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Pushover Analysis for Reinforced Concrete Portal Frames

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Abstract— This research provides a new methodology and a computer program in Java language to analyze reinforced concrete structures till failure and takes into consideration both geometric and material nonlinearities. The analysis uses the stiffness method as well as the finite element principles and analyses also the reinforced concrete cross section to calculate the member axial and flexural rigidities. The coordinates are updated each iteration and the axial load effect is considered in the geometrical stiffness matrix and also determines the failure criteria during the analysis. The development of plastic hinges throughout the building can be observed. The nonlinear equations are solved by the Newton Raphson method. A computer program (POA) is specially developed to solve different types of plane frames. It consists of the main and four classes. The application to the solution of one story as well as two story portal frames proved that the analysis is robust and the deviation to the experimental result is less than 3 %.

Index Terms— Nonlinear analysis, Material nonlinearity, Geometric nonlinearity, Reinforced concrete, Newton Raphson method, Portal Frame, Pushover Analysis

I. Introduction

Push-over analysis is a simple method for the nonlinear static analysis of building structures subjected to monotonically increasing horizontal loading and which is designed to be a part of new methodologies for the seismic design and evaluation of structures. In the classical analysis methods of plane framed structures, the axial and flexural rigidities are assumed to be constants. Such ideal conditions are unrealistic because the material behavior is actually nonlinear. The axial and flexural rigidities decrease with the increasing internal forces. The structure geometry is continuously changing with the varying applied forces too. A step-by-step nonlinear analysis method to invest up to failure is essential.

Parente et-al (2014) [1] formulated a co-rotational frame structural analysis program and used the principle of virtual work to find the material and geometrical nonlinear matrices and combined these matrices to form the tangent stiffness matrix for both local and global element and structural stiffness matrices. The cross section was divided to one to five layers according to the stress strain relations and used Gauss quadrature rule in the numerical integration.

Alaa et-al (2014) [1] in his thesis "Finite Element Analysis of Reinforced Concrete Joints" studied, used, and developed a computer program form the internet in Java Language named Jlimpo for the linear analysis of plane frame structures. He applied the program for the solution of statically determined joint frames. The program was used for the internal force calculations.

These internal forces were applied to anther program (Alaa Crack) for the cross section to calculate the cracking moment (M_c), the ultimate moment (M_u) and the crack length.

El Gendy (2012)[3] produced a computer program (NARC) for the frame structural analysis taking into consideration soil structure interaction. The material nonlinearity takes into consideration the distribution; number and diameter of the reinforcing steel used throw layered analysis for cross section. The soil was modeled using four different linear and nonlinear models.

El Hout et-al (1989) [4] developed a simple direct formulation and a computer program in Basic Language for the solution of plane frame structure taking into consideration both material and geometrical nonlinearities. In the material modeling, tension stiffening in concrete and compression for confined concrete as well as the strain hardening in steel were considered. The geometric stiffness considered axial load effect and updated the coordinates after each solution load step. The analysis was divided into two programs, one of them for the cross section (M-C) to find moment curvature relationship for each different cross section and the other for frame structure (RCF) to find load deflection relationship.

In this research, a new finite element model is developed in Java Language (Gehad POA) to analyze the nonlinear behavior of plane frame structures. The program takes into consideration both the material and geometrical nonlinearities. For the material nonlinearity, real stress-strain curves for confined and unconfined concrete and reinforcing steel in tension and compression up to failure are considered. The cross section is divided into equal number of layers where strains and stresses are calculated for each layer and summed up to find the axial and flexural rigidities to be used in the stiffness matrix. The

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geometrical nonlinearity is taken into considerations by updating the structural coordinates after each iteration, also the effect of axial load is considered in the geometrical matrix. The solution of nonlinear equilibrium equations is performed using the incremental-iterative Newton Raphson techniques. The solution can trace the load deformation up to failure of steel or concrete and through the formation of the plastic hinges. An object oriented program in Java is developed for the solution of the nonlinear equilibrium equations which is accurate, effective and easy to implement.

