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Research and Practice on Progressive Collapse and Building Strength in 2019 and 2020

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Abstract

A chain reaction or collapse-spreading in which, under certain causes, local damage occurs in a relatively small area of the structure and under certain conditions this local damage spreads to other parts of the structure and ultimately leads to overall collapse of the structure is called progressive collapse. The main feature of this phenomenon is that the final collapse is not commensurate with the initial collapse; severe incidents such as terrorist attacks, vehicle collisions and explosions, etc. often damage the building and collapse in one or more vertical bearing parts leads to a serious threat. The following article provides an overview of the main research and developments in 2019 and 2020 regarding progressive collapse such as progressive collapse assessment, experimental testing, and numerical modeling.

Keywords: progressive collapse; RC frames, beam connection; column loss scenario; steel frames; flange-plate connection

1. Introduction

A chain reaction in which, under specific causes, local damage occurs in a relatively small area of the structure and under certain conditions this local damage spreads to other parts of the structure and ultimately leads to total structural collapse is defined as progressive collapse.

Public and private buildings may be exposed to events such as hurricanes, gas explosions, tsunamis, earthquakes, plane crashes, vehicle collisions, explosions, and terrorist attacks. Such events usually damage the building and can lead to the complete collapse of the building. Progressive collapse in structures occurs mostly for the following two reasons:

A) Design and implementation errors.

B) Abnormal loads that cause local fracture or instability of the building.

After the demolition of the famous building of Ronan in 1968, some countries, such as Britain and Canada, set standards to prevent progressive collapse and breakdown. In 1976, the British Building Code required buildings to be designed to withstand non-intermittent damage by integrating structural members, and providing sufficient strength to withstand abnormal loads.

Progressive collapse design methods are as follows:

A) Accident control method

Since it does not increase the resistance of the structure against progressive collapse and on the other hand depends on factors that are beyond the control of the designer, so it is less used.

B) Indirect design method

This method is designed to create a general connection without considering abnormal loads and involves creating connections at the node points and increasing the ductility and indeterminacy of the system.

C) Direct design method

In this method, the occurrence of progressive collapse is specifically considered; this operation is done by designing it for abnormal loads or by

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assuming a specific local collapse, which is of two methods: specific local resistance and the alternative path method.

In the local resistance method, critical elements are designed to withstand abnormal loads. These elements must have sufficient strength and stiffness to withstand abnormal loads, and increasing the load coefficients can be one of the easiest methods. In alternative load path method, it is assumed that the structural member is disjointed separately and removed from the system before analysis. The loads carried by this member are transmitted to adjacent members. This procedure is repeated until the structure collapses or no additional collapse occurs.

In the case of framed buildings, through providing alternative load paths the risk of progressive collapse can be minimized by the following five robust mechanisms:

A) Bending of the beam where the column has failed.

B) Vierendeel behavior of the frame more than the failed column (Figure a1)

C) The buckling and arching effect of the beams in which the column has failed.

D) The use of large rotations and displacements and the membrane behavior of beams and slabs (Figure 1(b)).

E) The contribution of non-structural elements such as exterior walls and partitions (Figure 1(C))

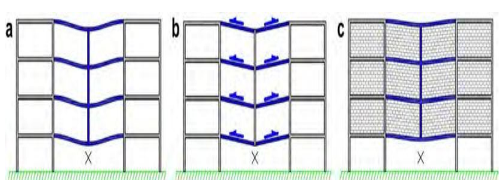

Figure 1: Alternative load paths: (a) Vierendeel action; (b) catenary action; and (c) contribution of non-structural elements.

2. Principles of the study and literature

2.1. Evaluating the potential of progressive collapse in resistant reinforced concrete fram

The purpose of this evaluation is to simulat behavior of special bending frame assemblie reinforced concrete (RC) under the column rer scenario through nonlinear finite element modeling. In addition to modeling the behavi nonlinear rate-dependent materials, bond-slip e in the concrete-steel joint are considered.

The contribution of compressive arc and l measures in the progressive collapse resistance c concrete structures has recently been studied experimentally and numerically by some researchers.

(Yu) and Tan examined the contribution of seismic details to the progressive collapse resistance of RC frames and found that CAA and rotational action increased the structural resistance to progressive collapse, significantly.

