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Analysis of building collapse under blast loads

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Abstract

The analysis of the structural failure of a reinforced concrete building caused by a blast load is presented in this paper. All the process from the detonation of the explosive charge to the complete demolition, including the propagation of the blast wave and its interaction with the structure is reproduced. The analysis was carried out with a hydrocode.

The problem analysed corresponds to an actual building that has suffered a terrorist attack. The paper includes comparisons with photographs of the real damage produced by the explosive charge that validates all the simulation procedure.

Keywords: Blast load; Reinforced concrete; Structural collapse; Numerical analysis

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1. Introduction

Due to different accidental or intentional events, related to important structures all over the world, explosive loads have received considerable attention in recent years. The design and construction of public buildings to provide life safety in the face of explosions is receiving renewed attention from structural engineers [1–3]. Such concern arose initially in response to air attacks during World War II [4–6], it continued through the Cold War [7] and more recently this concern has grown with the increase of terrorism worldwide [1–3]. For many urban settings, the proximity to unregulated traffic brings the terrorist threat to or within the perimeter of the building. For these structures, blast protection has the modest goal of containing damage in the immediate vicinity of the explosion and the prevention of progressive collapse. In this sense, computer programs simulations could be very valuable in testing a wide range of building types and structural details over a broad range of hypothetical events [3].

With the rapid development of computer hardware over the last decades, it has become possible to make detailed numerical simulations of explosive events in personal computers, significantly increasing the availability of these methods. On the other hand, new developments in integrated computer hydrocodes [8], such as AUTODYN software [9] used in this paper, complete the tools necessary to carry out the numerical analysis successfully. Nevertheless, two important features must be taken into account when performing computer blast resistance assessment of buildings. The first one is related to the need of experimental validation of both the models and the analysis procedures used. The second problem is related to the computational cost that makes almost impracticable, a realistic blast analysis of an actual reinforced concrete building with all its details.

Much research has been carried out in last years concerning the behaviour of structural elements and materials under blast loads. The experimental results about the behaviour of steel [10,11], concrete [12–14] and fibre reinforced [15] panels subjected to explosions can be found in the bibliography. Nevertheless, most of the results related to full-scale structures are those concerning structures that have actually suffered explosions [16]. One of the first tests was a full-scale blast test on a four-story building at the White Sands Missile Range in New Mexico as part of a research and development

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contract from the Defense Threat Reduction Agency (DTRA) to investigate measures to retrofit US Embassies and other critical structures worldwide against blast loads. At White Sands, DTRA Field Command has constructed a full-scale prototype of a concrete flat-structure called CST-1 to test windows, walls and structural elements under realistic threat conditions (i.e., the blast effects of large vehicle bombs [17]). Although there are still many uncertainties, material behaviour under blast loads has been widely studied experimentally [18–21] and many sophisticated numerical models have been proposed, especially for steel and concrete [22–30]. These models have been included in different computer programs [23,29,31], which can be used for the analysis of the blast behaviour of structural elements and small structures and validated with available experimental results. Nowadays, the analysis of a complete reinforced concrete structure taking into account all the reinforcement details is almost impracticable due to the elevated computational costs. Many simplifying assumptions should be done in order to perform the analysis [32,33] and most of these assumptions are related to constituents' materials that cannot be yet considered as individual materials but have to be interpreted as homogenised materials with average properties.

This paper is related to the effect of blast loads in reinforced concrete buildings and presents the results of the numerical simulation of the structural collapse of an actual building, the AMIA (Israel's mutual society of Argentina) building, that has suffered a terrorist attack that produced the demolition of part of it in 1994.

The location and magnitude of the explosive load were previously obtained from the blast analysis and comparison with actual damage of the complete block of buildings where the target was located [34] and are supposed to be known in this paper. It is assumed that the damage was caused by an explosive load equivalent to 400 kg of TNT placed in the entrance hall of the building.

In order to reproduce the structural collapse, the complete building was modelled, including the reinforced concrete structure and the masonry walls. Appropriate numerical models were used for the different materials that are in the structure. The mechanical properties of the materials were obtained from tests on parts of the actual structure. The constitutive model used for concrete was proved and calibrated with experimental results of a concrete plate subjected to blast loads [13]. A homogenised model was used for reinforced concrete.

In order to reproduce the complete phenomenon, the volume of air in which the structure was immersed was also modelled. The analysis began with the modelling of the detonation and propagation of the pressure wave inside the explosive and in the air in contact with the explosive. As this analysis must be performed with much detail, it was done in a previous stage in which a spherical explosive was modelled [34]. Then the results of this first analysis were mapped into the 3D model [9]. Starting from this point, the propagation of the blast wave in air and its interaction with the building was simulated. The complete collapse process was reproduced and compared with the rest of the actual building validating the analysis procedure and the assumptions made.