2- Formulation of the tangent stiffness matrix

2.1. Material Model

The material models adopted in the present work, are:-

2.1.1. Concrete in Compression

Two stress-strain relations were adopted to represent the behavior of compressed concrete: Hognestad curve[5] (Figure 1) for unconfined concrete and the curve recommended by Mander [6] for confined concrete. The relation is modeled by a parabolic curve up to the maximum strength $\left(f_c^{\prime}\right)$ followed by a descending linear till failure.

The equations of unconfined sections (without stirrups) are:-

For the parabola:

$$f_c = f_c' \left[\frac{2\varepsilon_c}{\varepsilon_{co}} - \left(\frac{2\varepsilon_c}{\varepsilon_{co}} \right)^2 \right] if \ 0 \le \varepsilon_c < \varepsilon_{co} \quad (1)$$

For the straight line:-

$$f_c = f_c' \left[1 - 0.15 \left(\frac{\varepsilon_c - \varepsilon_{co}}{\varepsilon_{cu} - \varepsilon_{co}} \right) \right] if \ \varepsilon_{co} \le \varepsilon_c < \varepsilon_{cu} \quad (2)$$

For confined sections (with stirrups), a similar model is considered as shown in the figure 1. It should be noticed that the crushing strain of a confined section is much higher than that of an unconfined section.

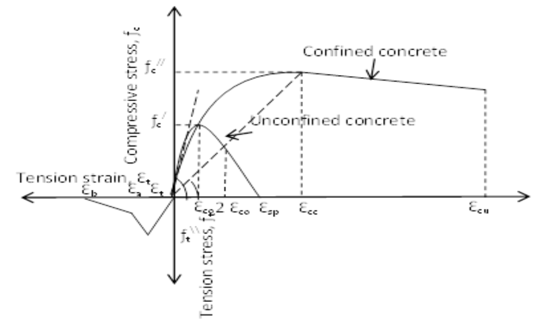

Fig.1. Stress-Strain Curves for Confined Concrete and unconfined concrete in Compression and Tension

For confined concrete, the stress in concrete $\left(f_c\right)$, corresponding to a strain $\left(\varepsilon_c\right)$ is given by

$$f_c = \frac{f_{cc}' x r}{r - 1 + x^r} \quad (3)$$

Where, $x = \dfrac{\varepsilon_c}{\varepsilon_{cc}}$ (4), $\varepsilon_{cc} = 0.002\left[1 + 5\left(\dfrac{f_{cc}'}{f_c'} - 1\right)\right]$ (5)

$r = \dfrac{E_c}{E_c - E_{sec}}$ (6), $E_{sec} = \dfrac{f_{cc}'}{\varepsilon_{cc}}$ (7), $E_c = 5000\sqrt{f_c'}\,(MPa)$ (8)

2.1.2. Concrete in Tension

It can be clearly observed that the tensile strength of concrete increases linearly with increasing strain up to cracking[6]. At cracking strain $\left(\varepsilon_{cr}\right)$, a small subsequent drop in tensile strength occurs. The tensile strength then decreases monotonically with increasing strain up to failure.

For $\varepsilon_t \le \varepsilon_{cr}$, $\quad f_t = E_t * \varepsilon_t \quad (9)$

For $\varepsilon_{cr} < \varepsilon_t \le \varepsilon_a$, $\quad f_t = f_{tu}\left(\dfrac{2\varepsilon_t + \left(\varepsilon_{cr} - 3\varepsilon_a\right)}{3\left(\varepsilon_{cr} - \varepsilon_a\right)}\right) \quad (10)$

For $\varepsilon_a < \varepsilon_t \le \varepsilon_b$, $\quad f_t = f_{tu}\left(\dfrac{\varepsilon_t - \varepsilon_b}{3\left(\varepsilon_a - \varepsilon_b\right)}\right) \quad (11)$

2.1.3. Reinforcing Steel

Figure 2 shows a typical stress-strain relationship of steel in tension and compression [8].

