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A NEW APPROACH FOR OPTIMUM DESIGN OF STRUCTURES UNDER DYNAMIC EXCIATION

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ABSTRACT

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. In order to obtain the optimum distribution of structural properties, 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 is shown that in general for a *MDOF* structure there exists a specific pattern for distribution of structural properties that results in an optimum seismic performance. The application of the proposed method for optimum seismic design of different structural forms such as truss-like structures and shear-buildings is presented.

Keywords: optimal strength pattern, performance-based design, seismic loading, ductility, optimum seismic performance

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. Consequently, the seismic codes are generally regarding the seismic effects as lateral inertia forces. 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. Use of forces as a design basis has remained more a matter of convenience than a representation of actual behavior during earthquakes. Many structures have apparently survived earthquakes capable of inducing inertia forces many times larger than those corresponding to their structural strength, if a linear response was assumed. The concept of ductility has then been introduced to reconcile this apparent inconsistency, and

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account for the anomaly survival with inadequate strength through nonlinear response. Subsequently, much research efforts have been directed to determining the available capacity of different structural systems, performing extensive experimental and analytical studies to determine their safe deformation capacity. This may be regarded as a more appreciation of the importance of deformation, as opposed to strength, in seismic design.

Indeed, a review of the history of seismic design indicates that initially design was purely based on strength, or force consideration. As the importance of deformation has come to better appreciation, the approach has been to attempt to modify the existing force based approach to include consideration of deformation, rather than to rework the procedure on more rational basis.

In the conventional seismic design, the pattern for distribution of structural properties such as strength, stiffness, and damping in a preliminary design is normally based on the presumption that the structure vibrates within its linear-elastic range [1]. Recent design guidelines, such as FEMA 356 [2] and SEAOC Vision 2000 [3], 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. This issue has been viewed by researchers from different angles.

Many experimental and analytical studies have been carried out to investigate the validity of the distribution of lateral forces according to seismic codes. Lee and Goel [4] analyzed a series of 2 to 20 storey 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. The consequences of using the code patterns on seismic performance have been investigated during the last decade [5,6,7]. Chopra [8] 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 [9]. 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 [10,11] 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 [9]. 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 [12,13], and led to the development of a new concept: optimum distribution pattern for seismic performance that is discussed in this paper.

2. OPTIMIZATION FOR DYNAMIC EXCITATION

2.1 Background

As discussed before, for decades seismic codes have regarded the seismic effects as lateral inertia forces. Consequently, in almost all optimization approaches developed for seismic design, the forces are regarded as static with a pre-assumed pattern of distribution such as triangular. In effect, these approaches are similar to the conventional optimization methods for design of structures subjected to static loadings. It is generally endeavored to induce a status of uniform deformation throughout the structure to obtain an optimum design as in Gantes et al. [14]. In an attempt for developing an optimization method for seismic design of steel frames, Gong et al. [15] used a set of story drift limits as performance objectives, and considered the seismic effects as static forces with parabolic distribution. From their results it can by concluded that the structural weight decreases as the deformation approaches to a uniform status. It should be noted that although in this procedure the effect of nonlinear behavior is considered, the seismic effects are regarded as external static forces rather than induced deformation. Therefore, the procedure is still remains similar to the conventional optimum design methods.

Some researchers have attempted to consider the effect of dynamic nature of seismic forces. Lee and Goel [4] proposed a design procedure using predefined performance targets. The procedure is based on minimizing the difference between the earthquake induced shear forces and the forces used for seismic design. Although within the linear range this concept seems to have a rather rational basis, the use of shear forces as a means of assessing the adequacy of design looses its weight in nonlinear ranges of vibration.

In his early attempts to establish and apply the performance-based method for seismic design of structures in late 1980's, the first author recognized the fact that several acceptable solutions could be obtained for a given set of objective targets such as ductility demands. This was later confirmed by the results of nonlinear dynamic analysis of shear buildings subjected to seismic excitations [10,11]. These early studies demonstrated that a conventional seismic design does not lead to a uniform distribution of ductility. Further investigations [12,13], suggested that we need to move towards a rather uniform distribution of ductility in order to reduce the ductility demand. Afterwards extensive studies have been conducted to highlight and establish a rational basis for this concept [16]. Indeed, in spite of those who assume the concept of uniform deformation as a performance objective, the authors are using it as a means for obtaining an optimum design. In the following, it has been attempted to substantiate the concept of uniform deformation and its application in optimum design for seismic excitation.

