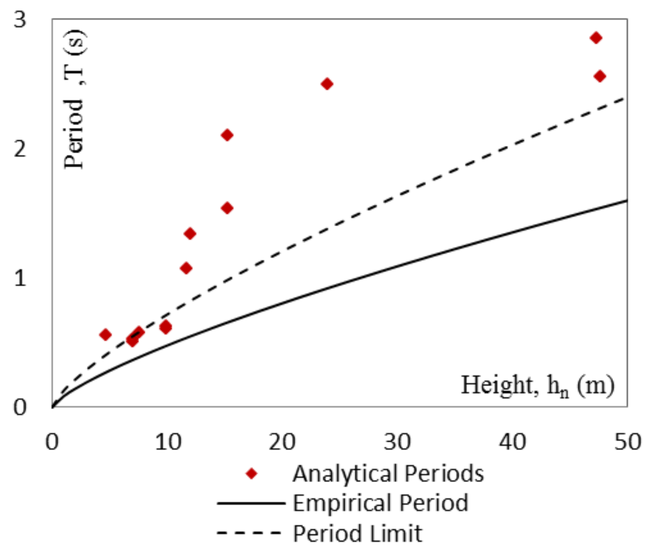
$$T_a = 0.0488(h_n)^{0.75} \quad \text{for all other structural systems} \tag{9}$$

$$T_a = 0.1N \qquad \text{for concrete or steel LLRS} \qquad (10)$$

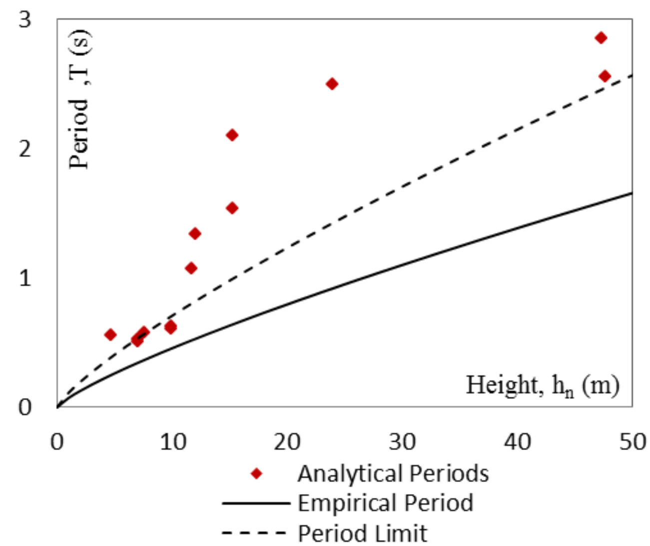
where T_a is the fundamental period of vibration, h_n is the height of the structure in meters, and N is the number of stories. Eq. (10) can be used for structures not exceeding 12 stories where the seismic force resisting system (SFRS) consists entirely of reinforce concrete or steel and the average story height is at least 3.0 meters.

ASCE 7-10 [3] limits the period obtained from an analytical model to a range from 1.4 to 1.7 T_a.

To assess the precision of the code calculated period, analytical models of 50 buildings were developed and their analytical periods were determined. The building database consists of 13 steel moment resisting frames, 21 steel braced frames, 20 reinforced concrete moment resisting frames, 26 reinforced concrete shear wall structures and 20 other structural systems including wood frame buildings and frame buildings with unreinforced masonry infill walls. The height of the buildings in the database varies from 3.84 meters to 47.7 meters. Each building was analyzed in both orthogonal directions, for a total of 100 calculated periods. The analytical periods of the structural systems versus the corresponding empirically determined periods given by NBCC 2015 [2] and ASCE 7-10 [3], including the code upper limits are presented in Fig. 1 to Fig. 5.



(a)

