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NONLINEAR DYNAMIC ANALYSIS OF CHIMNEY-LIKE TOWERS

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ABSTRACT

Evaluation of seismic vulnerability of very flexible structures such as high-rise petro-chemical and refinery stacks (chimney) and power plant chimneys is a challenging problem in earthquake engineering. Their equipments and structures are often considered as vital facilities and thus they must be fully functional after even very strong ground motions. From other point of view, numerical modeling of such mega-structures with numerous elements may not allow to consider all details of mechanical characteristics of consisting materials, particularly nonlinear performance of elements during large deformations. Therefore, a simplified model corresponding to dynamic characteristics of whole structure is substantially needed to investigate seismic performance and failure modes of these essential structures subjected to strong ground motions. The procedure for developing a 2-D simplified nonlinear model based on moment-curvature in some plane-sections of a 3-D sophisticated model but linear system having almost identical dynamic properties is discussed. If micro elements be used for structural modeling of the chimney like structures, static nonlinear analysis or dynamic linear analysis will be economically suitable for assessment investigation. But of the most important problems in earthquake behavior of the structures is hysteretic behavior of material and section properties. The significance of this study, if any, is mainly concentrated on model simplification that provides sufficient accuracy based on a nonlinear discrete model. Tous power-plant chimney is investigated numerically as an example.

Keywords: High-rise structures; seismic vulnerability; damage index; nonlinear model

1. INTRODUCTION

Some special structures such as high-rise towers and power plant chimneys and their equipments and/or in-charge facilities are often considered to be fully functional after even very strong ground motions. Consequently, seismic assessment for actual performance of structures during earthquakes, the nonlinear dynamic analysis is required. Then the damage

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indices of structure have to be calculated, using appropriate damage models since these indices could be numerical representation of damage states of the structures. However, numerical modeling of such mega-structures with numerous elements may not be allowed to include all details of mechanical characteristics of consisting materials, particularly nonlinear performance of elements during large deformations. Therefore, a simplified model corresponding to dynamic characteristics of whole structure is substantially needed to investigate seismic performance and failure modes of these essential structures subjected to strong ground motions.

In this paper a practical method to assessment chimney like towers such as smoke stacks, minarates and TV towers is presented based on nonlinear dynamic analysis considering appropriate hysteretic behavior of section and material properties. Damage analysis is presented as well. Such an analysis is required for fragility curve of the structure. Many nonlinear dynamic analyses are required for fragility cures. Only using simple reliable macro (beam-column element) model can achieve such a goal. Some examples of damaged towers in previous earthquakes show the necessity of the nonlinear dynamic analysis of these types of structures.

1.1 Seismic Behavior of Minarates

Classification and description of damage were documented for 64 historical and monumental structures surveyed after the August 17 (Kocaeli) and November 12 (Düzce), 1999 earthquakes (Turkey) by Sezen *et al.* [1-2]. The levels of damage recorded, construction type, and locations of the surveyed mosques and minarets were reported. Survey results indicated that the majority (70%) of the masonry and reinforced concrete minarets surveyed in Düzce was observed to sustain damage of intensity severe to collapse. Collapse of the minarets, which are tall slender towers, constituted a hazard to life and to surrounding buildings. Horizontal circumferential cracks and spalling of concrete near the bottom of minaret cylinder (Figure 1) were the common types of damage reported for reinforced concrete minarets. In most cases, damage was observed within a diameter above the intersection of the cylindrical portion of the minaret and the pyramid-shaped section providing the transition from a cube to cylinder. The location of failure in the reinforced concrete minarets that collapsed was found to be near the bottom of the cylinder where the longitudinal reinforcing bars were spliced (Figure 2). Less frequently, transition segment, mid-height of the cylinder (Figure 1), and top of the minaret were among the sections where minarets sustained significant damage.

1.2 Seismic Behavior of Chimneys

During the Ismit (Kocaeli) Earthquake of August 17, 1999, a 115 m. high reinforced concrete chimney or heater stack, located at the Tüpras Refinery, collapsed. The falling debris cut 63 pipes, which contributed to interrupted production for more than 14 months. This stack was designed and constructed according to international standards and is representative of similar structures at refineries throughout the world, including those in earthquake-prone regions. The opening was located about 1/3 of the height above the base and appeared to be the region of initiation of the collapse. The debris cut many lines, which fueled fires that shut down the refinery for months. The collapsed stack was 115 m. The remnants of the stack are shown more clearly in Figure 3, where it is evident that the failure

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occurred in the vicinity of the opening.