2.2 Inefficient Material

As discussed before, the use of distribution patterns for lateral seismic forces suggested by codes do not guarantee the optimum performance of structures. The current studies indicate that during strong earthquakes the deformation demand in structure does not vary uniformly. For instance, the steel frame shown in Figure 1 is designed in accordance with UBC1997 [9] and subjected to the Northridge earthquake of 1994 (CNP196). Non-linear dynamic analysis is conducted using the computer program DRAIN-2DX [17]. It is illustrated in Figure1 that deformation demand is not distributed uniformly. On the other hand, the maximum interstorey drift occurs almost at the top, and it decreases downward. Hence, it can be concluded that in some parts of the structure, the deformation demand does not reach the maximum level, and therefore, the capacity of the material is not fully exploited.

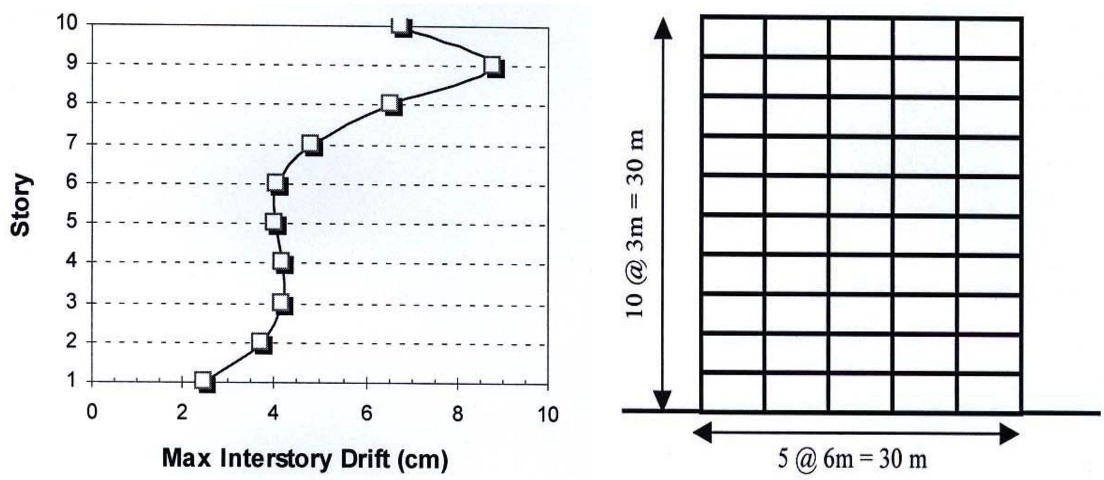

Figure 1. Inter-story drift distribution for a 10 story steel frame subjected to Northridge Earthquake 1994 (CNP196)

2.3 Principle of Strength-Deformation Reciprocal Relation

Early studies on shear building models have proved that it is possible to improve the performance of a model by shifting the material from strong to weak parts [12,13,16]. This results in a uniform distribution of deformation, and reduces the maximum deformation demand. The coincidence of uniform distribution of deformation with better seismic performance can be explained by the principle of strength-deformation reciprocal relation. The effect of variation of strength on seismic performance has been studied extensively [18,19]. These studies have lead to development of numerous strength- ductility (R-μ) relationships.