The aim of this evaluation is to develop finite element (FE) models to predict the behavior of SMRF RC assemblies under a column loss scenario.

2.1.1. Experimental program

To validate the FE model, a one-tier model with two SMRF RC scaled openings under a column removal scenario was used in this study. The RC columns rest on steel rods and are firmly attached to the laboratory floor. The building column removal scenario was simulated by removing the test column abutment and applying a sudden load.

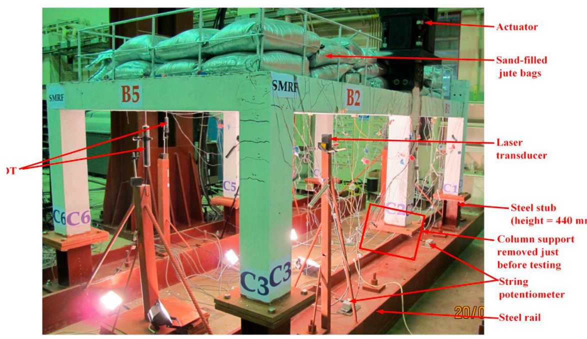

Figure 2: Instrumented testing specimen SMRF ready for.

2.1.2. Loading and border conditions

As shown in Figure (3), the displacement and rotation of the column bases are controlled in all three Cartesian directions. The plane of symmetry is modeled by inhibiting x-displacement and x and Z rotations in the plane.

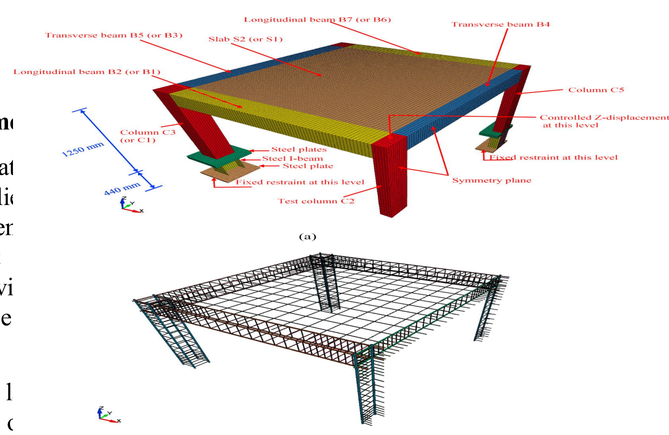

Figure 3: FE model for specimen SMRF: (a) FE mesh for one-half of specimen; (b) FE model of steel reinforcement for one-half of specimen

2.1.3. FE model setting

The modeling process and numerical analysis method have been scaled and validated using the model test results discussed here.

The bond-slip effect between the main bars and concrete beams and slab panels was included in a numerical model and the exponential coefficient of damage curve (hdmg) ranged from 0.05 to 0.15 in one step. Therefore, four FE models have been prepared for the SMRF test sample. The first case is for bond-slip effects with hdmg = 0.05. The second and third modes are 0.01 and 0.15 for the final coefficient of the curve (hdmg), respectively. Assuming a complete connection between all reinforcing bars and concrete, the fourth case is for analysis.

Figure 4 shows a comparison of load-location changes obtained from analysis and experiments. It can be seen that the assumption of a complete bond increases the stiffness and peak load, significantly.

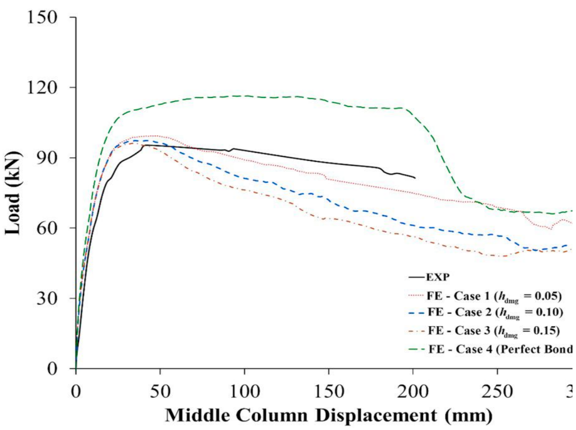

Figure 4: Comparison of experimental and FE load-displaceme envelopes for SMRF specimen

