SIMULATION OF SEISMIC DAMAGE TO STEEL BUILDINGS

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ABSTRACT: This work addresses the field of collapse analysis of steel framed structures under severe loading conditions. The Improved Applied Element Method (IAEM) has been presented and employed in the development of novel numerical solutions for the analysis of failure and collapse of large-scale structures under different hazardous loads. A series of numerical examples, including both geometric and material nonlinearity, are used for validation of the improved method. The case study, presented in this paper, shows different collapse mechanisms of steel frame structures under severe ground motions.

Key Words: Numerical simulation, Improved Applied Element Method, AEM, Collapse, Damage

INTRODUCTION

The 1994 U.S Northridge earthquake caused serious damage to modern steel structures. The brittle fractures of beam-to-column connections for the moment-frame buildings were widely observed (Miller, 1998). The damaged buildings were of various heights ranging from one story to 26 stories. One year later, in the Kobe earthquake (1995), nearly one thousand steel buildings were damaged, as well as 90 buildings being collapsed, 333 buildings being severely damaged, and 300 being slightly damaged (Nakashima et al., 1998 and Holguin, 1998). According to the FEMA (2000) report, modern steel-frame buildings, specially constructed to sway rather than fracture during an earthquake, are more vulnerable to collapse than had ever been considered. A design flaw could cause these often massive skyscrapers to crack, tilt and even collapse during violent shaking. To reduce such damage, it is important to understand its main mechanism. However, it is very difficult or practically impossible to perform damage tests for total collapse process of real scale steel structures, especially high rise buildings. Therefore, studying those phenomena requires powerful numerical tools that can extend the analysis up to complete failure.

To obtain full knowledge of the behavior of steel structures under severe ground motions, the current research is aimed at establishing a comprehensive numerical technique to evaluate and characterize the earthquake response as well as the characteristics and failure mechanisms of large-scale structures. The emphasis is placed on the collapse mechanisms and the associated behavior of structures and their members under large cyclic loading. In previous decades, considerable research efforts dealing with collapse analysis have been developed such as Rigid Bodies Spring Model (RBSM) (Toi and Yoshida, 1991), Extended Distinct Element Method (EDEM) (Meguro et al., 1991), combined FEM/DEM (Munjiza et al., 1995), and Applied Element Method (AEM) (Meguro and Tagel-Din, 2001). Nevertheless, none of them have yet been used for collapse analysis of steel structures. In order to guarantee decent accuracy of the solution in the case of modeling of steel structure using AEM, a very large number of elements will be required to extend the computer power and time needed for numerical simulation. Therefore, this paper describes the methodology of Improved Applied Element Method (IAEM) (Elkholy and Meguro, 2003 and Elkholy and Meguro, 2004), an efficient and accurate method for analyzing the failure and collapse of large-scale structures

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under hazardous loads.

IMPROVED APPLIED ELEMENT METHOD (IAEM)

IAEM is a newly developed method for structural analysis of large scale structural. It can follow total behavior of structures up to complete failure stage with high accuracy in reasonable CPU. In IAEM, each structural member is divided into a proper number of rigid elements connected by pairs of normal and shear springs uniformly distributed on the boundary line between elements. Two major extensions of the AEM (Meguro and Tagel-Din, 2001) have been implemented in IMEM: The first is improving the element type to use different thickness per each spring to be able to follow change of thickness in non-rectangular cross-sections. The second is using different thicknesses for calculating normal stiffness and shear stiffness in each pair of springs. The sort modifications allow modeling cross sectional geometric parameters of structural members using elements with large size. The value of normal and shear stiffness for each pair of springs can be determined as:

$$K_n^i = \frac{E \cdot d \cdot T_n^i}{a} \quad \text{And} \quad K_s^i = \frac{G \cdot d \cdot T_s^i}{a} \tag{1}$$

where: d is the distance between each spring; a is the length of the representative area; E and G are Young's and shear modules, respectively; T_n^i and T_s^i are the thickness represented by the pair of springs "i" for normal and shear cases, respectively.