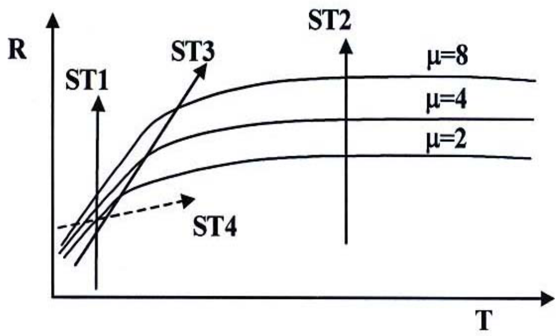

Figure 2. Typical R-μ–T relationship

Figure 2 shows a typical R-μ–T relationship. R is inversely proportional to strength. In this figure, three types of structures have been modeled. ST1 and ST2 represent low and high period structures, respectively. In these models, it is assumed that stiffness remains unchanged as strength varies. ST3 represents structures in which any decrease in strength is accompanied by a decrease in stiffness. This figure indicates that generally μ increases as strength decreases. It should be noted that in some exceptional cases this principle may be violated in low period range of ST3 type in which a decrease in strength is accompanied by an intensified decrease in stiffness (represented by the dotted arrow ST4 in Figure 2). Investigations have indicated that this rule is also applicable for MDOF systems [12].

2.4 Theory of Uniform Deformation

Consider the mentioned structure of Figure 1 in which the distribution of deformation is not uniform. If maximum story drift is taken as the failure criterion, the results would indicate that only some parts of the structure have failed. On the other hand, the deformation in the remaining parts is less than the maximum allowable limit. The Strength-Deformation Reciprocal Relation suggests that if the strength in these parts decreased, the deformation would increase. Hence, if the strength is decreased incrementally, we should eventually obtain a state of uniform deformation. At this point the material capacity is fully exploited. As any decrease in strength is normally accompanied by a decrease of material, a structure becomes lighter as deformation is distributed more uniformly as compared with a structure with non-uniform deformation. Therefore, in general it may be concluded that we need to reach the status of uniform deformation for optimum use of material. This is denoted as the *Theory of Uniform Deformation*.

3. OPTIMUM SEISMIC DESIGN OF TRUSS-LIKE STRUCTURES

The *Theory of Uniform Deformation* is examined by a conceptual example shown in Figure 3. The objective is to design a truss-like structure for sustaining four masses M1 to M4 by using any number of stud members connecting these masses to each other and to the supports A to E. This structure should not exceed a member ductility demand of 4 when subjected to the horizontal component of the Northridge Earthquake of 1994 (CNP196). A Rayleigh damping of 5% is assumed. No weight is considered for the masses and only the seismic forces are considered. Masses M1 to M4 are assumed to be 20, 5, 10, and 5 tons, respectively. Computer program Drain-2DX [17] is used for nonlinear dynamic analyses. At the starting point, a very general arrangement is chosen by considering all possible connections as shown in Figure 4. In the first step, an identical area of cross section of 1 cm^2 is assumed for all members. It is also assumed that the strength of each member is equal to (Af_y) in both tension and compression. The structure is subjected to the seismic excitation, and the ductility demand is calculated for all members. Subsequently, the area of cross section of all members is scaled until the maximum ductility demand reaches the target level of 4.

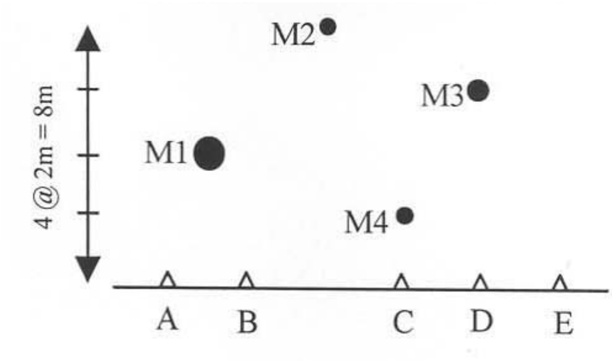

Figure 3. The position of masses and supports

The distribution of material and ductility demand at this stage is shown in Table 1. These results indicate that some members undergo much less deformation than others. This implies that the material is not fully exploited in some members.

