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Finite Element Modeling for Nonlinear Analysis of Multi-Story Multi-Bay RC Buildings subject to Progressive Collapse

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Abstract
Different mathematical F.E. models for a 5 story multi-bay Reinforced Concrete building have been analyzed for progressive collapse when subjected to the removal of one of its 1st floor primary columns. The analysis was carried out through LS-DYNA which is a general-purpose finite element program that runs on a broad range of computing platforms. It is a commercial version of the public domain code that originated as DYNA3D. Three nonlinear model systems for the building have been considered i. e. Model I which considers only the primary lateral-resisting system of the structure as frame elements, Model II which is developed from Model I by adding of flooring as shell element, and Model III which is the same as Model I but all primary lateral-resisting system of the structure was created as solid elements. For the sake of simplicity for Engineers, the paper examines the validity of applying popular commercial programs (such as SAP2000) for building local collapse analysis "progressive collapse" by comparing its results with the corresponding results of LS-DYNA program. The results have been presented and discussed.

Keywords: *Collapse, Multi-story, Multi-bay, RC Buildings, Dynamic Analysis, Nonlinearity, Finite Element.*

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I. Introduction

The progressive collapse term is used to describe the spread of a local failure like a chain reaction, which leads to the partial or total collapse of the building. The main feature of progressive collapse is that the total damage is disproportionate to the original cause. The interest in progressive collapse survey was dated back to three events: gas explosion occurred in an apartment on the 18th floor of a 23-story precast concrete building at Ronan Point – London in 1968, the explosion of a truck bomb in front of the Alfred P. Murrah federal building in Oklahoma City, in 1995 and the terrorist attack against World Trade Center towers, on September 11, 2001, [Ellingwood 2006]

Resistance of building structures to progressive collapse has been an essential task for the development of structural design codes. Many practicing engineers and academic researchers have been engaged in the prevention of progressive collapse. The current available methods in analyzing structural progressive collapse, [BS EN 1991-1-7 (2006), Ellingwood et al. (2007), Zhongxian and Yanchao (2008), UFC (2013), Stevens et al. (2013), Chen, et al. (2019)], could be classified into two major categories, namely the **direct simulation method**, and **uncoupled alternative load path method** with analysis of the structure by simulating various levels of damage by the removal of key load-carrying members. The direct simulation method yields reliable predictions of structural collapse, but it is extremely time consuming, and requires a profound knowledge of structural dynamics, damage mechanics, dynamic material properties and computational skills. Alternative load path method is relatively easy to be used but at the cost of low accuracy. An important issue for a building structure subjected to sudden column loss is its vulnerability to progressive collapse. Reliable methodologies to reduce the susceptibility to progressive collapse may include improving the structural integrity, increasing the specific local resistance for critical members and providing sufficient alternative load paths or redundancy. All the methodologies come to an identical purpose of enhancing the progressive collapse resistance. Hence, estimation of the collapse resistance for the column-removed building under vertical downward loadings may be necessary. Progressive collapse is a dynamic phenomenon; hence proposed analytical models have to capture the dynamic nature of the process.

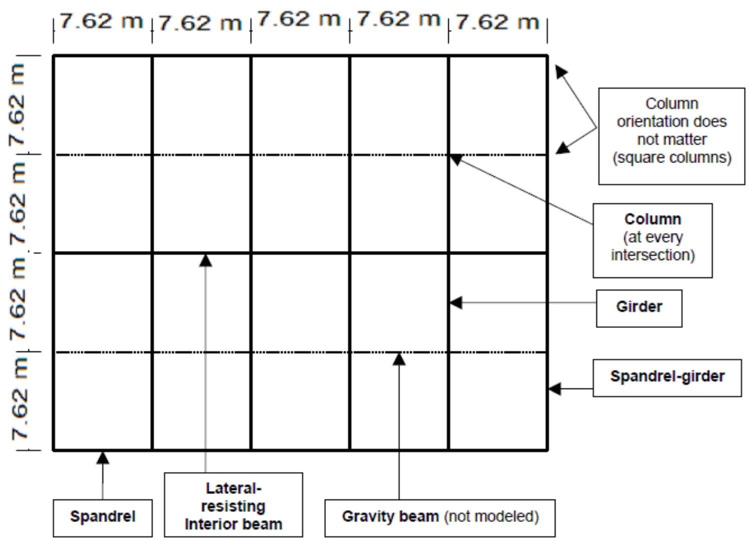
In present paper, different mathematical F.E. models for a five story multi-bay Reinforced Concrete building have been analyzed for progressive collapse when subjected to the removal of one of its 1st floor primary columns. The analysis was carried out using LS-DYNA finite element program that runs on a broad range of computing platforms. Three nonlinear model systems for the building have been considered i. e. Model I which considers only the primary lateral-resisting system of the structure as frame elements, Model II which is developed from Model I by adding of flooring as shell elements, and Model III which is the same as Model I but all primary lateral-resisting system of the structure was created as solid elements. The paper examines also the validity of applying popular commercial F.E programs (such as SAP2000) for building progressive collapse by comparing its results with the corresponding results of LS-DYNA program. The results have been presented and discussed.