Figure 1. Damage to cylinder mid-height

Figure 2. Collapse of minarets due to failures near the bottom of the cylinders

Figure 3. Remnants of failed stack

The investigation is focused on the dynamic response of the stack due to an earthquake motion recorded at a nearby site. Gould *et al.* [3] and Huang and Gould [4] presented interesting results of analysis of the Tüpras stack. They presented nonlinear static analysis of the collapsed stack using a demand-collapse comparison. The demand was represented by an acceleration-displacement response spectrum based on the recorded motion as well as some smoothed adaptations typical of design spectra, while the capacities were calculated from pushover curves using a nonlinear reinforced concrete finite element analysis. The results confirm that the stack could readily fail under the considered earthquake and were also consistent with the debris pattern. Pushover results shown in Figure 4 are cracking patterns for the model with and without opening in a step. Of course, a full nonlinear

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dynamic analysis would be necessary to more completely simulate the failure.

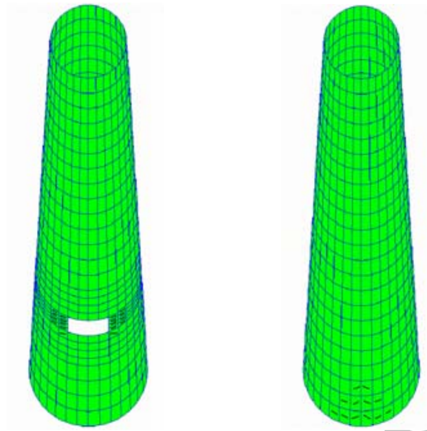

Figure 4. Cracking pattern for the model with and without opening [3]

1.3 Vibration Test

Yamamoto and Maeda [5] presented measured micro-tremor to acquire vibration characteristics, which was used to construct FE model for the superstructure by simulating base-fixed horizontal natural frequencies of the brick chimney of the height of 23m in Tokoname (Japan) which was built in 1922 (Figure 5). Soil-foundation interaction was treated by installing soil springs which were determined by simulating soil-coupled frequency. With the soil-coupled FE model, linear dynamic response analysis was carried out for four kinds of input ground motions. It was found that the base of the chimney will have damage at maximum acceleration from 41 to 77 Gal, should the collapse be caused by excessive tensile stress at mortal joints.

Figure 5. Appearance of the chimney

1.4 soil-structure interaction

Generally a fixed base model is adopted for seismic design when the effect of soil-structure

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interaction is considered negligible. When this effect is taken into account, the use is commonly made of a sway rocking model for the structures with spread or short-pile foundation. The SSI effect on the structure as well as piles, however, is considered significant in case of long piles in soft ground. In such a case, some coupled system should be adopted for the seismic design model. A finite element model would be powerful in evaluating the SSI effect if it were linear. Also as a simple model a lumped mass-spring-beam model can be considered appropriate for taking into account the major SSI effect as well as the material non-linearity in the structure and the ground [6]. Figure 6 shows the models mentioned above. Halabian and Kabiri have investigated the effect of soil structure interaction on the response of stack like structures [7].

