Structural Design of Buildings to Resist Blast and Progressive Collapse (Case Study: Main Substation Building Located at Esfahan Refinery Plant in Iran)

JAVAD YAZDANSETA, MAHBOBE TAHERI
Mapna Group Company
No.89, 6th St., North Naft St., Mirdamad Ave., Tehran
IRAN

Abstract: - The loading produced by blast events are typically much higher than the design loadings for which an ordinary structure is designed. These loadings, referred to as overpressures in the technical literature, are beyond the capacity of the structure and local failure of structural elements in the region of the explosion is likely. For such loadings, component and system ductility can be utilized to avoid system collapse. Progressive collapse occurs when a structure has its loading pattern, or boundary conditions, changed such that structural elements are loaded beyond their capacity and fail. The residual structure is forced to seek alternative load paths to redistribute the load applied to it. As a result, other elements may fail, causing further load redistribution. The process will continue until the structure can find equilibrium either by shedding load or by finding stable alternative load paths. As a case study for this paper the main substation building located at Esfahan refinery plant in Iran is considered. A nonlinear static analysis is used to assess accurately the post attack behavior of structural elements that are not removed from the building by the blast loads in their corresponding damaged states.

Key-Words: - Blast, Overpressures, Explosion, Progressive collapse, Load paths, Nonlinear Static Analysis

1 Introduction

Generally, for economic reasons buildings are not designed for load conditions to account for abnormal loads such as gas explosion, bomb explosion, vehicular collisions, aircraft collisions, tornados and the like. Thus when these buildings are subjected to such loads, they may sustain extensive damage.

Currently there are no formal blast performance criteria for buildings and making buildings blast resistant can be translated to making sure that partial and full collapse of the structure of these buildings are prevented. Some structures have highly indeterminate structural systems with more than one load paths to resist the applied loads. In these highly indeterminate systems with many redundant members, it is possible to eliminate a number of structural elements yet not to be able to collapse the structure or the floors. Such characteristic denoted as "progressive collapse" resistance that is discussed in more detail in this paper.

2 Progressive Collapse

Progressive collapse became an issue following the Roman Point incident HMSO [1], when a gas explosion in a kitchen on the 18th Floor of a precast building caused extensive damage to the entire corner of that building. Typical design strategies for collapse resistant buildings involve removal of one or more vertical load carrying elements and demonstrating that not more than specified portions of the building will be subject to collapse upon such occurrence.

Two general approaches are defined to reduce the possibility of progressive collapse including Alternate Path (AP) and Tie forces (TF) [2].

AP method is a direct design approach which requires the structure be capable of bridging over a missing structural element, with the resulting extent of damage being localized. In addition, damaged structures may have insufficient reserve capacities to accommodate abnormal load conditions. Therefore, it is critical to assess accurately the post attack behavior of structural elements that were not removed from the building by the blast loads in their corresponding damaged states. TF method is an indirect design approach that enhances strength, continuity and ductility redundancy by requiring ties to keep the structure together in the event of abnormal loading.

3 Case Study, Main Substation Building

As a case study for this paper the mail substation building located in Esfahan refinery plant is considered.

3.1 Introduction

Blast resistant buildings in Esfahan refinery plant are designed to withstand the surface explosion of a 500 lb "G.P.AERIAL BOMB" (i.e. : weight of TNT charge is about 50% of bomb weight) blasting at 10m (33ft) from any external wall of building [3]. It is assumed the pressure wave produced by such explosion and acted on the exterior of the building will cause the local column failure. The rest of structure is to resist the alternate path load combination by bridging over missing column. How long the building must remain standing following local damage is an issue in determining the appropriate load combination.

For the building under study horizontal and vertical tie forces are calculated and the Alternate Path method is applied to prevent local damage from propagating. CSI's most renowned software, i.e. SAP2000 is used for nonlinear static analysis of structure to investigate the redistribution of forces after the damaged column was instantaneously removed [4].