2.2. New structural details to improve the seismic performance and progressive collapse of RC frames

Conventional design methods usually focus on the design requirements of a particular hazard but ignore the interaction between different designs. For example, the design of progressive collapse of an RC frame often leads to increased reinforcement and flexural strength of the beams. As a result, the principle of seismic design of a weak beam of a strong column may be violated, which may lead to undesirable collapse states and weakening of seismic performance. To prevent these adverse effects of progressive collapse design on the seismic resistance of reinforced concrete frames, new structural details have been proposed in this study. The proposed detail method aims to simultaneously improve the seismic performance and progressive collapse of an RC frame against the progressive collapse without any additional reinforcement. A six-story RC frame is used as the main sample of the building for inspection. Both cyclical and progressive collapse sections have been performed to validate the performance of the proposed structural details. Based on the experimental results, the finite element (FE) models of the RC frame are created with different reinforcement designs. Seismic resistances and progressive collapse of different models have been compared based on incremental dynamic analysis (IDA) and nonlinear dynamic alternating path (AP) methods, respectively. The results show that the proposed structural details can effectively resolve the conflict between progressive and seismic collapse designs.

2.2.1. Experimental design

RC frame prototype design

The elevation and plan views of the prototype RC frame are shown in Figure (5).

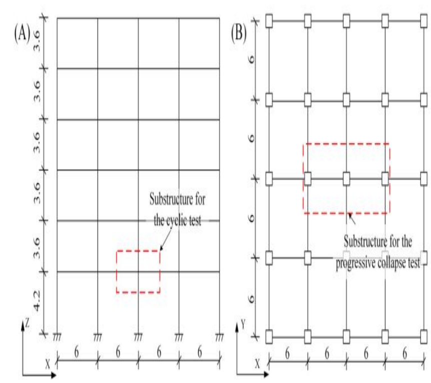

Figure 5: Elevation view (A) and plan view (B) of the six-story RC frame (unit: m) [Color figure can be viewed at *www.wileyonlinelibrary.com*]

This structure is first designed according to Chinese design regulations to create a normal RC frame, for example RC6. According to the numerical results, progressive collapse in the prototype building occurred when each column on the first to fifth floors was removed. Therefore RC6 can not meet the requirements of the progressive collapse design code. Therefore, RC6 is redesigned according to the binding force method provided in the DOD instruction and the new structure is named RD1.

A comparison between the conventional reinforcement design and the proposed structural details is shown in Figure 6.

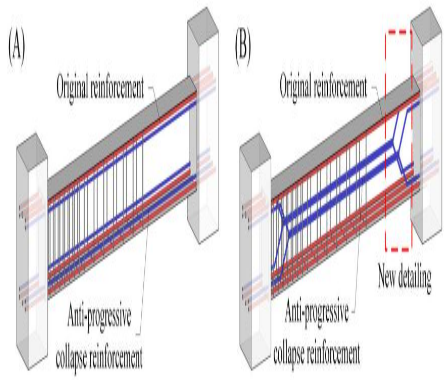

Figure 6: (A) Frame beam after progressive collapse design and (B) frame beam with new detailing [Color figure can be viewed at wileyonlinelibrary.com]

2.2.2. Conventional progressive collapse design

Typically, after the progressive collapse design is performed, the newly added reinforcement is usually placed at the top and bottom of the frame beam to prevent collapse, as shown in Figure 6a. According to the presented experimental results, the flexural strength under such a reinforcement design will increase by approximately 30% for RC6. Hence, this arrangement will undoubtedly increase the bending capacity of the beam. Therefore, it leads to the potential risk of weak column fracture under seismic measures.

To solve the above problem, the main challenge is to maintain the progressive collapse resistance of the beam without too much increase in its flexural strength. Therefore, the proposed detail technique rearranges the newly added reinforcement in the middle of the beam height away from the column face. In the joint, the reinforcing bars added are bent up and down at an angle of 〖45〗^° to the top and bottom of the beam and connected to the main reinforcement to pass through the joint area, as shown in Figure 6b. It is noteworthy that around the upward / downward bending reinforcement, the gradually changing height of the reinforcement changes the bending capacity of the beam along it. Under a seismic condition like an earthquake, the plastic hinge area can be relocated to sections that have reduced the flexural capacity by properly designing the cross-sectional strength in the upward / downward bending area of the reinforcement. The details of the cross section of the frame beam are also presented in Figure (7).