Table 1: The preliminary and final arrangement of members

Members	Preliminary Arrangement		Final Arrangement	
	Cross Section (cm^2)	Member Ductility	Cross Section (cm^2)	Member Ductility
1	1243.6	4.009	2363.8	3.998
2	1243.6	2.303	0.0	---
3	1243.6	1.077	3246.9	4.001
4	1243.6	1.173	0.0	---
5	1243.6	1.129	0.0	---
6	1243.6	1.735	0.0	---
7	1243.6	0.453	0.0	---
8	1243.6	0.580	0.0	---
9	1243.6	0.484	140.1	4.002
10	1243.6	0.650	0.0	---
11	1243.6	0.209	522.2	3.999
12	1243.6	0.764	330.0	4.001
13	1243.6	0.623	0.0	---
14	1243.6	0.485	0.0	---
15	1243.6	0.273	0.0	---

Members	Preliminary Arrangement		Final Arrangement	
	Cross Section (cm^2)	Member Ductility	Cross Section (cm^2)	Member Ductility
16	1243.6	1.191	691.2	4.000
17	1243.6	1.123	818.6	4.000
18	1243.6	0.189	0.0	---
19	1243.6	0.940	0.0	---
20	1243.6	1.800	1146.6	3.999
21	1243.6	0.457	10.9	3.997
22	1243.6	1.517	581.6	4.000
23	1243.6	1.233	0.0	---
24	1243.6	0.226	0.0	---
25	1243.6	0.690	0.0	---
26	1243.6	1.057	629.6	4.001
Cov		0.785		0.001
Weight	162.3 ton		52.7 ton	

Cov: Coefficient of variation

Considering the *Theory of Uniform Deformation*, it should be attempted to move towards a uniform ductility distribution demand to obtain a lighter structure. To accomplish this, the following optimization procedure is employed:

An arbitrary primary pattern is assumed for the distribution of structural properties that control the response of structure (such as strength, stiffness, and damping). Here, the cross section area is the only controlling parameter. Hence, as mentioned before, a uniform pattern is chosen.

The structure is subjected to the excitation, and the maximum deformation is calculated, and compared with the target value. The structural properties are then scaled, without changing the primary pattern, until the maximum deformation demand reaches the target value. This pattern is regarded as a feasible answer, and referred to as the first acceptable pattern. For the above example, member ductility represents the deformation demand, and the results of the first and the final steps are presented in Table 1.

The coefficient of variation (*cov*) of deformation distribution within the structure is calculated. If the *cov* is considered to be small enough, we can stop, and consider the pattern as practically optimum. Otherwise the analysis continues. The cov of the first acceptable pattern was determined as 0.785. It is decided that the *cov* is high, and the analysis should be continued.

At this stage the distribution pattern of structural properties is modified. Using the *Theory of Uniform Deformation*, the inefficient material is reduced until an optimum structure is obtained. To accomplish this, the positions where the deformation is less than the target value are identified, and the material is reduced accordingly. Experience has shown that this alteration should be applied incrementally in order to achieve convergence in the numerical calculations. Hence, the following equation is used in the present studies:

$$[(p_{sc})_i]_{n+1} = [(p_{sc})_i]_n \left[\frac{d_i}{d_{ti}}\right]^{\alpha} \tag{1}$$

Where d_i and d_{ti} are demand and target deformations at position i. $(P_{sc})_i$ is the structural control parameter, relating to position i. n denotes the step number. α is the convergence coefficient ranging from 0 to 1. For the above example, an acceptable convergence was obtained for a value of α=0.2. Considering the cross-section area, A_i, as the structural control parameter and member ductility,μ_i , as the deformation demand and substituting 4 as the target deformation for all members, the following equation is obtained:

$$[(A)_i]_{n+1} = [(A)_i]_n \left[\frac{\mu_i}{4}\right]^{0.2} \tag{2}$$

Using these modified cross sections; the procedure is repeated from step 2, until a new feasible pattern is obtained. It is expected that the *cov* of deformation distribution for the new 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 deformation prevails. Starting from a *cov* of 0.785, we reach a *cov* of 0.001 at the final step. A comparison of the results of primary and final steps in Table 1 leads to the following conclusions:

The weight of total material has decreased from 162.3 ton to 52.7 ton.

Member ductility demands in the final step have become remarkably uniform.

The method has been able to recognize and eliminate the redundant and inefficient members. Out of 26 members in the primary arrangement in Figure 4 only 11 members remain in the final step as shown in Figure 5.