3.2 Description of the Structure

The structure is a two-story reinforced concrete moment frame building. It is one bay by twelve bays in plan, with a 13.5 m x 5.5 m typical bay. The ground and first stories are 2.8m and 5.3m high respectively. (See figures in section 5 of this paper for building drawings)

3.2.1 Model Assumption

1. All connections are assumed to be moment connections.
2. Column to foundation connections are considered fully restrained.
3. Each floor is taken as a rigid diaphragm.
4. Material properties: concrete strength (fc') = 24 MPa, rebar yield strength (fy) = 300 MPai, modulus of elasticity of concrete (Ec) = 23390 MPa and modulus of elasticity of rebar (Es) = 210000 MPa.

3.2.2 Loading assumption

1. Dead load (DL) is equal to the self weight of members.
2. Live load (L).
3. Snow load (S).
4. Earthquake (E) load is determined according to the provisions of "Iranian
Code of Practice for Seismic Resistant Design of Buildings" known as standard
No.2800
The building is located in a seismic region and wind load is assumed not to control the design.
5. Blast load (BL), the effects of blast load is considered by removing failed columns and proceeding alternate path analysis.

3.2.3 Load combinations

ACI updated load combinations are used for preliminary design of building as follow [5]:
L.C.1: 1.4D
L.C.2: 1.2D+1.6L+0.5S
L.C.3: 1.2D+ (1.0E) + 0.5L+0.2S
L.C.4: 0.9D+ (1.0E)

The building is analyzed and designed for the above load combinations, the required tie forces are calculated and then the alternate path method is applied, after removal of failed columns, according to the load combination described below.

3.2.3.1 Factored Loads for Alternate Path Method

Various building standards recommend different load combinations for evaluating the capability of the damaged structure to bridge over damaged area. An appropriate load combination to base the reliability analysis of a building structural system with local damage is recommended as follow [6]:
Dead Load + Sustained Live Load+ (Daily Snow or Monthly Maximum Wind Load)
(Note: the wind load is not considered in this paper as a dominant lateral load)

Unified Facilities Criteria (UFC) considers a small lateral force 0.2W in the combination to ensure that the lateral stability of the damaged structural system is checked in the analysis. The same load factor is used for earthquake load as a lateral load in this paper. For static nonlinear analysis it recommends the factors to be doubled for those bays immediately adjacent to the removed column.

The AP analysis results are compared for two cases with and without doubled load factors. Also the dead load factor is kept as 1.2 (or 0.9 when dead load stabilizes the structural system), as with other load combinations. Therefore the building is checked for the following load combinations after notional removal of load-bearing column.

For the bays adjacent to the removed column:

2.0 [(0.9 or 1.2) D + (0.5 L or 0.2 S)] + 0.2 E

For the rest of the structure, the load combination is applied as follow:

(0.9 or 1.2) D + (0.5 L or 0.2 S) + 0.2 E

4 Preliminary Design Results

Building is analyzed and designed for ACI updated load combinations mentioned in section 3.2.3.

4.1 Member Sizes

The member sizes and the required reinforcements are set out in the tables 1&2 below.

Table 1: Reinforced concrete member sizes and reinforcement-first floor

Member Group	Dimensions	Bottom Reinf.	Top Reinf.
Spandrel-Girders	B=50 Cm D=70 Cm	22 Cm²	48 Cm²
Spandrels	B=30Cm D=40 Cm	10 Cm²	22 Cm²
Girders	B=50 Cm D=70 Cm	38 Cm²	77 Cm²
Corner Columns	B=50 Cm D=50 Cm	142 Cm²	
Mid Columns	B=60 Cm D=60 Cm	171 Cm²	