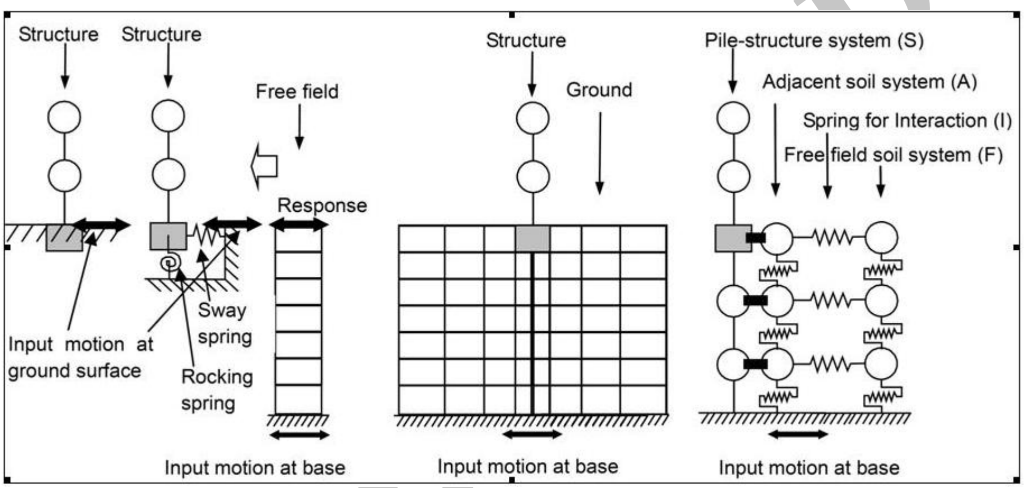

Figure 6. Models of preliminary analyses as candidates for the seismic design

In this paper, an exclusive review of relevant modeling and methodology for estimation of damage indices in special RC structures has been carried out. A procedure for developing a 2-D simplified nonlinear model based on moment-curvature in some plane-sections of a 3-D sophisticated model but linear system having almost identical dynamic properties is developed.

The significance of this study, if any, is mainly concentrated on model simplification that provides sufficient accuracy based on a nonlinear discrete model. Damage indices under different hazard levels of excitations provide the database for vulnerability and seismic risk analyses for the both elements and whole of structures. For instance, from relative displacement responses of nonlinear model, one may estimate the most probable modes of failure under MCE level of input excitation.

As noted above, the damage indices of structure have to be calculated, using appropriate damage models. It is because of the fact that damage indices are quantitative values for damage states of the structures. These models are usually based on the maximum deformations, hysteretic energies and structural deteriorations. Damage indices are appropriate tools for quantifying numerically the damage in structures sustained under earthquake loading. Many researchers have defined various damage indices. Damage indices may be defined locally for

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elements or globally for whole the structure. An extensive review of defined damage indices for various types of structures has been carried out [9-10]. Vulnerability of an existing structure based on seismic hazard analysis has been assessed [11].

Park-Ang damage index [12], considered in IDARC [13], is the most usual damage index for damage analysis of reinforced concrete structures. It can be calculated in the element, storey and overall scales. An important point is that the damage indices of stories are calculated based on hysteretic energy weighting factors and therefore the structural importance of beams and columns are the same. Also overall damage index of structure is calculated by summation of the storey damage indices on the basis of hysteretic energy of each storey.

Seismic vulnerability and damage analysis of special structures have been carried out successfully using IDARC program [14-15]. The key idea of structural modeling of the special structures is to develop a simplified 2-D model using beam-column elements based on moment-curvature in some plane sections. Appropriate results have been achieved by nonlinear dynamic analysis of these simplified models.

Some special structures such as high rise towers are often analyzed elastically using shell or solid elements, ignoring the effect of nonlinear properties. But it is clear that in order to understand the actual behavior of the structure, nonlinear dynamic analysis must be carried out. To investigate the structural behavior of towers under future earthquakes, the structure should be modeled as a way to perform the seismic behavior properly. The structural modeling can be carried out using various types of elements. Among these elements beam-column element is an appropriate type considering both the complexity of the structural performance and the time consuming of the analyses.

In this paper the seismic performance of Tous power-plant chimney, which is located in a seismically active zone, has been evaluated.

2. STRUCTURAL MODELING

As mentioned earlier, it is possible to model the structure using various types of elements. But it is clear that nonlinear dynamic analysis of structure is required for seismic vulnerability. It is possible to use various types off elements to model high rise cantilever type towers. The beam-column element is the most appropriate method for structural modeling, because of limitations in the computation time. Figure 7 shows the discrete model of the structure using lumped mass and beam-column elements. Beam-column elements are modeled considering flexural, shear and axial deformations. Flexural and shear components of the deformation are modeled using three parameter Park model described later. The axial deformation component is modeled using a linear-elastic spring.

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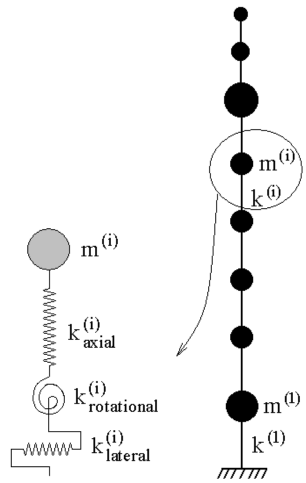
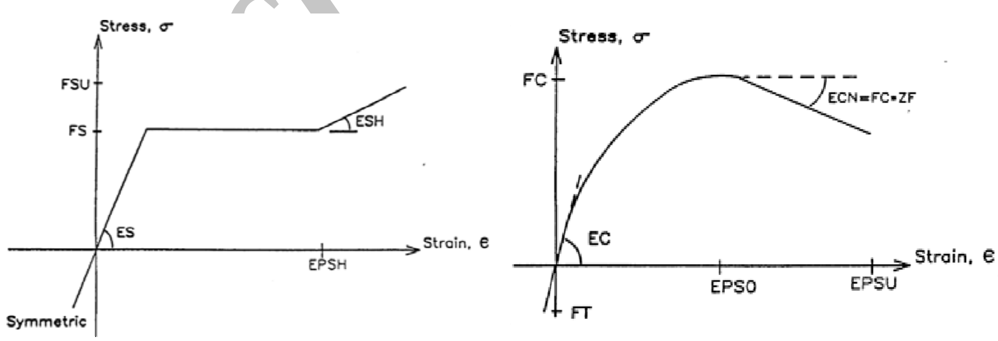


