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Experimental Investigation of Progressive Collapse of Steel Frames

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ABSTRACT

This paper reports two new tests conducted to augment available data highlighting the structural performance of multistory steel frames under progressive collapse. The investigated steel frames had different geometries, different boundary conditions, different collapse mechanisms, different damping ratios and different connections. Overall, the paper addresses how multistory frames would behave when subjected to local damage or loss of a main structural carrying element. The obtained results can form a data base for nonlinear finite element models. The deformations of the investigated steel frames and failure modes under progressive collapse were predicted from the finite element analysis, with detailed discussions presented.

Keywords: Experimental Investigations; Finite Element Model; Progressive Collapse; Steel Frames

1. Introduction

Existing multistory steel frames may be severely damaged owing to collapse of its main structural elements such as columns, beams, structural walls or bracing members. The collapse may occur as a result of explosions, blast or terrorist attacks. The damage or loss of a main structural carrying element leads to progressive failure of a significant part of the building or the whole building. As a result of the damage or loss of a main structural carrying element, the primary load-resisting system leads to redistribution of forces to the adjoining members. Due to the redistribution of forces, the stresses within the remaining structural elements such as other columns and beams would be changed and if the stresses exceed the yield stresses of the element it fails. This failure can continue from an element to another and eventually the building collapses. This failure is defined as progressive collapse of the multistory buildings. The initial collapse of the structural elements that initiates the overall collapse is sudden and dynamic involving significant geometric and material nonlinearities.

Steel frames are commonly used as efficient main structural supporting systems in multistory buildings. However, up to date, the detailed behavior of steel frames under progressive collapse is rarely found and there is a lack of information regarding the design of steel frames to overcome progressive collapse leading to the current investigation. Full-scale tests investigating progressive collapse of steel frames are quite costly and time-consuming.

Also, numerous simplified analysis methods are found to evaluate the potential of progressive collapse such as linear static, nonlinear static, linear dynamic and nonlinear dynamic analyses. Marjanishvili (2004) [1] discussed the advantages, disadvantages, limit and performance of each analysis method. It showed that the analysis methods varied from a simplest but very conservative linear analysis method to a sophisticated but most precise and realistic nonlinear dynamic analysis method. Based on Marjanishvili (2004) [1], the analysis results of the numerical investigations showed that the nonlinear dynamic analysis is easy to be conducted and provides the most precise solution. Also, it showed that the linear static analysis resulted in unconservative solution following the failure criteria of the linear static analysis that was specified by GSA guidelines (2003) [2]. Conducting a complex 3-D nonlinear dynamic analysis is time-consuming but provides accurate results.

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Nonlinear 3-D finite element models were developed using nonlinear software to conduct the analyses of steel frames under progressive collapse. For progressive collapse analysis, a nonlinear static analysis method employs a stepwise increment of amplified vertical loads which can be referred to as a vertical pushover analysis. The force redistribution within the steel frames under progressive collapse was investigated in this study and the failure modes were predicted. Progressive collapse is of particular concern since it may be disproportionate, *i.e.*, the collapse is out of proportion to initial local failure. After the progressive and disproportionate collapse of the Ronan Point apartment tower [3], prevention of progressive collapse became one of the main concerns of structural engineers, and code-writing bodies and governmental user agencies attempted to develop design guidelines and criteria that would reduce or eliminate the susceptibility of buildings to this form of failure. These efforts tended to focus on improving redundancy and alternate load paths, to ensure that loss of any single component would not lead to a general collapse. Improved local resistance for critical components and improved continuity and interconnection throughout the building (which can improve both redundancy and local resistance) can be more effective than improved redundancy in many instances. Through an appropriate combination of improved redundancy, local resistance and interconnection, it would be possible to greatly reduce the susceptibility of buildings to disproportionate collapse [2].