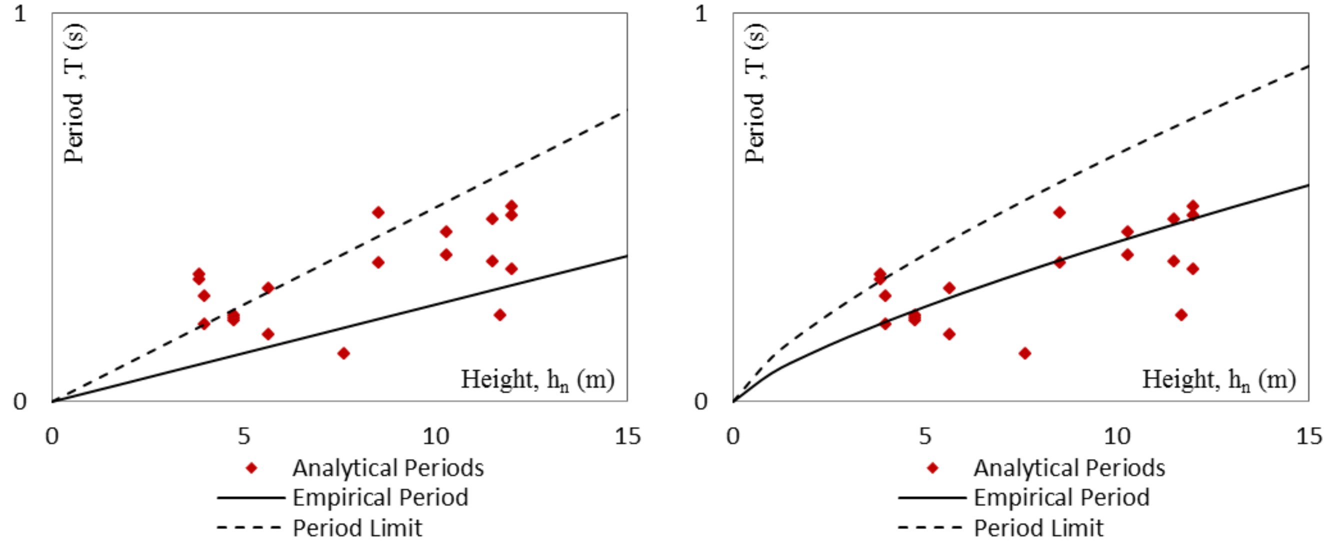


(b)

Fig. 1 –Steel moment resisting frames: analytical periods versus code empirical expression and the upper limits; (a) NBCC 2015, (b) ASCE 7-10

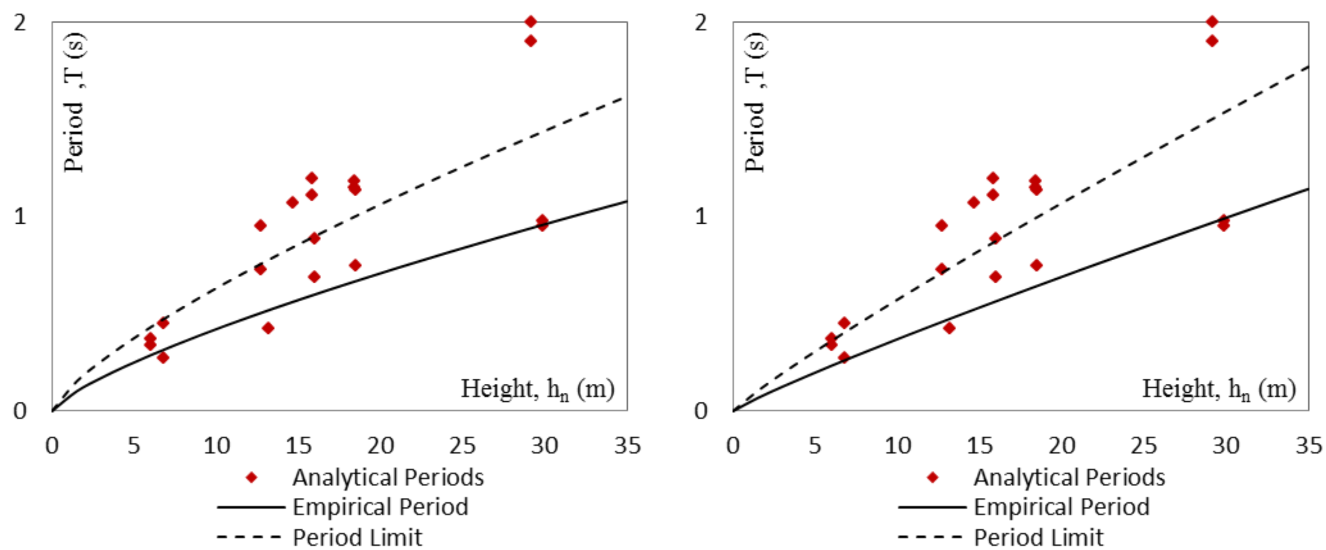
The results reported in this paper are part of the findings of an ongoing study for which the building database is still being updated and completed. Although the results at this stage cannot be used to propose methods for calculating the fundamental period of structural systems, they can certainly be used to assess the precision of the fundamental periods obtained from the empirical expressions proposed by NBCC 2015 [2] and ASCE 7-10 [3].

The limit imposed on the fundamental period by ASCE 7-10 [3], is dependent on the design spectral acceleration value at one second. The limiting factor C_u ranges from 1.4 to 1.7. In the current study, a value of 1.55 is assumed as the average value of C_u through Fig. 1 to 5.