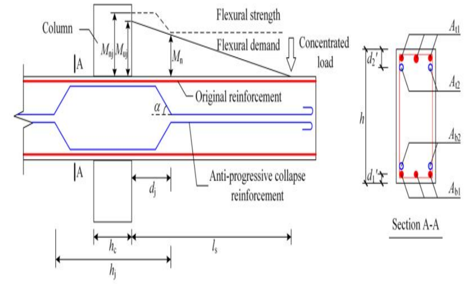

Figure 7: Design requirements for the detailing [Color figure can be viewed at *www.wileyonlinelibrary.com*]

2.2.3. Experimental commissioning of samples

To compare the seismic and progressive collapse performance of conventional reinforced concrete (RC) frame, reinforced concrete frame after progressive collapse design (RD) and reinforced concrete frame with the proposed new structural details (ND) of two substructures enclosed by the red fold line in Figures 5a and b are extracted from the building for cyclic and progressive collapse tests, respectively.

2.2.4. Seismic cyclic test

Laboratory setup for seismic cyclic tests is shown in Figure 8a.

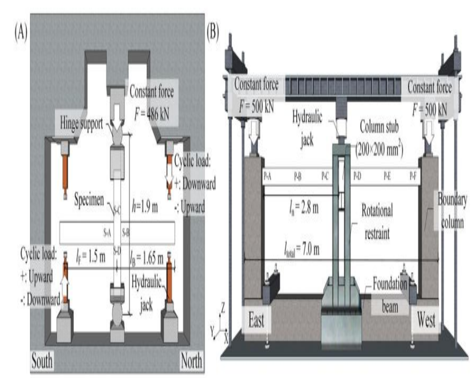

Figure 8: (A) setup for the seismic cyclic test and (B) setup for the progressive collapse test [Color figure can be viewed at *www.wileyonlinelibrary.com*]

2.2.5. Progressive collapse tests

Two substructures (of opening) on the ground floor of the RC frame, covered by a red fold line rectangle in Figure 5b, have been used for progressive collapse tests. Two fixed vertical forces of 500 KN are applied to the top of the boundary columns. The experiments were performed according to the AP method as specified in the progressive collapse design guidelines. A uniformly concentrated load is applied to the removed column to simulate the progressive collapse of the substructure under an intermediate column removal scenario.

Reinforcement details of P-RD1, P-RC6 and P-ND specimens are shown in Figure (9).

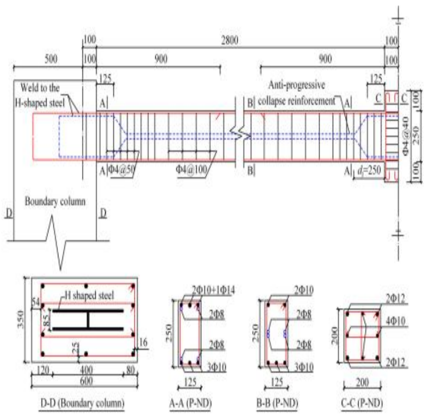

Figure 9: Reinforcement details of specimens P-ND (unit: mm) [Color figure can be viewed at *www.wileyonlinelibrary.com*]

Longitudinal reinforcements in the boundary columns are embedded in H-shaped steel, thus providing sufficient moment strength. Moment-rotation relationships of three seismic cyclic test specimens are shown in Figure 10a-c. Experimental observations show that the flexural strength of RD1 specimens is improved by almost 30%. By adopting the proposed structural details, the number of diametric cracks in the joint area of the ND specimen has been significantly reduced compared to the RD1 specimen for the same amount of end-displacement of the beam. Therefore, it shows that the damage in the joint area can be significantly reduced in length.