Astaneh (2002) [4] investigated the strength of a typical steel building and floor system to resist progressive collapse in the event of removal of a column. They tested a specimen of size 60 ft by 20 ft one-story steel building with steel deck and concrete slab floor and wide flange beams and columns. The connections were either standard shear tab or bolted seat angle under bottom flange and a bolted single angle on one side of the web. It was observed that after removing the middle perimeter column, the catenary action of the steel deck and girders was able to redistribute the load of removed column to other columns. The floor was able to resist the design dead load and live load without collapse. Damage to the system was primarily in the form of cracking of floor slab, tension yielding of the steel corrugated deck in the vicinity of collapsed column, bolt failure in the seat connections of the collapsed column and yielding of the web of the girders acting in a catenary configuration.

Astaneh (2003) [5] carried out an experimental investigation of the viability of steel cable-based systems to prevent progressive collapse of buildings. The tests were conducted on a full-scale specimen of a one-story building. One side of the floor of the specimen had steel cables placed within the floor representing new construction and the other side had cables placed outside as a measure of retrofit of the existing building. The author claimed that the test results showed that the system could economically and efficiently prevent progressive collapse of the floor in the event of removing one of the exterior columns.

Gravity load collapse of a reinforced concrete frame was studied by Moehle and Elwood [6,7]. Their studies found that residual axial capacity could prevent collapse of a building although shear failure in a concrete column had occurred. A formula using a shear-friction model was suggested to simulate additional axial load capacity after shear capacity was exhausted [6,7]. The study [6] also considered residual capacity of adjacent elements in analyses after a component fails and leads to redistribution of the applied loads.

Most flat plate buildings are prone to progressive collapse (Hawkins (1979)) [8]. Punching shear failure in a flat plate building was often observed even before yielding of the bottom reinforcement of slabs occurred. Mitchell and Cook (1984) [9] found that punching shear failure at exterior columns had a low possibility of leading to progressive collapse. However, unless continuous bottom reinforcement through a column or good anchorage was provided, tension membrane by slabs was not effective and could lead to catastrophic failure. Proper detailing of slab reinforcement at a column support enabled a damaged slab to hang from its support. Therefore, well detailed flat slab buildings were capable of resisting additional loads even after punching failure at a support region occurred.

A study by Malvar (2005) [10] reported behavior under blast load and suggested appropriate retrofit schemes. Both specific local resistance and alternate load paths were considered to rehabilitate the building. Although the suggested retrofit schemes were not necessarily applicable to all concrete buildings, the fundamental retrofit concepts to prevent progressive collapse were defined. The study recommended that exterior frames can be rehabilitated by providing specific local resistance using steel jacketing or wrapping with fiber material. Interior frames can be supported by developing load paths using adjacent components.

There is a lack of full-scale tests reported in the literature on progressive collapse of steel frames because they are quite expensive and time-consuming. Therefore, numerous small-scale tests were reported in the literature on progressive collapse. The small-scale tests can be conducted in labs and the result is considerable savings in time and money. Efforts to develop comprehensive and progressive collapse-resistant specifications have been hindered by a lack of both experimental and analytical information about progressive collapse, leading to the current investigation. On the experimental front, the rate of loading and the scale of the problem, *i.e.*, which involve a full system, have made testing difficult. On the other hand, numerical simulation is a challenging task because the collapse process involves modeling component and system behavior across several length scales. The main objective of this study is to conduct two new small-scale tests to augment available data on progressive collapse of steel frames and to use the results in developing nonlinear 3-D finite element models.

2. Model Description

The model is one tenth (1/10) scale, the frame consists of two bays of 0.5 m in two directions with 0.4 m for the height of the story. The slab is welded to the beam and the beams-to-connections are welded also to make a rigid connection. A superimposed load of 250 kg/cm^2 is applied for each floor. The small scale beams and columns are selected from commercial shapes. A hollow box section of dimensions 200 × 200 × 15 mm is selected for both columns and beams and for bracing system an equal angle 300 × 300 × 30 mm is selected. The steel frame is assumed to be built on fixed conditions such that no soil interaction or differential settlements need to be considered. The steel frames are designed for gravity loads only according to the Egyptian Code. Testing a small scale structure, hence, needs to pay a special care in planning stage to guarantee obtaining an accurate model. The steel frame is tested in two cases. The first one, edge column was removed and a dynamic load was applied on the frame as simulation of explosions, accidental overload, etc. The second one, internal column in the frame was removed followed by static collapse test under controlled displacement with the help of a hydraulic jack, see **Figure 1** for viewing the model after adding super imposed loads and is connected with instrumentations. It was considered that the columns were removed before the test. Observations of the building response following columns removal are presented.

