Review on Progressive Collapse Analysis of a Regular Structure

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Abstract: - Steel frame structures are constructed in seismic areas they are main targets of seismic activities. Due to such conditions nowadays, there is heavy demand of earthquake resisting steel frame structural design. Not only seismic activities but also due some of accidental failures, structure can fail. To analyze steel frame structure for different earthquake zones have to make model of steel structure using E-tabs software which can resist all types of loading such as dead load, live load, seismic load, using IS 800-2000 and IS 1893. In this study, we have selected a high-rise G+10 steel-framed structure. The structure is analyzed for seismic loading, due to which partial collapse or total collapse (progressive collapse) may occur which can be studied. From above analysis, we can study the type failure of structure under the guidelines of GSA for progressive collapse effect due to seismic load.

Key Words: — ***Progressive collapse, GSA guidelines, failure, steel structure, LSP, loading.***

I. INTRODUCTION

A structure undergoes Progressive Collapse when a primary structural element fails, resulting in the failure of adjoining structural elements, which in turn causes further structural failure. It is sometimes also called a disproportionate collapse, which is defined as a structural collapse disproportionate to the cause of the collapse. As the small structural element fails, it initiates a chain reaction that causes other structural elements to fail in a domino effect, creating a larger and more destructive collapse of the structure. A good example of progressive collapse is a house of cards; if one card falls near the top, it causes multiple cards to fall below it due to the impact of the first card, resulting in full collapse of the house of cards.

There are usually multiple factors that take place in order to initiate a progressive collapse. Improper communication between contractors and engineering documents can cause a progressive collapse. In this case, workers may not install specific structural elements properly that can lead to weakened structural members throughout the structure. Improper inspection or overlooking structural issues also leads to factors that initiate a progressive collapse.

In some cases, proper inspection may find a faulty member or connection yet may not properly document it or resolve the issue due to poor communication.

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In many multi-story buildings the lower floor has more headroom (so taller columns); and it often has more openings (so less walls); and it is usually stood on 'pinned' feet with no continuity. So the ground-to-first floor columns, which carry the biggest loads from the weight and the biggest cumulative sideways loads from the earthquake, are the longest and the least restrained and have the least end fixity.

II. LITERATURE REVIEW

Bruce R. Ellingwood (2002) studied on load and resistance factor criteria for progressive collapse. A progressive collapse initiates from a local structural failure and propagates, by a chain reaction mechanism, into a failure that involves a major portion of the structural system. The aftermath of the Roman point collapse in1969 saw numerous attempts in the 1970's to develop criteria for progressive collapse resistance. Improved building practices and design procedures to control the likelihood of progressive collapse are receiving renewed interest by standards organizations in the United States and elsewhere in the aftermath of the tragedy of September 11, 2001. Procedures for assessing the capabilities of a damaged structure to withstand damage without the development of a general structural collapse can be developed using concepts of structural reliability analysis and probability-based limit states design. This paper describes design strategies to minimize the likelihood of progressive collapse, and prospects for the implementations of general provisions in national standards such as ASCE Standard 7, Minimum Design loads for Buildings and Other Structures.

AbolhassanAstaneh-AsI (2007) studied Progressive Collapse Prevention of steel Frames with Shear Connections. This Steel Technical Information and Product Services (Steel TIPS) report provides information and technologies that can be used to protect steel buildings structures against progressive collapse in the event of removal of a column. It provides general information on progressive collapse of steel building structures. It provides information on progressive collapse behavior of steel frames with shear connections. The test consisted of removing the middle column of the exterior frame and pushing the joint at the top of the removed column down 19, 24 and 35 inches to measure the strength, stiffness and ductility of the structure as well as the connections. The steel frame with shear connections showed considerable resistance to progressive collapse after removal of a column. This was primarily due to the development of catenary force in the beams that were connected to the top of the removed column and to a lesser extent to membrane (catenary) action of the steel deck of the floors adjacent to the area of collapse. It discusses the research project conducted to investigate the use of steel cables to prevent progressive collapse of new steel building structures and develop design recommendations.

Kim and Kim (2009) utilized a macro-scale planner model to investigate the progressive collapse performance of Reduced Beam Section (RBS), Welded Cover Plated Flange (WCPF). Two types of steel moment frame buildings, designed for high seismic risk and moderate seismic risk were used in progressive collapse analysis. The building is about 3 storey and 6 stories high with various connection types. In this study, non-linear planner models which represented the perimeter moment frames of the buildings were used. The panel zones of all types of connections were modeled as rigid and distributed plastic hinge region was incorporated into all types of connections in order to mimic formulation of plastic hinges. The beam and column members were represented by nonlinear beam-column element provided by the open Sees and second order effect, the interaction between axial force and bending moment reaction could be considered by using the element. Nonlinear time-history seismic analysis, static pushdown analysis and nonlinear dynamic progressive collapse analysis were conducted using the proposed models. It was concluded that although the seismic performance of the three types of connections was similar, WCPF was the most effective in resisting progressive collapse, especially in structures located in moderate-seismic regions.