2. Computational model

The computational model was constructed following the structural and architectural plans of the actual building. For the dynamical analysis of building structures under seismic or wind loads, it is enough to model only the resisting structure. A blast analysis also requires the consideration of all the non-structural elements, specially the walls as they play an important role in the propagation of the pressure wave.

2.1. Parts of the model

The model is composed of the building and air volume occupied by the building.

A 3D Euler FCT [9](higher order Euler processor) subgrid was used for the air. The lateral faces of the underground level were supposed to have rigid surfaces and air flow out was allowed in the rest of the borders.

The complete building was modelled, from the underground level to the higher level. The resulting model is presented in Fig. 1. As it can be seen in this figure, the building was composed of three differentiable blocks: the front block, severely damaged by the blast load, the intermediate block and the back block. The mesh was refined and a more detailed modelling was used in the front block were the focus of the explosion was supposed to have been located and more damage was produced.

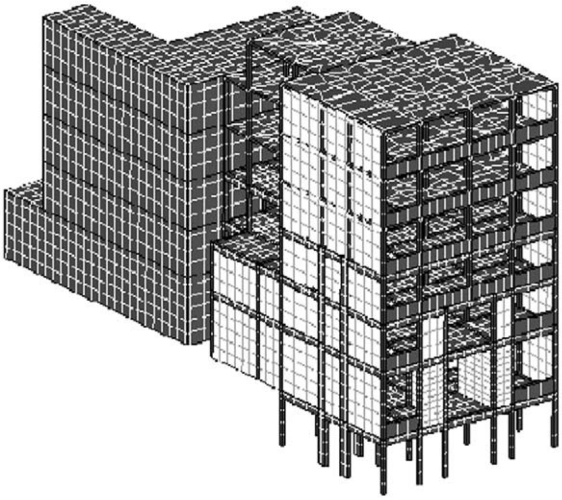

Fig. 1. Model for the complete building.

The model of the building was composed of the reinforced concrete structure and the masonry walls. A rigid floor in the bottom of the underground level was used to simulate the soil. The reinforced concrete structure was a frame structure composed by columns, beams and slabs. The mesh was finer in the front of the building and the lower levels. A coarse discretization was used for the back part of the building, not affected by the explosion and with no significance in the structural collapse.

The columns, beams and slabs were modelled with 3D solid elements that were solved with a Lagrange processor. The base of the columns were fixed in their base corresponding to the ground level in the back part of the building and the underground level in the front.

The exterior and interior walls of the first level, where the explosion was supposed to have taken place, were modelled with shell elements of 30 cm of thickness and solved with a Lagrange processor. As the walls in the building were in filled masonry walls, they were modelled as perfectly joined to the reinforced concrete frame. The loads were transferred to the frames through the common nodes.

2.2. Constitutive models

The mechanical properties of the different parts of the structure were obtained from the result of tests performed on proving corps extracted from the rests of the demolition and from parts of left structure.

2.2.1. Air

A fluid with the properties presented in Table 1 was used to model the air.

2.2.2. Reinforced concrete

Although reinforced concrete elements can be modelled as a combination of concrete and steel elements jointed together with the assumption of perfect bond, this type of model is prohibited for actual structures, as it requires a great number of elements. Moreover, the time step in explicit dynamic programs is directly related to the size of the elements. Elements of the dimensions of the actual reinforcement usually lead to extremely reduced time steps, making the analysis too slow.

Taking into account the above considerations, an approximate material model for reinforced concrete was defined to simulate the behaviour of reinforced concrete columns, beams and slabs that formed the resistant structure of the building. The model used is a homogenised elastoplastic material similar to concrete elastoplastic models but with higher tension strength to take into account the collaboration of the reinforcement to resist tension stresses. The model was calibrated with the results of the analysis of a reinforced concrete column of the first stage. These columns were chosen because they played an important role in the collapse mechanism of the building. They are square columns of 0.30 m side, 5 m of height and 4% of reinforcement. First, the columns were modelled and analysed identifying concrete and reinforcement elements, with a concrete and steel model, respectively, as in the actual reinforced concrete column. The analysis was repeated then using a reinforced concrete homogenised model for all the column elements. The properties of the homogenised model were determined to achieve the same strength under distributed static lateral loads as in the actual reinforced concrete column. The model was also tested under an explosive load of 400 kg of TNT located at a distance of 2.5 m to reproduce the same damage as in the actual reinforced concrete column. Both types of analysis corresponded to a tension strength of about the compression strength for the homogenised model and this fact justifies the use of the Von Mises model as the yielding and failure criteria. The compression strength of the homogenised model was taken as the compression strength of concrete obtained from tests performed on the rests of the structure.