Figure 1. The model after adding super imposed loads.

3. Test Instrumentation Used

The instrumentations used in the test include a display unit (laptop computer), data transmissions unit, transmission cables, and concentrated load applied on the removed column by a hydraulic jack. Five specifications of strain gauges are placed in different positions of the frame so that the concentrated load and the dynamic response, including strain and displacement of the frame, can be measured. The strain gauges attached to the columns are universal general purpose strain gauges with a resistance of 120 ± 0.3% Ohms, and have a strain range of ± 3%. They measure the strain in the vertical direction caused by the compressive and tensile forces. A set procedure is used to install the strain gauges on each column. The displacement of the frame in the vertical direction at the end of the failed column is measured using dial gauge.

Case1: Edge Column Removed

Since no vibration can occur at all unless dynamic loads are applied to the frame, a concentrated load by a hydraulic jack is used to vibrate the frame. The model is subjected to a concentrated load at the edge column removed = 20 Kn for constant time = 7 sec., then removed after that. Strain gauges also are fixed at the other columns, see **Figure 2**.

The output of maximum lateral deflection will be shown in **Figure 3**, it was observed that the maximum lateral deflection for the edge column in the first floor was 35 mm at 2 sec., immediately after the concentrated load is applied. After 2 sec., the maximum lateral deflection has fixed value at the end of the test (at the 7 sec.) 34 mm. This meant that the maximum deflection for the column will be in the first time of the applied load then fixed for a few sec. For the comparison between the experimental test and with the data get from sap 2000 analysis, the maximum lateral deflection for the edge column removed after applied load in the experimental test is higher than in the analysis by 16.6%, that is due to

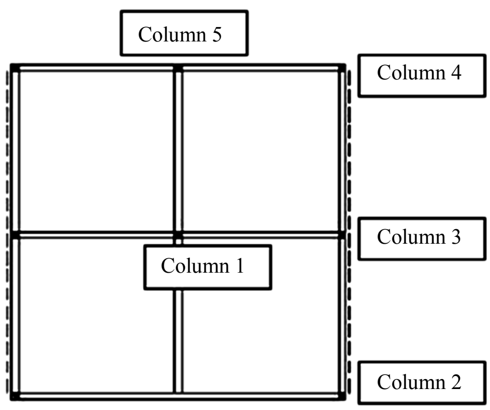

Figure 2. The fixed places of strain gauges for edge column removed.

the fixation points of the steel frame in the analysis is more specific so, get small value than the experimental test.

The strain for the different columns after applied load in the experimental test is shown in **Figure 4**. It was observed that the column No.2 near the edge of column removed having high value of strain than other columns this meant that the deformation of the column per unit of the original column is higher than the column No.3 and No.1 which having tension values. The column No.3 and No.1 almost having the same values of column No.4 and No.5 but having compression values, which meant that the maximum deformation and the redistribution of forces would be the maximum for the columns around the area of the removed column.

Case 2: Internal Column Removed

Load redistribution of the frame occurred after the bottom center column is removed. The measured and calculated results showed that the frame experienced only elastic deformations after the loss of the bottom column. Then a hydraulic jack is used to apply monotonically increasing static load to check the ultimate load, failure mechanism and the collapse-resistant behavior of the model frame, the static test setup. The displacement is controlled during the test, **Figure 5**. The model is subjected to a concentrated load, increasingly with time for 7 sec., until the column is damaged. A dial gauge is fixed at the column which is removed to measure the deflection. And strain gauges also were fixed at the other columns as in **Figure 6**. The steel frame columns and beams damaged and the resisting load of the frame decreased rapidly.