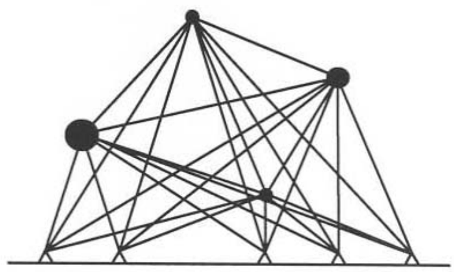

Figure 4. Preliminary arrangement of members

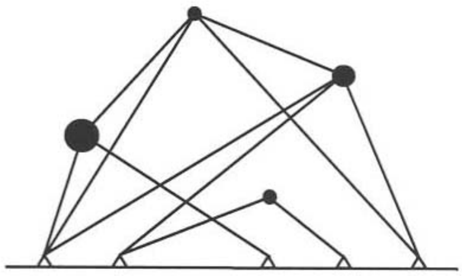

Figure 5. Final arrangement of members

4. OPTIMUM SEISMIC DESIGN OF SHEAR BUILDINGS

The *Theory of Uniform Deformation* can be easily adapted for evaluation of optimum patterns for shear buildings. To obtain such optimum patterns, in principle, the steps mentioned in the previous section are followed with some modifications. 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 storey shear force). Hence, for shear buildings, we can determine the optimum pattern for distribution of seismic lateral loads instead of distribution of strength. Let us assume that we want to evaluate the most appropriate lateral loading pattern to design a 10-story shear building with a fundamental period of 1 sec, so that it can sustain the Northridge earthquake of 1994 (CNP196) without exceeding a maximum story ductility demand of 4. In the example model, each floor is considered as a lumped mass and the total mass of the structure is distributed uniformly over its height as shown in Figure 6. The Rayleigh damping is adopted with a constant damping ratio 0.05 for the first few effective modes and non-linear dynamic analyses are conducted utilizing the computer program DRAIN-2DX [17]. Considering the *Theory of Uniform Deformation,* the following optimization procedure is used:

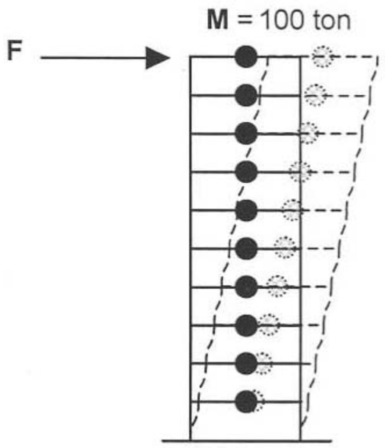

Figure 6. Primary load distribution pattern for the shear-building

1. Arbitrary patterns for primary height-wise distribution of strength and stiffness are considered. However, for shear buildings it is assumed that these two patterns are similar, and therefore, an identical pattern is assumed for both strength and stiffness. Here, the uniform pattern of Figure 6 is chosen for the primary distribution of strength and stiffness.

The stiffness pattern is scaled so as to attain a fundamental period of 1 sec.

Maximum ductility demand is calculated by performing nonlinear dynamic analysis for the given exaction. Subsequently, the strength is scaled (without changing the primary pattern) until the maximum deformation demand gets to a target value of 4. The resulting pattern is a feasible solution and can be considered as the first acceptable pattern. The first and final steps are illustrated in Table 2.

Table 2: The preliminary and final arrangement of Strength and Stiffness

Story	Preliminary Arrangement			Final Arrangement		
	Story Stiffness (ton.f/m)	Story Strength (ton.f)	Story Ductility	Story Stiffness (ton.f/m)	Story Strength (ton.f)	Story Ductility
1	176717	1753	4	256456	1433	3.99
2	176717	1753	2.46	241577	1350	3.99
3	176717	1753	1.78	219796	1228	3.99
4	176717	1753	1.41	194841	1089	4.00
5	176717	1753	1.38	170502	953	4.00
6	176717	1753	1.19	144584	808	3.99
7	176717	1753	0.98	118423	662	3.99
8	176717	1753	0.82	91522	511	3.99
9	176717	1753	0.59	66385	371	3.98
10	176717	1753	0.31	36515	204	3.99
Cov			0.719			0.001
Sum		17352			8610	

The *cov* (coefficient of variation) of story ductility distribution is determined. The procedure continues until *cov* decreases down to an acceptable level. For the first feasible pattern, the *cov* was determined as 0.719. The cove is considered to be high, and the analysis continues.