Dynamic analysis in IAEM

The general differential equation of motion, governing the response of structure in a small displacement range can be expressed as:

$$[M]\{\Delta\ddot{U}\} + [C]\{\Delta\dot{U}\} + [K]\{\Delta U\} = \Delta f(t) - [M]\{\Delta\ddot{U}_G\} \tag{2}$$

where: [M] is mass matrix; [C] is the damping matrix; [K] is the nonlinear stiffness matrix; Δf(t) is the incremental applied load vector; $\{\Delta\ddot{U}\},\{\Delta\dot{U}\},\{\Delta U\}$ and $\{\Delta\ddot{U}_G\}$ are the incremental acceleration, velocity, acceleration, and gravity acceleration vectors, respectively. The mass matrix and the polar moment of inertia of each element have been idealized as lumped at the element centroid. The lumped mass in each DOF direction can be calculated by summing the effect of small segmental masses represented by each spring considering the change of the springs' thickness. Equation (3) represents the value of lumped mass in each degree of freedom direction assuming that elements have rectangular shape.

$$\begin{bmatrix} M1 \\ M2 \\ M3 \end{bmatrix} = \begin{bmatrix} \frac{a \times b \times \rho}{nsp} \cdot \sum_{i=1}^{i=nsp} t_i^x \\ \frac{a \times b \times \rho}{nsp} \cdot \sum_{i=1}^{i=nsp} t_i^y \\ \frac{\rho}{nsp} \cdot \sum_{i=1}^{i=nsp} \left(\frac{t_i^x \times a^3 \times b}{12 \times nsp} + \frac{t_i^y \times a \times b^3}{12 \times nsp} \right) \end{bmatrix} \tag{3}$$

where: a and b are the element dimensions; ρ the density of the material considered.

Large displacement analysis with IAEM (Geometric nonlinearity)

The concept of large displacement analysis has been introduced by Tagel-Din and Meguro (2000). According to their concept, the AEM can follow the large deformation under both static and dynamic load by applying a slight change in the equation of motion Equation (4).

$$[M]\{\Delta\ddot{U}\} + [C]\{\Delta\dot{U}\} + [K]\{\Delta U\} = \Delta f(t) + R_m + R_G \tag{4}$$

Where: R_m represents the residual force vector due to cracking and incompatibility between strain and stress of each spring; and R_G the residual force vector due to geometrical changes in the structure during loading.

By assuming R_m and R_G equal to null and solving Equation (4) to get ΔU, the structural geometry can be modified according to the calculated incremental displacements. According to the modification of geometry of structure and checking the occurrence of cracks, new values for R_m and R_G can be calculated. By using new values of R_m and R_G to recalculate the incremental displacement ΔU, considering the stiffness changes due to cracking and yielding, analyzing the structure subjected to dynamic loading can allow us to follow both geometrical changes of the structure and rigid body motion during failure. The validity of the developed code had been demonstrated by several numerical examples. The verification examples indicate that IAEM shows excellent agreement with both theoretical and finite element results in linear static and dynamic load conditions (Elkholy and Meguro (2003) and Elkholy and Meguro (2004)).

Material nonlinearity

Over the past decades, numerous researchers have developed and validated various methods of performing the inelastic analysis on steel frames which can be categorized into two approaches:

(1) Plastic hinge based approach whish is considered the most direct and simplified approach for representing the material nonlinearity. All elements are assumed to remain elastic except at the places where zero length plastic hinges are allowed to form (Chen et al., 1996). This method accounts for inelasticity but it can't account for the spread of yielding through the section, therefore it is not possible to capture accurately member stability for beam-to-column problems (Liew et al., 1993).