Figure 7. Discrete structural beam element model

3. NONLINEAR MODELING OF MATERIALS

Softening of the concrete under deformation near its limit state of resistance is an important property that must be considered in nonlinear dynamic analysis. The moment–curvature relation method is one of the specialty approaches used to express the real behavior of concrete structures when deformed. This method could widely serve all general cross-sectional shapes that may be used as different geometric cross-sections in towers. The longitudinal concrete stresses can be found from the longitudinal concrete strains by using the appropriate concrete stress–strain relationship in compression. Figure 8 shows the relation between stress and strain for steel and concrete materials.



(a) (b)

Figure 8. Stress – strain relationship for steel (a) and concrete (b)

Assuming that there is appropriate reinforcement such that the collapse of the tower will be due to crushing of concrete in primary stress arising from axial load and flexure, it can be used from the moment - curvature diagram in order to damage analysis.

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4. PUSHOVER ANALYSIS

The nonlinear pushover analysis, or collapse mode analysis, is a simple and efficient technique to predict the seismic response of prior to a full dynamic analysis. A pushover analysis can establish the sequence of component yielding, the potential ductility capacity, and the adequacy of the lateral strength of the structures. The pushover analysis option performs an incremental analysis of the structure subjected to a distribution of lateral forces.

The pushover analysis may be carried out using force control or displacement control. In the former option, the structure is subjected to an incremental distribution of lateral forces and the incremental displacements are then calculated. In the latter option, the structure is subjected to a displacement profile, and the lateral forces that needed to generate the deformation are calculated. Typically, since the deformed profile is not known, and an estimate of the lateral distribution of forces can be made, force control is commonly used. For displacement control, the user must specify the target maximum deformation profile of the structure. This profile is internally divided by the number of steps specified by the user, and then incrementally applied to the structure. In the force control option the user must specify the maximum force distribution, or select one of the force distributions available in the program: Uniform Distribution, Inverted Triangular Distribution, Generalized Power Distribution Modal Adaptive Distribution.

The generalized power distribution was introduced to consider different variation of the storey accelerations with the storey elevation. This distribution was introduced to capture different modes of deformation, and the influence of higher modes in the response. The force increment at floor “i” is calculated according to:

$$\Delta F_i = \frac{W_i h_i^k}{\sum_{j=1}^{N} W_j h_j^k} \Delta V_b \tag{1}$$

where W_i and h_i are the storey weight and the storey elevation, respectively, and ΔV_b is the increment of the building base shear and k is the parameter that controls the shape of the force distribution. The recommended value for k may be calculated as a function of the fundamental period of the structure (T):

$$k = 1.0 \qquad for \quad T \le 0.5 \tag{2a}$$

$$k = 2.0 \qquad for \quad T \ge 2.5 \tag{2b}$$

$$k = 1.0 + \frac{T-0.5}{2} \qquad otherwise \tag{2c}$$

5. NONLINEAR DYNAMIC ANALYSIS

The nonlinear dynamic analysis is carried out using a combination of the Newmark-Beta

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integration method, and the pseudo-force method. The viscous damping matrix is calculated in the program using one of the following options: Mass or Stiffness proportional damping and Rayleigh damping. In the program IDARC the circular frequency corresponding to the first mode of vibration is used for the mass and stiffness proportional damping, while the circular frequencies corresponding to the first and second modes are used for the Rayleigh damping type. Under these conditions, mass proportional damping will yield a smaller damping ratio for the higher modes, while stiffness proportional and Rayleigh damping may result in a higher critical damping ratio for the higher modes.

