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# *SEISMIC RELIABILITY OF A PETROCHEMICAL STEEL STACK USING INCREMENTAL DYNAMIC ANALYSIS*

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## *ABSTRACT*

Incremental Dynamic Analysis (IDA) is a method that offers seismic demand and capacity prediction capability by using a series of nonlinear dynamic analyses under suitably scaled ground motion records. To apply the method, one needs to choose a suitable ground motion Intensity Measure (IM) and a representative Damage Measure (DM). In addition, proper interpolation and summarization techniques for multiple records need to be employed, providing the means for estimating the probability distribution of demand given intensity.
Limit-states, such as dynamic global system instability, can be naturally defined in the context of IDA, thus allowing annual rates exceedance to be calculated.

A performance-based design method enables designers to evaluate a graduate suit of performance levels for a structure in a given hazard level environment. One component of this framework is a probabilistic seismic demand model. A probabilistic seismic demand model relates ground motion Intensity Measures to structural Demand Measures. It is formulated by statistically analyzing the results of a suit of non-linear time-history analyses of typical structures under expected earthquake in the urban region.
To illustrate all the above concepts, the procedure has been applied to study the seismic reliability of a stack that has 76.2-meter height

## *1. INTRODUCTION*

In the evolving field of performance-base earthquake engineering, designers and owners are motivated to engineer structures to fulfill predetermined performance levels or objectives.
Previously, performance base design frameworks have addressed only the probabilistic evaluation of seismic hazards (FEMA-273 (FEMA 1996) and Vision 2000 (SEAOC 1995)). The resulting graduated arrays of performance levels are based on deterministic estimates of structural performance. The recent SAC Steel Project (FEMA 2000) provided a probabilistic

extension to the performance side, enabling simultaneous consideration of uncertainties in both demand and capacity.

The procedure described in this paper for evaluating the displacement demand hazard and the annual probability of reaching a collapse limit state for a model structure, has been developed as part of the SAC steel project (Cornell, 1997 and Wen, 1997). The procedure is presented generally and then demonstrated for a typical steel stack.

## 2. BACKGROUND

The final objective of the methodology devised by Cornell is the estimation of the annual probability of exceedance of a given level of inelastic response in a specific MDOF structure. The structure is located at a specific site with an associated potential seismic hazard. The response of interest can be any structural response parameter, representing either local or global structural demands (In this study displacement of three different location of structure). The procedure couples conventional Probabilistic Seismic Hazard Analysis (PSHA) with nonlinear dynamic structural analyses in order to determine the annual probability of exceedance of a given response parameter. This procedure is referred to as the Probabilistic Seismic Demand Analysis (PSDA).

### 2.1 Probabilistic Seismic Hazard Analysis (PSHA)

In view of the rareness of seismic events and the variability of their characteristics, it has become common to describe the seismic threat in probabilistic terms. The earliest work in this area has defined this threat in terms of some measure of the ground motion intensity, such as peak ground acceleration (PGA). More recent work has commonly sought to characterize the ground motion by its response spectral acceleration Sa, the peak acceleration the earthquake will induce in a 1DOF system with a specified period T and damping ratio $\xi$. Commonly, T and $\xi$ are chosen to describe the properties of the first mode of the building of interest. In any case, the aim is to describe the probability that the ground motion characteristic of interest (PGA, $S_a$, spectral velocity $S_v$, or spectral displacement $S_d$) is exceeded over a reference period of interest. For example, the probability that Sa exceeds a critical value $\mathbf{s}_{cr}$ in an arbitrary earthquake is formally written as

$$P[Sa > scr\ ] = P[Sa > scr \mid M = m,\ R = r]\ fM\ ,R\ (m,r)dmdr \qquad (1)$$

in which $f_{M,R}$(m,r) is the joint probability density function that describes the magnitude M and site-to-source distance R of an arbitrary event. Equation (1) is commonly evaluated numerically, and has become known as "PSHA"" or Probabilistic Seismic Hazard Analysis (e.g., Cornell, 1968). Final results are generally rescaled by the rate $v$ of all events being modeled, leading to a mean hazard rate H(s) associated with those seismic events that produce $S_a$>$\mathbf{s}_{cr.}$ As a simple parametric representation, it has been suggested (Cornell 1996) that the net result is often well approximated by a relationship of the form:

$$H(S_a) = P[S_a \geq s_a] = k_o s_a^{-k} \qquad (2)$$

The coefficients $k_0$ and k thus serve to characterize the seismic threat at a given site of interest. The approximation in Equation (2) has the advantage of being linear in log-log space and has been shown to be satisfactory over a range of spectral accelerations (Shome 1999)