Table 2: Reinforced concrete member sizes and reinforcement-second floor

Member Group	Dimensions	Bottom Reinf.	Top Reinf.
Spandrel-Girders	B=50 Cm D=60 Cm	19 Cm²	43 Cm²
Spandrels	B=30Cm D=40 Cm	10 Cm²	22Cm²
Girders	B=50 Cm D=60 Cm	32 Cm²	65 Cm²
Corner Columns	B=50 Cm D=50 Cm	102 Cm²	
Mid Columns	B=60 Cm D=60 Cm	108 Cm²	

4.2 Tie Force Check

After designing the reinforcement concrete moment frame building tie forces are calculated to ensure their requirements are met. The design tie strengths are considered separately from the forces that are typically carried by each structural element due to live load, dead load, earthquake load, etc.; in other words, the design tie strength of the element or connection with no other loads acting must be greater than or equal to the required tie strength. As the table 3&4 depict the concrete designed in previous section easily meets the tie force requirements.

Table 3: Required tie forces – first floor

Tie Type	Required Tie Force (KN)	Required Steel Area (Cm²)	Available Steel Area (Cm²)	TF> TR_{req}
Peripheral ties	28	1	22 Cm² Spandrel top 48 Cm² Spandrel-Girder	Yes
Internal ties	113.40	4	77 Cm² Girder top	Yes
Horizontal ties to columns	32	2	22 Cm² Spandrel top 48 Cm² Spandrel-Girder 77 Cm² Girder top	Yes
Vertical ties in columns	380	13	171 Cm²	Yes

Table 4: Required tie forces – second floor

Tie Type	Required Tie Force (KN)	Required Steel Area (Cm²)	Available Steel Area (Cm²)	TF> TR_{req}
Peripheral ties	28	1	22 Cm² Spandrel top 43 Cm² Spandrel-Girder	Yes
Internal ties	113.40	4	65 Cm² Girder top	Yes
Horizontal ties to columns	32	2	22 Cm² Spandrel top 43 Cm² Spandrel-Girder 65 Cm² Girder top	Yes
Vertical ties in columns	380	13	108 Cm²	Yes

5 AP Method Results

For each plan and each column removal, AP analysis is performed for each earthquake direction one at a time. For example, middle column in the ground floor is specified as the removal element and AP analysis is performed; another AP analysis is performed for the removal of the first floor middle column; another AP analysis is performed for the column next to the middle column in the ground floor, and so on.

The results of this method are presented for some columns alternatively being removed due to blast pressures. The fig.1 shows the redistribution of loads after removing the middle column in the ground floor due to the doubled load factors in the alternate path method load combination. The designed peripheral beams are to be strengthened for this case to resist the effect of blast pressure under amplified load factors. The nonlinear analysis is repeated for AP method when the load factors are not doubled. Fig. 2 depicts the effect of this load combination after removal of middle column. In these two figures the load combinations are for the earthquake in X direction.

