Response Reduction Factor of Building and Soil-Structure Interaction Effects

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Abstract: This study reviewed the recent developments in finding the response reduction factor for RC framed building and the influence of soil-structure interaction (SSI) effects in the various responses of the building. For Response Reduction Factor, the nonlinear analysis was done in order to capture all the hysteretic Energy beyond the elastic limit. Various approaches to pushover analysis and time history analysis have been mentioned in this review paper.

***Keywords* – Response Reduction Factor, Soil-Structure Interaction, Pushover Analysis, Time History Analysis.**

1. INTRODUCTION

Response Reduction Factor (R) is necessary to be found for existing structures or the structures that are going to be built on. In the case of new structures, R helps to find out how much amount of lateral force is to be resisted by the elastic state. Generally, 1/R part of the lateral force is resisted by the structural members till the elastic state. If all the parts of the lateral forces were to be resisted by the structural components of the complex irregular structures, high rise structures, or the structures that are built considering the SSI effect, then we could have used only linear elastic analysis. But it would result in very uneconomical sections. So, some parts of the lateral forces can be let for displacements due to the ductility of the ductile members due to the dissipation of hysteretic Energy. So, we can get economical sections and also the capacity based design of the structures if we can obtain the actual value of 'R.'

We can find different values of R in several building codes. But, those values of R were taken in codes due to observation of the performance of different structural systems in past strong earthquakes and the detailing process. But, the actual way of finding out R is not mentioned in Indian building codes. Only the serveacibility criteria for designing different structural members in different codes are given, including Indian code IS 456:2000 for flexural beams [29]. But, from paper [4], it has been found that R depends on several factors like Ductility Reduction Factor, Over-strength Factor (Reserve Strength of the structure), Material Over-strength Factor & Redundancy Factor. Also, if we use dampeners in our building structure, then R also depends on the damping factor [21]. So, the overall value of R will be the multiplication of all these factors.

1.1 *Effect of Soil Structure Interaction in finding R*

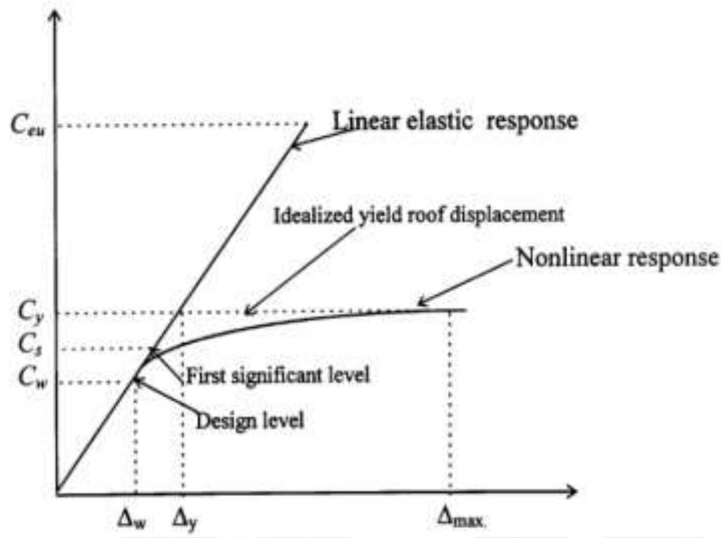
When the response of the soil influences the motion of the structure and the motion of the structure influences the response of the soil, then this condition is called Soil-Structure Interaction (SSI). There can be different types of soil. For a hard type of soil, due to SSI, there will be little increase in time period 'T' of the building. But for soft and clayey type soil, base stiffness is decreased, and 'T' increment can be very large, which can create a negative impact on the overall structure [1, 7]. So, in order to design a building for such a large 'T,' it is very important to find out 'R' because for such building structures having high 'T,' we also consider the ductility of the members.

There are mainly two kinds of SSI: Kinematic Interaction & Inertial Interaction. In Kinematic Interaction, due to EQ, there will be a free-field motion of the soil by which there will be Displacement of soil in a vertical and horizontal direction, but still, that free field motion of soil cannot displace the stiff foundation element. In Inertial Interaction, the weight of the superstructure transfers inertial force to the soil, causing deformation in soil. So, these effects have to be considered while finding out the value of the Response reduction factor (R).