(2) Plastic zone analysis in which the spread-of-plasticity of the member is assumed to be modeled by subdividing the frame members into several finite elements. Each element is subdivided into many fibers (Clarke, 1994). The plastic zone solution is known as an exact solution (Chan, 1989. This method has been used in IAEM whereas the connecting springs work as fibers. Once the strain of each spring is calculated, the stress state can be explicitly determined and the gradual spread of yielding traced.

Material Model

A simplified uniaxial bilinear stress-strain model with kinematic strain hardening is adapted for representing the normal stiffness component of structural steel, as shown in **Figure 1.** In this model, the plastic range remains constant throughout the various loading stages. Although, this is not an entirely realistic representation of the material behavior, it allows for the hardening to be included whilst keeping the formulation simple.

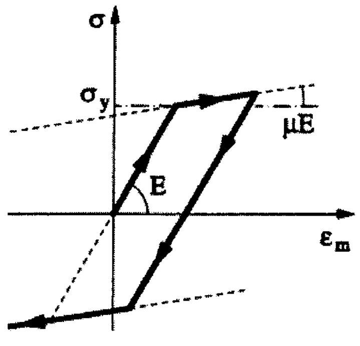

Figure 1 Material Model

VERIFICATION OF THE PROPOSED TECHNIQUE

The ultimate carrying capacity analysis for the rectangular portal frame shown in **Figure 2** had carried out. The frame was divided into 61 rigid elements. The cross section and material properties of the members are: A= 0.645×10^{-2} m^2, I=1.0886×10^{-4} m^4, E=209 kN/mm^2, Fy = 275.8 N/mm^2 and ν=0.30. The horizontal and vertical loads are applied as shown in **Figure 2**. The ultimate load capacity of the frame, according the experimental test that was carried out by Hodge (1959) was 133.0kN. However, based on IAEM, the maximum frame resistance is reached at load (P) of 136kN which is around 2% higher than the maximum recorded load during the experiment. The load-vertical displacement curve obtained by both IAEM and RBSM are plotted in **Figure 3** as well as the experimental data. **Figure 4** shows the location of the developed plastic zones which are represented as dark areas in the figure. The results demonstrate the good agreement with experimental and RBSM results (Ren et al., 1999). It can be also shown that unlike RBSM, the IAEM can follow the spread of yielding through the section. Therefore, it can capture member stability with enough accuracy for a wide range of beam-to-column problems.