(a) (b)

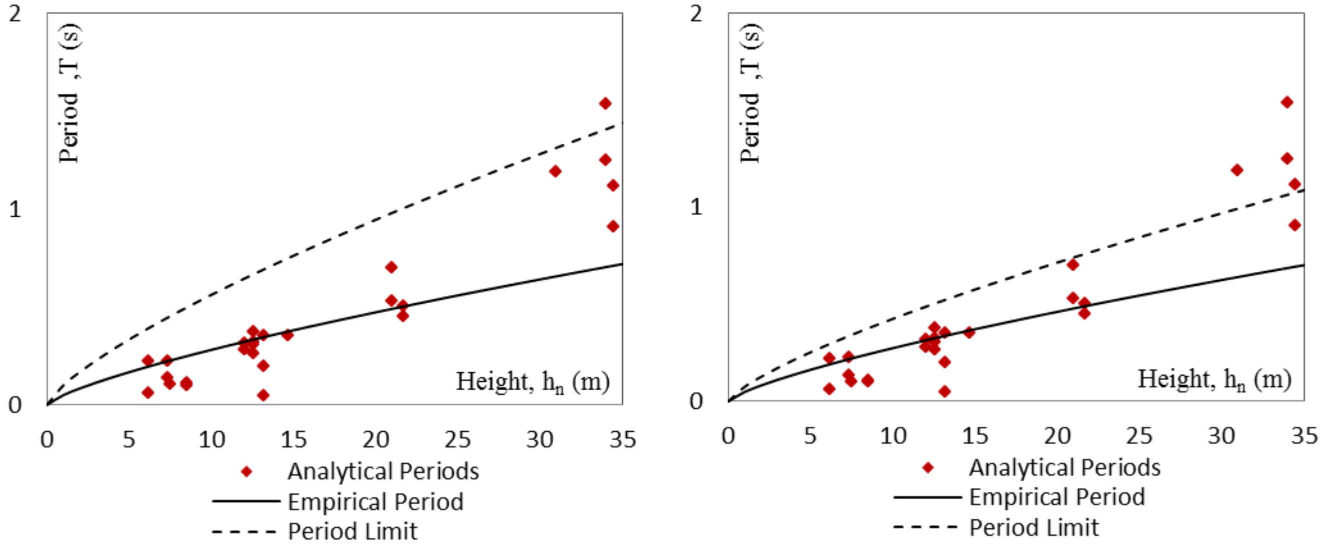
Fig. 2 – Steel braced frames: analytical periods versus code empirical expression and the upper limits; (a) NBCC 2015, (b) ASCE 7-10



(a) (b)

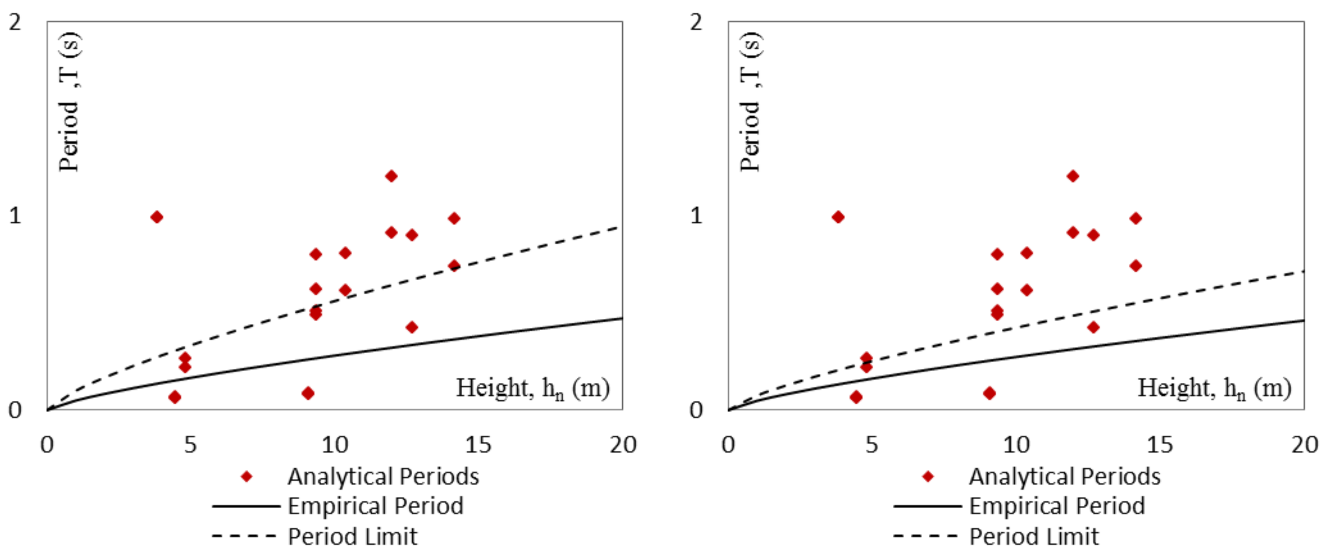
Fig. 3 – Reinforced concrete moment resisting frames: analytical periods versus code empirical expression and the upper limits; (a) NBCC 2015, (b) ASCE 7-10

In Fig. 1 to 5, the empirical equations for calculating the fundamental period of structural systems are illustrated with solid lines, while dashed lines are used to show the limits imposed on the fundamental period by the corresponding codes. For data points that lie within the two curves, the code-based calculated period is an accurate representation of the analytical period. For data points lying above the dashed lines, the code calculated period is lower than the actual period and therefore the design base shear will be overestimated, using the ESFP.



