Progressive Collapse of RC Buildings – Sustainable Analysis Procedures and Their Effects

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ABSTRACT

Progressive collapse is a chain reaction of failures that propagates through structure which is disproportionate to the original local failure. The progressive collapse of the World Trade Centre (WTC) towers has generated a worldwide concern of sustainability of multi-story buildings in the event of structural element(s) failure.

The prevention of progressive collapse lies primarily in the proper and effective analysis of the structural elements having high potential to progressivity. Different analysis methods are available such as linear static, non-linear static, linear dynamic and non-linear dynamic analysis. To minimize the progressive collapse risks, the structural system of the building should be able to tolerate the removal of one or more structural members and redistribute load carried by them on the surrounding members, so that disproportionate collapse would not take place.

Different agencies around the world are researching in this subject and have given guidelines time to time to sustain structures from progressive collapse.

This paper illustrates different analysis procedures, philosophies and the Codal provisions for this around the world. These analysis procedures are useful while analysing with different software packages. SAP2000 is one of the useful tools while doing nonlinear dynamic analysis as some of the peculiarities in SAP helps to complete the analysis in short time.

Keywords: Analysis procedures, FEMA, GSA, LRFD, Progressive collapse, SAP2000, UFC, WTC.

1. INTRODUCTION

A structure undergoes progressive collapse when a primary structural element fails, resulting in the failure of adjoining structural element, which in turn causes further structural failure similar to house of cards. The entire damage caused by this type of event is disproportionate to the initial damage. Terrorist attack is the most well-known cause one may remember but there are other reasons as well to trigger this event. The partial collapse of the 22-storey Ronan Point apartment tower in Newham (east London) in 1968 drew the interest of the research community towards this phenomenon for the first time. A gas explosion in a corner of the 18th floor blew out a load-bearing wall, which in turn caused the collapse of the upper floors due to the loss of support. The impact of the upper floors on the lower ones led to a sequential failure all the way down to the ground level. As a result, the entire corner of the building collapsed, as can be observed in Figure 1. This partial collapse was attributed to the inability of the structure to redirect loads after the loss of a load-carrying member. It is a particularly representative example since the magnitude of the collapse was completely out of proportion with respect to the triggering event.

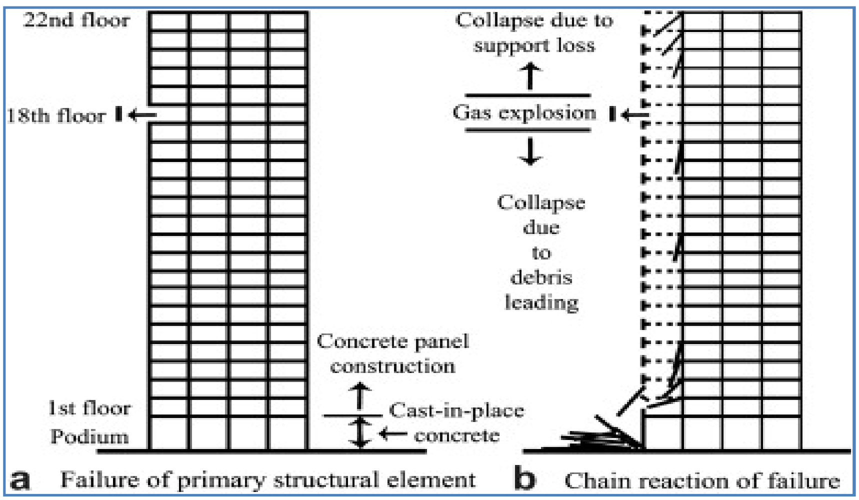

Fig. 1: Ronan Point Building in London 1968; after progressive collapse

2. NEED FOR ANALYSIS

The progressive collapse of the World Trade Centre (WTC) towers has generated a worldwide concern of the risks of progressive collapse in multi-story buildings. The prevention of progressive collapse lies primarily in the proper and effective analysis of the structural elements having high potential to progressivity.

Apart from blasts & terrorist attack following are few other reasons which could trigger this kind of collapse.

- Dynamic action and/or load concentration,
- Material deterioration or weak element(s),
- Overstrength or more ductility of material,
- Continuity or discontinuity,

- Series and parallel load transfer,
- Spatial orientation, size and slenderness of element(s).

There were two recent building collapses in Asia Region.