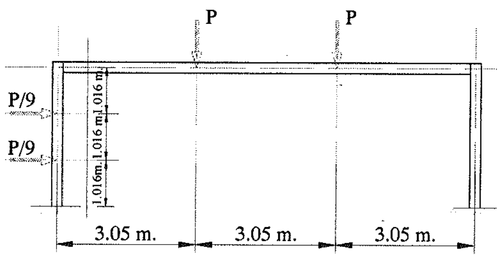

Figure 2 Analysis Model

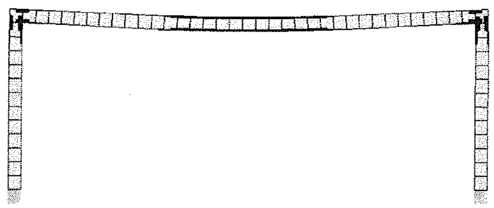

Figure 4 Location of plastic hinges

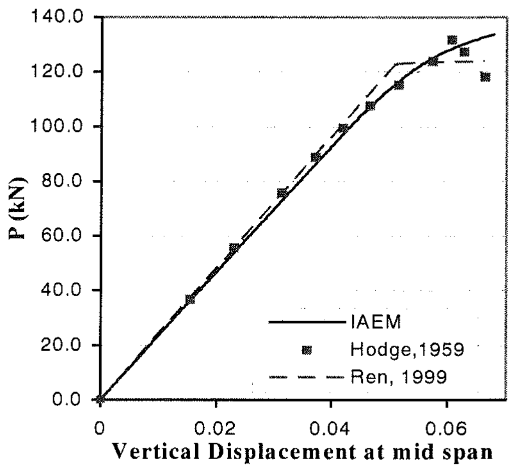

Figure 3 Ultimate load carrying capacity of the frame

CASE STUDY - COLLAPSE OF A NINE-STORY STEEL BUILDING

One of the main advantages of the analytical method is its versatility in parametric study of collapse cases. In this section, the IAEM is applied to investigate the validity of the proposed method in simulating progressive failure of steel structural buildings under hazardous load conditions. The collapsing process of a multi-story steel structure under severe ground motion conditions is presented in this section. The structure considered is a plane nine-story steel frame with three bays of 9.00m long, as illustrated in **Figure 5**. The typical height per story is 3.75m. The dimensions of the structural members are given in **Table 1**. In this frame, columns are bent about their major axes and rigid connections are assumed. The building was designed in accordance with the 1997 NEHRP recommended seismic provisions (Foutch and Yen, 2002). Young's modulus is taken as 205GPa and yield stress is 275 MPa and 355 MPa for beams and columns, respectively. Rayleigh damping with 5% damping for the first fundamental mode was assumed. Using IAEM, only 477 elements are utilized for modeling the whole structure.

Table 1 Cross sections assigned for a 9-story steel building

Story	Columns		Beams
	Exterior	Interior	
9	w14x342	w14x398	w21x62
8	w14x342	w14x398	w27x94
7	w14x398	w14x455	w33x118
6	w14x398	w14x455	w33x118
5	w14x455	w14x550	w36x150
4	w14x455	w14x550	w36x150
3	w14x455	w14x550	w36x150
2	w14x550	w14x550	w40x183
1	w14x550	w14x605	w40x183

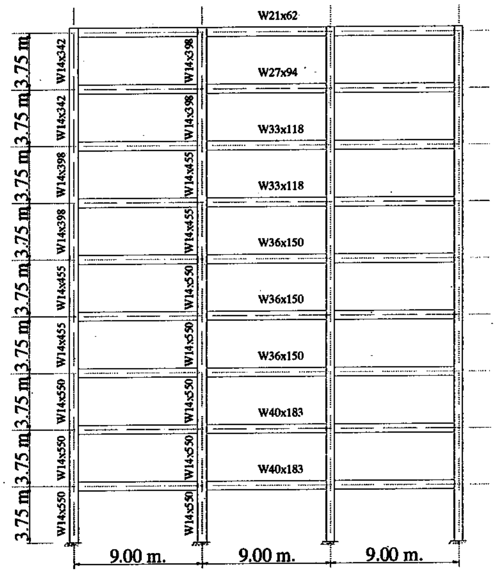

Figure 5 Nine-story steel frame

Seismic response

The inelastic dynamic analysis has been performed, which integrates step-by-step the differential equations of motion corresponding to a given seismic input. Both material and geometric nonlinearity has been considered. Displacement time history analysis has been conducted of combined horizontal and vertical components of the first 40 seconds of the Hyogoken-Nanbu Earthquake (1995). The PGA of the horizontal component (KOBE/KJM000) was 813gal and had a PGD of 17.68cm while the vertical component (KOBE/KJM-UP) had a peak ground acceleration of 336gal and a PGD of 10.29cm.

Collapse analysis

This paper illustrates a simulation of the building collapse under two different failure modes. The first failure is ground floor type failure as illustrated in **Figure 6**. A reduction of 40 % of steel strength of the columns at ground level and lack of ductility in column-to-beam connections were assumed. The intense shaking caused the failure of load bearing columns in the lower floor level and cause progressive failure. According to the figure, firstly the ground motion excitation resulted in the formation of plastic hinges at several locations. The zones that have plastic deformation are represented by dark color in the figure. From the figure, it can be noted that most of the plastic hinges formed in beams, instated of columns, is due to the strong column-weak beam design philosophy. With the progress of time and formation of enough plastic hinges, the weakness of the strength and the low ductility demand of the ground floor level produced a failure in the ground floor columns. The end stage of the failure, illustrated in **Figure 6**, shows a good agreement with a recorded collapse case of multi-story steel buildings due to Hyogoken-Nanbu, Kobe Earthquake (1995) (as shown in **Figure 7**).