(a) (b)

Fig. 4 – Reinforced concrete shear wall structures: analytical periods versus code empirical expression and the upper limits; (a) NBCC 2015, (b) ASCE 7-10



(a) (b)

Fig. 5 – Other structural systems including wood frames and moment frames with unreinforced masonry infill walls: analytical periods versus code empirical expression and the upper limits; (a) NBCC 2015, (b) ASCE 7-10

Fig. 1 and 3 indicate that the code-based period provides conservative estimates for the fundamental period of steel and concrete moment resisting frames. Both codes give fair estimates for the fundamental period of steel braced frames and concrete shear walls, as shown in Fig. 2 and 4. Yet, in all categories there are cases where the code fundamental period would be conservative, especially for taller buildings. The periods calculated for wood frame structural systems and frame buildings with unreinforced masonry infill walls, which lie in the category of "other structural systems" show the most deviations from the code calculated periods, as shown in Fig. 5. On average, both codes give conservative estimates for the fundamental period in more than one-third of the cases studied above. The seismic force is directly proportional to the fundamental period, and therefore, incorporating an accurate fundamental period of the structure in the seismic design can reduce the seismic loads significantly.

3. Limitations Associated with the Code-Based ESFP

The code-based equivalent static force procedure is based on a number of conservative assumptions, which tend to increase the seismic design base shear. Some of these assumptions including the building period estimation, failing to account for higher modes effects and the multi-directionality of earthquakes are described in the following sections.

3.1 Building period estimation

As shown in section 2, the code approach for calculating the fundamental period of the structure can be overly conservative. Accurate calculation of the fundamental period can significantly reduce the design earthquake loads on the structures, leading to more economical designs. Using either MRSA or a LTHA can make up for this limitation of the ESFP.

3.2 Contribution of higher modes

In irregular and tall structures, higher modes make a more significant contribution to the seismic response. In such structures, higher modes act to increase the seismic base shear. In regular structures, however, higher modes often act to decrease the seismic base shear. By accounting for the contribution of higher modes, in regular structures, the design earthquake load can decrease at times. It must be noted that whether the contribution of higher modes leads to an increase or a decrease in the seismic base shear, accounting for higher modes is the more accurate approach. Both MRSA and a LTHA take the contribution of higher modes into account and, therefore, make up for this limitation associated with the ESFP.

3.3 Multi-directionality nature of earthquakes

Earthquake excitations give rise to ground motions in all three Cartesian coordinates, X, Y and Z. For most structural systems, including regular structures which are the emphasis of this paper, vertical ground movement is not considered and only the horizontal components of the ground motions are considered in design. For structures in which the components of the SFRS are not oriented along the principal directions, the multi-directionality nature of earthquakes can lead to higher loads in the elements of the SFRS which are not oriented along two principal axes in plan. To account for this in design, NBCC 2015 [2] and ASCE 7-10 [3] recommend applying 100% of the calculated seismic load in one direction and 30% in the other. Such approach can be used both in ESFP and MRSA. For regular structures however, the multi-directionality aspect of earthquakes can reduce the lateral loads induced on the elements of the SFRS, since in regular structures, the orientation of the elements of the SFRS coincides with the buildings two principal axes.

The uniform hazard spectra are generated based on seismic events with a specific probability of exceedance during the service life of structures. NBCC 2015 [2] and ASCE 7-10 [3] provide uniform hazard spectra for the maximum considered earthquake (MCE) with 2% probability of exceedance in 50 years, or a return period of 2475 years. In the equivalent static force procedure provided by the codes of practice, the designer subjects the structural system to the design spectral acceleration along each principal axis and designs the structure for the lateral forces. This approach assumes that the earthquake peak ground acceleration (PGA) coincides with the orientation of the buildings principal axes. MRSA makes the same assumption, while accounting for the contribution of higher modes. By using this assumption, since the 2% in 50 years earthquake is applied in both orthogonal directions, the return period of the design earthquake would actually be much longer. In other words, the probability of occurrence for a seismic event which matches the UHS of the MCE and is oriented in the same directions as the structures principal axes is much lower than 2% in 50 years.

To better understand this, the ground acceleration of a number of historical seismic events are mapped in Fig. 6, in which the horizontal axis shows the ground acceleration in the X direction and the vertical axis shows the ground acceleration in the Y direction. The records are taken from the PEER ground motion database [4]. The ground accelerations for Chuetsu-oki Japan earthquake in 2007, Imperial Valley – 06 earthquake in 1979, Darfield New Zealand earthquake in 1979 and Chi Chi Taiwan seismic event in 1999, are shown in Fig. 6 (a), (b), (c), and (d), respectively. A circle with a radius equal to the PGA is also shown for each seismic event. Assuming that the buildings principal axes are oriented along the directions of the two horizontal accelerometers, it can be seen that the direction of the ground acceleration is random and the (PGA) does not coincide with either direction of the buildings principal axes.