Modeling the hysteretic behavior of structural elements is one of the core aspects of a nonlinear structural analysis program. The three parameter Park hysteretic model is as part of the original release of IDARC. The hysteretic model incorporates stiffness degradation, strength deterioration, non-symmetric response, slip-lock, and a trilinear monotonic envelope. The model traces the hysteretic behavior of an element as it changes from one linear stage to another, depending on the historey of deformations. The model is therefore piece-wise linear. Each linear stage is referred to as a branch. Figures 9 shows the effect of various parameters on the shape of the hysteretic loops.

6. DAMAGE ANALYSIS

In order to damage analysis, it is necessary to quantify the structural damage and therefore damage index must be defined. Many damage indices have been defined. The most usual damage index is the Park-Ang model. It is defined as combination of maximum deformation and hysteretic energy [12]:

$$DI = \frac{\delta_m}{\delta_u} + \frac{\beta}{\delta_u P_y} \int dE_h \tag{3}$$

in which δ_m is the maximum deformation of the element (nonlinear dynamic analysis), δ_u is the ultimate deformation (push-over analysis), β is a model constant parameter (0.1-.015), $\int dE_h$ is the hysteretic energy absorbed by the element during the earthquake, P_y is the yield strength of the element. Park-Ang damage model can be extended to the storey and overall scales. Park-Ang damage indices for various damage states are shown in Table 1.

Table 1: The relation between damage index and damage state

Degree of Damage	Damage Index	State of Building
Slight	<0.1	No Damage
Minor	0.1-0.25	Minor Damage
Moderate	0.25-0.4	Repairable
Severe	0.4-1.0	Beyond Repair
Collapse	>1.0	Loss of Building

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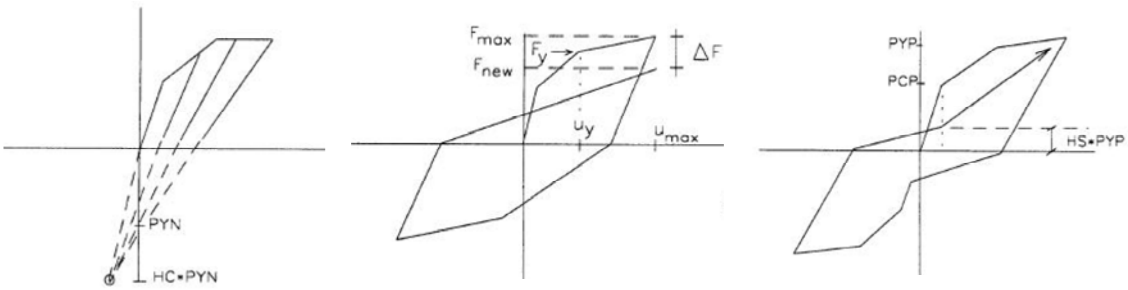

stiffness degradation strength deterioration pinching

Figure 9. Control parameters for the three parameter hysteretic model [15]

7. CASE STUDY

The chimney has cylindrical shape with 100m height, and external diameter of 10 meter. The cylindrical shell thickness is 80cm from base to the elevation of 17.5m and then it is reduced to 30cm in upper part. The schematic view of the chimney has been presented in Figure 10.

Figure 10. Schematic view and photograph of chimney

Seismic hazard analysis has been performed to determine the peak ground acceleration (PGA) in both case studies. For instance for return periods of 75, 500, 1000 and 2500 years at the site of chimney, PGA values resulted in 0.13, 0.26, 0.33, and 0.43g, respectively.

In order to determine the periods and the effects of different modes of the structure, eigenvalue analysis must be performed. For the studied chimney the periods of the first three modes associated to 1.67, 0.30 and 0.11 sec, respectively. It is important to note that the mode shapes and frequencies of the structure were measured using Ambient Vibration Survey (AVS) [16]. The first and second frequencies resulted in 1.48 and 0.29 sec

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respectively. The third mode of AVS test has been associated to the torsional mode, and therefore it is not comparable with numerical results. Figure 11 shows the flexural mode shapes. It is clear from the first mode that the considered power distribution of lateral force is appropriate for pushover analysis.