F. NateghiAlahi and N. Parsaeifard (2010) studied and analyzed of Seismic Progressive Collapse in one storey Steel Buildings to navigate the initial damage towards specific parts of the structural a corner-column was intentionally weakened. Then, push over analysis is carried out on the three dimensional model of the building and the behavior of structure, such as deformations are studied and the energy absorption of the frames are investigated and finally the collapse pattern of the building is obtained. In this paper progressive collapse potential of a special moment resisting steel buildings was investigated under earthquake action. A three dimensional model of the structure with an initially damaged corner-column was analyzed by increasing lateral loads, through nonlinear static procedure At the next steps, damaged frame and the nearby one support much deformation in comparison with the other ones, which can be due to torsion in structure as the effect of shifting the stiffness center to another point far from the damaged column. Another one-story building with five frames at both directions was modeled to have better perception about the behavior of one-story buildings. Linear elements were used to models the columns and beams and plastic hinges to define the non-linear behavior of the elements.

H.R. Tavaoli and A. RashidiAlashti (2012) made an attempt to investigate and study whether MRF steel structures that have been designed based on seismic codes, are able to resist progressive collapse with damaged columns in different locations under seismic loading. For this purpose, 3-D and 2-D push-over analysis of structure is carried out. The progressive collapse potential has been assessed in connection with 5 and 15-story buildings with 4 and 6 bays by applying the alternate load path method recommended in UFC guidelines. In contrast with 3-D models, two dimensional frames represent a higher sensitivity to base shear reduction and element removal. In the case of middle column removal, the structural is more robust than in a corner column removal situation. The influence of storey number, redundancy and location of critical eliminated elements has been discussed.

G. Taraa and A. Pinteaaa(2012) made an attempt to investigate and evaluated of multi-storey moment-resisting steel frames with stiffness irregularities using standard and advanced pushover methods. The standard pushover procedure is restricted to single-mode responses, a valid supposition for symmetrical or low-rise buildings, where the responses is dominated by the fundamental vibration mode. The standard pushover procedure becomes misleading when the response of the structure is influenced by higher vibration mode. This is a case of tall or non-symmetrical buildings. Several pushover procedures, able to take into account the effect of the higher vibration modes; have been lately developed to overcome this drawback. He compared between standard, advanced pushover analysis and the exact result obtained by nonlinear time history analysis. The analyses have been conducted on a series of moment-resisting steel frames with stiffness irregularities, with different no of stories, designed according to E8 and the Romanian Seismic Design Code for Romania.

III. Procedure As Per GSA Guidelines

Limitations on the use of LSP:

The use of the LSP is limited to structures that are 10-stories or less and that meet the following requirements for irregularities and Demand-Capacity Ratios (DCRs).

If there are no structural irregularities as defined as defined below, a linear static procedure may be performed and it is not necessary to calculate the DCRs. If the structure is irregular, a linear static procedure may be performed if all of the component DCRs determined are less than or equal to 2.0. If the structure is irregular and one or more of the DCRs exceed 2.0, then a linear static procedure cannot be used.

Loading:

Due to the different methods by which deformation-controlled and force-controlled actions are calculated, two load cases will be applied and analyzed: one for the deformation-controlled actions, and one for the force-controlled actions, as specified here. Live load reduction is allowed, if the requirements are met.

Increased Gravity Loads for Floor Areas Above Removed Column or Wall. Apply the following increased gravity load combination to those bays immediately adjacent to the removed element and at all floors above the removed element

$$G_{LD} = \Omega_{LD} [1.2 D + (0.5 L \text{ or } 0.2 S)] \quad (3.1)$$

Where,

G_{LD} = Increased gravity loads for deformation-controlled actions for Linear Static analysis

D = Dead load including façade loads (lb/ft^2 or KN/m ^{2})

L = Live load including live load reduction, not to exceed the maximum of 50-lb/ft^2 or 244-kN/m ^{2}

S = Snow load (lb/ft^2 or KN/m ^{2})

Ω_{LD} = Load increase factor for calculating deformation-controlled actions for Linear Static analysis; use appropriate value for framed or load-bearing wall structures.

Gravity Loads for Floor Areas Away From Removed Column or Wall. Apply the following gravity load combination to those bays not loaded with G_{LD}

$$G = 1.2 D + (0.5 L \text{ or } 0.2 S) \quad (3.2)$$

Where G = Gravity loads

Load Case for Force-Controlled Actions Q_{uf}

To calculate the force-controlled actions, simultaneously apply the following combination of gravity loads. Increased Gravity Loads for Floor Areas Above Removed Column or Wall. Apply the following increased gravity load combination to those bays immediately adjacent to the removed element and at all floors above the removed element

$$G_{LF} = \Omega_{LF} [1.2 D + (0.5 L \text{ or } 0.2 S)] \quad (3.3)$$

Where

G_{LF} = Increased gravity loads for force-controlled actions for Linear Static analysis

D = Dead load including façade loads (lb/ft2 or KN/m2)

L = Live load including live load reduction , not to exceed 50lb/ft^2 or 244-kN/m ^{2}

S = Snow load (lb/ft^{2}or KN/m ^{2})

Ω_{LF} = Load increase factor for calculating force-controlled actions for Linear Static analysis; use appropriate value for framed or load-bearing wall structures.

Gravity Loads for Floor Areas Away From Removed Column or Wall.

Use Equation 3.2 to determine the load G.

IV. Conclusion

This study can give us detail idea of failure of steel structure due to earthquake loading. Type of failure of structure can also be studied. Failure of structure due to earthquake lading can guide us for design of steel structure. The conclusion, which is derived from this project, is only for steel structures, as model in this project is considered to be steel frame structure. Analysis done in this project is only for G+ 10 structures made of only steel section. Results obtained from this project are only valid for G+ 10 structures. Results varies as per location of structure for example change in location of structure may change design of structure due to earthquake load or wind load.

Model selected for structure have specific dimension any change in dimension will change the analysis and hence result of the project.

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