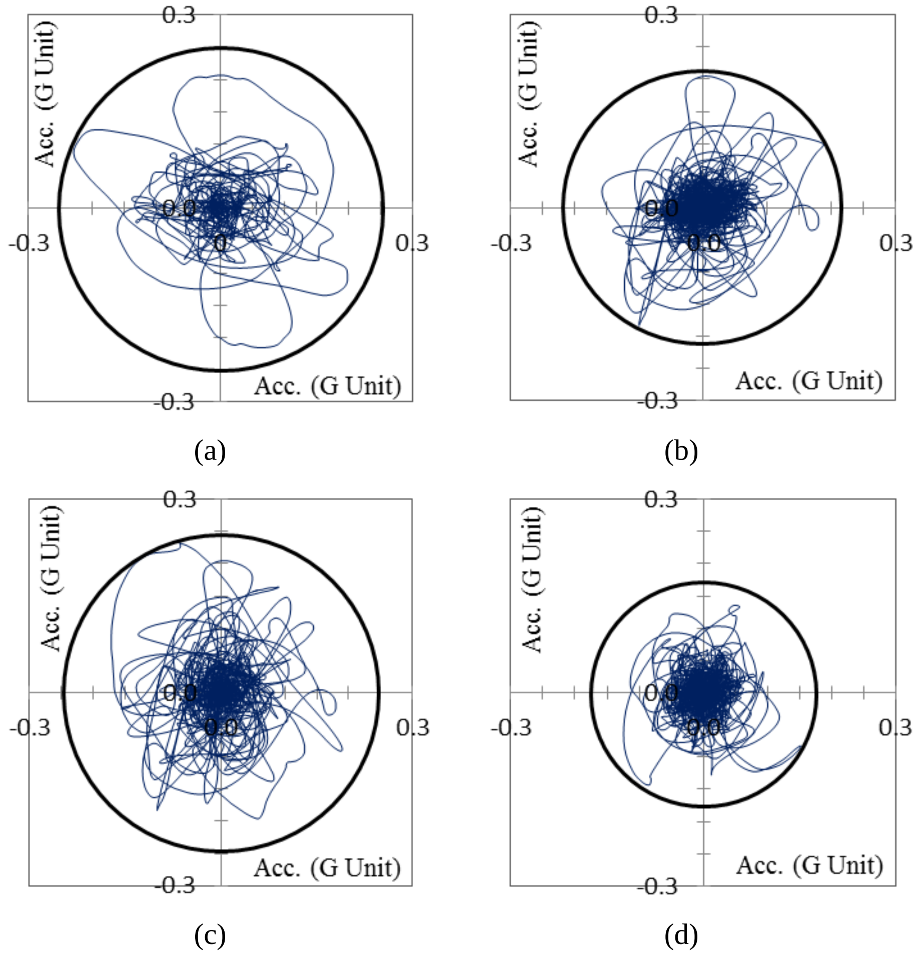


Fig. 6 – Ground accelerations during selected historical seismic events (a) Cheutsu-oki Earthquake, Japan, 2007, (b) Imperial Valley – 06 Earthquake, 1979, (c) Darfield Earthquake, New Zealand, 2010, and, (d) Chi Chi Earthquake, Taiwan, 1999.

By performing a three dimensional time history analysis and subjecting the structural system to a sufficient number of pairs of ground motions, matched to the design UHS, the multi-directionality aspect of earthquakes is taken into account. For irregular structures and structures with elements of the SFRS not oriented along the principal axes, this approach has the advantage that the building is tested in multiple directions. For regular structures, using a LTHA generally reduces the base shear in each principal direction, since the total base shear caused by the seismic event matching the design UHS now consists of two orthogonal components occurring concurrently along the buildings principal axes. It is important that the building is subjected to a sufficient number of ground motions to examine the structure in different directions. For instance, if a building is subjected to the Imperial Valley – 06 earthquake, shown in Fig. 6 (b), it will be subjected to the PGA in three different directions. Using another seismic event such as Cheutsu-oki earthquake, shown in Fig. 6 (a) will subject the building to the PGA in another direction. Using the mean response quantities for the design of the building obtained from multiple seismic events, as described above, leads to a design with reduction in the seismic load for regular structures.

3.4 Limited response quantities

The other limitation of the ESFP is that the outputs of such analysis are limited to force quantities and displacements. By performing a LTHA, the designer will have knowledge of other response quantities such as absolute acceleration, relative acceleration and velocity time histories. While these response quantities, obtained from a LTHA, are limited to the elastic response of the structure, they still give an understanding of the dynamic behavior of the structure. For instance, acceleration and velocity time histories can be used to assess the performance of equipment sensitive to these response quantities.