- 1100+ killed in Building collapsed in Dhaka, Bangladesh on 23rd April 2013.
- 74 killed in Building collapsed in Mumbai, India on 4th April 2013.

These incidences lead to think over the reasons in the progressive collapse with respect to above reasons as well as think for solutions to prevent such man made calamities.

3. AGENCIES WORKING IN THIS AREA

Progressive collapse of World Trade Center (WTC) towers has generated worldwide concern in the researchers and there was a wave to get in to the exact reasons in the last decade. There are some agencies working on guidelines for design of progressive collapse, but no one had specified a particular approach or guideline for the analysis. Below is list of few agencies which have given some guidelines.

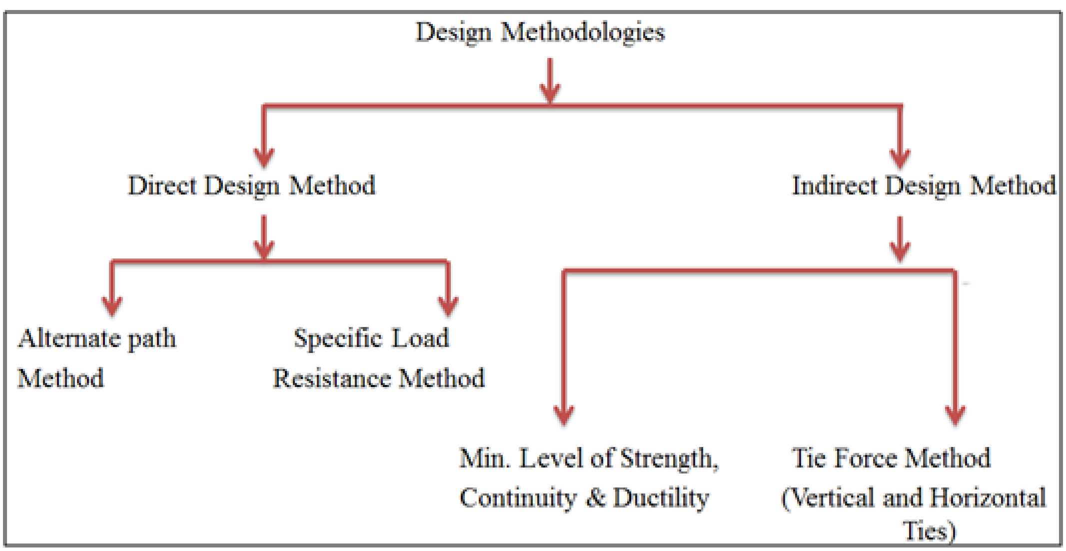

Fig. 2: Design Methodologies

- ASCE 7 – 05 provide guidelines related to direct and indirect design methods.
- FEMA – 427
- GSA Guidelines (2003)
- UFC 4-023-03 (2009)

- European Standards provides strategies for safeguarding civil engineering work against accidental actions.
- EN – 1991-1-7

4. DESIGN METHODOLOGIES

The principle of taking precaution in design to limit the effects of local collapse shall be realistic and can be satisfied economically. From a public-safety viewpoint it is reasonable to expect all multi-storey structures to possess general structural integrity comparable to that of properly designed, conventionally framed structure. There are a number of ways to obtain resistance to progressive collapse. A distinction is made between direct and indirect design, and the following approaches are defined.

5. INDIRECT DESIGN

With Indirect Design, resistance to progressive collapse is considered implicitly "through the provision of minimum levels of strength, continuity and ductility". Structural integrity has been considered with general design guidelines in code provisions by most of the national codes. These guidelines include:

- Good plan layout,
- Integrated system of ties,
- Changing span directions of floor slabs,
- Load-bearing interior partitions,
- Catenary action of the floor slab,
- Beam action of the walls,
- Redundant structural systems,
- Ductile detailing,
- Additional reinforcement for blast and load reversal, if the designer must consider explosive loads.

6. DIRECT DESIGN

Direct Design approaches include "explicit consideration of resistance to progressive collapse during the design process…" These include:

- Specific Local Resistance
- Alternate Path Method (APM)

Specific local resistance method requires that the building, or parts of the building, provide sufficient strength to resist a specific load or threat whereas alternate path method requires that the structure should be capable of bridging over a missing structural element, with the resulting extent of damage being localized. This method follows the LRFD philosophy or Limit State approach by employing load factor combination for extraordinary events and resistance factors to define design strengths.