II. Description Of The Analyzed 5-Story Rc Building

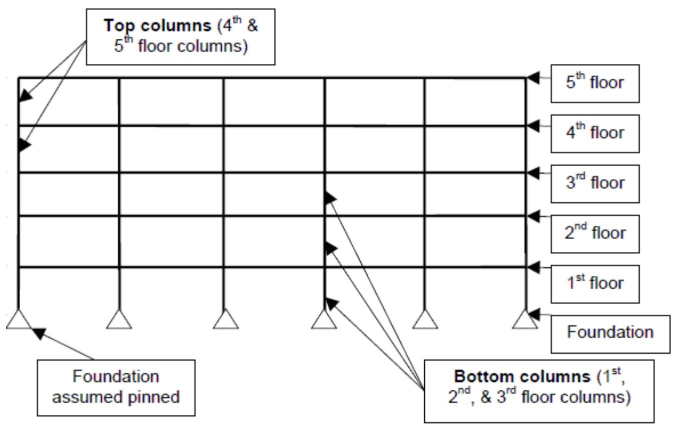
The structure is a five-story reinforced concrete moment frame building from UFC (2005). It is four bays by five bays in plan, with a 7.62 m x 7.62 m typical bay. The function of this building is office use only. Figure 1 shows plan and elevation for building dimensions. Material properties and reinforced concrete member sizes and reinforcement for the building are illustrated in tables 1 and 2 respectively.

Table (1) Material Properties

Concrete strength "FC"	34500 kN/m²
Rebar yield strength "FY"	413700 kN/m²
Modulus of elasticity of concrete "EC"	24855578 kN/m²
Modulus of elasticity of rebar "ES"	2.0E+8 kN/m²
Shear modulus "G"	10356491 kN/m²
Poisson's Ratio of concrete "PR"	0.2
Poisson's Ratio of rebar "PRS"	0.3



(a) Plan



(b) Elevation

Figure (1) Reinforced Concrete Building [1]

Table (2) Reinforced Concrete Member Sizes and Reinforcement

Member Group	Dimensions	Bottom Reinf.	Top Reinf.
Spandrels	B = 0.51 m D = 0.61 m	0.00114 m²	0.00155 m²
Interior Beams	B = 0.51 m D = 0.61 m	0.00114 m²	0.00155 m²
Girders	B = 0.51 m D = 0.76 m	0.00156 m²	0.0029 m²
Spandrel-Girders	B = 0.51 m D = 0.76 m	0.00156 m²	0.0021 m²
Bottom Columns	0.51 m x 0.51 m	0.0052 m²	
Top Columns	0.51 m x 0.51 m	0.0041 m²	

III. Mathematical Models And Load Cases

Model I: This model considers only the primary lateral-resisting system of the structure. The model was created by using SAP 2000 and LS-DYNA programs.
Model II: This model is developed from Model I by add modeling of flooring as shell element. The model was also created by SAP 2000 and LS-DYNA programs.
Model III: This model is the same as Model I but all primary lateral-resisting system of the structure was created as solid elements by LS-DYNA program only. This is because material nonlinearity is not available in SAP2000 program.
The total Dead Load (D) is equal to DL+SDL+CL where:

- Dead Load (DL) is equal to the self weight of the members, it was assumed to be 2.6 KN/m²
- Super-imposed Dead Load (SDL) is equal to 1.67 KN/m². SDL includes partitions, ceiling weight, and mechanical loads
- Cladding Load (CL) is equal to 2.6 KN/m and is applied only on perimeter beams.

Live Load (L) is equal to 2.4 KN/m²

For this building, it was assumed that one of the 1st floor columns did not meet vertical tie force requirements. Because vertical tie forces were not met by this interior column, the alternate path method must be used with this column being removed, UFC (2005).

a) Static Analysis Load Case

For Nonlinear Static analyses of all construction types, apply the following amplified factored load combination to those bays immediately adjacent to the removed element and at all floors above the removed element, DoD (2005).

$$2.0\,[(0.9 \text{ or } 1.2)\,D + (0.5\,L)] + 0.2\,W \qquad (1)$$

For the rest of the structure, apply the load combination in equation (2).

b) Dynamic Analysis Load Case

For Nonlinear Dynamic analyses of all construction types, apply the following factored load combinations to the entire structure:

$$(0.9 \text{ or } 1.2)\,D + (0.5\,L) + 0.2\,W \qquad (2)$$

Where: D = Dead load (kN/m2), L = Live load (kN/m2), W = Wind load (kN/m2)

IV. Nonlinear Analysis Using Ls-Dyna Program

For the five stories building shown in figure 1, three mathematical F.E. models were created using LS-DYNA program. Model I (Frame element), Model II (Frame and shell elements) and Model III (solid element without floor) were created using LSDYNA program, figures 2, 3 and 4 respectively.

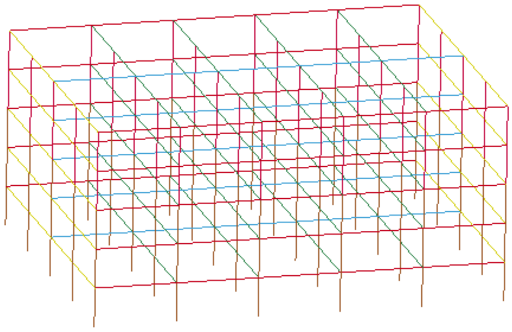


Figure (2) View of mathematical modeling of multi-bay five story RC building LS-DYNA Model I (frame element only)

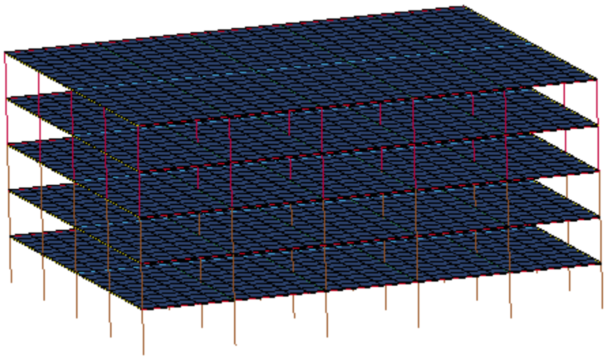


Figure (3) View of mathematical modeling of multi-bay five story RC building LS-DYNA Model II (frame and shell elements)