Considering the *Theory of Uniform Deformation*, the distribution pattern is modified. To achieve this, the stories where the ductility demand is less than the target values are identified, and weakened by reducing the strength and stiffness. Similar to Equation 1, the following equation is used for the good convergence:

$$[V_i]_{n+1} = [V_i]_n \left[\frac{\mu'_i}{\mu'_{ti}} \right]^{\alpha} \tag{3}$$

Where μ'_i is ductility demand at i[th] story, and μ'_{ti} is the target ductility assumed as equal to 4 for all stories. V_i is the shear strength of the i[th] story. n denotes the step number. α is the convergence coefficient ranging from 0 to 1. For the above example, an acceptable convergence has been obtained for a value α=0.1. At this stage, a new pattern for heightwise distribution of strength is obtained. As mentioned before, the same pattern is used for heightwise distribution of stiffness. Now 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 rather uniform ductility demand is achieved. The story ductility patterns for preliminary and final designs are compared in Figure 7. This figure indicates the efficiency of this method to reach the status of uniform ductility demand.

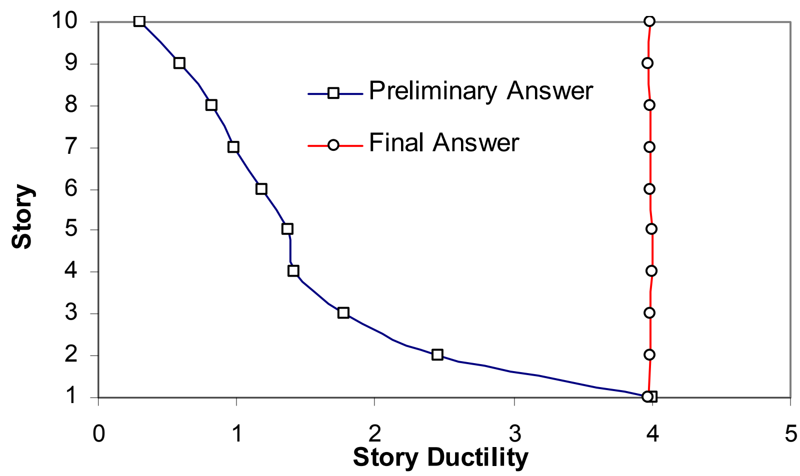

Figure 7. Primary and Final distribution pattern for story Ductility, 10 story shear building with T=1 Sec and μ_{ti}=4, Northridge 1994 (CNP196)

Table 2 illustrates the results of analysis for the first and final step. Figure 8 demonstrates the variation of *cov* and the total strength from the first feasible pattern toward the final one. It can be concluded that the proposed method has a good capability for converging to the optimum solution. As shown in Figure 8, the total strength decreases up to 40% in five steps. The figure also indicates that the decrease in *cov* is accompanied by a decrease in total strength. Here the total strength is in proportion to the total weight of the seismic resisting system. These results are in agreement with the *Theory of Uniform Deformation*.