1.2 *Pushover Analysis for Response Reduction Factor (R)*

If we have to consider the nonlinearity of the materials, we have to perform Non- linear analysis of the building. Pushover analysis is one type of nonlinear static analysis process. There are two ways of doing a pushover analysis. They are either force-controlled or deformation controlled. In the force-controlled method, structural members are pushed to a certain defined force level. Force-controlled pushover analysis is, however, suitable for non- ductile components whose capacities are governed by strength. In the Displacement controlled method, structural members are pushed to a specified displacement. The horizontal lateral force that is obtained due to the distribution of base shear is used generally to match the required Displacement. Displacement controlled pushover analysis is generally done for ductile components that can tolerate inelastic deformations. Generally, the strength of the building is governed by gravity loading. Therefore, for the lateral forces developed in the building due to the distribution of base shear along with the height of the building due to the horizontal earthquakes, it has been found that Displacement Controlled Pushover Analysis is much more effective to use than force-controlled pushover analysis.

The general idea of getting a response reduction factor from any type of pushover analysis is that after running the analysis, we get the pushover curve, which is the plot of the base shear vs. roof displacement. And, by definition, the response reduction factor is a factor by which the actual base shear force should be reduced to obtain the lateral design force during an earthquake. That means we can get all the parameters required for finding the response reduction factor from the pushover curve.



<Fig: Pushover Curve> [4]

1.3 ***Time History Analysis for Response Reduction Factor(R)***

Time History Analysis is a dynamic method of doing structural analysis where the real-time earthquake ground motion data like frequency content of the earthquake, amplitude content such as peak ground acceleration (PGA), peak velocity, peak displacement, sustained maximum acceleration and velocity, effective design acceleration as well as motion duration data are being used to generate demand parameters required. With the help of these demand parameters, we can predict the accuracy of R obtained from Pushover Analysis. Also, the capacity curve can be obtained from incremental dynamic time history analysis for the exact solution, and hence, R can be found out from the capacity curve as in Pushover analysis [18].

2. LITERATURE REVIEW

2.1 ***Soil Structure Interaction***

In addition to the two components of SSI - kinematic and inertial - originally coined by Whitman, Roesset also discussed direct and substructure approaches to perform SSI analyses [24]. In the direct approach of soil modeling, the entire soil, foundation & superstructure were modeled and analyzed in a single step. Mercado et al. (2019) evaluated SSI effects in tall buildings using direct approaches & described the importance of including SSI in numerical simulations as opposed to using conventional fixed-base building models. It was observed that SSI effects are properly analyzed by an indirect approach [22]. Though it could give the most accurate result for soil parameters, it was very difficult to model soil with interaction with foundation & superstructure in a single step. The main difficulties lay in finding the stiffness of the soil. Later on, the Winkler model was developed in 1867 A.D for modeling of soil behavior. He gave the relation between bearing pressure of soil and deflection of soil with the help of stiffness of soil. Stiffness of soil was represented by springs and was called as modulus of subgrade reaction of springs (ks). Winkler's idealization represented soil medium as a system of identical but mutually independent, closely spaced, discrete, linearly elastic springs. After comparing the behavior of the theoretical model & actual foundation, it was seen that the Winkler model suffered from a complete lack of continuity in supporting medium soil [3]. Later on, for Non-linear characteristics of soil deformation, different models were approached by different investigators like Filanenko Borodich Model, Hetenyi's model, Pasternak model & Kerr Model [3]. Asrat Worku (2007) also derived spring formulas for short and long-term static soil deformation [6]. There have been numerous parametric studies to date to establish the significance of considering SSI in structural design. Most of them suggest geometry of superstructure, foundation characteristics, soil modulus, and shear wave velocity profile in stratified deposits be the ones that affect their seismic response [24].