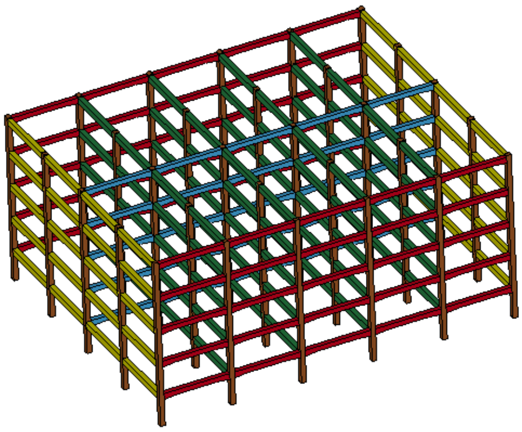


Figure (4) View of mathematical modeling of multi-bay five story RC building LS-DYNA, Model III (solid element without floor)

a) Material Constitutive Models

An important aspect of finite element modeling is the establishment of material constitutive models, which represent the real behavior of the structure in question. The material models in the LS-DYNA constitutive model library are more than capable of accurately simulating the actual material behavior in the model.

1) Reinforcement Concrete Beam Material

This is Material Type 174, for Hughes-Liu beam elements only. The material model can represent plain concrete only, reinforcing steel only, or a smeared combination of concrete and reinforcement. This material is used in model I and model II for beams and columns as shown in figure (5)

```
$                          (7) MATERIAL CARDS
*MAT_RC_BEAM
$MATERIAL NAME:CONCOL
$---+----1----+----2----+----3----+----4----+----5----+----6----+----7----+----8
$      MID        RO      EUNL        PR        FC       EC1      EC50     RESID
         1 2.500E+03   3.0E+10       0.2     3.0E+7    0.0020   3.50E-3     0.200
$       FT     UNITC                                  ESOFT     LCHAR    OUTPUT
    3.0E+6  1.00E-06                                3.0E+10       0.0
$    FRACR   YMREINF   PRREINF   SYREINF   SUREINF      ESHR       EUR    RREINF
     0.005   2.0E+11       0.3   5.0E+08   7.0E+08     0.025      0.05
```

Figure (5) Material input card for reinforced concrete beams element

2) Plastic kinematic / Isotropic Material

This is Material Type 3. This model is suited to model isotropic and kinematic hardening plasticity with the option of including rate effects. It is a very cost effective model and is available for beam (Hughes-Liu), shell, and solid elements. This material is used in model II for slabs. Figure (6) shows the material input card for slab concrete element.

```
*MAT_PLASTIC_KINEMATIC
$MATERIAL NAME:CONSLAB
$      MID        RO         E        PR      SIGY      ETAN      BETA
         2 2.500E+03 3.000E+10 2.000E-01 4.000E+07 0.000E+00 0.000E+00
$      SRC       SRP        FS        VP
 0.000E+00 0.000E+00 0.005E+00 0.000E+00
```

Figure (6) Material input card for reinforced concrete slab

3) Reinforcement Concrete solid Material (Concrete Winfrith model)

The primary constitutive model applied was the Concrete Winfrith model, Material type 084, which is suitable for columns, beams and slab elements. Degenerated eight-node solid elements with variable size were used to model the concrete volume. The Winfrith material model is capable of representing plain concrete, reinforcement bars, and concrete with smeared reinforcement, which is predominantly used in the global model. Figure (7) shows the material input card for solid concrete element used in Model III

```
$---+----1----+----2----+----3----+----4----+----5----+----6----+----7----+----8
$                          (7) MATERIAL CARDS
*MAT_WINFRITH_CONCRETE
$MATERIAL NAME:MAT 84 - WINFRITH CONCRETE MODEL
$C1      MID        RO        TM        PR       UCS       UTS        FE     ASIZE
           1 2.500E+03 3.000E+10 2.000E-01 3.000E+07 3.000E+06 0.100E+03 1.000E-02
$C2        E        YS        EH    UELONG      RATE      CONM      CONL      CONT
  0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 1.000E+00 1.000E+00 1.000E+00
$C3     ESP1      ESP2      ESP3      ESP4      ESP5      ESP6      ESP7      ESP8

$C4       P1        P2        P3        P4        P5        P6        P7        P8
```

Figure (7) Material input card for solid concrete element

b) Procedure of Progressive collapse Analysis using LS-DYNA

A nonlinear dynamic analysis has been carried out using the previously mentioned three F.E. models. A load combination [1.2D + 0.5L] was uniformly applied as vertical gravity load on the entire floor spans of the treated building for sufficient time until the structure becomes under equilibrium. For this building, it was assumed that one of the 1st floor columns did not meet vertical tie force requirements, so the alternate path method must be used with this column being removed, UFC (2005). To initiate progressive collapse, this column is removed suddenly at specified time during the analysis. The member forces of the column, which is to be removed to initiate progressive collapse, were computed before its removable. Then, the column was replaced by the point loads equivalent to its member forces. In order to simulate the phenomenon that the column was abruptly removed, the member forces were suddenly removed after a certain time had elapsed while the gravity load remained unchanged. The member forces were increased linearly for two seconds until they reached their full amount, kept unchanged for one second until the system reached stable condition and then suddenly removed to initiate progressive collapse.

