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OPTIMUM STRENGTH DISTRIBUTION FOR SEISMIC DESIGN OF TALL BUILDINGS

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SUMMARY

This paper examines the effects of strength distribution pattern on seismic response of tall buildings. It is shown that in general for a MDOF structure there exists a specific pattern for height wise distribution of strength and stiffness that results in a better seismic performance in comparison with all other feasible patterns. This paper presents a new optimization technique for optimum seismic design of structures. In this approach, the structural properties are modified so that inefficient material is gradually shifted from strong to weak areas of a structure. This process is continued until a state of uniform deformation is achieved. It is shown that the seismic performance of such a structure is optimal, and behaves generally better than those designed by conventional methods. The optimization algorithm is then conducted on shear-building models with various dynamic characteristics subjected to a group of severe earthquakes. Based on the results, a new load pattern is proposed for seismic design of tall buildings that is a function of fundamental period of the structure and the target ductility demand. The optimization method presented in this paper could be useful in the conceptual design phase and in improving basic understanding of seismic behavior of tall buildings.

Keywords: Tall buildings; Ductility demand; Optimum strength and stiffness distribution; Seismic codes; Performance-based design

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1. INTRODUCTION

Seismic design is currently based on force rather than displacement, essentially as a consequence of the historical developments of an understanding of structural dynamics and, more specifically, of the response of structures to seismic actions and the progressive modifications and improvement of seismic codes worldwide. Although design procedures have become more rigorous in their application, this basic force-based approach has not changed significantly since its inception in the early 1900s. Consequently, the seismic codes are generally regarding the seismic effects as lateral inertia forces. The height wise distribution of these static forces (and therefore, stiffness and strength) seems to be based implicitly on the elastic vibration modes (Green, 1981; Hart, 2000).

Recent design guidelines, such as FEMA 356 and SEAOC Vision 2000, place limits on acceptable values of response parameters, implying that exceeding of these acceptable values represent violation of a performance objective. Further modifications to the preliminary design, aiming to satisfy the Performance Objectives could lead to some alterations of the original distribution pattern of structural properties. As structures exceed their elastic limits in severe earthquakes, the use of inertia forces corresponding to elastic modes may not lead to the optimum distribution of structural properties.

Many experimental and analytical studies have been carried out to investigate the consequences of using the code patterns on seismic performance of tall buildings (Anderson et al., 1991; Gilmore and Bertero, 1993; Naeim et al., 2000; Ventura and Ding, 2000). Lee and Goel (2001) analyzed a series of 2 to 20 story frame models subjected to various earthquake excitations. They showed that in general there is a discrepancy between the earthquake induced shear forces and the forces determined by assuming distribution patterns. Chopra (2001) evaluated the ductility demands of several shear-building models subjected to the El- Centro Earthquake of 1940. The relative story yield strength of these models was chosen in accordance with the distribution patterns of the earthquake forces specified in the Uniform Building Code (UBC). It was concluded that this distribution pattern does not lead to equal ductility demand in all stories, and that in most cases the ductility demand in the first story is the largest of all stories. The first author (1995, 1999) proportioned the relative story yield strength of a number of shear building models in accordance with some arbitrarily chosen distribution patterns as well as the distribution pattern suggested by the UBC1997. It is concluded that: (a) the pattern suggested by the code does not lead to a uniform distribution of ductility, and (b) a rather uniform distribution of ductility with a relatively smaller maximum ductility demand can be obtained from other patterns. These findings have been confirmed by further investigations (Moghaddam et al., 2003; Moghaddam and Hajirasouliha, 2004; Karami et al., 2004), and led to the development of a new concept: optimum distribution pattern for seismic performance that is discussed in this paper. An effective optimization algorithm is developed to find more rational criteria for determination of design earthquake forces. It is shown that using adequate load patterns could result in a reduction of required structural weight and a more uniform distribution of deformations.

2. MODELING AND ASSUMPTIONS

Among the wide diversity of structural models that are used to estimate the non-linear seismic response of building frames, the shear-beam models are the one most frequently adopted. In spite of some drawbacks, they are widely used to study the seismic response of multi-story buildings because of their simplicity and low computational requirements, thus permitting the performance of a wide range of parametric studies (Diaz et al., 1994). Lai et al. (1992) have investigated the reliability and accuracy of such shear-beam models.

To investigate the seismic behavior of tall buildings, shear-building models with fundamental period varying from 1 sec to 3 sec, and target ductility demand of 1 (elastic), 1.5, 2, 3, 4, 5, 6 and 8 have been used in the present study. In the shear-building models, each floor is assumed as a lumped mass that is connected by perfect elastic-plastic shear springs. The total mass of the structure is distributed uniformly over its height as shown in Figure 1. The Rayleigh damping is adopted with a constant damping ratio 0.05 for the first few effective modes. In all MDOF models, lateral stiffness is assumed as proportional to shear strength at each story, which is obtained in accordance with the selected design lateral load pattern.

Fifteen selected strong ground motion records, including 6 components of Imperial Valley 1979 and 9 components of Northridge 1994 are used for input excitation as listed in Table 1. All of these excitations correspond to the sites of soil profiles similar to the S_D type of UBC 1997 and are recorded in a low to moderate distance from the epicenter (less than 45 km) with rather high local magnitudes (i.e., M_l>6.6). Due to the high intensities demonstrated in the records, they are used directly without being normalized.

The above-mentioned models are, then, subjected to the seismic excitations and non-linear dynamic analyses are conducted utilizing the computer program DRAIN-2DX (Prakash et al., 1992). For each earthquake excitation, the dynamic response of models with various fundamental periods and target ductility demands is calculated.

2.1. Lateral loading patterns

In most seismic building codes (Uniform Building Code, 1997; NEHRP Recommended Provisions, 1994; ATC-3-06 Report, 1978; ANSI-ASCE 7-95, 1996; Iranian Seismic Code, 1999), the height wise distribution of lateral forces is to be determined from the following typical relationship:

$$F_i = \frac{w_i h_i^k}{\sum_{j=1}^{N} w_j h_j^k} \cdot V. \tag{1}$$

Where w_i and h_i are the weight and height of the i^{th} floor above the base, respectively; N is the number of stories; and k is the power that differs from one seismic code to another. In some provisions such as NEHRP-94 and ANSI/ASCE 7-95, k increases from 1 to 2 as period varies from 0.5 to 2.5 second. However, in some codes such as UBC 1997 and Iranian Seismic Code (1999), the force at the top floor (or roof) computed from Equation (1) is increased by adding an additional force F_t=*0.07TV* for a fundamental period *T* of greater than 0.7 second. In such a case, the base shear *V* in Equation (1) is replaced by $(V-F_t)$.