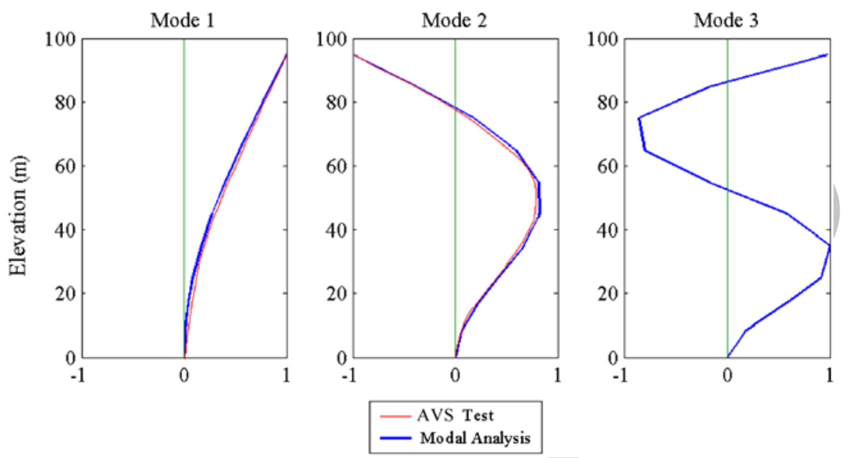

Figure 11. Mode shapes of chimney

The results of the pushover analyses for various types of lateral force have been presented in Figure 12. As shown in this Figure, the uniform distribution of lateral load gives the maximum base shear. Also the base shear of the triangle distribution is more than that of the power distribution for the power factors equal to 1.5, 1.625 and 2 that are corresponding to the periods of 1.5, 1.75 and 2.5 sec, respectively. Note that the fundamental period of the structure is 1.67 sec. It is seen that the ultimate strength of the structure is approximately 19% of weight.

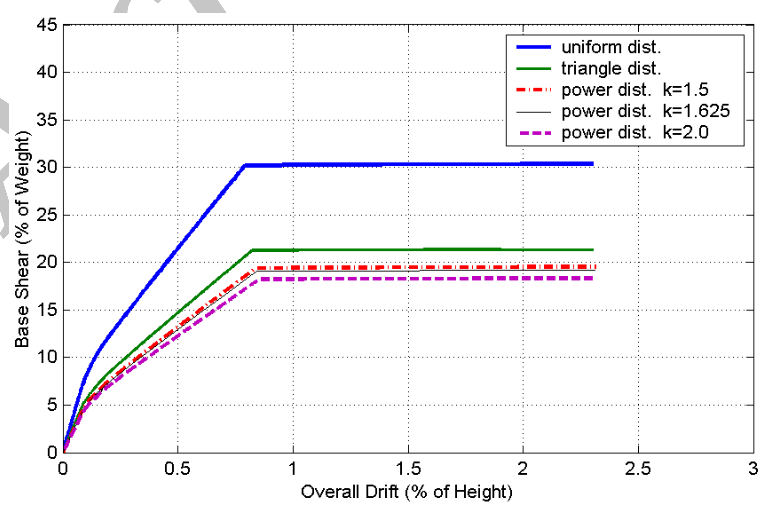

Figure 12. Comparison of pushover analyses of chimney

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For all kinds of distribution proposed for lateral loads, the mechanism is similar to Figure 13, in which the plastic hinge occurs in the elevation 17.5m, the section that thickness changes sharply.

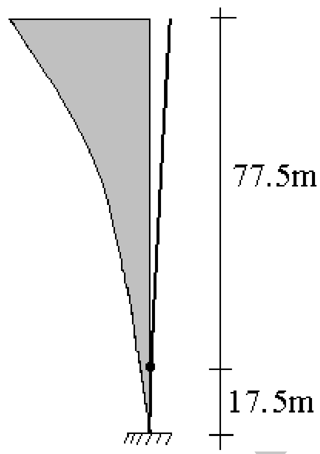

Figure 13. Failure Mode of chimney (position of plastic hinge)

Nonlinear dynamic analysis has been performed under the input motions shown in Figure 14.