As per the LRFD approach, the design strength must be greater than or equal to the required strength:

$$\boldsymbol{\Phi\, Rn \geq \Sigma\, \gamma i\, Qi} \qquad \text{Eq. 1}$$

Where $\Phi\, Rn$ = Design Strength

$\Sigma\gamma i\, Qi$ = Required Strength

In APM designation of primary and secondary components is utmost important as primary components are structural elements which have capacity to resist progressive collapse and transfer load from one structural element to other. Secondary elements are the nonstructural elements which may or may not take any part in load transfer when primary structural element fails.

7. GENERAL GUIDELINES

Intention should be such that the structural elements should be sound or strong enough to distribute/transfer load to other structural elements after failure of particular structural element. So that primary element failure could not be the cause of entire structure collapse or a disproportionately large part of structure. Although, it is not practically possible to design every structure to resist progressive collapse but structures can be designed to limit the effect of local failure of structural element and to prevent or minimize progressive collapse. In this analysis procedure it has been assumed that initial local damage is undefined and so the event causing the initial local damage.

Following are general guidelines which must be followed in all the analysis procedures.

- To model, analyze and evaluate a building – use three-dimensional assembly of elements and components. No two-dimensional models are permitted.
- Include stiffness and resistance of only the primary elements and components.
- Secondary members and elements analyzed separately. If they are added in model then stiffness should not be considered.
- Use the guidance of chapter 5 to 8 of ASCE41 to create a model.

- Expected material properties such as yield strength, ultimate strength, weld strength, fracture toughness, elongation, etc, shall be based on mean values of tested material properties. Lower bound material properties shall be based on mean values of tested material properties minus one standard deviation.

Types of Analysis Procedures

- Linear Static (LS) Procedure
- Nonlinear Static (NS) Procedure
- Nonlinear Dynamic (ND) Procedure

8. LINEAR STATIC (LS) PROCEDURE

This procedure is applicable for the structures which meets the following requirements for irregularities and Demand Capacity Ratios (DCRs).

- Significant discontinuities in load transfer path such as transfer girder, out of plane offset of primary vertical element.
- Strength and/or stiffness of columns and connections other than exterior corner column shall not be less than 50%.
- The vertical lateral-load resisting elements are not parallel to the major orthogonal axes of the lateral force-resisting system, such as the case of skewed or curved moment frames and load-bearing walls.
- There is no need to calculate DCR if particular structure meets irregularity criteria. Also if DCRs calculated by following equation 2 exceeds 2.0 then this procedure cannot be used.

$$DCR = Q_{UDLim}/Q_{CE} \qquad \text{Eq. 2}$$

Where, Q_{UDLim} = Design Strength

Q_{CE} = Expected Strength

- **Loading in Linear Static Procedure**

Increased Gravity Loads for Floor Areas Above Removed Column: Apply the following increased gravity load combination to those bays immediately adjacent to the removed element and at all floors above the removed element; see Figure 3.

$$G_{LD} = \Omega_{LD} \text{ or } \Omega_{LF} \, [(0.9 \text{ or } 1.2)\, D + (0.5\, L \text{ or } 0.2\, S)] \qquad \text{Eq. 3}$$

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

D = Dead load

L = Live load

S = Snow load (lb/ft^2 or kN/m^2)

Ω_{LD} = Load increase factor for deformation-controlled actions for LS analysis; use appropriate value for framed structures see table 1.

Ω_{LF} = Load increase factor for calculating force-controlled actions for Linear Static analysis; use appropriate value for framed structures see table 1.

Material	Structure Type	Ω_{LD}, Deformation-controlled	Ω_{LF}, Force-controlled
Steel	Framed	0.9 m_{LIF} + 1.1	2.0
Reinforced Concrete	Framed[A]	1.2 m_{LIF} + 0.80	2.0
	Load-bearing Wall	2.0 m_{LIF}	2.0

Table 1: Load Increase Factor

Gravity Loads for Floor Areas Away From Removed Column: Apply the following gravity load combination to those bays not loaded with $\boldsymbol{G_{LD}}$; see Figures 3.