Karami et al. (2004) introduced an "optimum" loading pattern as a function of the period of the structure and target ductility. This loading pattern is a rectangular pattern accompanied by a concentrated force αTV at the top floor, where α is a coefficient that depends on the fundamental period, *T*, and the target ductility, μ_t. Based on the nonlinear dynamic analyses on shear-building models subjected to twenty-one earthquake ground motions; the following expression is suggested for α (Karami et al., 2004):

$$\alpha = (0.9 - 0.04\mu_t).e^{-(0.6+0.03\mu_t)T} \tag{2}$$

In this study, the adequacy of the above loading patterns to seismic design of tall buildings is investigated

2.2. Modal analysis

The modal superposition method is a general procedure for linear analysis of the dynamic response of structures. In various forms, modal analysis has been widely used in the earthquake-resistant design of special structures such as very tall buildings. In this method, the distribution of the seismic lateral forces over the building is based on properties of the natural vibration modes (Chopra, 2001).

The peak value of any response quantity, R_{n0}, of the n^{th} mode contribution can be expressed as:

$$R_{n0} = R_n^{st}.S_{a,n} \tag{3}$$

where R_n^{st} denotes the modal static response and $S_{a,n}$ is the spectral acceleration for the n^{th} mode. For a given *N* story building, the effective weight, β_n, of the n^{th} mode is governed by:

$$\beta_n = \frac{(\sum_{i=1}^{N} W_i \phi_{in})^2}{\sum_{i=1}^{N} W_i \phi_{in}^2} \tag{4}$$

where $W_i = m_i .g$ is the weight of the i^{th} story and ϕ_{in} represents the mode shape corresponding to the n^{th} mode. Consequently, the total base shear V_n for the n^{th} mode is calculated from:

$$V_n = \frac{S_{a,n}}{g} \beta_n \tag{5}$$

And the force F_{in} at the i^{th} story can be obtained by following equation:

$$F_{in} = \frac{W_i \phi_{in}}{\sum_{i=1}^{N} W_i \phi_{in}} V_n \tag{6}$$

Because the peak modal responses do not occur at the same time for each mode of vibration, the maximum story shears have been combined by the SRSS (square root sum of squares) rule. According to the most seismic building codes, a modal response spectrum analysis shall be performed for the structures using sufficient modes to capture 90% mass participation. In the present study, we investigate how the adequacy of design loading patterns with one, two, and three modes included vary with the ductility demand imposed by the ground motion.

3. HEIGHT WISE DISTRIBUTION OF DUCTILITY DEMANDS

It is generally endeavored to induce a status of uniform deformation throughout the structure to obtain an optimum design as in Gantes et al. (2000). Karami et al. (2004) showed that for a given earthquake, the weight of seismic resistant system required to reach to the prescribed target ductility is correlated with the *cov*, the coefficient of variation, of the story ductility demands and the two minimize simultaneously. Therefore, they concluded that the *cov* of ductilities could be used as a means of assessing the adequacy of design load patterns to optimum use of material.

To investigate the efficiency of conventional loading patterns to lead to the equal ductility demands in all stories, shear-building models with various periods and ductility demands are subjected to 15 selected ground motions (Table 1). In each case, strength and stiffness are distributed within the stories according to the lateral load pattern suggested by UBC 1997. Subsequently, the stiffness pattern is scaled to obtain the prescribed fundamental period. Maximum ductility demand is then calculated by performing non-linear dynamic analysis for the given exaction. By an iterative procedure, the total strength of the model is scaled (without changing it's distribution pattern) until maximum ductility demand gets to the target value with less than 1 % error. Finally, *cov* of the story ductility demands is calculated for each case. Figure 2 illustrates the average of *cov* obtained in 15 earthquakes versus fundamental period and for various target ductility demands. Based on the results presented in Figure 2, it is concluded that, in average, using the strength pattern suggested by UBC 1997 leads to an almost uniform distribution of ductility demands for the structures within the linear range of behaviour. However, the adequacy of conventional load patterns is reduced in non-linear ranges of vibration. It is shown that increasing the target ductility is always accompanied by increasing in *cov* of story ductility demands. While conventional loading patterns suggested by most seismic codes are not a function of the target ductility demand, Figure 2 indicates that in the structures with long fundamental period (i.e. greater than 1 sec), *cov* of ductilities is more dependent on the maximum ductility demand than the fundamental period of the structure.

4. CONCEPT OF THEORY OF UNIFORM DEFORMATION

As discussed in previous section, using distribution patterns for lateral seismic force suggested by seismic codes does not guarantee the optimum performance of structures. Current study indicates that during strong earthquakes the deformation demand in structures does not vary uniformly. Therefore, it can be concluded that in some parts of the structure, the deformation demand does not reach the allowable level of seismic capacity, and therefore, the material is not fully exploited. If the strength of these strong parts decreases, the deformation would be expected to increase (Riddell et al., 1989; Vidic et al., 1994). Hence, if the strength decreases incrementally, we should eventually obtain a status of uniform deformation. At this point, the material capacity is fully exploited. As the decrease of strength is normally obtained by the decrease of material, a structure becomes relatively lighter in the case of uniformly distributed deformations. Therefore, in general it can be concluded that a status of uniform deformation is a direct consequence of the optimum use of material. This is considered as the Theory of Uniform Deformations (Moghaddam and Hajirasouliha, 2004). This theory is the basis of the studies presented in this paper.

5. OPTIMUM DISTRIBUTION OF DESIGN SEISMIC FORCES

The Theory of Uniform Deformation can be employed for evaluation of optimum distribution of structural properties for shear building like structures. To accomplish this, an iterative optimization procedure has been adopted. In this approach, the structural properties are modified so that inefficient material is gradually shifted from strong to weak areas of a structure. This process is continued until a state of uniform deformation is achieved. It should be noted that there is a unique relation between the distribution pattern of lateral seismic forces and the distribution of strength (as the strength at each floor is obtained from the corresponding story shear force). Hence, for shear buildings, we can determine the optimum pattern for distribution of seismic lateral loads instead of distribution of strength. Assume that one is to evaluate the most appropriate lateral loading pattern to design a 10-story shear building, shown in Figure 1, with a fundamental period of 1 sec such that it sustains the Northridge earthquake of 1994 (CNP196) without exceeding a maximum story ductility demand of 4. Considering the Theory of Uniform Deformation, the following optimization procedure is used:

1. Arbitrary initial patterns are assumed for height wise distribution of strength and stiffness. However, for shear building models we can assume that these two patterns are similar, and therefore, an identical pattern is assumed for both strength and stiffness. Here, the uniform pattern in is chosen for the initial distribution of strength and stiffness.
2. The stiffness pattern is scaled such that the structure has a period of 1 sec.