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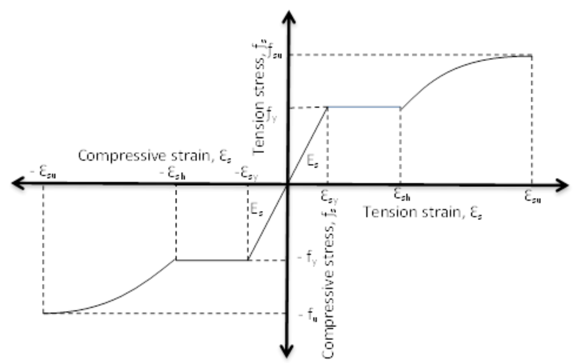

Fig.2. Stress-Strain Curve for Reinforcing Steel

For $\varepsilon_s < \varepsilon_y$ $\qquad F_s = \varepsilon_s * E_s$ (12)

For $\varepsilon_y < \varepsilon_s < \varepsilon_{sh}$ $\qquad F_s = f_y$ (13)

For $\varepsilon_{sh} < \varepsilon_s < \varepsilon_{su}$

$$F_s = f_y \left\{ \frac{(m*k_1+2)}{(60*k_1+2)} + \frac{k_1(60-m)}{2*(30*r+1)^2} \right\} \quad (14)$$

In which:

$$m = \frac{\left\{ \left[\left(\frac{f_u}{f_y} \right) (30*r+1)^2 \right] - 60*r - 1 \right\}}{(15*r^2)} \quad (15),$$

$r = \varepsilon_{su} - \varepsilon_{sh}$ (16), $k_1 = \varepsilon_s - \varepsilon_{sh}$ (17)

2.2. Cross Section Models

The nonlinear model of reinforced concrete sections subjected to strain (ε_n), and the curvature (κ) is developed in this section, based on the following assumptions:-

- Strain distribution is linear along the section while the stress distribution is nonlinear.

- The bond between concrete and reinforcing steel is perfect.

Figure 3 shows a reinforced concrete section subjected to an axial force (N) at the centroid of the cross section and a bending moment (M) with the corresponding strain and stress distributions.

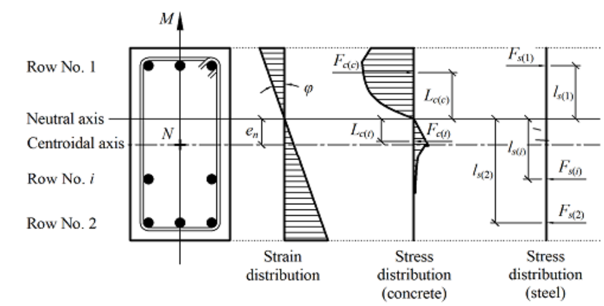

Fig.3. Strain and Stress Distributions on RC section

The cross-section is divided into a number of equal concrete strips as shown in Figure 3. The strains for each strip are functions of the cross section curvature (κ) and the strain (ε_n), while the stresses, in turn, are functions of the strains. The mechanical properties of the cross section (EA & EI) on each concrete strip are numerically integrated in order to obtain the stress and strain acting on that strip.

$EA = \int_A E_t dA$ (18), $EI = \int_A E_t y^2 dA$ (19)

2.3. Strain Displacement Model

Two-node beam element having four degrees of freedom; one lateral and one rotational at each node, beside the axial effect at each node to have six degree of freedom system (Figure 4).

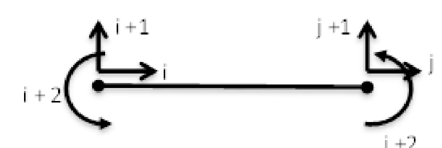

Fig. 4 Degree of freedom of element

Hermitian shape functions are used for the two-node beam element (Equations 20, 21, 22, 23).

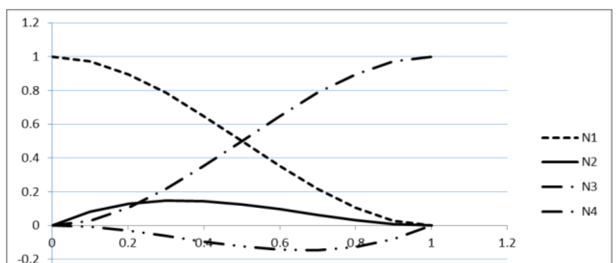

Fig.5. Hermitian Function

$$N_1 = 1 - 3\left(\frac{x}{L}\right)^2 + 2\left(\frac{x}{L}\right)^3 \quad (20), \quad N_2 = x\left(1 - \frac{x}{L}\right)^2 \quad (21)$$

$$N_3 = 3\left(\frac{x}{L}\right)^2 - 2\left(\frac{x}{L}\right)^3 \quad (22), \quad N_4 = x\left(\left(\frac{x}{L}\right)^2 - \frac{x}{L}\right) \quad (23)$$