Figure 7 shows the maximum lateral deflection gets from sap 2000 analysis case is higher than the one get from experimental test by 27%, the difference between the analysis and the experimental due the rate of loading and the scale of the problem, *i.e*. that it involves a full system, has made testing difficult. From **Figure 7**, the maximum lateral deflection gets at the ultimate load capacity 50 KN for the case of loading where the load is applied increasingly with time the maximum deflection would be measured and get around 35 - 36 mm. The strain for the different columns after applied load is shown in **Figure 8**. It was observed that the column No.1 near the damaged column having higher value of strain

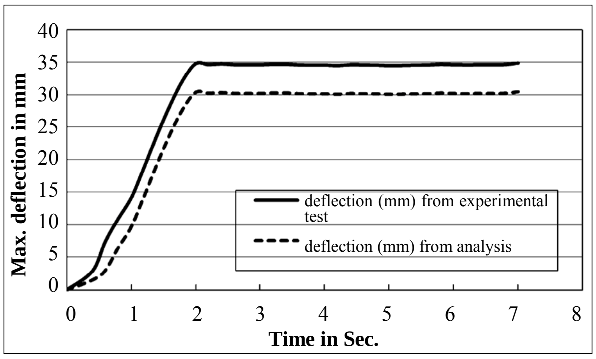

Figure 3. The maximum lateral deflection for edge column removed.

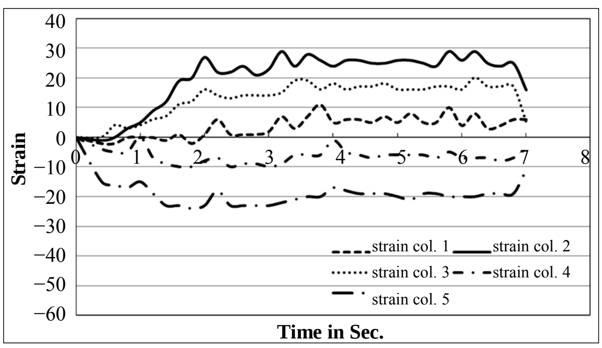

Figure 4. The strain for different columns for edge column removed.

Figure 5. The model of internal column removed.

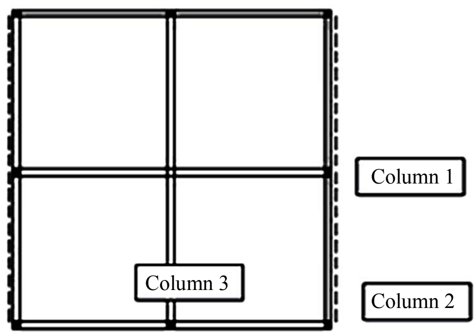

Figure 6. The fixed places of strain gauges for internal column removed.

than other columns that meant that the deformation of the column is higher than the other columns and having tension value and for column No. 3 has tension value and for column No. 2 having the compression value. The failure mode for internal column removed presented in **Figure 9**.

4. Nonlinear Analysis

A nonlinear structural problem is one in which the building's stiffness changes as it deforms. All physical buildings exhibit nonlinear behavior. Linear analysis is a convenient approximation that is often adequate for design purposes. There are three sources of nonlinearity in structural mechanics simulations: material nonlinearity, geometric nonlinearity, boundary nonlinearity. Most metals have a fairly linear stress/strain relationship at low strain values; but at higher strains the material yields, at which point the response becomes nonlinear and irreversible this source is defined as material nonlinearity. For geometric nonlinearity is related to changes in the geometry of the building during the analysis. Geometric nonlinearity occurs whenever the magnitude of the displacements affects the response of the building. This may be caused by large deflections or rotations. Finally boundary nonlinearity can occurs if the boundary conditions change during the analysis. For linear analysis, the two primary source are the stress/strain relationship & the deformation behavior. The stress is assumed to be directly proportional to strain and the structure deformations are proportional to the loads.

In this paper, the linear analysis material and geometric are carried out.