3. The structure is subjected to the given excitation, and the maximum story ductility is calculated, and compared with the target value. Consequently, the strength is scaled (without changing the primary pattern) until the maximum deformation demand reaches the target value. This pattern is regarded as a feasible design, and referred to as the first acceptable pattern. For the above example, story strength and maximum story ductility corresponding to the first feasible answer are given in Table 2.
4. The *cov* (coefficient of variation) of story ductility distribution within the structure is calculated. The procedure continues until *cov* decreases down to a prescribed level. The *cov* of 0.719 was obtained for the first feasible pattern, which is high, hence the analysis continues.
5. At this stage the distribution pattern is modified. Using the Theory of Uniform Deformation, the inefficient material should be reduced to obtain an optimum structure. To accomplish this, stories in which the ductility demand is less than the target values are identified and weakened by reducing strength and stiffness. Investigations show that this alteration should be applied incrementally to obtain convergence in numerical calculations. Hence, the following equation is used in the present studies:

$$[S_i]_{n+1} = [S_i]_n \left[\frac{\mu_i}{\mu_t} \right]^{\alpha} \tag{7}$$

Where μ_i is the ductility demand at ith story, and μ_t is the target ductility taken to be 4 for all stories. S_i is the shear strength of the ith story, n denotes the step number, and α is the convergence coefficient ranging from 0 to 1. For the above example, an acceptable convergence has been obtained for a value of 0.2 for α. Now, a new pattern for height wise distribution of strength and stiffness is obtained. The procedure is repeated from step 2 until a new feasible pattern is obtained. It is expected that the *cov* of ductility distribution for this pattern is smaller than the corresponding *cov* for the previous pattern. This procedure is iterated until *cov* becomes small enough, and a status of rather uniform ductility demand prevails. The final pattern is considered as practically optimum.

Story ductility pattern for preliminary and final answers are compared in Table 2. According to the results, the efficiency of utilizing this method to achieve a structure with uniform ductility demand distribution is demonstrated. Figure 3 illustrates the variation of *cov* and total strength from first feasible answer toward the final answer. Figure 3 shows the efficiency of the proposed method that resulted in reduction of total strength by 41% in only five steps. It is also shown in this figure that the proposed method has the property of converging to the optimum pattern without any oscillation. It can be noted from Figure 3 that decreasing the *cov* is always accompanied by reduction of total strength. Here the total strength is proportional to the total weight of the seismic resisting system. Furthermore, the results confirm the efficiency of the Theory of Uniform Deformation.

Table 2 shows the results of analysis for the first and the final step. The height wise distribution of strength can be converted to the height wise distribution of lateral forces. Such pattern may be regarded as the optimum pattern of seismic forces for the given earthquake. Figure 4 enables one to compare this optimum pattern with the conventional lateral load patterns suggested by seismic design codes. The results indicate that to improve the performance under this specific earthquake, the frame should be designed based on an equivalent static lateral load pattern relatively different from the suggested conventional code patterns, e.g. that of UBC 1997 guideline.

5.1. Effect of initial pattern on the optimum load pattern

As described before, an initial strength distribution is necessary to begin the optimization algorithm. In order to investigate the effect of this initial load (or strength) pattern on the final results, for the previous example four different initial load patterns have been assumed:

1) A concentrated load on the roof level;
2) Triangular distribution similar to the UBC code of 1997;
3) Rectangular distribution; and

4) An inverted triangular distribution with maximum lateral load on the first floor and the minimum lateral load at the roof level.

For each case, the optimum lateral load pattern was derived for Northridge 1994 (CNP196) event. The comparison of the optimum lateral load pattern for each case is depicted in Figure 5. As shown in this figure, the optimum lateral force pattern is not dependent on the initial strength pattern. However, the convergence speed of the algorithm is to some extent dependant on this initial pattern. This conclusion has been confirmed by analysis of several different shear buildings and ground motions.

6. ADEQUACY OF SEISMIC LOADING PATTERNS

To investigate the validity and accuracy of the proposed optimization method, the foregoing procedure has been applied to find the optimum pattern for shear-building models with various fundamental periods and target ductility demands subjected to 15 synthetic earthquakes representing a design spectrum. These seismic motions are artificially generated using the SIMQKE program, having a close approximation to the elastic design response spectra of UBC 1997 with a PGA of 0.44g as shown in Figure 6. In this study, the maximum story ductility is considered as the failure criterion, implying that exceeding of the target ductility represents violation of the performance objective. Therefore, according to the Theory of Uniform Deformation, it is expected that seismic performance be improved by a uniform distribution of ductility demands. It is demonstrated in previous section that the proposed method is very efficient to reach to the uniform distribution of ductility demands.

To evaluate the weight of the seismic resistant system for MDOF structures, it is assumed that the weight of lateral-load-resisting system at each story, W_{Ei}, is proportional to the story shear strength, S_i. Therefore, the total weight of the seismic resistant system, W_E, can be calculated as:

$$W_E = \sum_{i=1}^{n} W_{Ei} = \sum_{i=1}^{n} \lambda \cdot S_i = \lambda \cdot \sum_{i=1}^{n} S_i \quad (8)$$

where λ is the proportioning coefficient. It is of interest to compare the required structural weight for structures of identical period and ductility ratio that have been designed for different seismic loading patterns, and, therefore, assess the relative adequacy of the chosen loading patterns. The loading pattern that corresponds to the minimum required structural weight would be regarded as the most adequate loading pattern. To accomplish this, the total weight of the seismic resistant system has been calculated for shear-building models with various fundamental period and target ductility demands, designed according to UBC 1997, NEHRP 1994, Karami et al. (2004) proposed load pattern and loading patterns with one, two, and three modes included; subjected to 15 synthetic earthquakes. Subsequently, the ratio of required structural weight to the structural weight of the corresponding optimum model, $(W_E)/(W_E)_{opt}$, has been calculated for all cases. Figure 7 shows the median values of $(W_E)/(W_E)_{opt}$ as a function of ductility demand, and for shear-building models with fundamental period of 1, 2 and 3 sec. This figure has been obtained by averaging the responses of 15 synthetic earthquakes. According to the results illustrated in Figure 7, it is concluded that:

1. Having the same period and ductility demand, structures designed according to the optimum load pattern always have less structural weight compare to those designed conventionally. Therefore, the adequacy of optimum loading patterns is emphasised.
2. In the elastic range of vibration (μ_t=1), the total structural weight required for the models designed according to the seismic design guidelines are in average 10% above the optimum value. It is also shown that, in average, there is not a big difference between different conventional code type load patterns when structures remained in elastic range. Hence, it can be concluded that for practical purposes, using the conventional loading patterns could be satisfying within the linear range of vibrations.
3. Increasing the ductility demand is generally accompanied by increasing in the structural weight required for the conventionally designed models compare to the optimum ones. This implies that conventional loading patterns loose their efficiency in non-linear ranges of

vibration. It is illustrated that for high levels of ductility demand, the required structural weight for the models designed according to NEHRP 1994 could be more than 70% above the optimum weight.