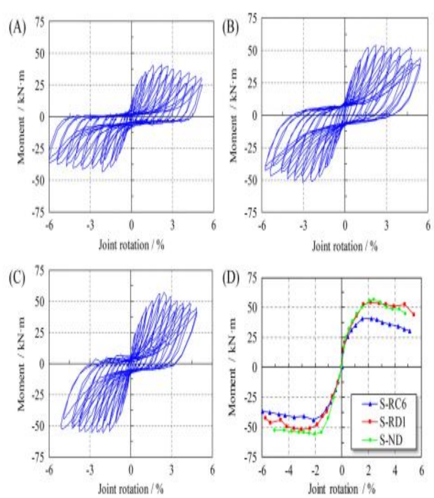

Figure 10: Moment-rotation relationships of (A) S-RC6, (B) S-RD1, and S-ND, and (D) comparison of the backbone curves [Color figure can be viewed at *www.wileyonlinelibrary.com*]

2.3. Studying progressive collapse in dual systems

In this study, the progressive collapse of a steel structure with a dual system has been studied by considering the column removal scenario. According to this goal, two lateral load-resistant systems have been modeled in ABAQUS software through removing different columns and the performances of these systems against progressive collapse have been compared with each other. The results showed that the most critical columns are those located near the outer frame of the structure.

Two numerical examples were tested and analyzed, for both which, two types of column removal algorithms were implemented: the new mark method and the Hilbert-Hughes-Taylor method. The results show that when the distances are not small enough, the response is less considered until the safety results are obtained. But when the time intervals tend to be close to zero, both algorithms get almost the same results. According to overall results of the analyses for each solution algorithm, the time interval size should be significantly less than the column removal time.

2.3.1. Numerical models

Nonlinear dynamic analysis is used as the most accurate method. The models used in this study are two-story frames with a floor height of 3 meters.

Figure 11 shows the design and position of the lateral load-bearing system. A total of six numerical models have been designed and analyzed, in which the structural system for the first three models is a double steel frame with a steel shear wall and the next three models are a double steel frame with a concentrically bracing system.

According to the design shown in Figure 11, the columns in positions B-2, B-3 and A-3 are removed as a progressive collapse scenario in each model.

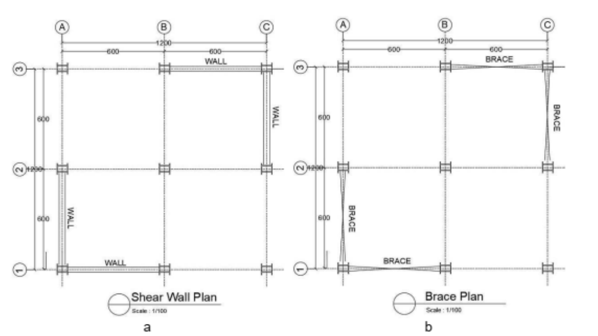

Figure 11: Plan view of models; (a) Steel frame with steel shear wall; (b) Steel frame with a concentrically bracing system.

Figure 12 shows the maximum displacement each model against time.

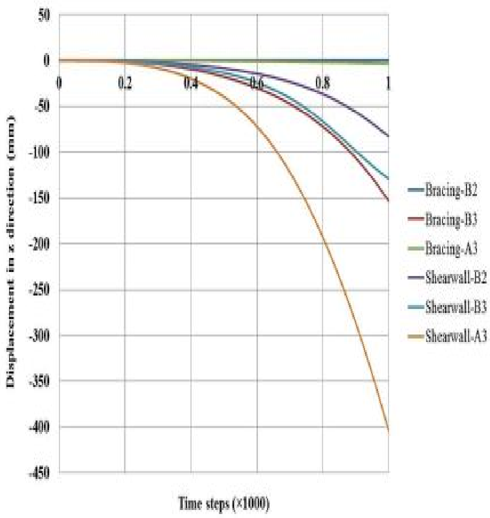

Figure 12: Maximum displacement of each model during column collapse.

2.4. Progressive collapse behavior of steel frame substructures with different beam-column connections

This study presents experimental efforts in steel frame substructure with bolted flange plate (BFP) joints subject to progressive collapse. It has been shown that the connection with welded flange plates can lead to greater flexural strength than bolt and nut connections.

Attempts were made to investigate the effect of compressive arc action (CAA) and catenary action (CA) on structural strength with the aim of developing an analytical, experimental or numerical model to predict the progressive collapse strength of each specimen.

Previous studies have shown that for steel frames, the response to the progressive collapse of a structure depends mainly on the ability to connect between the main elements of the structure.