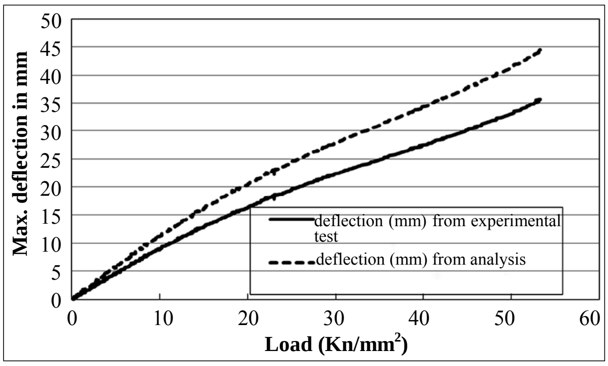

Figure 7. The maximum lateral deflection for internal column removed.

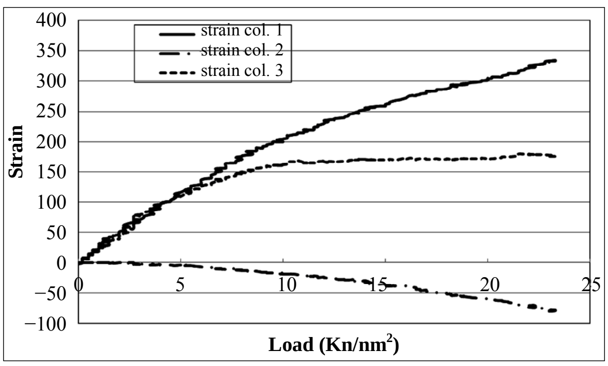

Figure 8. The strain for different columns for internal column removed.

Figure 9. The failure mode for internal column removed.

5. Finite Element Model

A finite element model of the analyzed frame has been created by sap 2000; the beams and columns element defined as frame section and were divided into number of subdivided elements as shown in **Figure 10**. The frame has two equal spans in two directions. In this report, two cases are considered: a removal of an edge column, and internal column. The slabs are also divided as in **Figure 10** to subdivided slabs The self-weight for two cases is considered and a superimposed load of 250 kg/cm^2 is

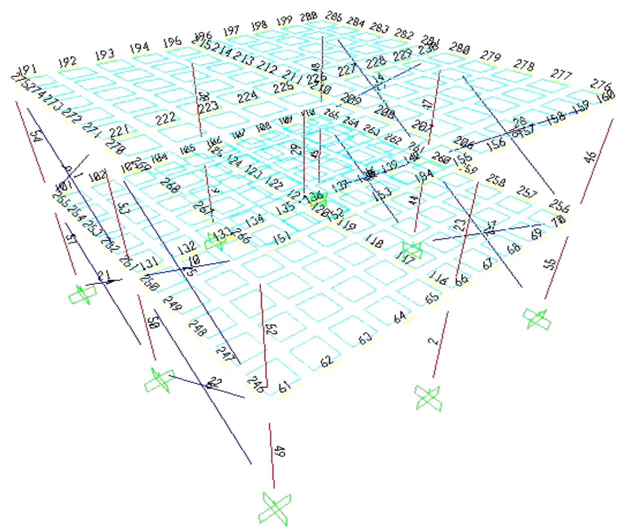

Figure 10. Finite element model of the analyzed frame in SAP 2000—element numbers.

applied for each floor. The superimposed load is modeled as a uniformly distributed linear load applied to the slab. The columns would be fixed end conditions. The linear static analysis is carried out for two cases.

6. Conclusion

This study has reported two tests conducted on steel frames under progressive collapse. The tests have augmented previous investigations on this field and provided detailed information regarding the behavior of the frames when subjected to a significant loss of main structural elements such as column. The test results were comprised of failure modes, time-displacement relationship and stresses in the adjacent elements. The tests results were used to verify nonlinear finite element models developed in this study. Also, existing information previously analyzed by other researchers has been used to verify the finite element models. The comparison between the experimental results and the existing results in the literature with finite element results obtained in this study showed that the developed model simulates the behavior of steel frames well. It showed that the maximum lateral deflection measured for the edge-column-removed case was higher than that when predicted numerically because the fixation points of the steel frame were not fully rigid. It also showed that the column adjacent to the removed column underwent higher strains than other columns, which implied the redistribution of forces from the removed column to the nearest columns.

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