c) Results of progressive collapse analysis using LS-DYNA program

<u>(Case 1)</u> - Model I (frame element only)

Figure (8) shows isometric shape for vertical deformation of the RC building at time 4.0 sec with maximum vertical displacement of 2.19 m. Figure (9) provides time history diagram for the vertical displacement of the RC building during progressive collapse analysis

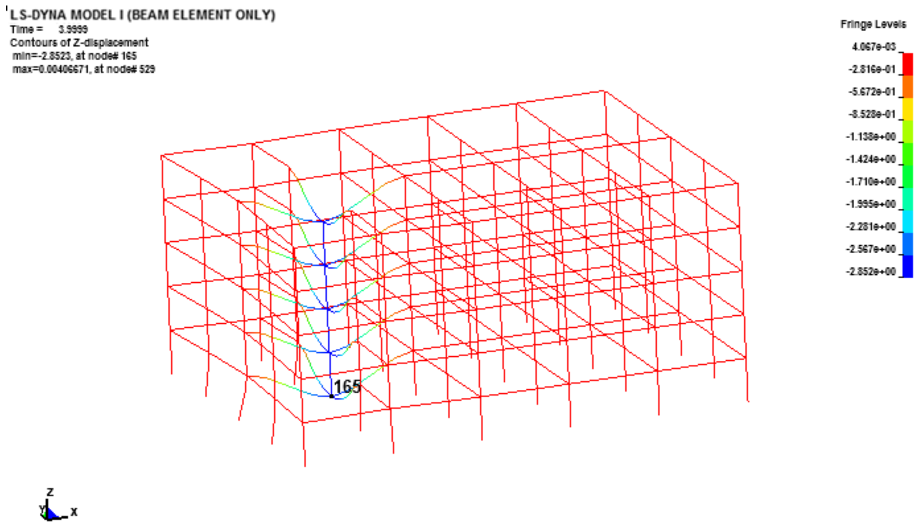


Figure (8) Isometric shape for vertical deformation, LS-DYNA Model (I), time=4.0 sec.

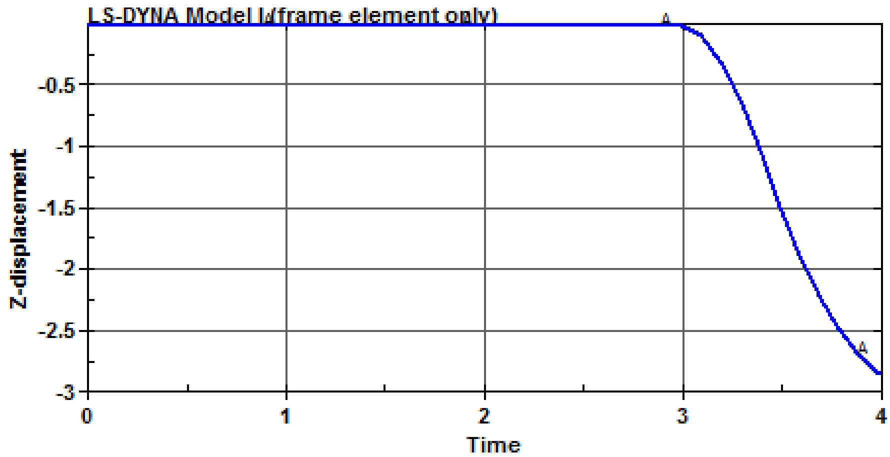


Figure (9) Time history diagram for vertical displacement of point above removed column, LS-DYNA Model I (frame element only)

<u>Case (2)</u> - Model II (frame and shell elements)

In this analysis, adding the floor model makes the building more stiffer to resist the progressive collapse after removal of the column. Figure (10) shows isometric shape for vertical deformation of the RC building at time 4.0 sec. Figure (11) provides time history diagram for the vertical displacement of the RC building during progressive collapse analysis with maximum vertical displacement about 0.088 m at time = 3.5 sec.

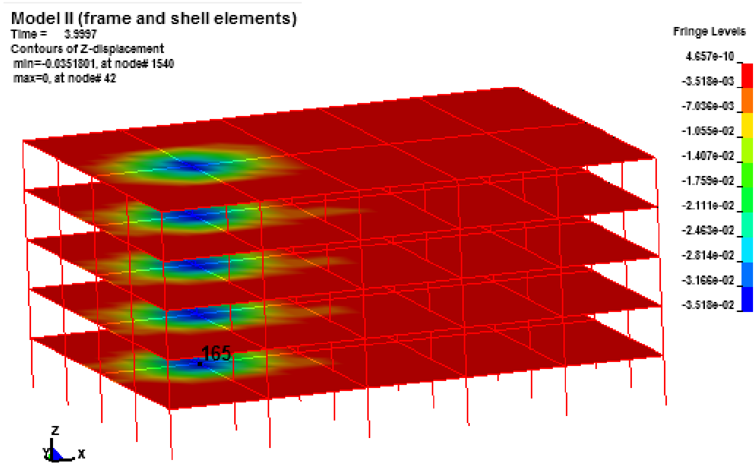


Figure (10) Isometric shape for vertical deformation, Model II (frame and shell elements)