4. Loading pattern proposed by Karami et al. (2004), in average, results better than code type loading patterns for tall buildings in highly inelastic ranges (i.e. $\mu_t \geq 3$), however; it looses its efficiency for the buildings behave almost linearly (i.e. $\mu_t \leq 2$).
5. The required structural weight for the models designed according to the loading patterns with two or three modes included are generally smaller than those designed with UBC 1997 and NEHRP 1994 loading patterns. However, in non-linear ranges of vibration the results are in average 30% above the optimum weight. It is shown that significant improvement is achieved by including response contributions due to the second mode, however, the third mode contributions do not seem especially important.

7. MORE ADEQUATE LOADING PATTERNS TO DESIGN TALL BUILDINGS

It is well known that there are many uncertainties in seismic loading and seismic design of structures. One of the most random parameters is the seismic event that might occur in a place; therefore, the selection of only one ground motion for seismic design of a structure might be a great risk. As described before, to improve the performance under a specific earthquake, structures should be designed in compliance with an optimum load pattern different from the conventional patterns. This optimum pattern depends on the earthquake, and therefore, it varies from one earthquake to another. However, there is no guarantee that the structure will experience seismic events, which are the same as the design ground motion. While each of the future events will have its own signature, it is generally acceptable that they have relatively similar characteristics. Accordingly, it seems that the designed model with optimum load pattern is capable to reduce the deformation demands experienced by the model after similar ground motions. It can be concluded that for design proposes, the design earthquakes must be classified for each structural performance category and then more adequate loading pattern must be found by averaging optimum patterns corresponding to every one of the earthquakes in each group.

To verify this assumption, 15 strong ground motion records with the similar characteristics, as listed in Table 1, were selected. Time history analyses have been performed for all earthquakes and the corresponding optimum pattern has been found for shear-building models with various fundamental periods and target ductility demands. Consequently, about 1000 optimum load patterns have been determined at this stage. For each fundamental period and ductility demand, a specific matching load distribution has been obtained by averaging the results for all earthquakes. Subsequently, these average distribution patterns used to design the given shear building models. Then the response of the designed models to each of the 15 earthquakes was calculated. In Figure 8, the ratio of required structural weight to the optimum weight, (W_E) / $(W_E)_{opt}$, are compared for the models designed with the UBC 1997 load pattern, average of optimum load patterns, and Karami et al. (2004) proposed load pattern. This figure has been obtained by averaging the responses of shear-building models with fundamental period of 1 sec to 3 sec, subjected to 15 earthquake ground motions. It is illustrated in Figure 8, having the same period and ductility demand, structures designed according to the average of optimum load patterns require less structural weight compare to those designed conventionally as well as those designed according to the Karami et al. (2004) proposed load pattern.

Karami et al. (2004) proposed load pattern (Equation (2)) suggests that the value of the concentrated force to be applied at the top of the structure decreases as the period of vibration increases. Considering the influence of higher modes effects, this is opposite to what one would expect for multi-story buildings with linear behaviour. Therefore, it is expected the Karami et al. (2004) proposed loading pattern would not be adequate for small level of inelastic behaviour. It is shown in Figure 8 that using Karami et al. (2004) proposed load pattern results better than

UBC1997 load pattern for the structures in highly inelastic ranges (i.e. $\mu_t \geq 3$), however; it looses its efficiency for the structures with linear or nearly linear behaviour (i.e. $\mu_t \leq 2$).

The effectiveness of using average of optimum load patterns to reduce required structural weight is demonstrated in Figure 8 for both elastic and inelastic systems. However its efficiency is more obvious for the models with high ductility demand. It is shown in this figure that using the average load pattern for seismic design of tall buildings could be resulted in more than 50% reduction in the total structural weight compared to using conventional load patterns. Such a load pattern is designated as 'more adequate load pattern'. The proposed approach can be utilized efficiently to determine more adequate load patterns for any set of earthquakes with similar characteristics. At present, the seismic load patterns suggested by most seismic codes do not depend on the ductility demand of the structure. However, the present study shows that more adequate loading patterns are a function of both the period of the structure and target ductility demand. According to the results of this study, more adequate loading patterns for seismic design of tall buildings could be illustrated in two different categories as follows:

- **Parabolic load pattern**

As shown in Figure 9, in general, parabolic load patterns are appropriate to seismic design of tall buildings with fundamental period longer than 1 sec and small ductility demand (i.e. $\mu_t \leq 3$). It should be noted that the rectangular pattern accompanied by a concentrated force at the top floor, which is suggested by Karami et al. (2004), could be similar to this load pattern. However, it is shown above that Karami et al. (2004) proposed load pattern is not adequate to design tall buildings with linear or nearly linear behavior (i.e. $\mu_t \leq 2$). As Figure 9 indicates, for the same ductility demand, loads at the top stories are increasing as the fundamental period of the structure increases. It is also shown in Figure 9 that, in general, increasing the ductility demand results in decreasing the loads at the top stories and increasing the loads at the lower stories. For higher levels of ductility demand, optimum load patterns corresponding to the models with fundamental period longer than 1 sec, change to the hyperbolic shape.

- **Hyperbolic load pattern**

As illustrated in Figure 10, more adequate load patterns are in hyperbolic shape for structures with high levels of ductility demand ($\mu_t \geq 3$) and fundamental period longer than 1 sec. It is also shown in this figure that increasing the ductility demand results in decreasing the loads at the top stories and increasing the loads at the lower stories. It can be noted from Figure 10 that for the optimum loading patterns corresponding to the structures with long periods and high levels of ductility demand ($T \geq 2.5$ sec and $\mu_t \geq 5$), loads assigned to the lower stories could be greater than those assigned to the top stories. Therefore in this case, optimum loading patterns are completely different with the conventional lateral loading patterns suggested by most seismic codes (e.g. triangular pattern). However, it should be mentioned that this condition is beyond the most practical designs.