### 2.2 Probabilistic Seismic Demand Analysis (PSDA)

Unfortunately, Sa does not provide sufficient information to determine the precise response of actual buildings, which generally show both nonlinear and multi-degree-of-freedom (MDOF) behavior. Indeed, there is a growing trend toward "performance-based" seismic design, in which a range of increasingly rare hazards are specified, together with a correspondingly increasing

level of permissible "damage" (i.e., nonlinear behavior). This implies the need for explicit recognition, and statistical quantification, of the degree of nonlinear behavior.
Extension of conventional PSHA, to directly describe the seismic demands 'D' of a complex, nonlinear structure, has become known as probabilistic seismic demand analysis (PSDA) (Cornell 1996, Luco et. al. 1998). The power of this method hinges on the following observations. First, given knowledge of the ground motion's intensity, as measured by Sa at the building's first mode, the nonlinear behavior is often found to be not substantially influenced by additional ground motion parameters (e.g., M, R, duration). While somewhat counterintuitive, this result has been demonstrated by detailed comparisons (Shome 1999, Shome et. al. 1996), at least in the context of non-degrading systems. This result permits Sa to be used as a powerful, scalar descriptor used to summarize the ground motion threat. Secondly, in practice, considerable variability exists in Sa, which reflects the elastic demand (of a simplified, 1DOF building).
The median relationship between spectral acceleration and drift is established be performing nonlinear dynamic analyses of the model structure for numerous ground motions at different levels of intensity, as measured by spectral accelerations. The spectral acceleration at the fundamental period of the structure for each ground motion is simply obtained from its elastic response spectrum. The response of the model structure subjected to each earthquake record provides the corresponding drift. The functional relationship between median drift and spectral acceleration is taken to be:

$$\hat{D} = a(S_a)^b \qquad (3)$$

Where $\hat{D}$ is the median drift response and $S_a$ is the spectral acceleration. The exponent b is included to capture potential "softening" of the nonlinear relationship between spectral acceleration and median drift. Utilizing this relationship, an expression for the drift hazard rate, $H_D(d)$, can be expressed as (Cornell 1996, Barroso).

$$H_D(d) = P[D \geq d] = H(S_a^d)\exp\left[\frac{1}{2}\frac{k^2}{b^2}\beta^2_{D\backslash S_a}\right] \qquad (4)$$

Where:

$H_D(d)$ is the spectral acceleration hazard (or mean annual frequency of exceeding d).

$S_a^d$ is Spectral acceleration "corresponding to" the drift demand level d.

$\beta_{D\backslash S_a}$ is Dispersion measure (standard deviation of the natural logarithm of data) for drift.

k is Coefficients for linear regression of hazard H ($S_a$ ) on intensity Sa in the

b is Regression coefficients for linear regression of drift demand D on intensity Sa in the logarithmic space

Finally probability of collapse is produced by:

$$P_{PL} = P[C \leq d] = H(S_a^{\hat{C}})\exp\left[\frac{1}{2}\frac{k^2}{b^2}\left(\beta^2_{D\backslash S_a} + \beta^2_C\right)\right] \qquad (5)$$

Where:

PPL is the annual probability of the performance level not being met

$\beta_C$ is Dispersion measure for drift capacity C

$S_a^{\hat{C}}$ is Spectral acceleration "corresponding to" the median capacity.

For more details you can see reference No.5.