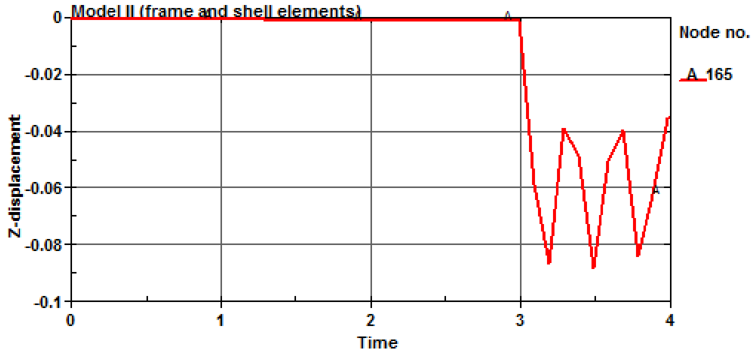
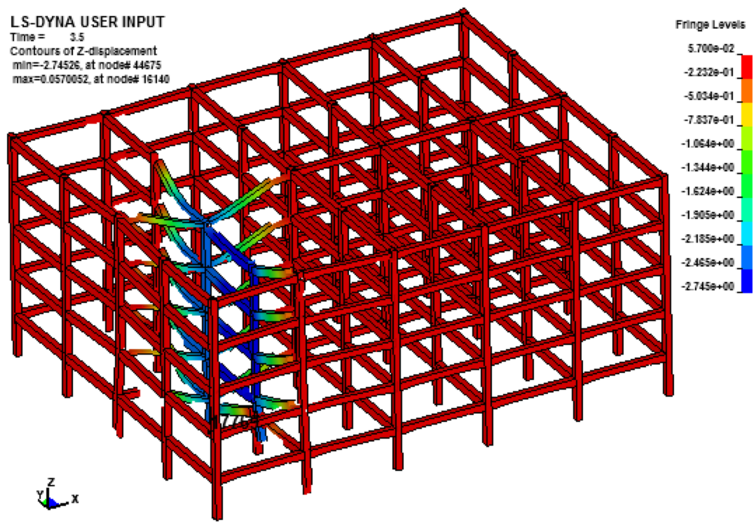


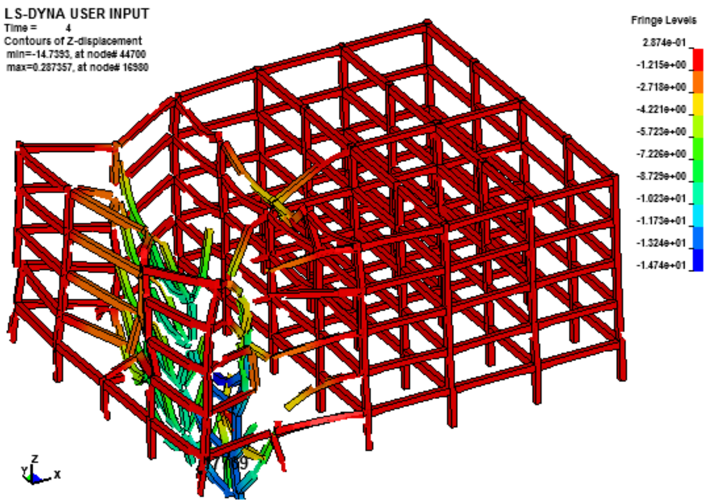
Figure (11) Time history diagram for vertical displacement of point above removed column, LS-DYNA Model II (frame and shell elements)

<u>Case (3)</u> - Model III (solid element model without floor)

Figure (12) illustrates the progressive collapse shapes of RC building at time 3.5, and 4.0 sec respectively. Figure (13) provides time history diagram for the vertical displacement of the RC building during progressive collapse analysis.



(a) Time = 3.5 sec



(b) Time = 4 sec

Figure (12) Collapse scenario shapes for Model (III) at different times

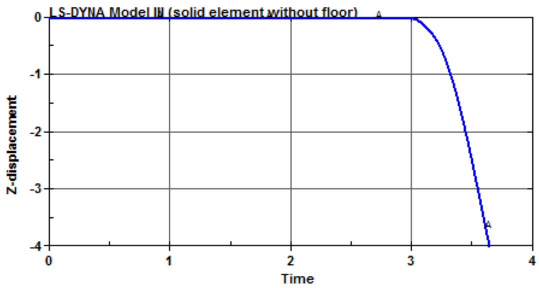


Figure (13) Time history diagram for vertical displacement of point above removed column, LS-DYNA Model III (solid element without floor)

V. NONLINEAR ANALYSIS USING SAP2000 PROGRAM

The SAP name has been synonymous with State-of-the-art analytical methods since its introduction over 30 years ago. SAP2000 follows in the same tradition featuring a very sophisticated, intuitive and versatile user interface powered by an unmatched analysis engine and design tools for engineers. It is one of the most popular software available for use by engineers of different specializations.

Models I and II previously analyzed using LS-DYNA were once more simulated using SAP2000:

<u>Model I</u> <u>Figure (14)</u>: which considers only the primary lateral-resisting system of the structure as frame elements

<u>Model II</u> <u>Figure (15)</u>: This is developed from Model I by adding of flooring as shell elements

Model III which is the same as Model I but all primary lateral-resisting system of the structure was created as solid elements was not produced by SAP2000 because material nonlinearity is not available in SAP2000 program