It should be noted that there is not a definite boundary between two categories of more adequate load patterns and they convert to each other very smoothly. While more adequate load patterns could be very different in their shape, it is possible to establish some general relationships. According to the illustrated results, increasing the fundamental period is usually accompanied by increasing the loads at the top stories caused by the higher mode effects. On the other hand, in general, increasing the ductility demand results in decreasing the loads at the top stories and increasing the loads at the lower stories. By changing both the fundamental period of the structure and the target ductility demand, these two contrary effects are combined with each other.

More adequate load patterns introduced in this paper are based on the 15 selected earthquakes, as listed in Table 1. However, discussed observations are fundamental and similar conclusions have been obtained by further analyses on different models and ground motions (Hajirasouliha, 2004). Despite obvious variation between the adequate load patterns proposed for different conditions, for each story there is generally a specific relationship between the optimum load pattern, fundamental period of the structure, and target ductility demand. Based on the results of this study, the following equation has been suggested:

$$F_i = (a_i T + b_i)\mu_t^{(c_i T + d_i)} \tag{9}$$

Where F_i is the optimum load component at the i^{th} story; T is the fundamental period of the structure; μ_t is target ductility demand; a_i,b_i,c_i, and d_i are constant coefficients at i^{th} story. These coefficients could be obtained at each level of the structure by interpolating the values given in Table 3. Using Equation (9), the optimum load pattern is determined by calculating optimum load components at the level of all stories. The comparison of the load patterns obtained by Equation (9) and the corresponding load patterns obtained by nonlinear dynamic analysis is shown in Figure 11. As shown in this figure, the agreement between Equation (9) and analytical results is excellent and this equation has good capability to demonstrate different categories of optimum load patterns. Hence, the proposed expression can be efficiently used to determine optimum load patterns for seismic design of tall buildings. The optimization method introduced in this paper, can be used for any set of earthquakes, and can provide an efficient optimum performance-based seismic design method for building structures. However, more adequate loading patterns proposed in this paper should prove useful in the conceptual design phase, and in improving basic understanding of seismic behavior of tall buildings.

8. CONCLUSIONS

1. This paper presents a new method for optimization of dynamic response of structures subjected to seismic excitation. This method is based on the concept of uniform distribution of deformation.
2. It is shown that using the load pattern suggested by seismic codes does not lead to a uniform distribution of deformation demand in tall buildings, and, it is possible to obtain uniform deformation by shifting the material from strong to weak parts. It has been shown that the seismic performance of such structure is optimal. Hence, it can be concluded that

the condition of uniform deformation results in optimum use of material. This has been denoted as the Theory of Uniform Deformation.

3. By introducing an iterative method, Theory of Uniform Deformation has been adapted for optimum seismic design of shear buildings. It is concluded that this can efficiently provide an optimum design. It has been demonstrated that there is generally a unique optimum distribution of structural properties, which is independent of the seismic load pattern used for initial design.
4. For a set of earthquakes with similar characteristics, the optimum load patterns were determined for a wide range of fundamental periods and target ductility demands. It is shown that, having the same story ductility demand, models designed according to the average of optimum load patterns have relatively less structural weight in comparison with those designed conventionally.
5. Based on the results of this study, a more adequate load pattern is introduced for seismic design of tall buildings. This pattern is a function of the fundamental period of the structure and the target ductility. It is shown that the proposed loading pattern is superior to the conventional loading patterns suggested by most seismic codes.

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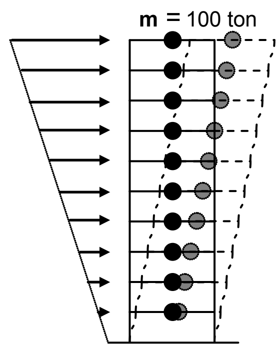

Figure 1. Typical 10-story shear building model

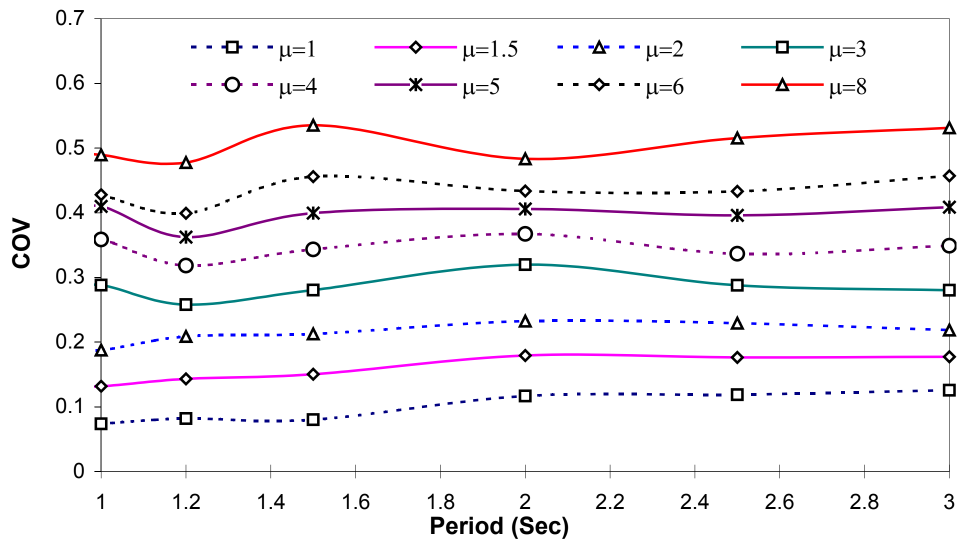

Figure 2. *Cov* of story ductility demands, average of 15 earthquakes

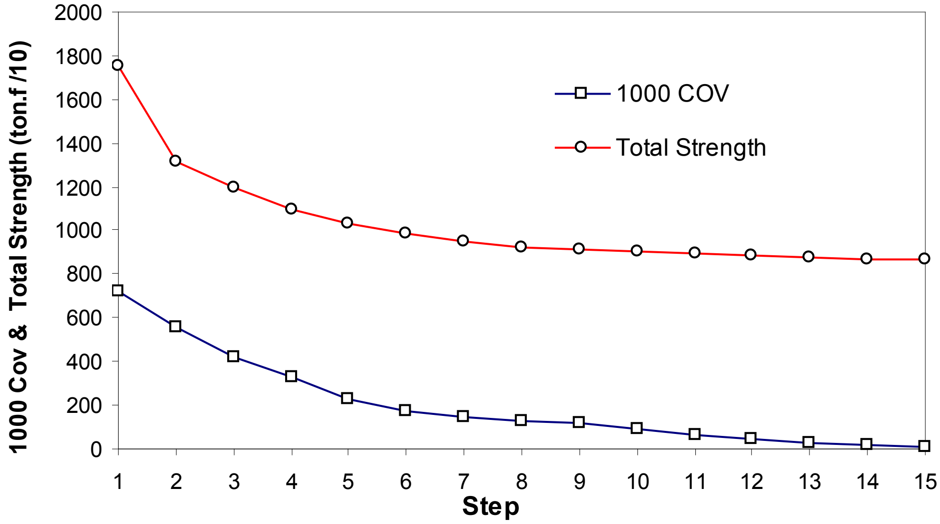

Figure 3. *Cov* of story ductility demands and total story strength for feasible answers, 10-story shear building with T=1 Sec and μ_t=4, Northridge 1994 (CNP196)