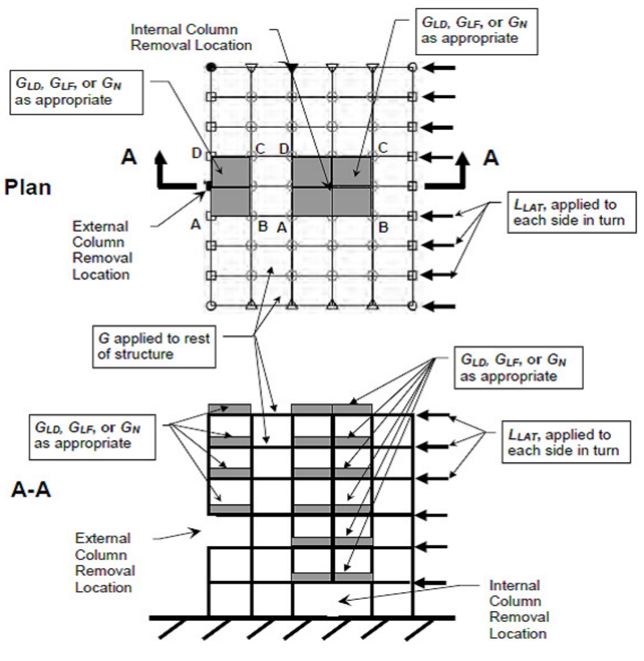

Figure 3: Loads and Load Locations for External and Internal Column Removal for Models.

$G = (0.9 \text{ or } 1.2)\ D + (0.5\ L \text{ or } 0.2\ S)$ Eq. 4

Where G = Gravity loads

Lateral Loads Applied to Structure: Apply the following lateral load to each side of the building one side at a time, i.e., four separate analyses must be performed, one for each principal direction of the building, in combination with the gravity loads G_{LD} and G.

$L_{LAT} = 0.002\Sigma P$ Eq. 5

Where L_{LAT} = Lateral load

0.002ΣP = Notional lateral load applied at each floor; this load is applied to every floor on each face of the building, one face at a time

ΣP = Sum of the gravity loads (Dead and Live) acting on only that floor. Load increase factors are not applicable.

9. NONLINEAR STATIC (NS) PROCEDURE

There are no DCR and geometric irregularity limitations on the use of this procedure. Similar loading shall be used as per LS procedure except Ω_N which is dynamic increase factor as per ASCE 41.

Material	Structure Type	Ω_N
Steel	Framed	$1.08 + 0.76/(\theta_{pra}/\theta_y + 0.83)$
Reinforced Concrete	Framed	$1.04 + 0.45/(\theta_{pra}/\theta_y + 0.48)$
	Load-Bearing Wall	2

Table 2: Dynamic Increase Factor for Nonlinear Static Analysis

Apply the loads using a load history that starts at zero and is increased to the final values. Apply at least 10 load steps to reach the total load. The software must be capable of incrementally increasing the load and iteratively reaching convergence before proceeding to the next load increment.

10. NONLINEAR DYNAMIC (ND) PROCEDURE

There are no DCR and geometric irregularity limitations on the use of this procedure. Create a model of the entire structure, including the wall section and column that are to be removed during the analysis. Include the stiffness and resistance of primary components. Use the stiffness requirements of ASCE 41 Chapters 6 to create the model. Discretize the load-deformation response of each component along its length to identify locations of inelastic action. The force-displacement behavior of all components shall be explicitly modeled, including strength degradation and residual strength, if any. Model a connection explicitly if the connection is weaker or has less ductility than the connected components, or the flexibility of the connection results in a change in the connection forces or deformations greater than 10%. Apply the following combination of gravity and lateral loads per the loading procedure.

Gravity Loads for Entire Structure: Apply the following increased gravity load combination to entire structure.

$$G_{ND} = (0.9 \text{ or } 1.2)\ D + (0.5\ L \text{ or } 0.2\ S) \qquad \text{Eq. 4}$$

11. APPLICATION OF HINGE AND NONLINEAR EFFECT

For nonlinear static (NS), and nonlinear dynamic (ND) with direct-integration time-history analyses, simulation of post-yield behavior is possible by assigning concentrated plastic hinges to frame and tendon objects. Elastic behavior occurs over member length, and then deformation beyond the elastic limit occurs entirely within hinges, which are modeled in discrete locations. Inelastic behavior is obtained through integration of the plastic strain and plastic curvature which occurs within a user-defined hinge length, typically on the order of member depth. To capture plasticity distributed along member length, a series of hinges may be modeled. Multiple hinges may also coincide at the same location.