2.4.1. Overview of laboratory tests

In this experiment, two 2-story steel frame substructures with the same $\frac{1}{3}$ scale are prepared, which represent part of a prototype of a real office building (Figure 13). It can be seen that the lower middle column of the sample has been removed to adapt to the onset of progressive collapse.

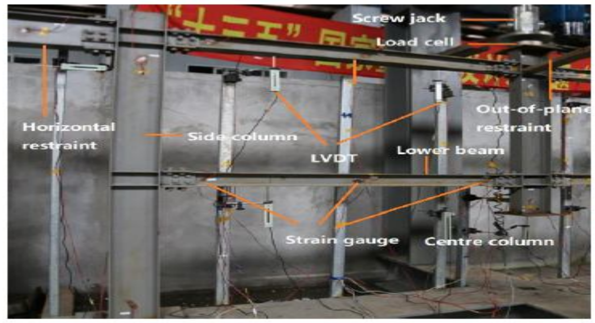

Figure 13

The configuration of the beam-column connections is shown in Figure (14). In the connection of the tabs and the shear flange, they are connected to the column flange by corner welding and are screwed to the beam through M16 bolts.

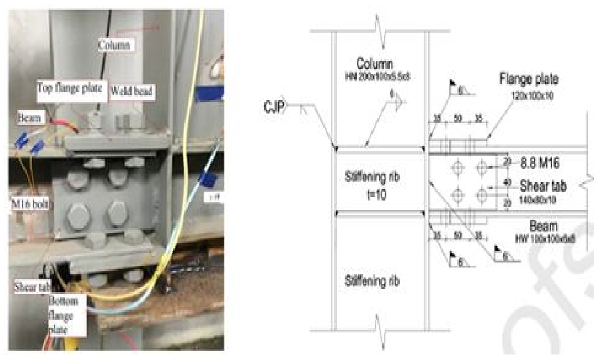

Figure 14

2.4.2. Summary of experimental results

The extension of the applied load against the displacement of the intermediate connection of the MJD specimen is shown in Figure 16. In Figure 16, FP shows the vertical capacity when the moment of complete plastic yield of the beam is achieved and the rotation is calculated by MJD along the aperture of the beam. It can be seen that the curves are ascending linearly in the early stages. With increasing beam yield load (point A- A') cracks were observed in the upper flange plate of the lower western beam (Figure a-15). The plastic joint forms in the beam, with more cracks in the bottom plate indicating that the compressive arc action (CAA) stops and CA begins (point B- B').

Under the CA stage, the existing cracks continue to expand as the load increases (Figure 15-b). When the final load reaches point C- C', a sever collapse occurs suddenly in the weld on the flange of the upper beam of the lateral connection (Figure 15-c). Then, with the complete detachment of the upper beam from the side column, the frame resistance is drastically reduced and the test is terminated to avoid any possible danger. (Point D- D').

(a) point A (b) point B (c) point C

Figure 15

The resistance of the sample is shown in Figure (16), which is based on the strain gauges installed on it. It can be seen that the axial force is negligible in the early stages and the strength of the frame is mainly provided by the bending action of the beam.

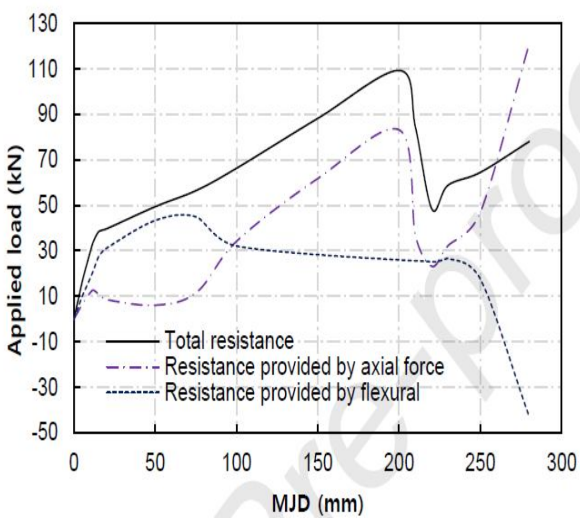

Figure 16: Resistance development diagram for the sample

3.4.3. Numerical simulation

Since conventional solid modeling strategies can be time consuming, a number of simplified simulation methods have been used to perform progressive collapse analysis. After comparing the existing modeling methods, the structural action of the load-bearing samples was predicted using ABAQUS software with models based on the shell elements. In this model, steel components and welds are modeled by three-dimensional shell elements, respectively. An overview of the frame model is shown in Figure (17).