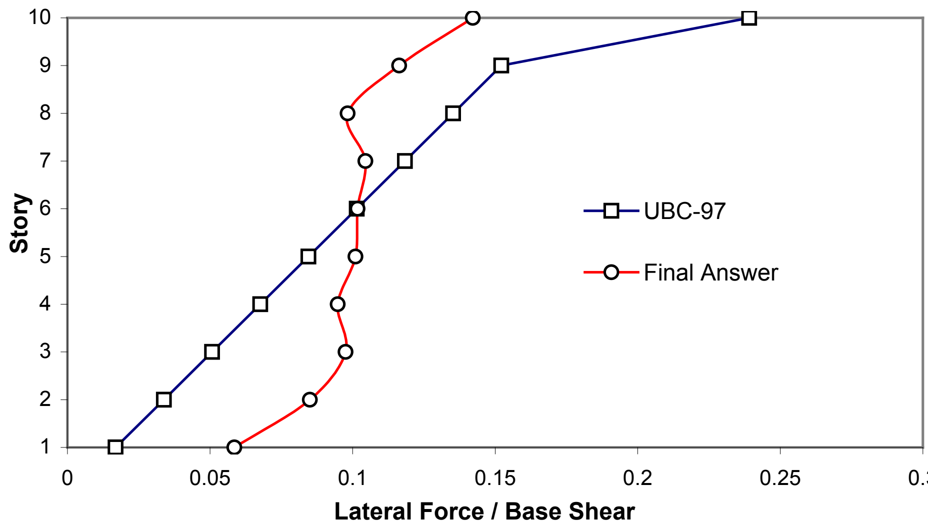

Figure 4. Comparison of UBC 1997 & optimum lateral force distribution, 10-story shear building with T=1 Sec and μ_t=4, Northridge 1994 (CNP196)

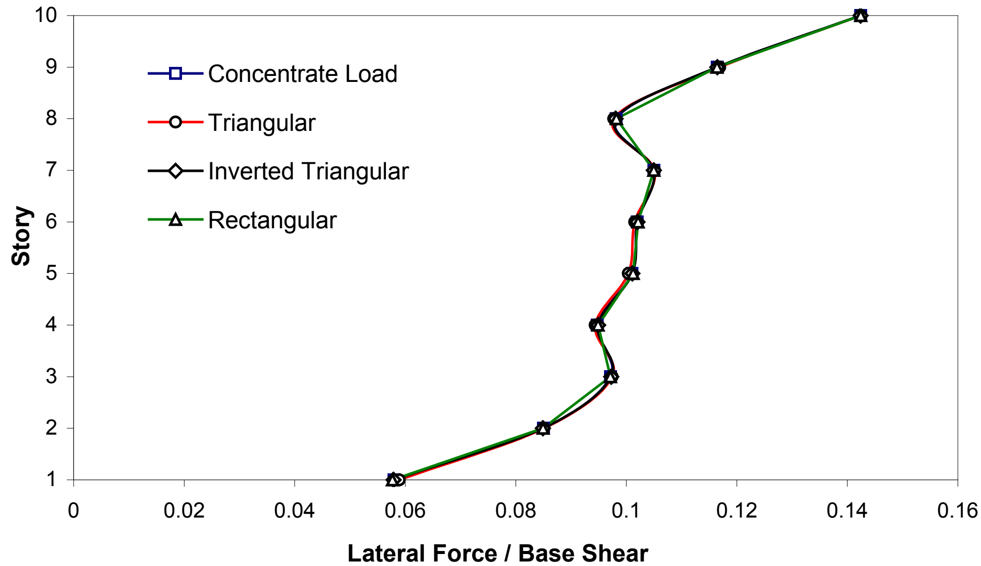

Figure 5. Optimum load pattern for different initial strength distributions, 10-story shear building with T=1 Sec and μ_t=4, Northridge 1994 (CNP196)

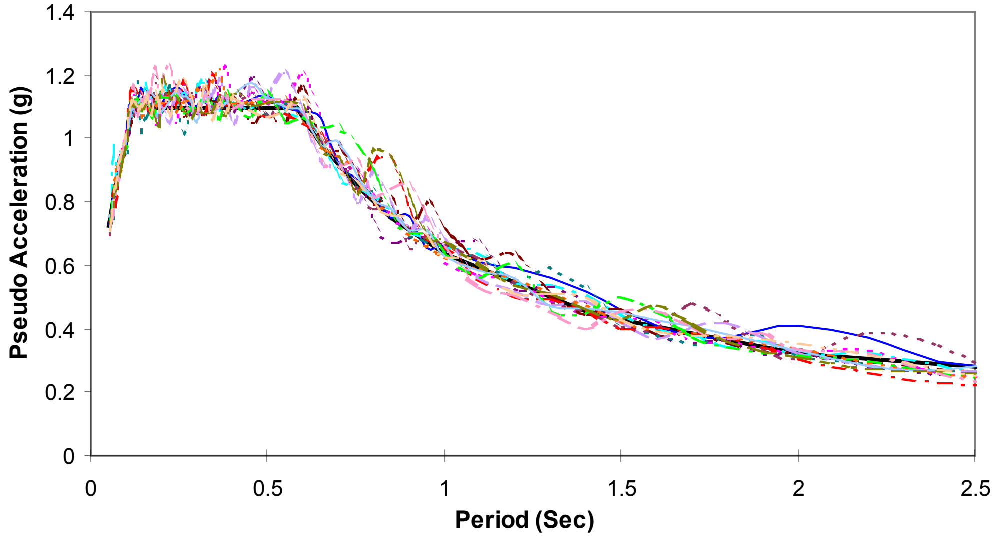

Figure 6. UBC design spectrum and response spectra of 15 synthetic earthquakes (5% damping)