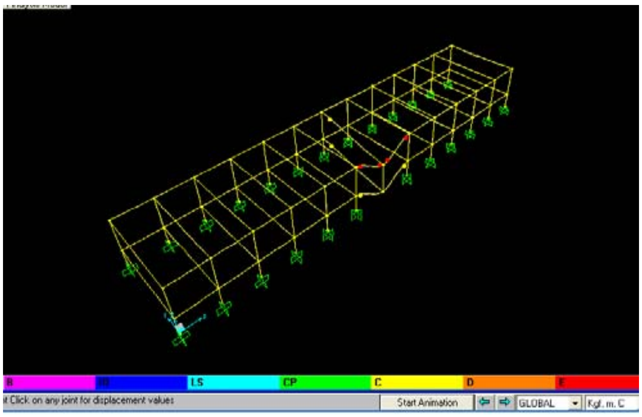

Fig.1: AP method result after removal of middle column in ground floor

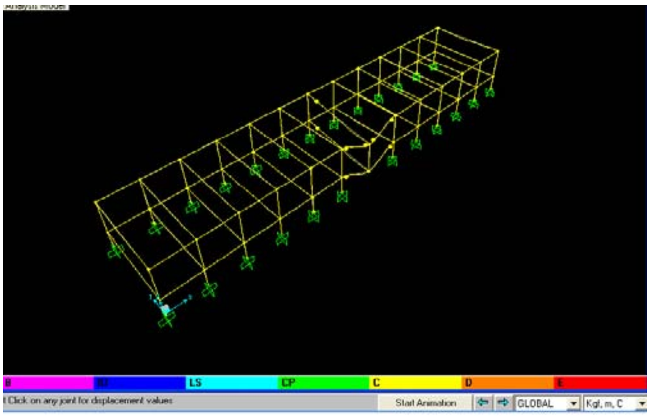

Fig.2: AP method result after removal of middle column in ground floor (load factors are not doubled)

Figs. 3&4 show the analysis results after removal of one of columns in the first floor in two earthquake directions. After performing first nonlinear analysis the plastic hinge was formed in the beam connected to the removed column and put the member in the D-E area. The beam is strengthened and the analysis is repeated for which the results are shown in these two figures.

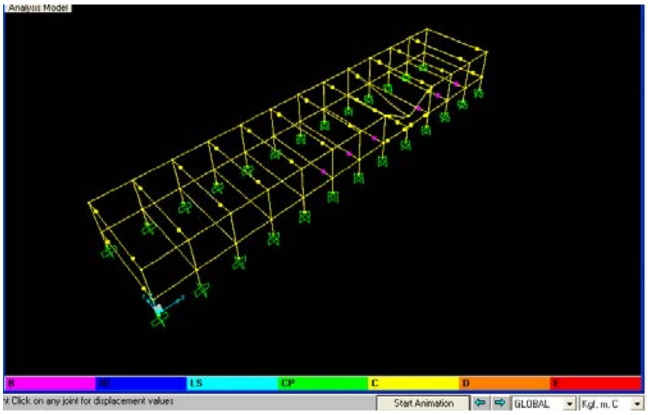

Fig.3: AP method result after removal of first floor column (earthquake in X direction)

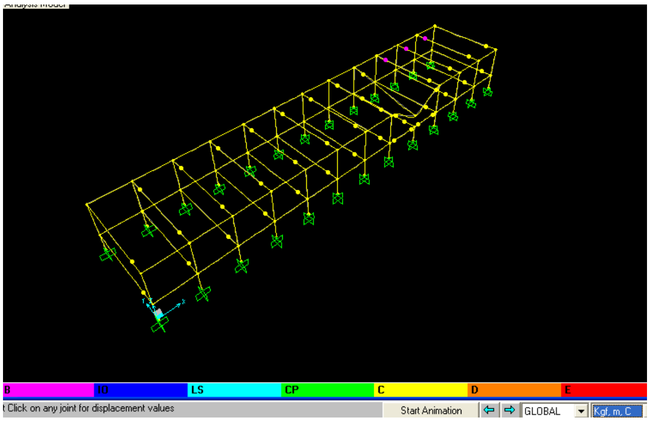

Fig.4: AP method result after removal of first floor column (earthquake in -Y direction)

6 Conclusion

The purpose of this study was to see that the preliminary design of building subject to typical loads is to be changed to make the building blast resistant. A number of buildings in Esfahan refinery plant are to be designed blast resistant and the analysis path method can be used for each of them to prevent progressive collapse due to abnormal blast loads.

Because of removal of damaged column the members that originally spanned a single bay must now span two bays and they have to be strengthened to develop positive moments.

Furthermore, discontinuities will cause concentration loads and must be avoided.

It can be concluded that applying the amplifying load factors by some international standards for the load combinations in AP method is a good idea for the static linear and nonlinear analysis.

Column connections to foundation should be checked for additional flexure that might result from load redistribution as a consequence of the loss of a structural element.

Loss of column will increase the loads of footings under adjacent columns as load redistribution, therefore check is to be done to make sure the ultimate bearing strength is not exceeded. The thickness for foundations will be necessary to be checked to avoid punching failure.

References:

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