The behavior of soil is predicted based on the engineering properties of soil, vertical soil profile, and the alignment of the ground surface. The soil properties such as unit weight (γ), shear modulus (Es), Poisson's ratio (v), effective cohesion (c), friction angle (y) play a very important role in representing soil behavior. It was observed in certain studies that reduction in soil moisture content causes an increase in the lateral soil resistance [23]. The empirical relations for γ, c & y are given by Bowels based on SPT penetration N values and type of soils. Also, from the theory of elasticity, we can get the other parameters of soil like Bulk modulus (k), shear modulus (G), and stiffness modulus (M). [35]

$$K = E/\,(3(1-2v))$$
$$G = E/\,(2(1+v))$$
$$M = E\,(1-v)/\,((1+v)\,(1-2v))$$

In a homogeneous isotropic half-space of soil with the horizontal ground surface, we can estimate the Poisson's ratio from

$$v = (1-\sin y)/\,(2-\sin y)$$

where y is the angle of internal friction, and $-1 < v < 0.5$ has to be satisfied. This equation uses elasticity theory and Jaky's formula for the stress ratio, i.e.

Sx / Sz = 1-siny

where Sx is horizontal stress, and Sz is vertical stress. [35]

The USA governmental agency NEHRP has given the average shear wave velocity for different types of soil.

[Table 1: NEHRP Soil Types Based on Shear Wave Velocity of upper 30m] [10]

Soil Types	Rock/ Soil Description	Average shear wave velocity (V_{s30}) m/s
A	Hard rock	> 1500
B	Rock	760-1500
C	Dense soil/soft rock	360-760
D	Stiff soil	180-360
E	Soft soil	< 180
F	Special soils requiring special evaluation	

Many investigators also gave different empirical formulas for finding the modulus of subgrade reaction.

No.	Investigator	year	Suggested formula
1	Winkler	(1867)	$k_s = \frac{q}{\delta}$
2	Biot	(1937)	$k_s = \frac{0.95E_s}{B(1-\nu_s^2)}\left[\frac{B^4E_s}{(1-\nu_s^2)EI}\right]$
3	Terzaghi	(1955)	$k_s = k_{s1}\left(\frac{B+B_1}{2B}\right)$
4	Vesic	(1961)	$k_s = \frac{0.65E_s}{B(1-\nu_s^2)}\sqrt[12]{\frac{E_sB^4}{EI}}$
5	Meyerhof and Baike	(1965)	$k_s = \frac{E_s}{B(1-\nu_s^2)}$
6	Selvadurai	(1984)	$k_s = \frac{0.65}{B}\cdot\frac{E_s}{(1-\nu_s^2)}$
7	Bowles	(1998)	$k_s = \frac{E_s}{B_s(1-\nu_s^2)mI_sI_f}$

Fig. 1 - (a) Different Empirical Formulas for Modulus of subgrade reaction of soil [15]

For soil bearing pressure and soil settlement, a plate load test was used to be done and is in practice to date. But, it was realized that finding the soil parameters by the plate load test required a lot of time and money. However, Bowles developed another empirical relation for finding the modulus of subgrade reaction, which was theoretical based empirical relation which just depended on footing dimensions and SPT 'N' values of soil [17], i.e.

Modulus of subgrade reaction, $k = 40 * S.F. * Qa$

- For Footing width 4 ft or less,
  Qa = (N/2.5)/K
- For Footing width >4ft,
  Qa = (N/4)[(B+1)/B²)/K

Where,
N is SPT value
K= 1+0.33(D/B)<=1.33
D= depth from ground to footing bottom in feet.

Fig.1-(b) Allowable bearing capacity empirical formula given by Bowles [17]

However, the aforementioned formula is given by Bowles still limited for 25mm soil settlement only and mainly used for a linear type of soil. For nonlinear soil, finding the stiffness of the soil is not a straightforward process because it depends on the width and depth of the loaded area. The more the width of the footing, the more the soil has to be mobilized by that width of the footing. Also, there will be a change in stress in soil due to variation in depth of the loaded area [3]. Lab tests for finding the behavior of non- linear soil didn't consider the propagation of waves, which could also affect the behavior of soil. So, some researchers used a series of response spectra of ground motions, PGV and PGV, for actual nonlinear soil modeling in a particular place.

While talking about vertical soil profile, which is also known as soil layers, it is always layered in the actual ground surface, and soil properties vary from layer to layer, and it is considered to remain constant for a particular layer for the purpose of analysis. [23]. For example, in the Winkler approach, which considers the soil as a layered system to study the lateral resistance of the structure, the soil pressure, also called as soil bearing capacity 'q', is related to the soil settlement 'y' through the modulus of subgrade reaction 'ks' of the particular layer [3].

q = ks * y.