Another well observed failure mode is the intermediate soft floor type of failure. This failure mechanism had been widely observed for many multi-story steel buildings due to the Kobe Earthquake (1995), as illustrated in **Figure 8**. The sequence of intermediate soft-story failure based on IAEM simulation is illustrated in **Figure 9**. The collapse had been initiated due to the same assumption of weakness of columns and reduction of ductility at intermediate floor level. The weakness of columns and the intensity of the ground motion develop inelastic behavior through the formation of yielding zones at the connections between beams and columns. Developing plastic zone hinges permit free lateral displacement of frame to occur and initiate the failure.

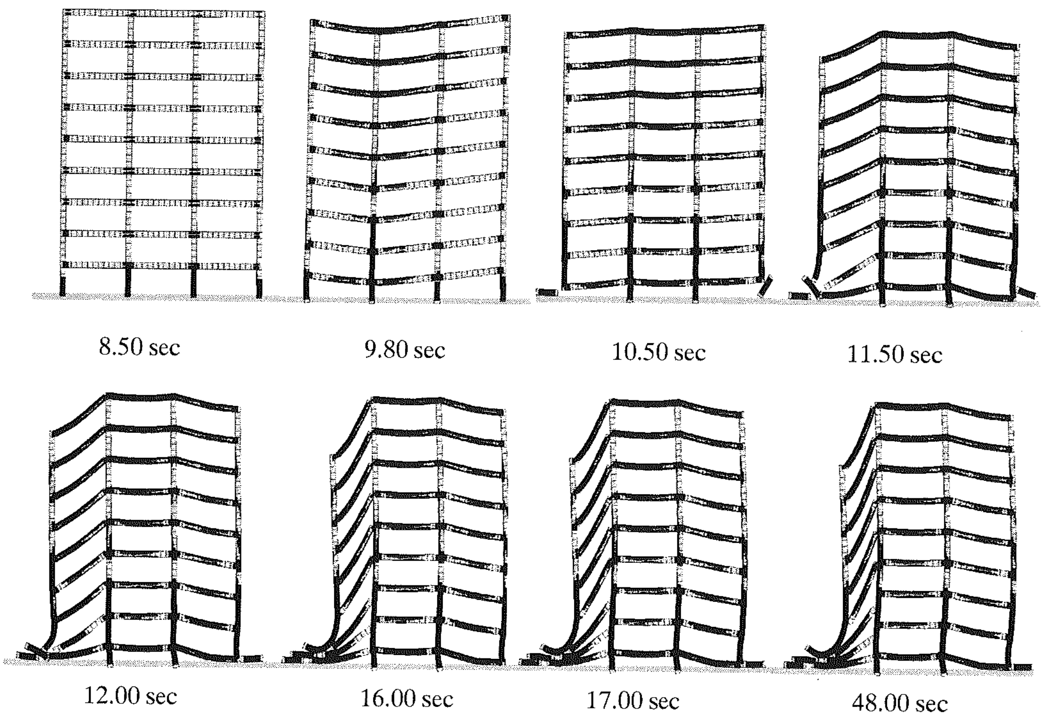

Figure 6 Ground soft-story collapse mechanism

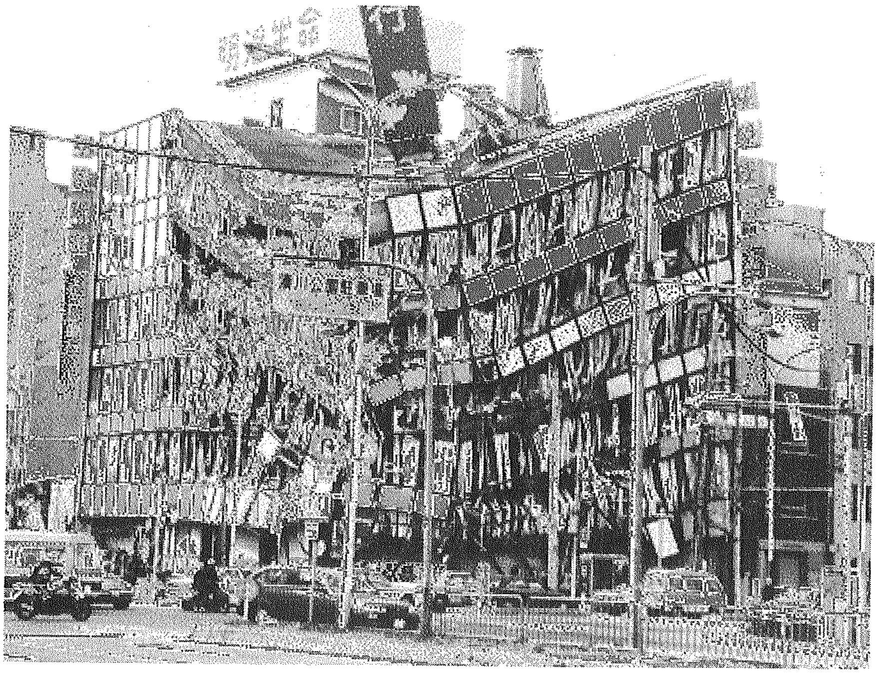

Figure 7 Collapsed steel building during Kobe Earthquake, 1995 (by K. Meguro)

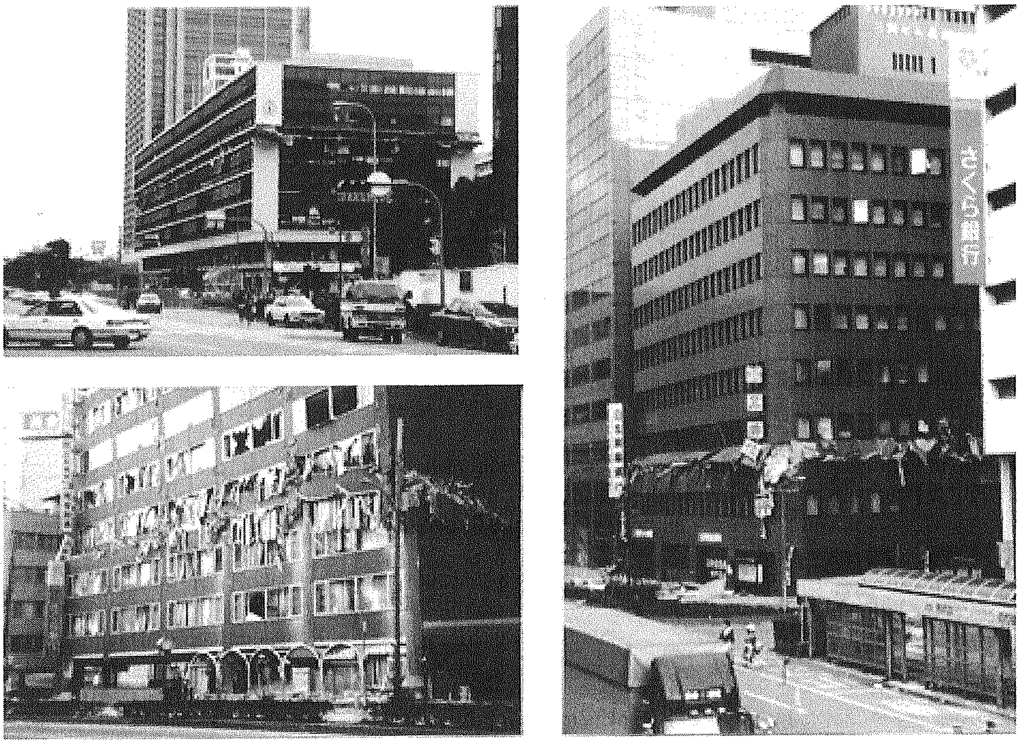

Figure 8 Collapsed commercial buildings during Kobe Earthquake, 1995 (by K. Meguro)