4. Limitations Associated with the Modal Response Spectrum Analysis

To make up for the limitations associated with the ESFP, MRSA has been widely used. By performing a MRSA, the elastic period of the structure is calculated accurately. Further, the contribution of higher modes is taken into account. Both of these reduce seismic loads in regular structures. While MRSA has proven to be a very computationally efficient improvement to the ESFP, it still uses approximations. Wilson (2010) [5] presents some of the limitations associated with this method. The limitations associated with the MRSA are outlined in this section.

4.1 Multi-directionality nature of earthquakes

To account for simultaneous earthquake motions, in buildings with non-orthogonal SFRS, design guidelines recommend applying 100% of the base shear obtained from the MRSA in one direction and 30% in the other. This is a tested approach to predict the lateral demand on members of the SFRS that are not oriented along the principal axes of the building. However, there are no methods to account for the reduction in base shear in regular structures as discussed above.

4.2 Limitation of response quantities

As was the case for the ESFP, MRSA does not provide the designer with any measure of acceleration and velocity time-histories that the structure will experience.

4.3 Estimation of modal peak response quantities

Modal response spectrum analysis accounts for the contribution of higher modes. In this method the maximum displacements and force response quantities associated with each mode of vibration are calculated, using the UHS. Afterwards, the response quantities are superimposed. For superimposing the results, the designer can use a number of methods. The classical and perhaps the most conservative approach is to use the absolute sum of the maximum response quantities obtained for each mode. This assumes that the peak response quantities in all modes happen at the same point in time, which evidently leads to very conservative design parameters. The alternative common approach is to use the Square Root of the Sum of the Squares, SRSS, of the peak response quantities in each mode of vibration. This approach assumes that the peak modal response quantities are statistically independent. For three dimensional structural systems in which a large number of periods of vibration are close, this assumption is not justified [5]. The third approach is to use the Complete Quadratic Combination, CQC, to add the peak modal response quantities. This method was first introduced by Wilson et al [6]. The key characteristic of the CQC method is the ability to distinguish the relative signs of peak modal responses and, therefore, eliminate some of the errors that will be observed by using absolute sum and SRSS methods.

Regardless of the method used to superimpose the results, the modal response spectrum analysis will produce positive maximum response quantities that are not a function of time. As a consequence a plot of the deformed shape of the structure under MRSA has little meaning and cannot reliably be used to assess nonstructural damage. Further, in the design of structural elements under combined axial load and bending moments, the maximum force quantities are taken into account, assuming that the maximum axial load and bending moments will occur at the same time. This can lead to conservative designs.

5. Provisions for Conducting a Linear Time-History Analysis

LTHA reduces the uncertainties, errors and conservatisms associated with the ESFP and MRSA, within the bounds of a linear elastic analysis. Design guidelines propose certain provisions for carrying out a LTHA and appropriate methods for the selection and scaling of ground motion time-histories. Some of the provisions provided by NBCC 2015 [2] and ASCE 7-10 [3] and NBCC 2010[1] are discussed below.

5.1 Linear time-history analysis procedure in ASCE 7-10

ASCE 7-10 [3] recommends using at least 7 seismic events (ground motion pairs) for doing a LTHA. If at least 7 ground motion pairs are used, the average of the response quantities from each seismic event can be used in the design. If less than 7 ground motion pairs are used in the analysis, the maximum response quantity must be used in the design. The minimum number of ground motions to be used in the analysis is limited to 3. Appropriate ground motions must be selected from seismic events with distances, magnitudes and fault mechanisms consistent with the seismological characteristics of the region. If sufficient number of ground motion pairs is not available, the designer is allowed to use appropriate simulated records. The records must be scaled, using an identical scaling factor for both horizontal components, so that their SRSS match the design UHS. For each scaled ground motion pair, the SRSS of the pseudo-acceleration response spectra (PSA) of the horizontal components must be constructed. Each ground motion must be scaled so that the average of the SRSS spectra from all seismic events would be equal to or above the target design UHS, over the period range of interest. The period range of interest is given as $0.2T$ to $1.5T$, where T is the fundamental period of the structure.

Force response parameters obtained from each analysis are multiplied by the structure's importance factor, I_e, and divided by the force modification factor, R, according to the system SFRS. Displacement response parameters must be multiplied by the deflection amplification factor, C_d, and divided by the force modification factor, as defined by ASCE 7-10 [3]. For each seismic event, if the base shear, V_i, is less than 85% of the minimum static base shear, V_{min}, according to ASCE 7-10 [3], force response quantities must be scaled up with the factor V_{min}/V_i. In addition, displacement response quantities must be multiplied by the factor, $0.85V_{min}/V_i$.

5.2 Linear time-history analysis procedure in NBCC 2010

General guidelines for the selection and scaling of earthquake records are provided in the NBCC 2010 Commentary [7]. The guidelines for ground motion selection and scaling are somewhat general and a reference has been made to ASCE 7-10 [3] for more specificity.