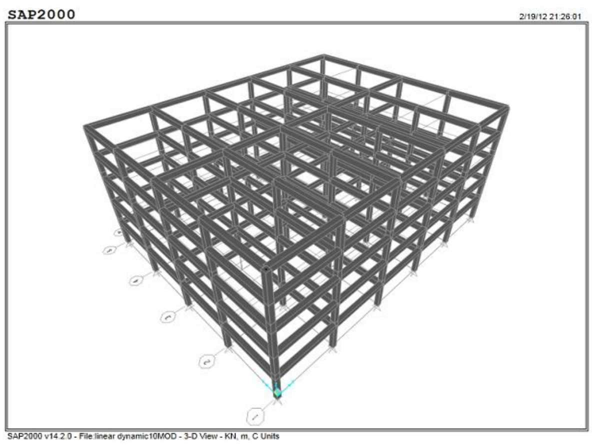


Figure (14) Five story building create by SAP2000 program, Model I (Frame element only)

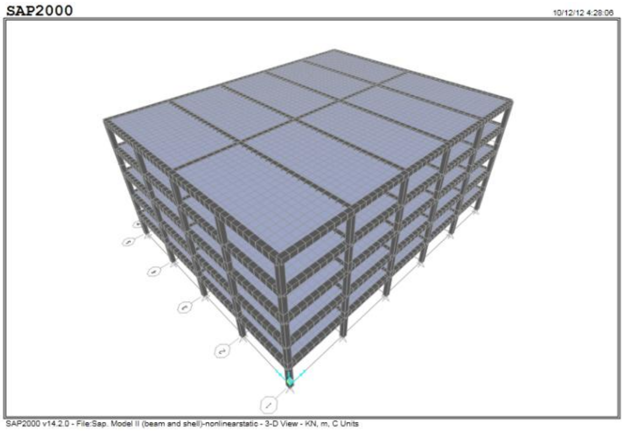


Figure (15) Five story building create by SAP2000 program, Model II (Frame and shell elements)

a) Material Models

For the nonlinear analysis, plastic hinges are allowed to form along the members. These hinges are based on maximum moment values. However, only moments can cause a plastic hinge to form in flexural members, and only the axial-moment interaction (PMM) can cause a plastic hinge to form in a column. Any shear or torsion values that would cause a hinge to form would result in an immediate failure.

Theoretically hinges can occur anywhere along the beam. However, for simplifying the models, hinges are allowed to occur at the ends of each member and at the midspan of the flexural members (most probable locations). The plastic hinge properties are adapted from the reinforced concrete member rotation requirements of, figure (16). It should be noted that reinforced concrete member allowable nonlinear capacity is based on absolute rotation, independent of member section properties. The parameters shown in table on figure (17) are based on tables 6–7 of FEMA-356 (2000).

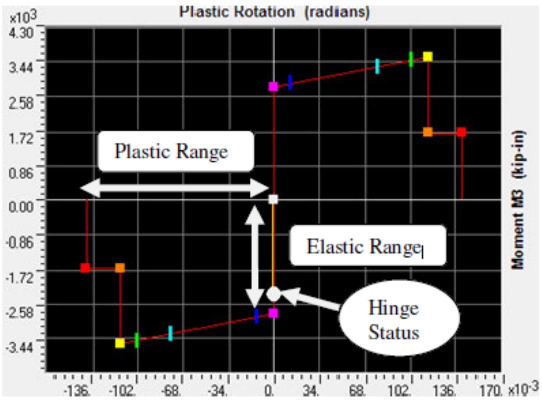
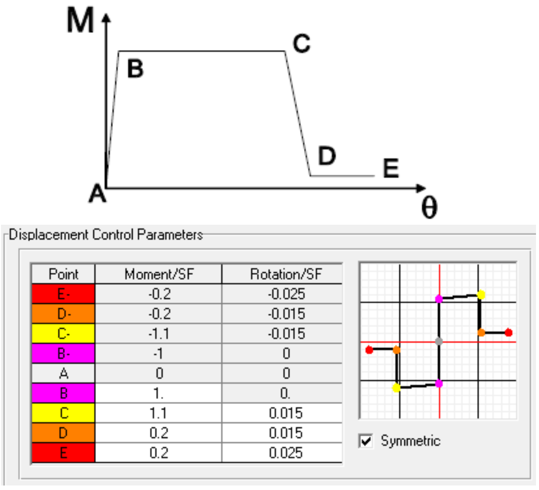


Figure (16) Description of the hinge status for nonlinear analysis



Point	Moment/SF	Rotation/SF
E-	-0.2	-0.025
D-	-0.2	-0.015
C-	-1.1	-0.015
B-	-1	0
A	0	0
B	1.	0.
C	1.1	0.015
D	0.2	0.015
E	0.2	0.025

Figure (17) Nonlinear Hinge Properties

b) Procedure of Progressive collapse Analysis using SAP2000

Nonlinear static and dynamic analysis methods were applied to investigate the progressive collapse potential of the RC building.

For nonlinear static analysis, the load combination 2(1.2DL + 0.5LL) was used as vertical load in the span where a column was removed, and the load combination 1.2DL + 0.5LL was applied in the other.

For nonlinear dynamic analysis, the load 1.2DL + 0.5LL was uniformly applied as vertical load in the entire span as shown in figure (18). In order to carry out dynamic analysis the member forces of a column, which is to be removed to initiate progressive collapse, were computed before it is removed. Then, the column was replaced by the point loads equivalent of its member forces as shown in figure (19). In order to simulate the phenomenon that the column was abruptly removed, the member forces were suddenly removed after a certain time had elapsed while the gravity load remained unchanged, where the variables P, V, and M denote the axial force, shear force, and bending moment, respectively, and W is the vertical.

In this study, the member forces were increased linearly for two seconds until they reached their full amount, kept unchanged for one second until the system reached stable condition and then suddenly removed to initiate progressive collapse.