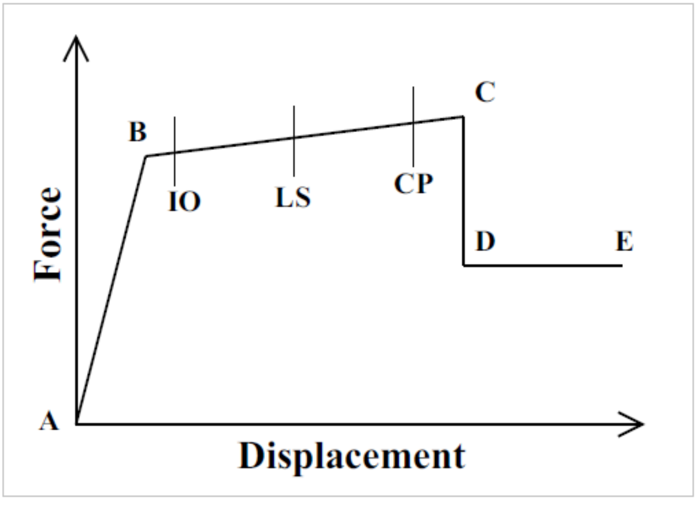

Fig. 4: Force-Deformation Relation

Plasticity may be associated with force-displacement behaviors (axial and shear) or moment-rotation (torsion and bending). Hinges may be assigned (uncoupled) to any of the six DOF. Post-yield behavior is described by the general backbone relationship. The modeling of strength loss is discouraged, to mitigate load redistribution (which may lead to progressive collapse) and to ensure numerical convergence. CSI Software automatically limits negative slope to 10% of elastic stiffness, though overwrite options are available. For informational purposes, additional limit states (IO, LS, CP) may be specified which are reported in analysis, but do not affect results. Unloading from the point of plastic deformation follows the slope of initial stiffness.

Both P-M2-M3 hinges and fiber hinges are available to capture coupled axial and biaxial-bending behavior. The P-M2-M3 hinge is best suited for nonlinear static pushover, whereas the fiber hinge is best for hysteretic dynamics. Difference between FEMA and Caltrans hinge has been described in more detail.

Table 3: Difference between FEMA & Caltrans Hinge

Sr. No.	Description	MPH (FEMA)	FPH (Caltrans)
1.	Non Linearity	Material	Both
2.	Behavior	Rigid-Plastic	Elastic Plastic
3.	Moment Rotation Relation	P-M Interaction not considered	Considered
4.	PMM interaction and deformation	P-M Interaction considered but not deformation.	Both PMM interaction and deformation
5.	Effect	Rigid-Elastic-Plastic	Elastic-Elastic-Plastic

Frame hinges must be specified to model nonlinear frame behaviour. Nonlinear material parameters are then associated with hinge response, including the interaction surface and the moment-rotation curves which describe post-yield behaviour. When implementing fiber hinges, material definition controls the stress-strain relationships of individual fibers.

12. CONCLUSION

Different analysis procedures studied with reference to some technical papers. It is utmost important to choose particular analysis procedure based on engineering judgment so that there should not much difference in actual or experimental results and the software analysis. Following points needs to be considered in analysis so that structural health assessment could be done in effective manner.

- Linear Static and Nonlinear static procedures apply heavy penalties in terms of load increase factor.
  - This could be useful where software unable to handle material and geometric nonlinearity.
- Nonlinear dynamic analysis with direct integration time history analysis could give nearest possible loading & could provide correct behavior.
  - Software should be capable of doing time history analysis with direct integration.
  - Software should have ability to add plastic hinges to structural element.
- Loading applied to the structure in different analysis procedures is lesser than the actual analysis & design.
- Nonlinear effect of floor slab needs to be taken in to account.

REFERENCES

[1] David Stevens; Brian Crowder; Doug Sunshine; Kirk Marchand; Robert Smilowitz; Eric Williamson; Mark Waggoner, M.ASCE "DoD Research and Criteria for the design of Buildings to resist Progressive Collapse" Journal Of Structural Engineering © ASCE / September 2011.

[2] SAP2000. (2009). SAP2000 three dimensional static and dynamic finite element analysis and design of structures, analysis reference, version 14.1, Computer and Structures, Inc., Berkeley, CA.

[3] ASCE (2006b) "Seismic rehabilitation of existing buildings" ASCE/SEI 41-06, Reston, VA.

[4] Dept. of Defense (DoD). (2009). "Design of buildings to resist progressive collapse." UFC 4-023-03, Washington, DC.

[5] FEMA 427 / December 2003