The second derivative of the shape functions is the strain-displacement matrix $[B]$ and is given by:

$$[B] = \frac{d^2}{dx^2}[N] \quad (24),$$

$$[B] = \begin{bmatrix} -6\dfrac{(L-2x)}{L^3} \\ -2\dfrac{(2L-3x)}{L^2} \\ 6\dfrac{(L-2x)}{L^3} \\ -2\dfrac{(L-3x)}{L^2} \end{bmatrix} \quad (25)$$

For the straight beam the stiffness matrix $[K_m]$ is:

$$[K_m] = [B]^T * EI * [B] \quad (26)$$

The element stiffness matrix [k]is 4 by 4 matrixes and EI parameter is obtained from the cross-section. The EI parameter is calculated from cross section model.

Curvature (κ) is the second derivative of the displacements (u), as seen in Equation (27).

$$\kappa = \frac{d^2\bar{u}}{dx^2} \quad (27)$$

$$u(x) = [\bar{u}]_{1*4}[N]_{4*1} = \overline{u_{11}}N_{11} + \overline{u_{12}}N_{21} + \overline{u_{13}}N_{31} + \overline{u_{14}}N_{41} \quad (28)$$

$$\kappa(x) = [\bar{u}]_{1*4}[B]_{4*1} = \overline{u_{11}}B_{11} + \overline{u_{12}}B_{21} + \overline{u_{13}}B_{31} + \overline{u_{14}}B_{41} \quad (29)$$

At this step a 4x4 stiffness matrix is constructed. The axial degrees of freedom are added to the system. As the flexural stiffness softens via the displacements, also the axial stiffness decreases.

$$[k_m] = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & -\frac{12EI}{L^3} & \frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{2EI}{L} \\ -\frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & -\frac{12EI}{L^3} & -\frac{6EI}{L^2} & 0 & \frac{12EI}{L^3} & -\frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{4EI}{L} \end{bmatrix} \quad (30)$$

2.4. Force Displacement Model

If a structure is in the state of stable equilibrium and the small displacement theory is valid, then there is a relationship between the deformations of the structure and the applied load system. The load displacement relationship can be expressed by:

$$[k_T]_i * \{d\delta\}_i = \{dF\}_i \quad (31)$$

Where,

$[k_T]_i$ = tangential stiffness matrix for the ith element,

$\{d\delta\}_i$ = displacements vector for the ith element, $\{dF\}_i$ = force vector for the ith element, the tangential stiffness matrix $[k_T]$ is composed of two components and can be written as:

$$[k_T]_i = [k_m]_i + [k_g]_i \quad (32)$$

Where,

$[k_m]_i$ *and* $[k_g]_i$ = material and geometrical stiffness matrix for the ith element, respectively,

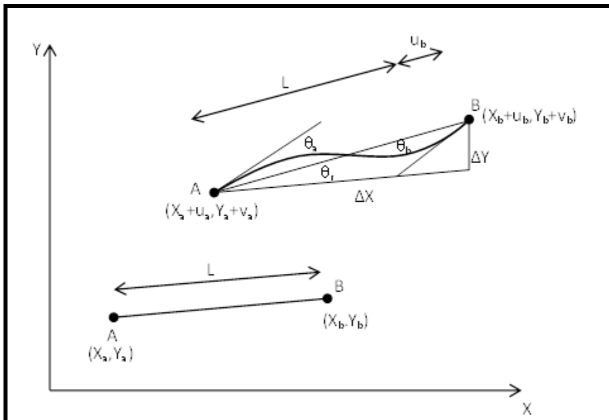

Fig.6. update coordinates

The geometric stiffness matrix $[k_g]$ for frame element is given by: -

$$\left[k_g\right]=\frac{\pm P}{L}\begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{6}{5} & \frac{L}{10} & 0 & -\frac{6}{5} & \frac{L}{10} \\ 0 & \frac{L}{10} & \frac{2L^2}{15} & 0 & -\frac{L}{10} & -\frac{L^2}{30} \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -\frac{6}{5} & -\frac{L}{10} & 0 & \frac{6}{5} & -\frac{L}{10} \\ 0 & \frac{L}{10} & -\frac{L^2}{30} & 0 & -\frac{L}{10} & \frac{2L^2}{15} \end{bmatrix} \qquad (33)$$

Where,

± P = tensile or compressive axial force,

L= length of frame element.