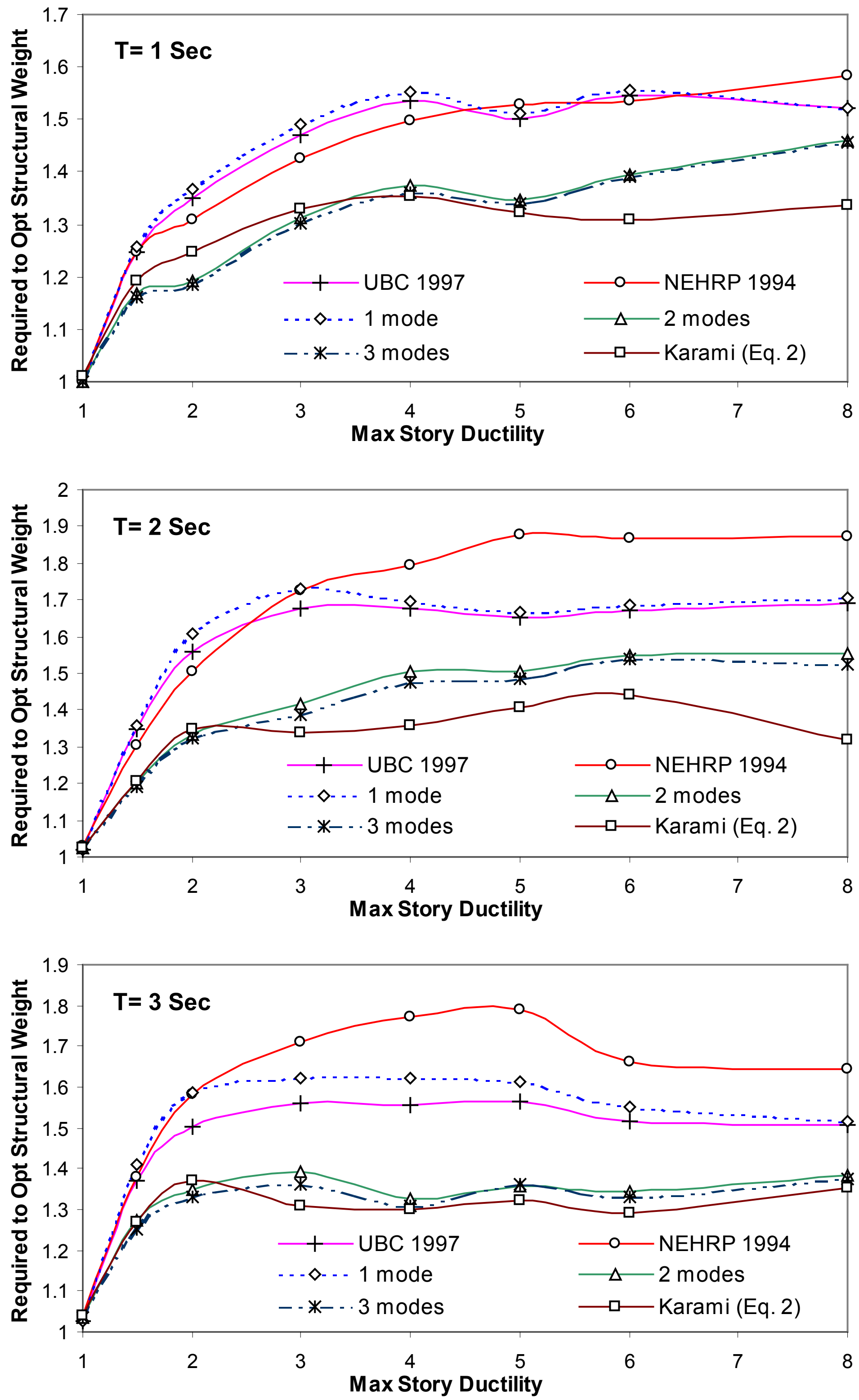

Figure 7. The ratio of required structural weight to the optimum weight, $(W_E) / (W_E)_{opt}$, for the models designed according to different load patterns, Average of 15 synthetic earthquakes

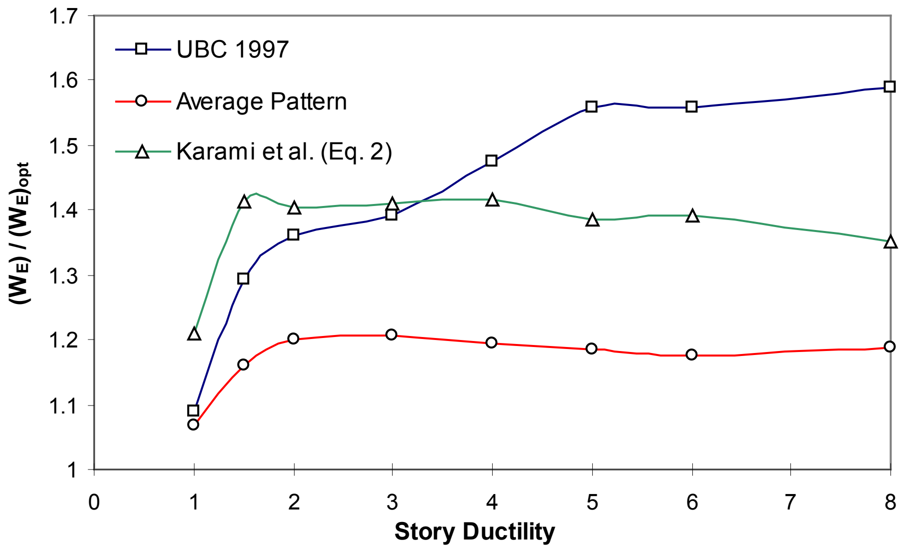

Figure 8. The ratio of required structural weight to the optimum weight, (W_E) / (W_E)$_{opt}$, for the models designed with the UBC 1997 load pattern, average of optimum load patterns, and Karami et al. (2004) proposed load pattern, Average of 15 earthquakes.