While talking about the alignment of the ground surface, generally, a level ground surface is assumed in SSI. But when the ground surface is sloping, it increases the bending moment. Hence, the sloping ground surface should be taken into consideration while analyzing the soil-structure system [23].

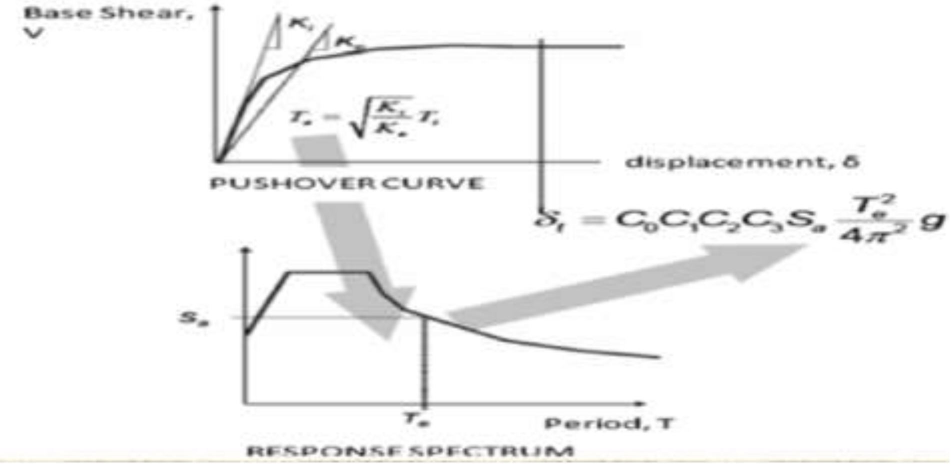
2.2 ***Pushover Analysis (PA)***

Starting from the seventies, pushover analysis has been performed by several methods. Standard guidelines were developed in America for pushover analysis ATC-40 in 1996 and FEMA-273 in 1997 [5]. The first known method was the Capacity Spectrum Method (CSM) started by Freeman and co-workers. Here, real structures were represented as an equivalent single degree of freedom (SDOF) model. Target displacement was found in CSM by intersecting the capacity curve with the demand curve [27]. For the improvement of nonlinear static seismic analysis, FEMA 440 was developed in 2005. Later on, the Saudi Building Code (SBC 301) also satisfied acceptance criteria for the aforementioned methods by FEMA 273, ATC 40, and FEMA 440 [8]. However, no guidelines are available for pushover analysis by Indian Standard Codes till yet. By CSI back in 2014, Dr. Graham Powell mentioned that the CSM used by ATC 40, established in 1996, was found to be very inaccurate [13]. Several Displacement coefficient methods were also developed, such as FEMA 356 displacement coefficient method and FEMA 440 displacement modification method, where target displacement or displacement demand was calculated by applying its formula [26, 28]. However, modified CSM was also developed, which could be comparable with the accuracy of DCM [13]. All these methods used for PA used the equivalent SDOF model [16]. However, there were some limitations of these all methods, like there were not the inclusion of higher modes in PA. Not only that but, also standard guidelines for the inclusion of building torsion were not available [16]. This led to the development of Multimode PA. But, still, multimodal pushover analysis could not capture the entire seismic response because it was found that distortion of the capacity curve takes place by taking roof displacement as a reference point. The author S. Soleimani et al. (2017) has mentioned that the authors Antonio & Pinho had concluded adaptive force-based PA has a relatively minor advantage over Modal Pushover Analysis (MPA), which is nonadaptive procedures. In order to address the distortion of the capacity curve, an Energy-based pushover analysis was developed. The Energy based pushover analysis (EMPA) is similar to MPA. In contrast to MPA, the work done by the lateral loads & torques through EMPA was considered as an index to compute the Displacement of corresponding equivalent single degree of freedom (EDOF) system. But, the application of EMPA was limited to 2D structures only [19]. The development of multi-mode adaptive displacement-based pushover analysis using SQRSS or CQC combination rules depending upon the closeness of the modal responses also helped well to consider even small changes in the seismic load distribution along with the height of the structure at each step of PA [9]. However, the drawback of adaptive pushover analysis is that the load pattern has to be updated in each step of analysis in accordance with the changes in characteristics of the structure due to nonlinearity. The latest method of 2020, known as E-DVA method, tried to include both the torsion effect and multi-mode effect by using alpha factors while doing a linear combination of modes [14]. However, the implementation of this method has not yet been found in any commercial software.