It is clearly observed that the geometrical stiffness matrix $\left[k_g\right]$ depends on the axial force P. It expresses the decrease in the flexure stiffness due to the presence of a compressive axial force. The negative sign corresponds to a compressive axial force, and vice versa.

Equation (33) is formed in an incremental form since the use of the geometric matrix to capture the second-order effects requires a stepwise application of the applied loads. This matrix should be updated, at every load step, based on the resulting axial forces in the frame elements.

3- Solution Procedure and Flow Chart

The solution of the nonlinear equilibrium equations is performed using Newton Raphson technique.

3.1. Newton Raphson Method

In this technique, the load is divided into load steps, and then an iterative procedure is adopted to achieve equilibrium as shown in figure (7).

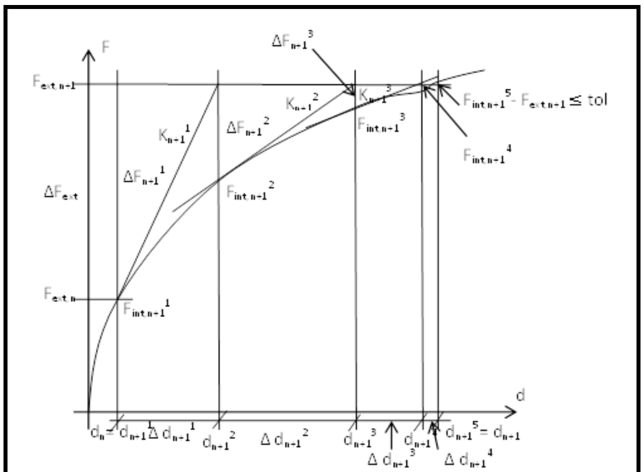

Fig.7. Newton Raphson Method

As shown in figure 7, the applied load is divided into steps and the external load (f_{ext}) incremental by using (Δf_{ext}) and number of step (n_{step}). Frist, the initial stiffness matrix $\left(k_0\right)$ is used to calculate the nodal displacements and the internal forces (f_{int}), the differences between external and internal force (Δf_{int}) is corrected by the Newton Raphson technique to reach the equilibrium position within a satisfy tolerance (tol).

The load continues to increase until the structure collapses by any of failure criteria.

3.2. Failure Criteria

Two type of failure are considered in this research, these are:-

- Crushing failure of concrete: - it is assumed to occur when the maximum compression strain is exceeded.
- Steel yielding: - it is assumed to carry when the ultimate tension strain of the reinforcing steel is exceeded.