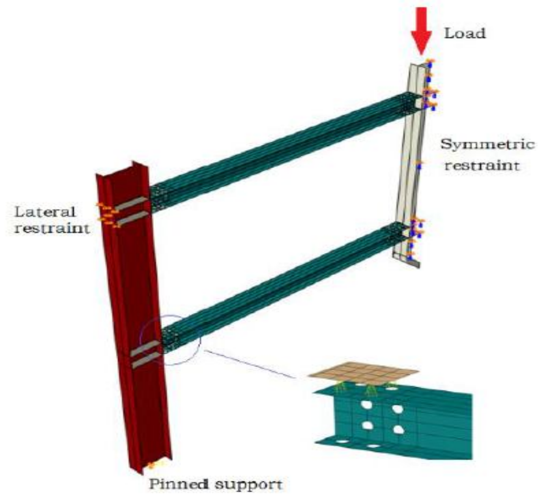

Figure 17: View of the FE model for example with BFP connections.

Conclusion

In the first study [1], the following results are evident:

A) The FE model used in the first study to evaluate the progressive collapse strength of RC flexural strength frames is recognized as suitable. This demonstrates the validity of the FE modeling approach that may be reliably used in future research to evaluate the progressive collapse of RC intermediate flexural frame structures, given the sudden column removal scenario.

B) Bond-slip effects of the concrete and longitudinal bars of beams and slabs connected to the removed column should be modeled.

C) Modeling RC slabs in three-dimensional assemblies significantly increases flexural capacity and collapse strength.

D) Applying axial load on the columns will increase the flexural stiffness of the columns and thus cause more restrictions on external joints and increase the flexural capacity.

In the second study [2], the following results are evident:

Events like earthquake and progressive collapse are two important hazards that affect the safety of RC frames. New structural details have been proposed to resolve the differences between the progressive and seismic collapse designs of reinforced concrete frames. Both cyclic and progressive collapse tests of RC substructures were performed to evaluate the effectiveness of the proposed structural details. Based on the test results, the following results were obtained:

A) Experimental results showed that although the existing progressive collapse scheme can effectively increase the resistance twice as much as the progressive collapse of RC frames, the new reinforcement created in the beams can make the connecting columns vulnerable to seismic cycle loads. After adopting the proposed structural details, earthquake damage can be reduced in the joint area and columns.

B) In the progressive collapse experiments of RC substructures under the middle column removal scenario, the newly added progressive collapse reinforcement, which has been rearranged according to the proposed details, can effectively improve the chain strength of the sample in such a way that meets the regulations by using the usual progressive collapse plan.

In the third study [3], the following results are evident:

A) Whenever an unusual external load is applied to a structure, such as a vehicle collision or an improvised explosive device, the most critical columns are those closest to the external frame of the structure.

B) The results of the analysis show that the removal of internal columns in bracing frames instead of corner columns is more important. In other words, the middle columns of the outer frames are more vulnerable than the corner columns.

C) Columns that are connected to the removed columns through beams get the maximum load in the redistribution of structural load and the effect of adjacent columns has the greatest among columns. It is interpreted that the extra capacity in the columns adjacent to the removed column has the most important effect in order to prevent progressive collapse.

D) The displacement at the location of the removed column in X-shaped bracing structures is less than the corresponding shear wall system. Also, the overall displacement of X-shaped bracing structures is less than the shear wall system.

E) Progressive collapse potentially decreases as the stiffness of the structure increases.

In the fourth study [4], the test results show that the acceptance criterion in ASCEH is very conservative in predicting the rotational capacity of the sample with BFP joints, and for steel frames that are not involved in shear collapse of the screw during progressive collapse, a computational model based on shell element with simplified coupling of screw surface and collapse criterion were found to be suitable for evaluating the strength and collapse state of the structure. This method can lead to a more efficient solution than the solid modeling strategy.

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