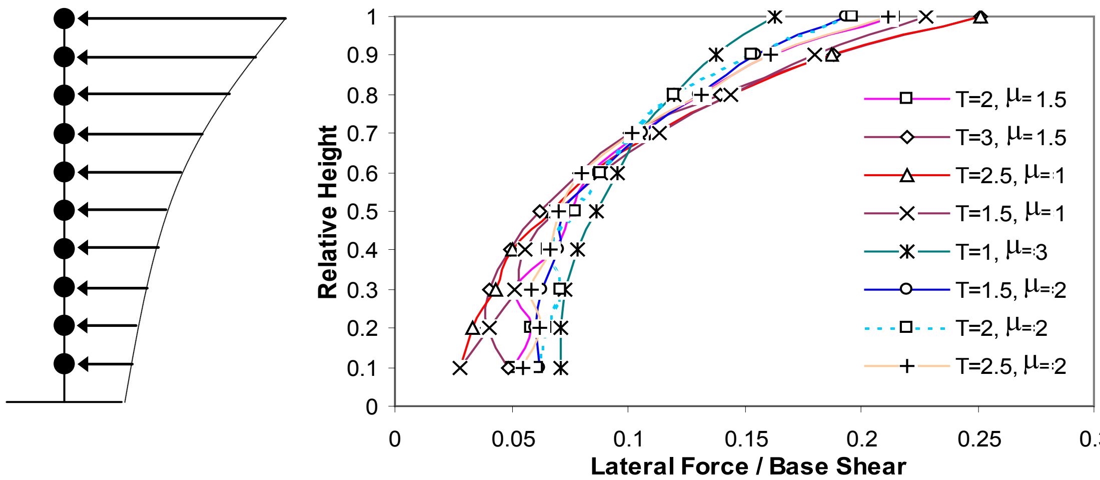

Figure 9. Optimum load patterns in parabolic shape

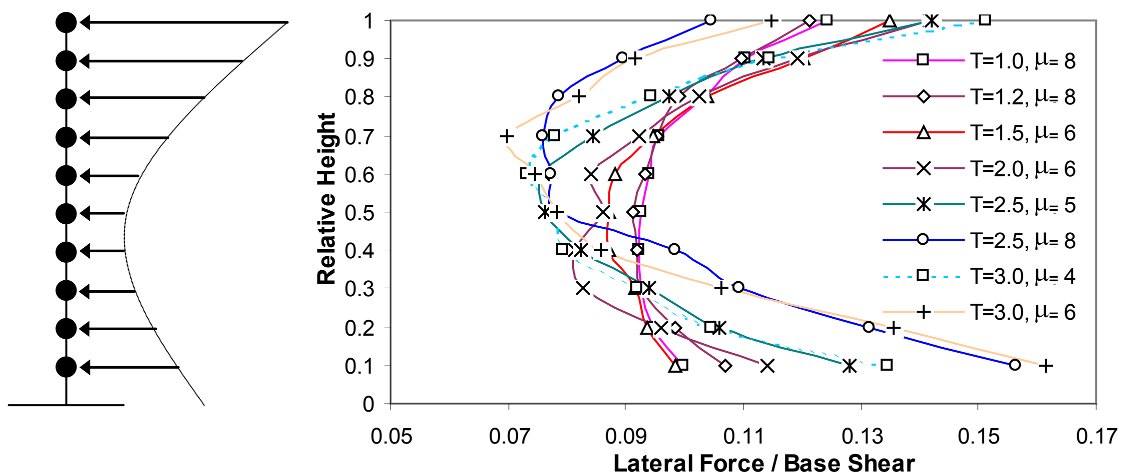

Figure 10. Optimum load patterns in hyperbolic shape

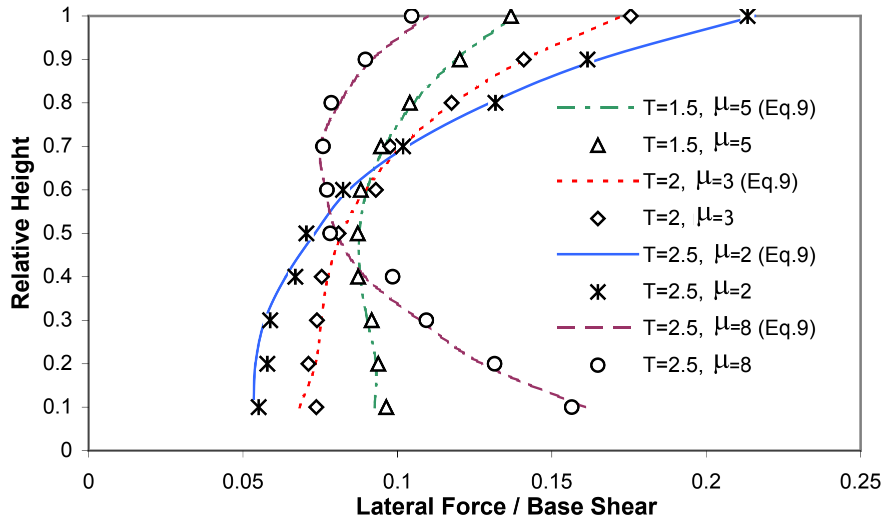

Figure 11. Correlation between Equation 9 and analytical results

Table 1. Strong ground motion characteristics

	Earthquake	Station	Ms	PGA (g)	USGS Soil
1	Imperial Valley 1979	H-E04140	6.9	0.49	C
2	Imperial Valley 1979	H-E04230	6.9	0.36	C
3	Imperial Valley 1979	H-E05140	6.9	0.52	C
4	Imperial Valley 1979	H-E05230	6.9	0.44	C
5	Imperial Valley 1979	H-E08140	6.9	0.45	C
6	Imperial Valley 1979	H-EDA360	6.9	0.48	C
7	Northridge 1994	CNP196	6.7	0.42	C
8	Northridge 1994	JEN022	6.7	0.42	C
9	Northridge 1994	JEN292	6.7	0.59	C
10	Northridge 1994	NWH360	6.7	0.59	C
11	Northridge 1994	RRS228	6.7	0.84	C
12	Northridge 1994	RRS318	6.7	0.47	C
13	Northridge 1994	SCE288	6.7	0.49	C
14	Northridge 1994	SCS052	6.7	0.61	C
15	Northridge 1994	STC180	6.7	0.48	C

Table 2. The preliminary and final arrangement of strength and stiffness

Story	**Preliminary Arrangement**		**Final Arrangement**	
	Story Strength (ton.f)	**Story Ductility**	**Story Strength (ton.f)**	**Story Ductility**
1	1753	4	1435	3.98
2	1753	2.46	1351	3.99
3	1753	1.78	1229	3.99
4	1753	1.41	1089	4.00
5	1753	1.38	953	4.00
6	1753	1.19	808	3.99
7	1753	0.98	662	3.99
8	1753	0.82	512	3.99
9	1753	0.59	371	3.97
10	1753	0.31	204	3.99
Cov		0.719		0.002
Total Strength	17532		8614	

Cov: Coefficient of variation

Table 3. Constant coefficients of Equation 9 as a function of relative height

Relative height	a	b	100 c	100 d
0	-5.3	38.8	23.7	39.9
0.1	-8.2	49.0	22.2	29.6
0.2	-10.6	59.2	19.6	18.4
0.3	-12.7	70.5	16.5	9.8
0.4	-12.3	81.0	9.8	5.4
0.5	-10.5	91.3	4.0	2.2
0.6	-8.4	103.2	0.1	-1.4
0.7	-0.8	114.6	-5.4	-3.9
0.8	10.3	127.2	-8.5	-7.2
0.9	26.1	140.9	-10.7	-10.0
1	49.8	157.0	-12.5	-12.1