3.3. Flow chart

Figure 8 shows the steps of the method as a flow chart

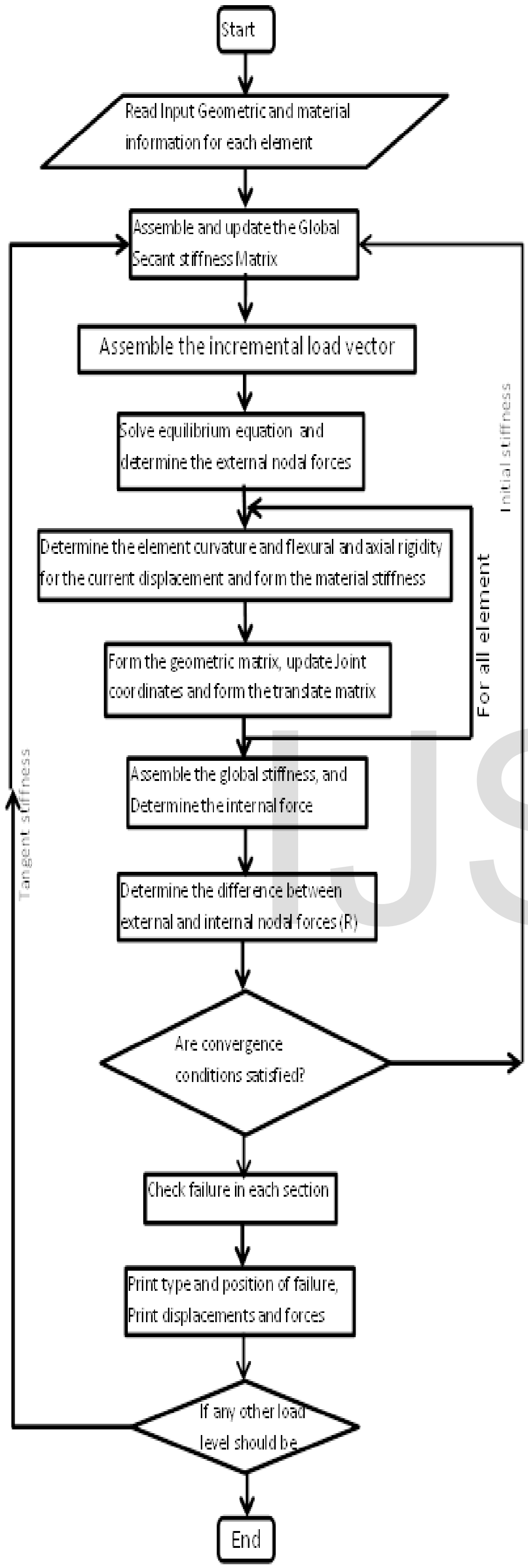

Fig.8. flow chart

4- COMPUTER PROGRAM

POA (pushover analysis) is a computer program developed to carry out the analysis in this research. The program was written in Java Language [9] and is easy to implement, efficient in obtaining the results, simple and robust.

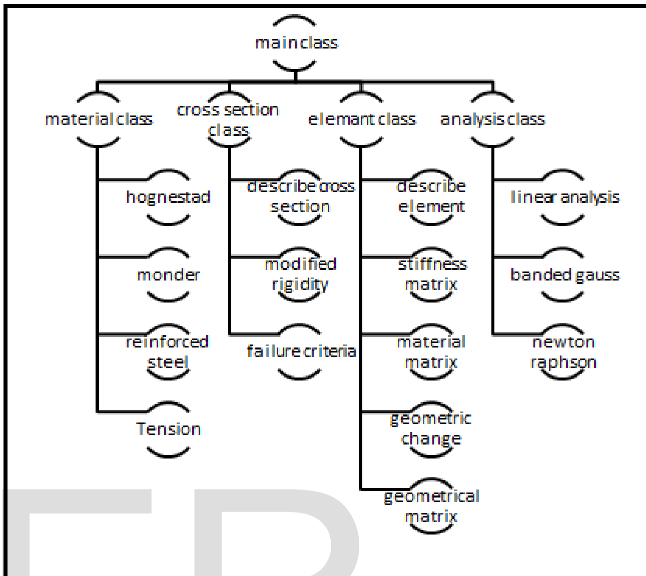

Fig.9. Main Package

As shown in Figure 9 shows that the main package consists of four classes beside the main class and each had been tested separately before putting together. These classes are:-

4.1. Main Class

All these classes are organized to form the main class which analyses the structure. This works as the diver for all classes. It calls four classes.

4.1.1. Material Class:-

This class defines the properties of material and the stress- strain relationship for concrete and steel. It was built with seven material model methods in the program.

- Hognestad method: - for unconfined concrete in compression.

- Monder method: - for confined concrete in compression.

- Reinforced steel: - which describes stress- strain curve for reinforcing steel with three models: - bilinear model, tri-linear model and strain harden model, is used in this research.

- Tension method: - which describes stress- strain curve for concrete in tension.

There were other methods to describe stress- strain curve as needed.