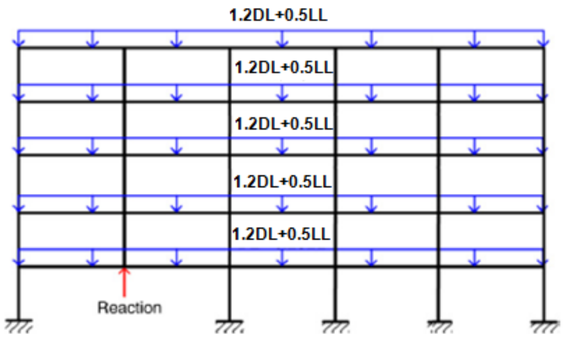


Figure (18) Distribution of load combination

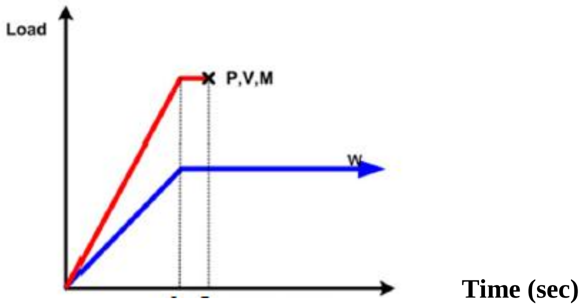


Figure (19) Dynamic load (P, V, M) and static load (W)

c) Results of progressive collapse analysis using SAP2000 program

<u>Case (1)</u> - Model I (frame element only)

Throughout the nonlinear dynamic analysis, the status of hinge in terms of elastic/plastic behavior at the girders connected to the top of removed column is presented in figure (20). It can be observed in this figure that the hinge stress status is <u>over the material yielding value</u> and therefore the building is going to collapse. The maximum vertical displacement for this case of analysis reaches 1.35 m at 4.0 sec., figure (22), this is consistent with the dynamic analysis for progressive collapse presented by Kim and Hong S. (2009).

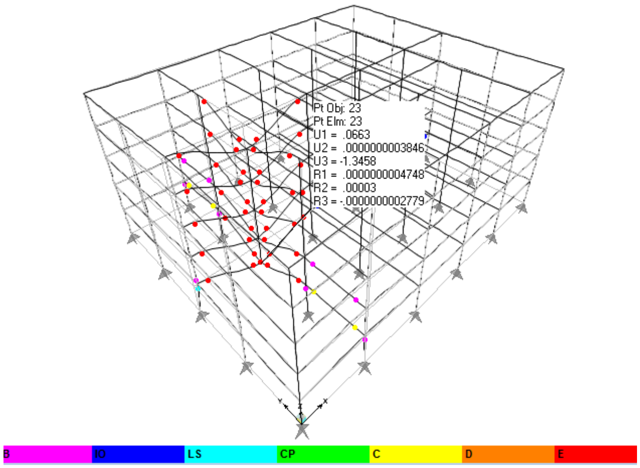


Figure (20) Isometric shape for vertical deformation, Model I (frame element only), SAP2000 analysis (at time = 4 sec)

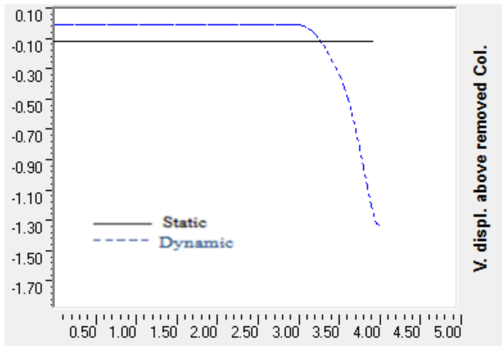


Figure (21) Time history diagram for vertical displacement of the point above removed column, Model I (frame element only), SAP2000 analyses

<u>Case (2)</u> Model II (frame and shell elements)

Figure (22) shows the Isometric Structure deformed shape of dynamic nonlinear analysis for model II (frame and shell elements). In this figure, the hinge stress status is under the material yielding value and therefore the building is not going to collapse. The maximum value of vertical displacement of the point above removed column is 0.088 m at 4.0 sec., figure (23).

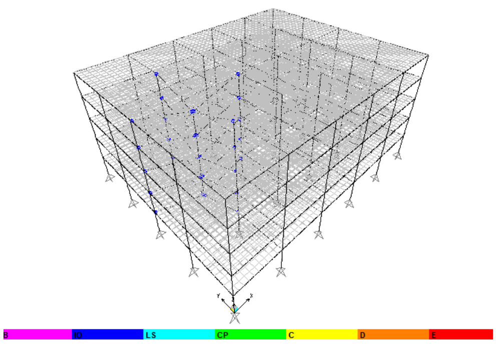


Figure (22) Isometric shape for vertical deformation, Model II (frame and shell elements), dynamic nonlinear analysis

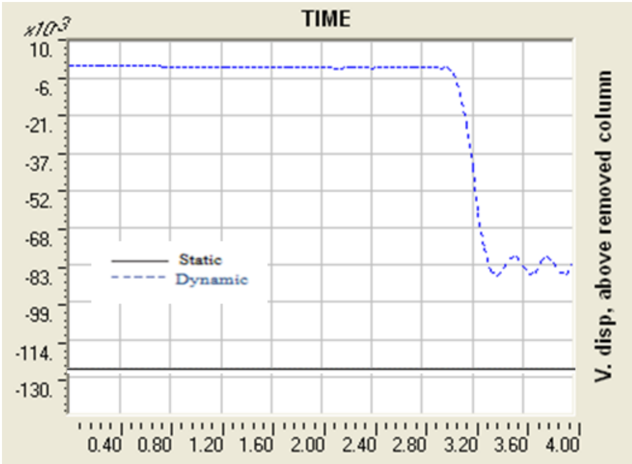


Figure (23) Time history diagram for vertical displacement of point above removed column, SAP2000 Model II (frame and shell elements)

<u>Case (3)</u> - Model I (frame element only with modified properties)
To avoid that the hinge stress status is over the material yielding value, the cross sections and reinforcement of the beams which have plastic hinge were modified as shown in table (3).