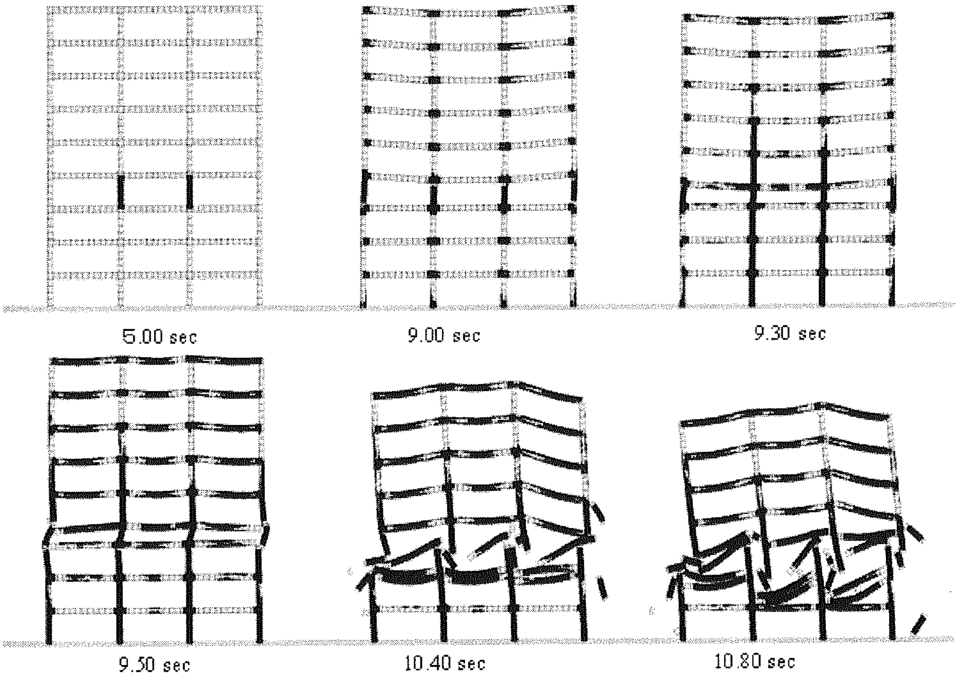

Figure 15: Sequences of soft-story failure of multi-story steel framed structure

From the results, it can be concluded that the collapse of large scale structures due to earthquakes can be performed with sufficient accuracy by using the well-verified and calibrated analysis tool (IAEM). The calculation time required for the simulation of complete failure required only approximately one and half hours on a personal computer. This was due to the simplification of the IAEM w hich assumes a much less number of elements compared to traditional methods. Such a minimal requirement of computational time, with acceptable accuracy, can be considered as a unique advantage of this model.

CONCLUSIONS

This paper has attempted to briefly trace the development of the IAEM for analyzing the entire behavior of large scale steel structures up to total failure. The main feature of this tool is to use as few elements as possible to model each structural component and to obtain a realistic representation of material and geometric non-linearity. The results indicate that the improved method is capable of accurately analyzing the ultimate load-carrying capacity of steel structures. Numerical examples showing the accuracy, efficiency, and the range of application are presented. The program is a useful tool for performing intensive parametric studies to achieve a deeper understanding of structural behavior of steel structures under strong ground motions. Our method can help engineers to investigate the performance of even high-rise buildings under different hazardous loads. The mechanism of progressive failure and the effect on the neighboring buildings can also be simulated.

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