According to NBCC 2010 [1], force response parameters determined from each separate analysis must be multiplied by the importance factor of the structure, I_e, and divided by the product of the ductility-related force modification factor, $R_{d,}$ and the over-strength force modification factor, R_o, based on the SFRS. Displacement response parameters can be obtained from the elastic seismic response of the structure without any reduction factors. For each seismic event in regular structures, if the base shear, V_i, is less than 80% of the static base shear, V_{st}, according to the ESFP in NBCC 2010 [1], all response quantities must be scaled up with the factor $0.80V_{st}/V_i$. For irregular structures, the base shear of each seismic event must be higher than the base shear obtained from doing the ESFP. If not, all response quantities must be scaled up with the factor, V_{st}/V_i.

5.3 Linear time-history analysis procedure in NBCC 2015

Guidelines for selection and scaling of ground motions are more specific in NBCC 2015 Commentary [8]. It is recommended that 11 ground motion pairs are used in the time-history analysis. The average of the response quantities obtained from the three ground motions that produce the maximum response must be used for design. The period range over which the ground motions must match the target UHS is specified as $0.2T$ to $1.5T$, where T is the fundamental period of the structure. However, the lower bound on the period range does not need to be less than the period associated with the mode of vibration, making the total modal contribution higher than 90%. The provisions for scaling of the response parameters are identical as those given in NBCC 2010 [1].

6. Linear Time-History Analysis of Case Study Buildings

To assess the benefits of performing a linear time-history analysis in practice, the lateral seismic demands on four case study buildings are investigated, using the ESFP, MRSA and LTHA. The results are presented, compared and discussed. The buildings were originally retrofitted to meet the requirements of the NBCC 2010 [1] and, therefore, the provisions provided by NBCC 2010 Commentary [7] are adopted for the selection and scaling of ground motion records and doing a LTHA. Throughout the analysis, 5% damping is assumed for the first and the second mode and direct time step integration of the equations of motion is carried out. In the following paragraphs, brief information about the selected records and the buildings are provided.

6.1 Selected Records

Each building is subjected to seven records and the mean response is determined. Where possible, ground motion records are selected from historical events, provided by the PEER ground motion database [4]. In the absence of historical records, synthetic ground motions have been used. Using finite fault method, Atkinson [9] generated a comprehensive database of simulated records compatible with eastern and western sites of Canada. These records are used in the analysis where historical records were not available. The selected records are scaled to match the target design UHS of the site. Filiatrault et al. [10] provide descriptions and examples of the adopted scaling method.

6.2 Buildings descriptions

Building I is a 12 story reinforced concrete structure located in on site class ‘E’. The peak ground acceleration and the short period spectral acceleration of the site are 0.32g and 0.64g, respectively. The building’s height is 34.5 meters and reinforced concrete shear walls form the primary elements of the SFRS. The floor system of the structure consists of two-way reinforced concrete slabs and the building has a foot print of 2268 m^2 in plan. The fundamental period of the structure obtained from the analytical model is 1.118s in one direction and 0.909s in the other direction.

Building II is an industrial one story building which has a total foot print of 3333 m^2 in plan. The structure is 9.2 meters tall. The primary SFRS of the building is formed by steel braces. The roofing system consists of light wood panels and wood trusses in both the orthogonal directions. The wood trusses also act as the secondary SFRS. The site PGA and the short period spectral acceleration are 0.31g and 0.61g, respectively. The structure is on site class ‘D’. The structure’s analytical periods in two principal directions are 1.26s and 0.88s.

Building III is a two-story steel building. The SFRS is formed by concentric chevron braces. The roofing system consists of steel decks with concrete topping supported on steel joists. The building is located on site class ‘D’ and the site PGA and short period spectral acceleration are 0.32g and 0.64g, respectively. The building has a total foot print of 2220 m^2 in plan and is 8.5 meters in height. The buildings fundamental period, obtained from the analytical model is 0.49s and 0.36s in two orthogonal directions.

Building IV is a three-story concrete structure with irregularity in height. Friction dampers form the primary SFRS. Gravity loads are resisted by concrete two-way slabs spanning between concrete moment frames. Concrete moment frames also act as the secondary SFRS. The building has a foot print of 680 m^2 and a height of 11.5 meters. The structure’s analytical periods are 0.37s and 0.36s in two orthogonal directions. PGA and short period spectral acceleration for the site are 0.5g and 0.92g, respectively. Seismic base shears on these structures are determined, using the ESFP, MRSA and a LTHA.

7. Results of the Analyses

The results of the analyses in terms of structures base shears are summarized in Table 1. For ease of comparison, the elastic base shears are reported without applying any design factors such as importance factor, force modification factors, etc. Building IV is an irregular structure according to NBCC 2010 [1] for which a dynamic analysis is mandatory. However, for the purpose of having complete results, its base shear using an ESFP is reported as well.