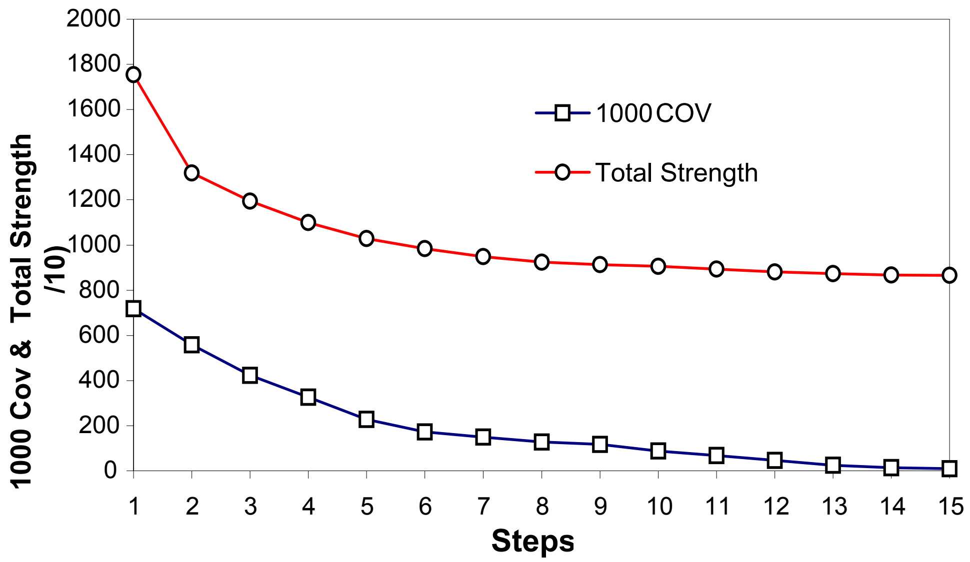

Figure 8. Cov of story ductility factors and total story strength for feasible patterns, 10 story shear building with T=1 Sec and μ_{ti}=4, Northridge 1994 (CNP196)

The height wise distribution of strength can be converted to the distribution of lateral forces. Such pattern may be regarded as the optimum pattern of seismic forces for the given earthquake. As shown in Figure 9, this would enable us to compare this optimum pattern with the conventional lateral load distribution suggested by codes for seismic design.

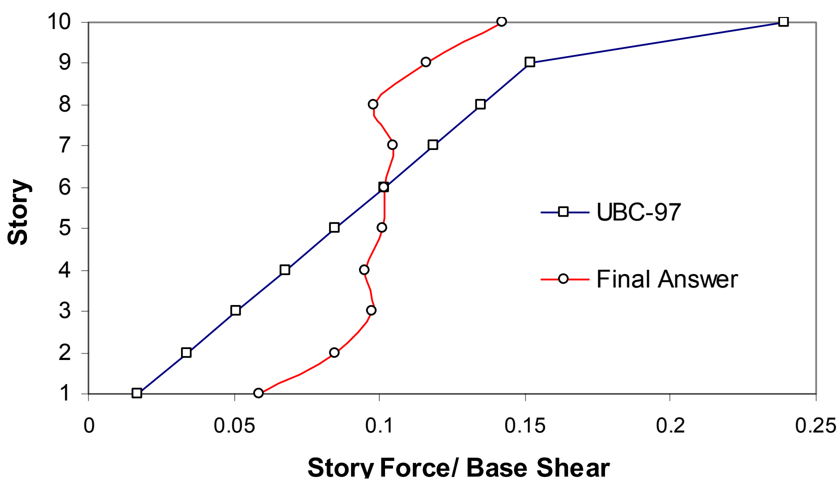

Figure 9. Comparison of UBC-97 & Optimum lateral force distribution, 10 story shear building with T=1 Sec and μ_{ti}=4, Northridge 1994 (CNP 196)

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 result, for the previous example four different initial load patterns have been assumed:

A concentrated load on the roof level

Triangular distribution similar to the UBC code of 1997 [9]

Rectangular distribution

An inverted triangular distribution with the 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 10.