. Before running any type of PA, we need to define vertical loads and lateral load pattern. For defining vertical loads, the previous IS 1893:2002 code used summation of dead load of the structure plus 25% of total imposed loads. But newer Indian Standard code IS 1893:2016 has used summation of dead load plus 25% of live load up to 3KN/m2 plus 50% of the live load exceeding 3KN/m2 [30]. Also, for considering nonlinear geometric structures, the P-delta effect must be included while defining vertical loads.

The conventional way of selecting the lateral load pattern is based on fundamental or first mode shape. But it is a well-known fact that Lateral load pattern varies at every instant of time during an earthquake. Therefore, it is very difficult to capture the entire seismic response by defining a single-mode lateral load pattern. So recently, PA is being performed using different types of lateral load patterns by different authors. Most of the lateral load patterns were determined from the conventional modal response spectrum analysis, which is applicable only up to the elastic range. So, based on the type of structures, a new lateral load pattern can be proposed, but it has to be verified against the Non -linear Time History Analysis (NTHA) results [18]. Example: The authors M. Bhandari et al. (2018) had proposed a new lateral load pattern (LLP) suitable for base-isolated frames. They had used four LLP. They used LLP1, which was proportional to shape of the 1st mode. In order to capture the participation of higher modes, they had used LLP2, which was an SRSS combination of 1st three mode shapes. Similarly, LLP-3 and LLP-4 were derived by modifying the uniform lateral load pattern [18]. FEMA has provided uniform and triangular lateral load patterns. According to the author Sadegh Etedali et al. (2015), in a low period concrete special moment-resisting frame (SMRF), the uniform load pattern proposed by FEMA was found to be a suitable load pattern, but with increasing time period, the load patterns recommended by FEMA could not provide accurate results [25].

After displaying the pushover curve, recently, most of the Pushover methods use the bilinear approximation of the actual pushover curve. FEMA440, ATC-40, and EC-8 have changed the pushover curve into a capacity curve for representing bilinear approximation. However, the P-695 document used a pushover curve directly to represent a bilinear approximation. So, the P-695 way of representing the bilinear curve is the easy way. For representing Energy-based pushover analysis, FEMA 440 has also given provision for the Tri-linear approximation curve [19].

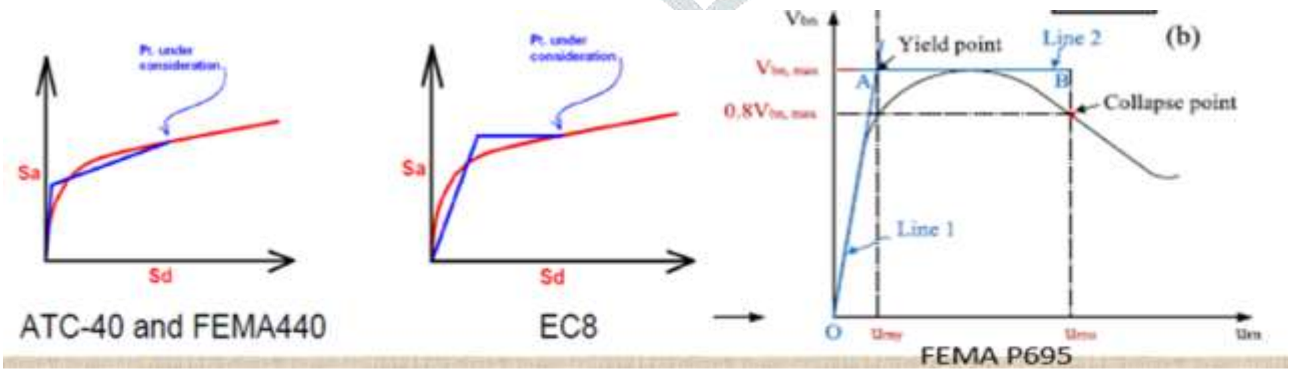


<Fig: Target Displacement formula as per FEMA 356 and FEMA 440 displacement modification method> [8]