Table 1 – Seismic base shears for each building obtained from different analysis methods

Analysis Method	Building I		Building II		Building III		Building IV*	
	X (kN)	Y (kN)	X (kN)	Y (kN)	X (kN)	Y (kN)	X (kN)	Y (kN)
$ESFP_X$	78907.7	0.0	2525.3	0.0	10302.4	0.0	12999.0	0.0
$ESFP_Y$	0.0	91657.8	0.0	2525.3	0.0	10302.4	0.0	12999.0
RS_X	63818.0	38.9	2020.6	2.7	8422.7	351.6	15641.2	1037.0
RS_Y	38.9	75381.2	4.5	2020.6	351.6	10836.3	1037.0	14259.2
MEAN LTHA	52358.4	62853.4	1900.2	1281.6	6766.9	8150.6	11789.4	11154.9

**Building IV is an irregular structure in height*

In buildings I, II and III, by doing a MRSA and capturing the actual fundamental period of vibration as well as the contribution of higher modes, the base shear is reduced. According to NBCC 2010 [1], this reduction must be limited to 20% of the base shear calculated using the ESFP. As expected and since building IV is classified as an irregular structure, doing a MRSA will in fact increase the seismic base shear.

Performing a LTHA will further reduce the base shears. In all cases, the response quantities had to be scaled up to meet the minimum values as outlined by NBCC 2010 [1]. For buildings I, II, and III, the base shears obtained from a LTHA were scaled up to be at least 80% of the ESFP base shear. However, for building IV, the LTHA base shear was scaled up to 100% of the ESFP base shear. It must be noted that while the same minimum limit must be imposed on the base shear as MRSA, in LTHA, it is the resultant of the base shear that is scaled up to meet the minimum code requirement. This means that the components of base shear acting along the buildings orthogonal directions would be smaller than the base shear in each direction by doing a MRSA. This is due to the multi-directionality nature of earthquakes. Further, the maximum base shear in the X direction does not necessarily occur at the same time as the maximum base shear in the Y directions occurs. This will improve the seismic design in buildings that are sensitive to torsion.

It is of interest to note that the Y direction base shear for building II, obtained from doing a LTHA, is much less than the original base shear obtained from the ESFP. The primary reason for this is the fact that the period of the structure in the Y direction is significantly longer. However, the code limit on the fundamental period of the structure dictates the use of identical periods in both orthogonal directions. Therefore, the static base shears in both directions are identical. After doing a MRSA, the base shear in the Y direction is, similarly, substantially less. However, since the building is analyzed in each orthogonal direction independently, again the response spectrum base shear needs to be scaled up to 80% of the ESFP base shear. However, when doing a LTHA, the resultant base shear is scaled up and, therefore, the artificial increase in the base shear in the Y direction is avoided. However, it is of great importance that in the ground motion database there is sufficient number of records with maximum PGAs in different directions to assess the buildings performance in different directions.

8. Summary and Conclusions

The study presented in this paper investigates the benefits of using linear time history analysis for determining the seismic lateral demand on structural systems. The limitations associated with the ESFP and the MRSA were outlined. Further, using a large building database, the adequacy of the empirical equations predicting the fundamental period of structures were investigated. In the end, the results of 4 case study buildings analyzed using different methods of seismic analysis were presented and compared. The study leads to the following conclusions:

1. Seismic lateral demands determined using the equivalent static force procedures outlined by the codes of practice, are accompanied by a number of conservative assumptions. These assumptions are due to uncertainties in the UHS, complicated nature of earthquake excitations, complexity of the nonlinear response of structural systems and the fact that they are developed to be applicable to a large group of buildings.

2. Some of the assumptions associated with the ESFP include the estimation of the fundamental period of structures, the exclusion of the higher modes response contribution and the assumption that the direction of the earthquakes PGA will coincide with the buildings principal axis. These assumptions lead to conservatively high seismic forces. In addition, ESFP does not provide any measure for response quantities such as velocities, accelerations, etc.
3. By performing a MRSA, the building period is determined accurately and the contribution of higher modes is taken into account. In regular structures, this will often decrease the base shear. However, MRSA has several shortcomings including the fact that in modal analysis the maximum response quantities in the form of positive values are produced. This will make the structure's deformed shape to have very little meaning. In addition, for the design of members under combined axial loads and bending moments, both maximum values are reported which leads to conservative designs.
4. By performing a LTHA, other conservative assumptions are reduced and the base shear will drop further. The maximum force quantities will not necessarily occur at the same time and the outputs are functions of time. Specifically, maximum base shears along both orthogonal directions do not occur concurrently. Also, the axial load and bending moment in beam-column elements are reported realistically and not as maximums. The designer is also capable to report the elastic velocity and acceleration time-histories. The use of linear time-history analysis, compared to doing a MRSA, offers advantages for both regular and irregular structures.
5. Performing a LTHA is computationally expensive and requires more time and improved knowledge, compared to doing the ESFP or the MRSA. However, it can lead to significant reduction in the construction costs in regular structures, while having a design that is within the boundaries of code requirements.

9. References

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