Table 1
Properties of air

State equation: ideal gas
γ=1.41
Reference density: 1.225E$-$03 g/cm^3
Reference temperature: 2.882E + 02 °K
Specific heat (C.V.): 7.173E + 02 J/kg K

The mechanical properties of the homogenised model used for reinforced concrete are shown in Table 2. This concrete model with increased tension capacity was used for all reinforced concrete elements.

2.2.3. Masonry

An elastoplastic model, similar to those models frequently applied for plain concrete, but with reduced strength, was used to reproduce the behaviour of masonry walls. The principal mechanical properties of masonry are presented in Tables 3 and 4.

Table 2
Mechanical properties of reinforced concrete

State equation: linear
Reference density: 2.750 g/cm^3
Bulk modulus: 3.527E + 07 kPa
Strength model: Von Mises
Shear modulus: 1.220E + 07 kPa
Elastic limit: 1.000E + 04 kPa
Failure criteria: principal stresses
Failure stress: 1.000E + 04 kPa

Table 3
Mechanical properties of masonry

State equation: linear
Reference density: 2.400E + 00 g/cm^3
Bulk modulus: 7.800E + 06 kPa
Strength model: Mohr Coulomb
Shear modulus: 2.6E + 06 kPa
Elastic limit: according to Table 4
Failure criterion: principal stresses
Failure tension: 1.000E + 03 kPa

Table 4
Evolution of the elastic limit of masonry with mean pressure

Pressure (kPa)	Elastic limit (kPa)
0.000E + 00	1.000E + 04
3.600E + 04	3.130E + 04
1.460E + 05	1.140E + 05
2.700E + 05	1.140E + 05

2.3. Erosion

Detonation is a type of reaction of the explosive that produces shock waves of great intensity. If the explosive is in contact near a solid material, the arrival of the explosive wave to the surface of the explosive generates intensive pressure waves that can produce the crushing or the disintegration of the material. This shock effect is known as brisance effect [4].

If the explosive is surrounded by air, a pressure wave that can fracture masonry and concrete structures is generated.

Both brisance effect and fracture produce discontinuities in the material. In order to reproduce this type of effects, the erosion model of AUTODYN [9,29,30] was used to remove from the calculus the cells that have reached certain criteria based on deformations. When a cell is eliminated, its mass is retained and concentrated in its nodes that begin to behave as free masses conserving their initial velocity. This erosion model represents a numerical remedial to great distortion that can cause excessive deformation of the mesh. For this reason, its application to the simulation of a physical phenomenon requires the calibration with experimental results.

Erosion models were used both for the reinforced concrete structure and for masonry walls. For the calibration of the erosion model used for concrete, the experimental results of concrete slabs lying on the ground and subjected to explosions over them were used [13]. The slab was tested with different amounts of explosive. The testing set up is shown in Fig. 2a and the final state of the slab after three explosions over it is shown in Fig. 2b.

The slab was simulated with different erosion models, erosion limits and meshes in order to define the appropriate erosion criteria and erosion limit. Fig. 3 shows the results of one of the numerical tests. The light grey circles represent the eroded nodes.

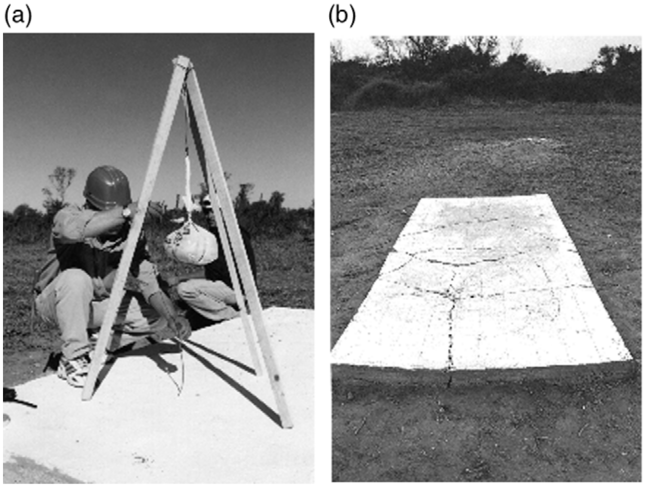

Fig. 2. Blast test of a concrete slab. (a) Location of the explosive over the slab, (b) final damage pattern.

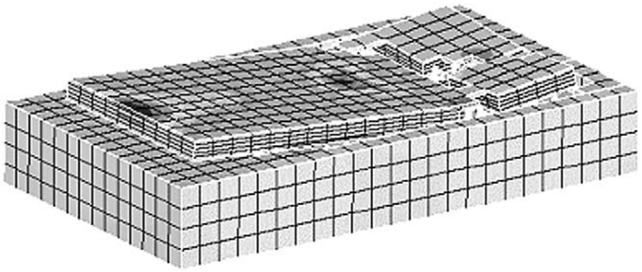

Fig. 3. Pressure contours after the first blast (4 kg of TNT 0.5 m over the slab).