## 3. PROBLEM DEFINITIONS

#### 3.1 Class of Structure

Typical old refinery steel stacks are selected as the class of structures. A class is characterized by geometry, components, and methods of design. Ideally, each of these parameters can be investigated in a parameter sensitivity study.
The structure has 76.2 meter height, 150 ton weight, fundamental period equal to 1.8s, and viscose damping ($\xi$=0.05).
The structure is modeled as a lump mass structure with simple elastic-plastic behavior elements. For damage monitoring displacement of three location of structure were selected. Joint 12 represents location that has maximum difference in diameter, joint 21 represents location that has maximum difference in thickness, and joint 36 represents top of structure's displacement.

#### 3.2 Earthquakes

Two suites of ten time history were selected to represent ground motions in structure's site. Ten time history represents hard soil characters and another ten is for soft soil. Each earthquake is scaled to $S_a(T=1.8s, \xi=0.05)$ up to 1 g.
An important note regarding the earthquake sets is that they should be used only as a set, and not individually or as *small* sub-sets as representative of the probability levels specified. At any particular period the median spectral acceleration of the set may match the target value reasonably well; however, any individual record may have a value quiet different than the expected target spectral acceleration.

#### 3.3 Incremental Dynamic Analysis

Example IDA curves are shown in Fig. 1. Usually plotted in linear scale, each curve is for one ground motion as it is incremented. A single dynamic pushover analysis entails performing multiple nonlinear dynamic analyses for a model structure subjected to an earthquake record, which is incrementally scaled. The result is a dynamic pushover curve, which relates the scale factor for the earthquake record and the drift response of the model structure. From the dynamic pushover curve, the maximum drift angle limit corresponding to the transition point when the analytical response of the model structure becomes "unstable" (when the dynamic drift response increases drastically for a relatively small increase in ground motion intensity), or when the apparent "stiffness" (the slope of the dynamic pushover curve) decreases radically, may be used as a measure of the maximum drift angle capacity.
The capacity point may be difficult to identify. For example in this study there is no flat line in IDA curves (as usually seen in structures with higher periods). These kinds of structures are called happy structure.
For solving this problem the static overturning capacity of structure was assumed to be structural capacity (it occurs at Sa=0.5g).