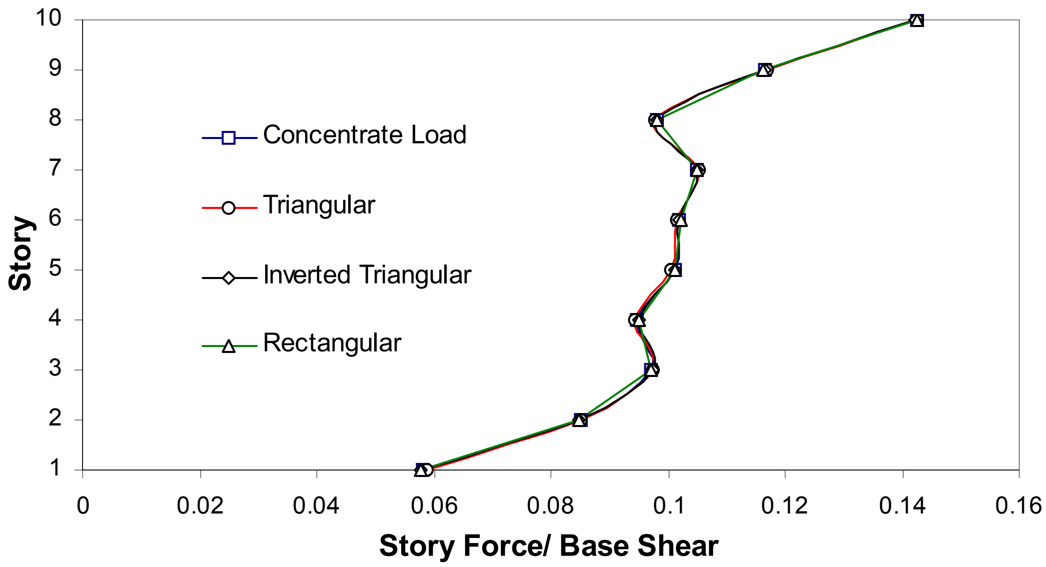

Figure 10. Optimum lateral force distribution for different initial load patterns, 10 story shear building with T=1 Sec and μ_{ti}=4, Northridge 1994 (CNP196)

As shown in Figure 10, 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 additional shear buildings and ground motions.

To investigate whether or not these findings are dependant on the selected seismic excitation, the following seismic records are also applied to the foregoing 10-storey shear building model: (1) The 1994 Northridge earthquake CNP196 component with a PGA of 0.42g, (2) The 1979 Imperial Valley earthquake H-E08140 component with a PGA of 0.45g, (3) The 1992 Cape Mendocino earthquake PET090 component with a PGA of 0.66g, and (4) A synthetic earthquake record generated to have a target spectrum close to that of the UBC1997 [9] code with a PGA of 0.44g.

Subsequently, the optimum strength-distribution patterns corresponding to these excitations are determined, and compared with a typical conventional seismic load pattern suggested by the UBC1997 [9] in Figure 11. The figure indicates that for the same ductility demand, the optimum design requires less strength as compared with the conventional design.

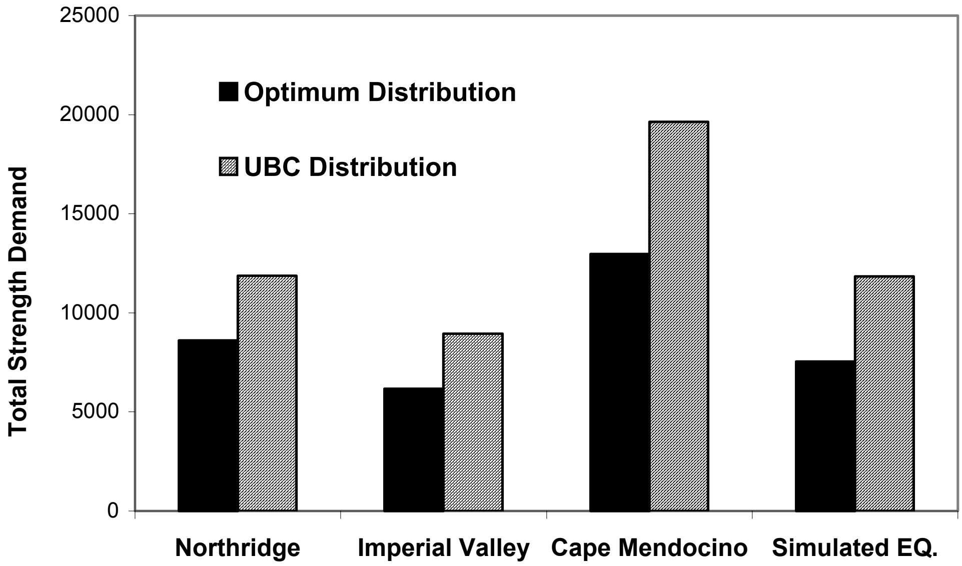

Figure 11. Comparison of total strength demand for UBC-97 and Optimum distribution, 10 story shear building, T=1 Sec and μ_{ti}=4

5. CONCLUSIONS

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.

It is shown that using the strength pattern suggested by seismic codes does not lead to a uniform distribution of deformation demand, 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.

By introducing an iterative method, Theory of Uniform Deformation has been adapted for topology optimization in seismic design of truss-like structures. It is shown that this method can reduce the required structural weight by eliminating the redundant and inefficient members.

With some modifications, 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.

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