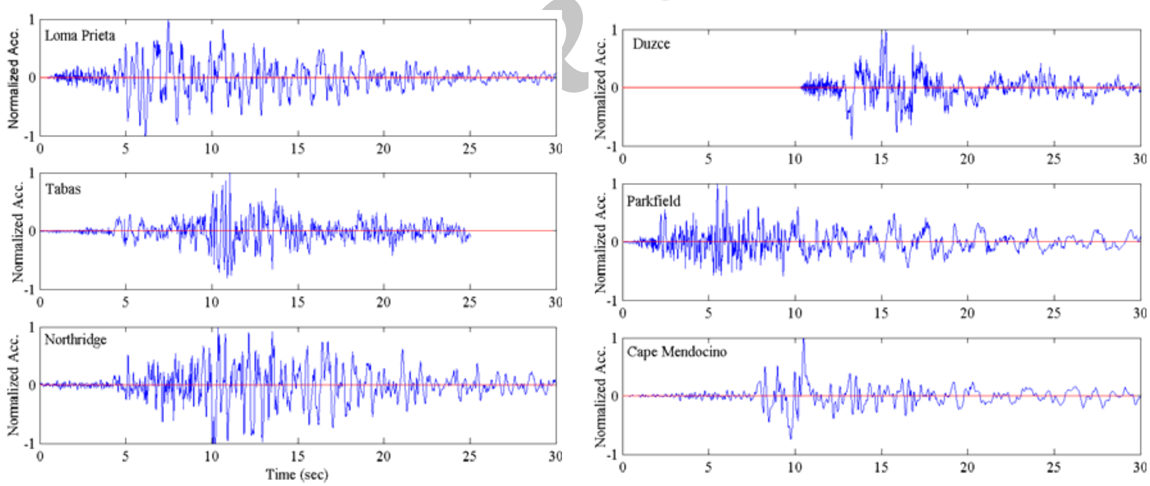

Figure 14. Acceleration time history records

Nonlinear dynamic analysis has been performed under the input motions shown in Figure 14. Time history responses at top of the structure for input motion scaled to PGA level of 75 years return period are given in Figure 15.

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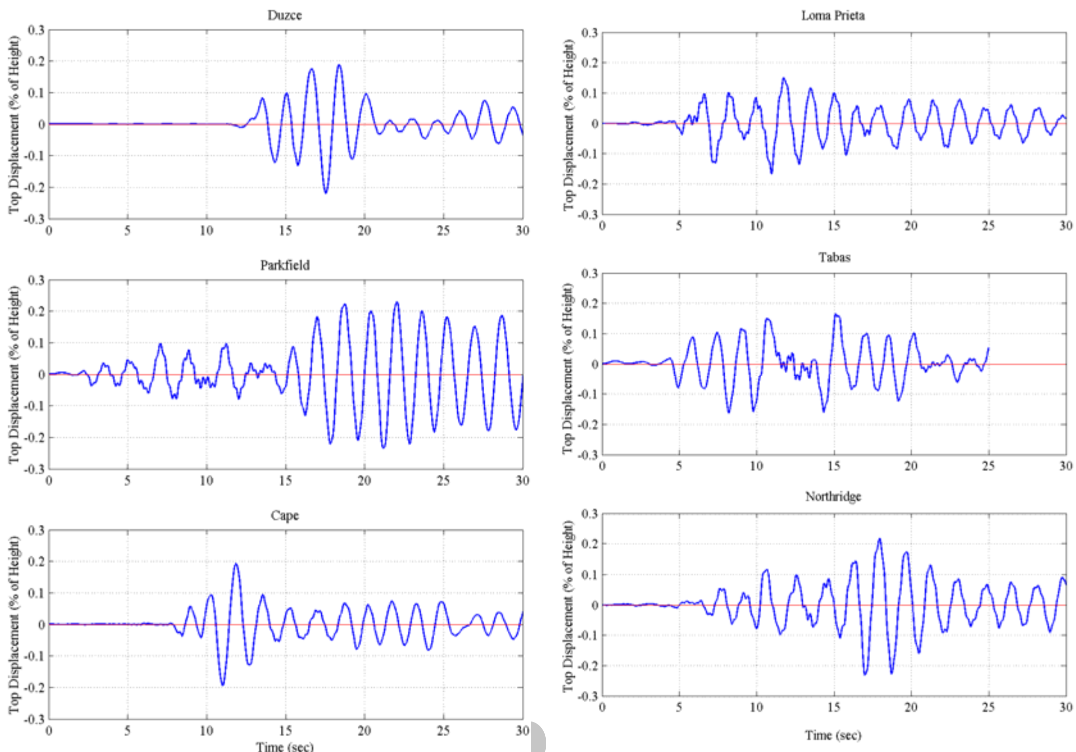

Figure 15. Time history responses at top of the structure for input motion scaled to PGA level of 75 years return period

The effect of input motion is observed clearly. Overall drift of the structure for PGA values of 0.1g-0.6g has been shown in Figure 16. It is seen that for PGAs less than 0.3g, there is no considerable difference among the responses of various input motions, but for PGAs greater than 0.3g, the input motion characteristic has a strong influence on the response of the structure.