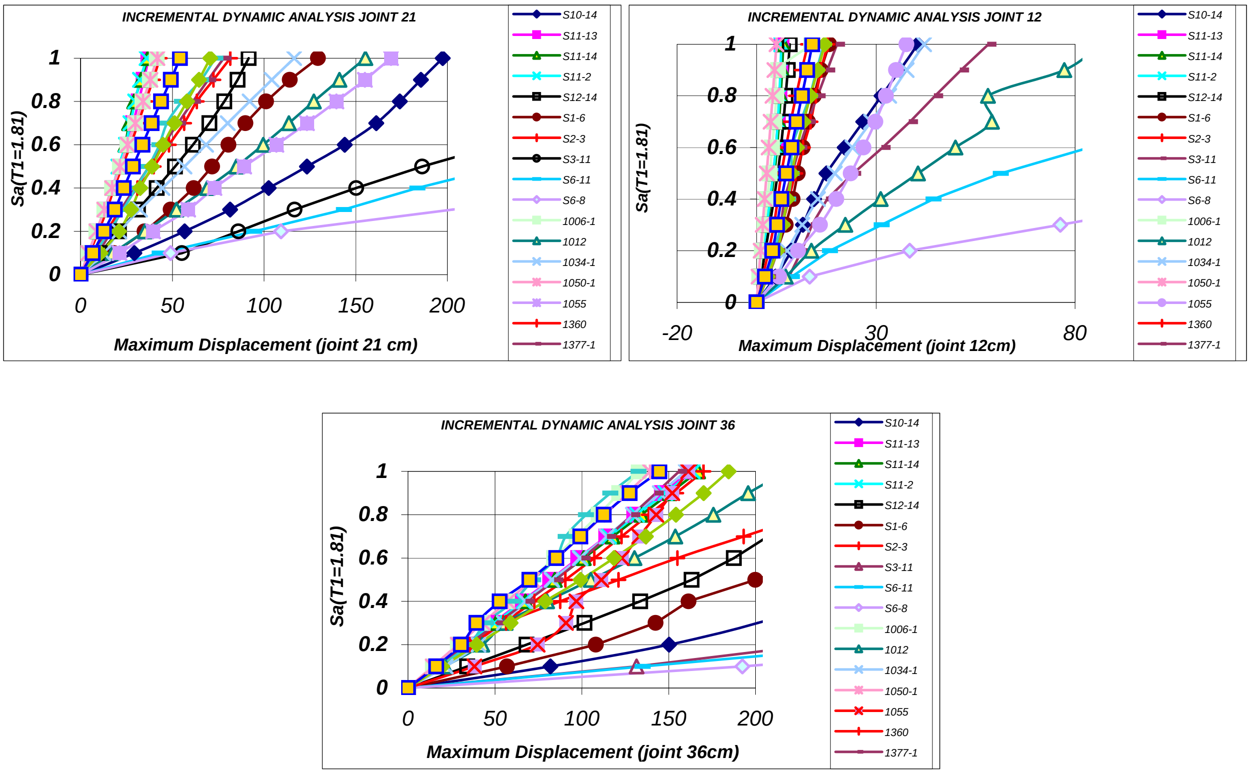


***Figure 1.*** Incremental Dynamic Analysis for three different location of structure (joint 12,21,36)

## *4. CONFIDENCE LEVELS*

Results of confidence levels are shown in figures 2-4. In three different situations the capacity of structure is assumed to be 0.1g, 0.5g, 1.0g and confidence levels that are results of joints 12, 21, 36 are plotted. Left figure shows confidence levels for soft soil type and right figure is for hard one.

According to the figures for soft soil, confidence levels of three joints are approximately the same but for hard soil they are showing some differences.

By increasing capacity from 0.1g to 1.0g the upper joint of structure is resulting better confidence level. However it is simple to select a damage point for a complex structure, it is important to know that different selections may cause different results.