Table (3) Properties of Redesigned Members

Member Group	Dimension	Bottom Reinforcement	Top Reinforcement
Interior Beams	B = 0.51 m D = 0.71 m	0.001419 m²	0.001935 m²
Girders	B = 0.51 m D = 0.81 m	0.00195 m²	0.00361 m²

The nonlinear dynamic analysis for model I after modification and the status of hinge in terms of elastic/plastic behavior at the girders connected to the top of removed column are presented in figure 24. It can be observed in Figure (24) that the hinge stress status is under the material yielding value. The maximum value of vertical displacement of the point above removed column is 0.018 m, figure (25).

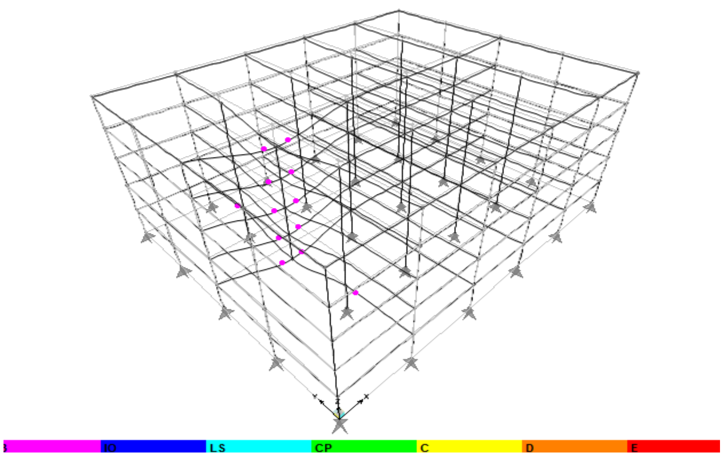


Figure (24) Isometric Structure deformed shape of dynamic nonlinear analysis for model I after modification (at time = 4 sec)

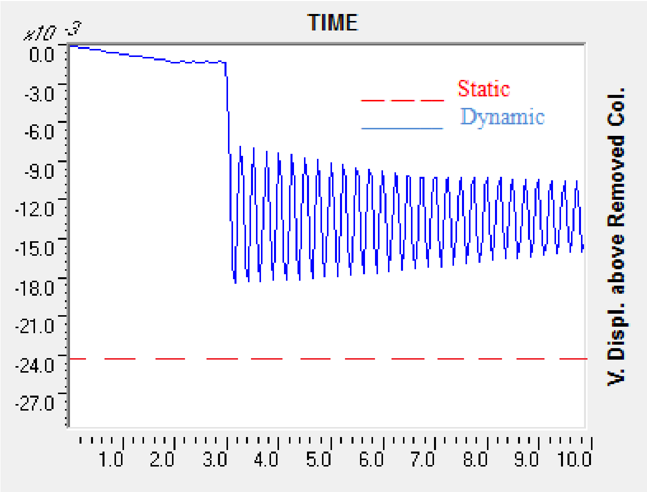


Figure (25) Time history diagram for vertical displacement of point above removed column, model I after modification

VI. Discussion of Results

1) Nonlinear dynamic progressive collapse analysis F.E. model I (frame element only) created by both LS-DYNA and SAP2000 programs, gives the same behavior of collapse (failure occurred in the two models), however the resultant maximum vertical displacement of point above removed column from the two programs are different, figures (9) and (21) respectively.

2) Nonlinear dynamic progressive collapse analysis F.E. model II (frame and shell elements) created by LS-DYNA and SAP2000 programs, gives the same behavior of deformation (no failure), and the resultant maximum vertical displacement of point above removed column from the two programs is almost the same from both, figures (11) and (23) respectively.

3) Adding floor model in Model II (frame and shell elements) makes the model stiffer to resist the collapse when removing the column. Thus, no clear failure occurred in the studied model.

4) Model I (with frame element only) have the same behavior of collapse as model III (with solid elements only), however model III gives more details for member stress distribution and building collapse scenario, figures (8) and (12). It should be noticed that model III by LS-DYNA requires more time consumption in nonlinear dynamic analysis.

5) The nonlinear dynamic analysis by SAP2000 gives in all studied cases in this paper trusted results. In contrast, equivalent nonlinear static analysis do not give real results when the hinge stress status is over material yielding value and the building is going to collapse.

VII. Conclusion

SAP2000 program is general purpose civil-engineering software in the market. The majority of engineers used program SAP2000 for the analysis and design of structures and therefore it can be easy used for progressive collapse analysis. On contrast, LSDYNA is an advanced general-purpose multi physics simulation software package used for simulating complex structures and it can be used by expert engineers only to give the complete collapse scenario.

The present study indicates that using nonlinear dynamic analysis by SAP2000 program is sufficient for the design purpose of structures against progressive collapse. However, this program did not give the complete collapse scenario of the structures. On the other hand, using LSDYNA program for progressive collapse analysis of structures gives complete collapse scenario of the structures.

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