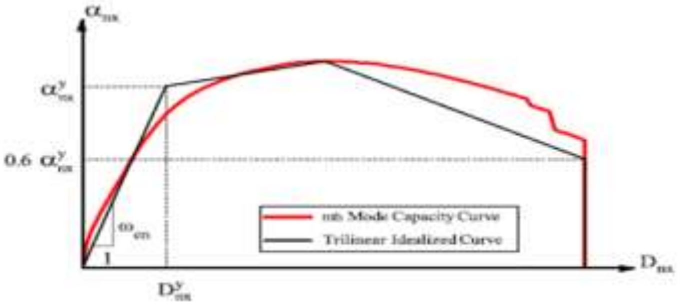
Table 10-1 Coefficients for Use in Equations for Effective Damping

Coefficient	Current Specification	Modification	Purpose of Coefficient
C_1	1.0 for $T_e \ge T_s$ $[1.0 + (R-1)T_s/T_e]/R$ for $T_e < T_s$	$C_1 = 1 + \frac{R-1}{aT^2}$ where a = 130 for site class B 90 for site class C 60 for site class D	Convert max. elastic displacement to estimate for inelastic system
C_1 (with short T "cap")	1.5 for $T_e < 0.1$s interpolating to 1.0 for $T_e \ge T_s$	Not recommended	
C_2 (degrading systems)	Immediate Occupancy 1.0 Life Safety 1.3 for $T \le 0.1$ interpolating to 1.1 for $T \ge T_s$ Collapse Prevention 1.5 for $T \le 0.1$ interpolating to 1.2 for $T \ge T_s$	$C_2 = 1 + \frac{1}{800}\left[\frac{R-1}{T}\right]^2$ recommended only for structures with significant stiffness and/or strength degradation	Hysteretic pinching Cyclic degradation
C_2 (non-degrading systems)	1.0	1.0	
C_3	$1.0 + \frac{\lvert\alpha\rvert(R-1)^{3/2}}{T_e}$	Eliminate in favor of strength limit	P-Δ In-cycle degradation

<Fig: Coefficients used in current specification (FEMA 356) & modified specification (FEMA 440) for target displacement formula> [28]



<Fig: Bi-Linear Approximation given by different codes>



<Fig: Tri-Linear Approximation given by FEMA 440 for energy based Pushover Analysis> [19]

Pushover analysis is also being used in performance-based design to meet the certain performance objectives. A hinge deformation pattern can be observed. The study on the Sequence of plastic hinge formation is less investigated to date. Supriya R. Kulkarni et al. (2018) mentioned that the Sequence of plastic hinge formation has a great influence on PA results and also concluded that displacement characteristics of the structure change, but the collapse load doesn't change due to variation in the Sequence of plastic hinge formation. They also concluded that variation in hinge formation sequence leads to more uncertainties than by the variation in strength and stiffness parameters [20].

2.3 ***Time History Analysis***

Ground motion parameters are the main things that is included in the Time History Analysis. Theory of Dynamics had already given the equilibrium equations of motion:

$$m\,\frac{\partial^2 u}{\partial t^2} + c\,\frac{\partial u}{\partial t} + k\,u + m\,\ddot{u}_g = 0$$

< Fig: Equilibrium equation of ground motions>

The above differential equations of ground motion needed to be solved. The solution of the above equation was obtained by standard procedures. Central Difference Method, Houbolt Integration Method (developed by Houboult in 1950), Wilson Ɵ method (developed by Wilson et al. in 1973), Newmark Method generally for linear time history analysis, and later on, the Direct Integration Method for non- linear time history analysis was developed [12]. The direct Integration method was found to be the most effective method, among other methods. In the Direct Integration method, the equilibrium equations of motion are fully integrated as a structure is subjected to dynamic loading. Integration is performed at every time step of the input record regardless of the output increment. [31]. Recently, ATC 58 guidelines for non- linear dynamic analysis were developed in 2009 for seismic performance assessment of new and existing buildings, including fragility models [5]. Also, new guidelines ATC 72-1 and PEER 2010 was developed in 2010 for time history analysis for tall buildings [5]. The important input ground motion records or parameters that are required to be found during THA are Amplitude components, Frequency components, and Duration components.