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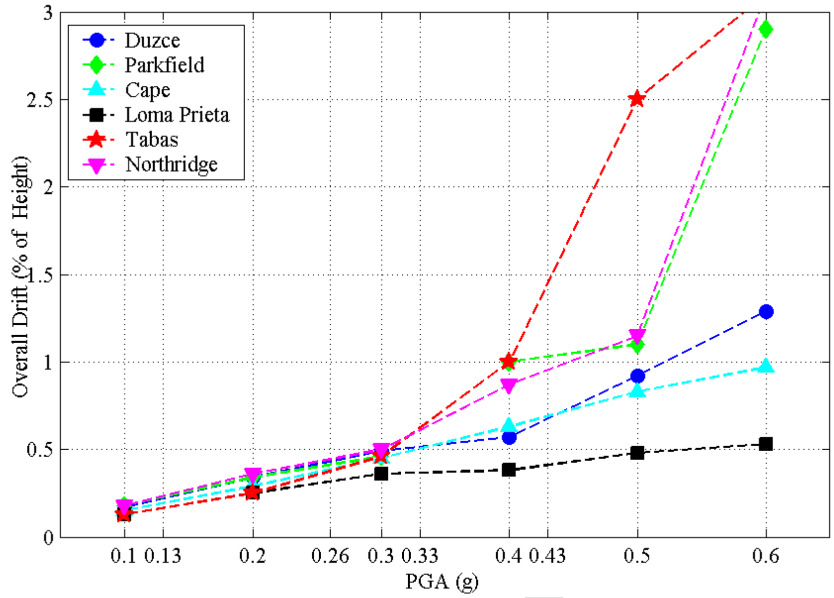

Figure 16. Relation between PGA and top displacement ratio

Global damage index of the structure for PGA values of 0.1-0.6g has been shown in Figure 17. It is observed that for PGAs greater than 0.2g, the input motion has an obvious influence on the damage of the structure. Also for PGAs greater than 0.6g structural collapse may occur under some records consisting of low frequency content.

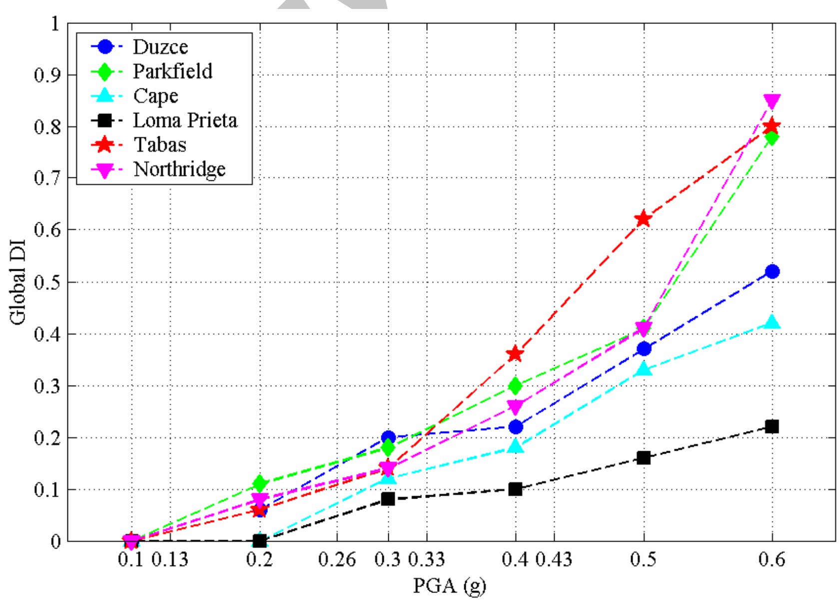

Figure 17. Relation between PGA and global damage index

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8. CONCLUSIONS

The nonlinear dynamic analysis is essentially needed for seismic assessment in evaluation of actual performance of complicated structures during earthquakes. Then the damage indices of structure had to be calculated, using appropriate damage models, since these indices would be numerical representation of damage states of the structures. However, numerical modeling of such mega-structures with numerous elements would not be allowed to include all details of mechanical characteristics of consisting materials, particularly in large deformations. Therefore, a simplified model corresponding to dynamic characteristics of whole structure was developed to investigate seismic performance and failure modes of these essential structures subjected to strong ground motions. The simplified model provided sufficient accuracy based on a nonlinear discrete model. To verify the proposed modeling procedure, two case studies were investigated numerically. Acceleration time histories scaled to different hazard levels were used as input excitations. Among the results, distribution of damage indexes could verify the most probable mode of failure under the severe excitation.

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