4.1.2. Cross Section Class:-

This class is used to define properties and behavior of cross section. It was built with three methods

- Describe Cross Section method: - which is used to define properties of cross section and the number, diameter and distribution of steel bars.
- Modified Rigidity method:- which is used after calculating strain and curvature in elements and this method determine and draw strain distribution, and from there calculate stress distribution on section hence determine real axial and flexural rigidity to find the modified stiffness element.
- Failure Criteria method: - which is used to check the failure due to crushing of concrete, or yielding of steel, at the end of each load step.

4.1.3. Element Class:-

This class is used to describe the element and form its stiffness matrix. It consists of six methods.

- Describe Element method: - which is used to define element, joints, coordinates, loads, and boundary condition information: - the start and end joint, its coordinates, and assign the recorder cross section, and define boundary condition.
- Initial stiffness matrix method: - which is used to build the initial stiffness matrix for element.
- Geometric change method: - which is used to modify the coordinates of joints and update the length and the slope angle of element hence forms the transformation matrix.
- Material Matrix method: - which is used to incorporate new axial and flexural rigidity form cross section with strain-displacement matrix to form material matrix $\left[k_m\right]$ for element.
- Geometrical Matrix method: - which is used to include the effect of the axial load and form the geometrical matrix $\left[k_g\right]$ for element.
- Tangent Stiffness Matrix method: -which is formed by adding the material and geometrical matrices and transforming to global axes matrix with both and sum them to build tangent stiffness matrix for element.

4.1.4. Analysis Class:-

This class is the most important class in this package, because it defines the type of analysis. It consisted of three methods.

- Gauss Banded method: - which solve banded equilibrium equations using Gauss method.
- Linear Analysis method: - which assemble the global stiffness matrix, apply the boundary conditions, solve the equilibrium equations, determine nodal displacement and end forces.
- Newton Raphson method: - which solve linear analysis, calculate strain-displacement matrix, move to cross section and modify rigidity, and move to element to calculate tangent stiffness matrix, finally solve and determine the nodal displacement and search for convergence.

5- Application:-

Two portal frames tested experimentally were chosen as a check for our formulation and computer program (POA), these are:-

5.1. Single bay one story portal frame:-

This concrete frame model was tested in [10]. Geometry, cross-sections and material properties are presented in Figure 10. It has span of 2.00m and a story height of 1.64 m and the cross section dimension is 12.5*15 cm for the column cross section and 12.5*20 cm for the beam cross section. The other material parameters used in the nonlinear analysis were estimated as described in Section 4. The compressive strength is fc = 13 MPa, and Ec = 20,000 MPa, and the yield strength is fy = 535 MPa for steel reinforcement and the Young's modulus is Es = 200,000 MPa.

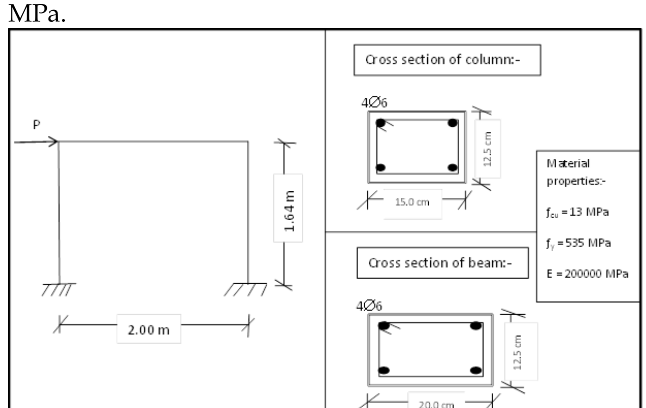

Fig.10. Material & Geometric properties for Frame 1

The Force Control Method was used for nonlinear analysis with increments of 1 KN for the horizontal force of top-left node.

The load (base shear)-displacement curve is shown in Figure 11. These results shown that the models can adequately represent the structural behavior of this frame. The POA prediction for ultimate load was found to be 2.1% lower than the experimental result at 300 mm deflection.