The amplitude Components include Peak ground acceleration (PGA) for high frequency (which is further classified into peak ground horizontal acceleration (PGH) and peak ground vertical acceleration (PGV)), Peak ground velocity (PGV) for intermediate frequency, Peak Displacement (PD) for low frequency, Sustained maximum acceleration and velocity and Effective Design Acceleration.

The frequency components include Fourier Spectra & Power Spectra, which corresponds to the frequency content of GM itself, and Response Spectra corresponds to the influence of Ground Motion on structures with different first Eigen periods. Motion Duration represents the time required for release of accumulated strain energy along fault, thus increases with increase in magnitude of the earthquake. Relative Duration & Bracketed duration are the two types of motion duration.

These seismic input ground motion records is represented in terms of properly defined time series (example: Accelerograms) which needs to be consistent with seismic hazard at the site. In many building codes, this idea is associated with the concept of “Spectrum Compatibility” [32]. Seismic hazard at site is generally represented in probabilistic terms. Three types of accelerograms are developed till date. The first one is the natural set of records which are selected from the strong ground motion databases. Nowadays, real or natural earthquake databases can be found over the web, which allow to interactively search events and retrieve waveforms in digital forms with prescribed characteristics. The second type of accelerogram is Synthetic Accelerograms which are generated through complex mathematical models of the seismic source and wave propagation phenomena. Also, empirical ground motion prediction equations were developed by using regression analysis of databases of observed strong ground motion which accounts for possible nonlinear behavior of near surface soil [11]. The third type of accelerogram is artificial accelerogram which are generally used to fulfill the gap of recorded accelerograms. For the generation of artificial accelerograms, target spectrum and envelope type were defined and calculation were based on algorithm but before target spectrum and envelope type, synthetic accelerogram which is simulated by the user was defined which was compatible with Target spectrum [33]. Finally, after defining target spectrum and envelope type, Fourier transformation was applied to change from time domain to the frequency domain and correction to the accelerogram was carried out simultaneously. After the correction, again the accelerogram was returned to the time domain by applying inverse Fourier transformation & convergence were checked and decision was made whether further correction is required or not [33].

However, the main drawback of direct integration method is that it takes a lot of computational time. So, for practical purpose new method known as Fast Non Linear Analysis (FNA) Method was developed where implementation of Ritz vector instead of Eigen vector makes the less computational time by appropriately trimming beginning and end of acceleration record and down-sampling the remaining parameters while conserving significant frequency characteristics of the original record including its S-phase [31,34]. The parameter to identify leading and trailing segments of the signal to be trimmed is the maximum roof displacement of an equivalent SDOF system. This parameter is selected over other parameter such as Areas Intensity because it represents the characteristics of both the ground motion and structural response [34]. CSI commericial Software Company also recommends to do time history analysis by FNA method over direct integral method [31]. Also, different speedup algorithm techniques have been developed recently to improve computational efficiency for time history analysis. Zheng He et al. (2017) proposed the new speedup algorithm for super tall buildings in nonlinear time history analysis by utilizing optimizing and parallel computing techniques which depends on computer hardware condition by speeding Jacobian factorization with the help of INC, PSTP and PF algorithms [2]. The effects in performance of the proposed speedup algorithms needs a further investigation [2].

3. CONCLUSION

This review paper included the way for finding the Response reduction factor (R) of the building by several methods. Since, Non Linear Time History Analysis is time consuming and results of time history analysis are highly sensitive to methods of selecting and scaling ground motion records, Pushover analysis is widely being used for finding R with the help of plot of base shear vs roof displacement. Several approaches for Pushover analysis and their suitability were mentioned in this review paper. The Sequence of plastic hinge formation is the area which is less investigated till date.

This review paper also talked about SSI effects on the response of the overall building structure. Several empirical formulas for identifying soil parameters were included. It was found that the time period of the building increases due to consideration of SSI. So, the future research topic can be made for finding response reduction factor of the building including soil structure interaction.

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