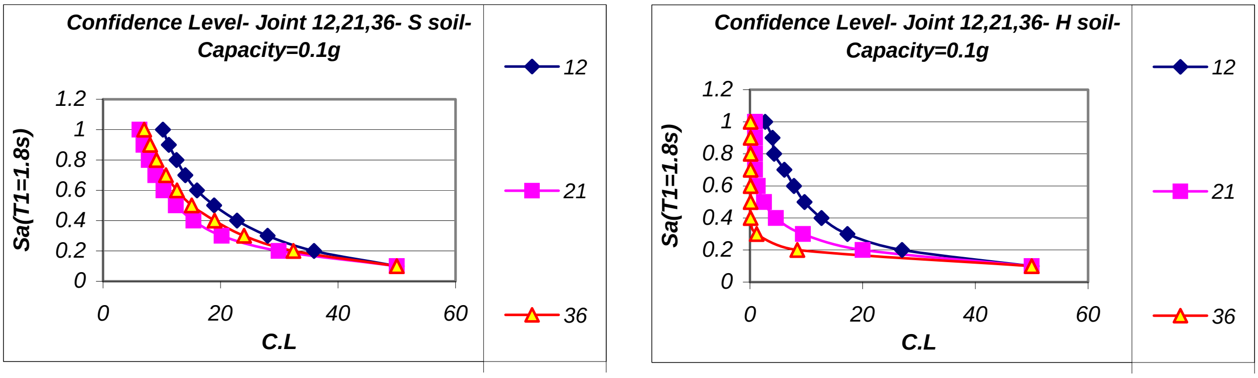


***Figure 2.*** Results of confidence level by assuming capacity=0.1g, and comparison between different joints and soils

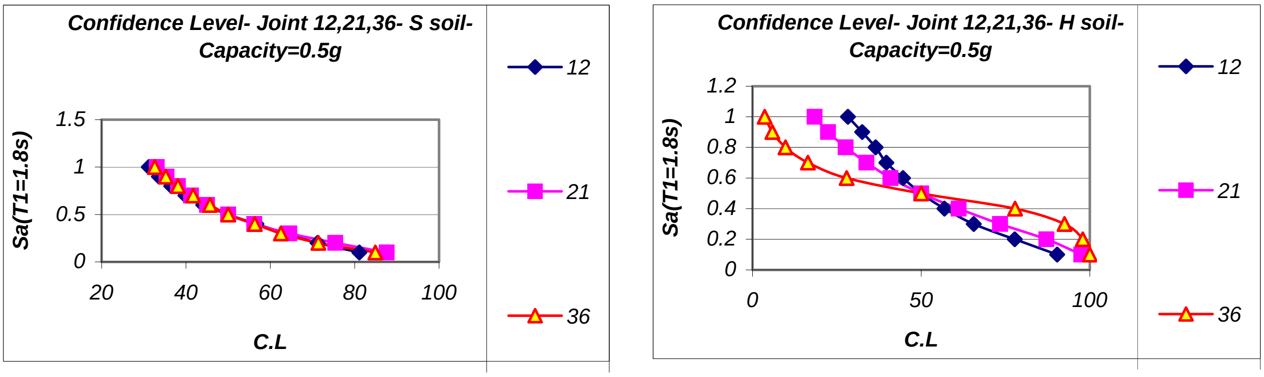


***Figure 3.*** Results of confidence level by assuming capacity=0.5g, and comparison between different joints and soils

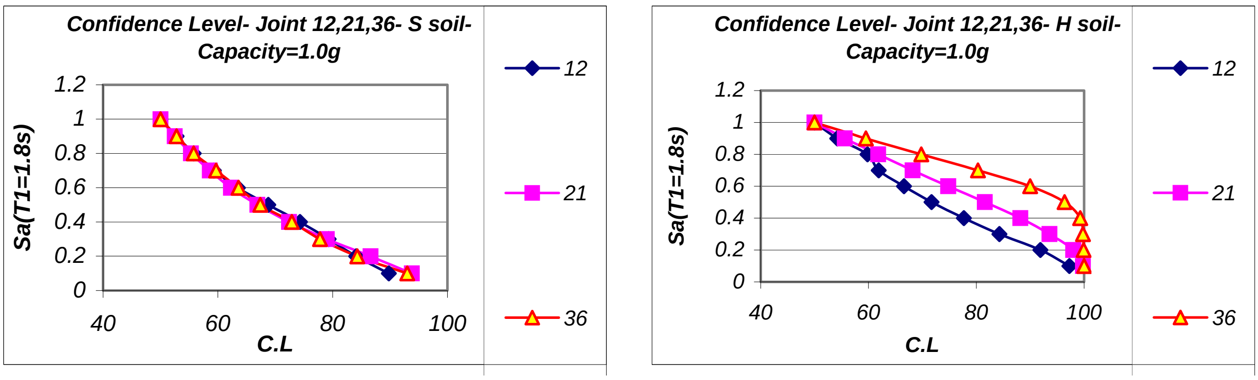


***Figure 4.*** Results of confidence level by assuming capacity=1g, and comparison between different joints and soils

## 5. CONCLUSION

The IDA study is now a multi-purpose and widely applicable method and its objectives, only some, include:

1. Thorough understanding of the range of response or ”demands” versus the range of potential levels of a ground motion record

2. Better understanding of the structural implications of rarer / more severe ground motion levels

3. Given a multi-record IDA study, how stable (or variable) all these items are from one ground motion record to another.

***Table 1.*** *Results of confidence levels vs. performance objective.*

| | Joint 12 | | Joint 21 | | Joint 36 | |
|---|---|---|---|---|---|---|
| | Soft Soil | Hard Soil | Soft Soil | Hard Soil | Soft Soil | Hard Soil |
| Probability of Exceedance 2% in 50 years | 42 | 64 | 43 | 37 | 43 | 22 |
| Probability of Exceedance 10% in 50 years | 60 | 81 | 60 | 67 | 59 | 85 |
| Probability of Exceedance 50% in 50 years | 76 | 94 | 81 | 92 | 78 | 99 |

Results of confidence levels for different joints and soils are shown in table 1. Performance objectives are assumed to be three probability of exceedance.
Two criteria for adequacy of structure was taken as:

1. Confidence level must be greater than 50%
2. Probability of exceedance must be greater than probability of failure. For example for probability of exceedance 2% in 50 years earthquakes, probability of failure must be less than 0.0004.

According to the table 1 the structure is safe for probability of exceedance 50% in 50 years earthquakes in any conditions (for different soils and joints) and for other objectives safety criteria is not satisfied.

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