	The ultimate load (KN)	Displacement (mm)
Test	14.71	300
POA	14.4	300
deviation	2.1%	

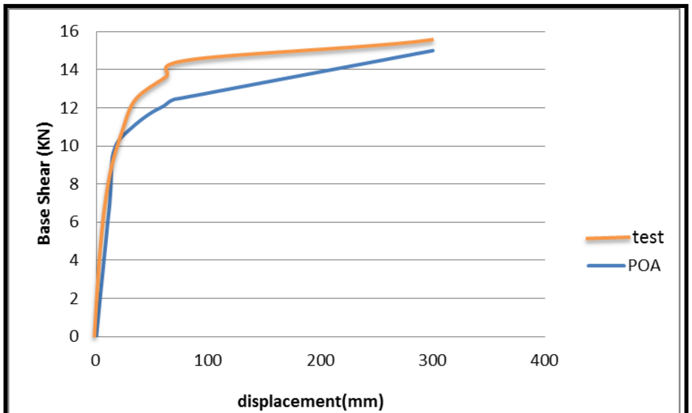

Fig.11.Base shear vs Displacement for Frame1

5.2. Single bay two story frame test:-

Figure 12 shows a model for single-bay two-story frame, which was tested in [11]. The frame was designed with a span of 2.1m and a story height of 1.5 m for each story. The lateral load was applied such that 1/3 and 2/3 of the total load on the lower and top story, respectively.

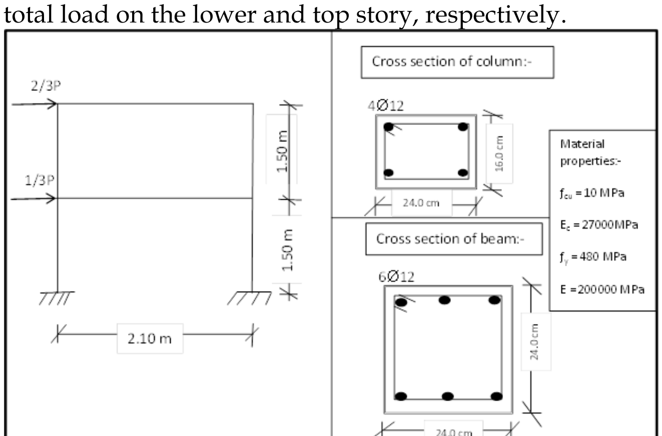

Fig.12. Material & Geometric properties for Frame 2

The concrete had a compressive strength of 10 MPa. The φ12 mm reinforcing steel bars, used as longitudinal reinforcement in all members were found to have a yield strength of 480 MPa, and a modulus of elasticity of 200,000 MPa. The lateral load was applied monotonically until the ultimate capacity of the frame was achieved.

Figure 13 shows comparisons between the predicted load (base shear) deflection responses using the POA program and the experimental results, [11]. It can be clearly observed that the analytical and experimental results are in a good agreement. The POA prediction for ultimate load was found to be 2.8% higher than the experimental result at 40 mm deflection.

	The ultimate load (KN)	Displacement (mm)
Test	34.51	40
POA	35.48	40
deviation	2.8%	

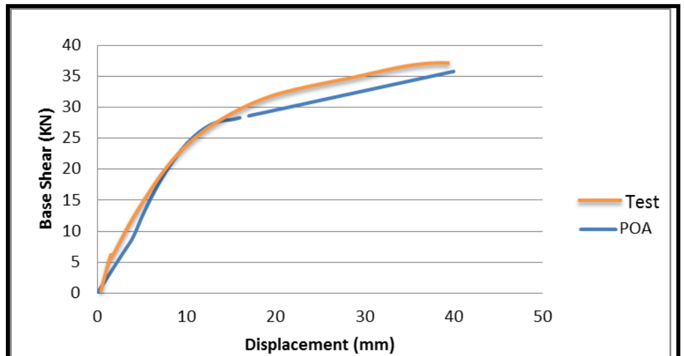

Fig.13. Base Shear vs. Displacement for Frame 2

6- Conclusions:-

In this research, a general methodology and a computer program in Java Language for the nonlinear analysis of framed structure is developed. It was shown that the method is simple, efficient, accurate and robust. The material nonlinearity includes diffident material models for concrete and steel and forms the material stiffness matrix and includes diffident failure criteria. For the geometrical nonlinearity, it updates the coordinates and takes the effect of axial loads. The solution of nonlinear analysis performed using the incremental - iterative Newton Raphson method. The method is applied to the pushover analysis of both one story and two story portal framed structures with maximum load deviation less than 2.8% of the experimental results.

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