For the calibration of the erosion model for masonry, data found in the bibliography on the pressure and impulse that produce damage and demolition of masonry walls were used [1,2,4].

3. Blast analysis

3.1. Introduction

The analysis of the structural collapse of the building was performed in two stages. The first part of the analysis consists on the simulation of the explosion itself from the detonation instant [34] and the second part consists on the analysis of the effect and interaction with the building of the blast wave generated by the explosion. Only the load produced by the air blast wave was considered in the analysis. The ground motion generated by the explosion was not taken into account because the explosive was supposed to be placed 1 m above the ground level, for example on a vehicle, isolated from the ground and so the ground shock can be diminished [4,35].

Fig. 4. Mesh used for the generation of the explosion (400 kg TNT).

3.2. Explosion generation

The initiation of the explosion and the propagation of the blast wave in the air in contact with the explosive were first simulated. For this purpose, it was assumed that, until the blast wave encounters a rigid surface, the problem has spherical symmetry and can be treated as 1D. The results of this analysis were later mapped in the 3D model representing the building and the surrounded air volume [9]. In this way, the computational cost of the analysis was drastically reduced.

The initial detonation and expansion of the sphere of 400 kg of TNT was modeled in a 1D, spherically symmetric model of 1 m radius with a JWL equation of state, as illustrated in Fig. 4 [9,34]. Part way through the detonation process, and to avoid numerical errors, the material model for the high explosive was modified. The 1D expansion analysis continues until just prior to impingement of the blast wave on the rigid surface.

The TNT and air material data are shown in Table 5. The default density gave a radius 388.4 mm for a 400 kg spherical charge of TNT. Fig. 5 illustrates the remapping of the 1D analysis to the 3D model of the building.

3.3. Structural analysis

To study the structural behaviour of the building, the propagation of the blast wave and its interaction with the building was analysed. For that purpose, an interaction algorithm between the Lagrange (building) and Euler (air) processors was used [9].

As illustration of the role played by the interaction of the blast wave with the building, the propagation of the blast wave in a building without walls is compared with the propagation of the same blast wave in the actual building with walls in Fig. 6. The velocity field is represented in the air and pressure contours are plotted on the solids. It may be noted the alteration of the blast wave produced as a result of the multiple reflections on the walls, even though they were brittle elements. Because of the reflections on the walls, the blast wave lost its spherical shape and increased its destructive effect in vertical direction.

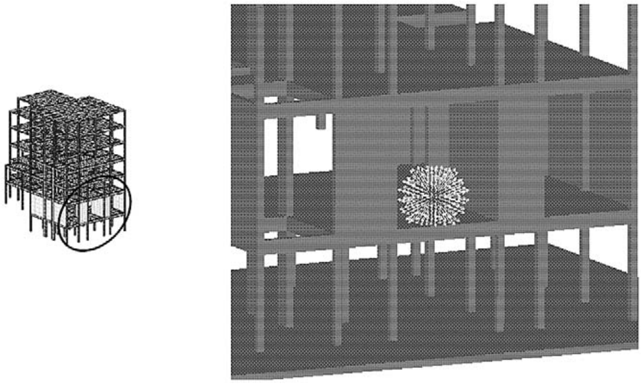

Fig. 5. Remapping to the 3D model (0.2 ms).

In the analysis of the building collapse, the solid–solid interaction [9] between the different parts of the structures that fell down was also taken into account.

4. Results

The results obtained for an explosive load of 400 kg of TNT located 1 m above the ground level, 1 m inside the entrance hall and 1 m to the right of the axis of the building are shown in Fig. 7. The magnitude of the explosive load and its location were obtained from a previous analysis described in Ref. [34]. For the sake of visualization, only the front and the intermediate blocks of the building are shown. The first moments following the detonation are shown in Fig. 7a. The erosion of most of the columns and walls in the first stage and the underground stage, in the front block, is showed in Fig. 7b. The failure of the highest slab and the loosening of the front block are clear in Fig. 7c. The free fall of the front

Table 5
Material properties for the explosion generation [9]

Material	Air	TNT	TNT (ideal)
Equation of state	Ideal gas $\gamma = 1.4$ $\rho = 1.225 \times 10^{-3}$ g/cm^3 Ref. energy = 0.0 mJ Press. shift = 0.0 kPa	JWL Standard Library data	Ideal gas $\gamma = 1.35$ $\rho = 1.0 \times 10^{-4}$ g/cm^3 Ref. energy = 0.0 mJ Press. shift = 0.0 kPa
Initial conditions	$\rho = 1.225 \times 10^{-3}$ g/cm^3 Ref. energy = 2.068×10^5 mJ/mg	Default	From detonation model/remap data

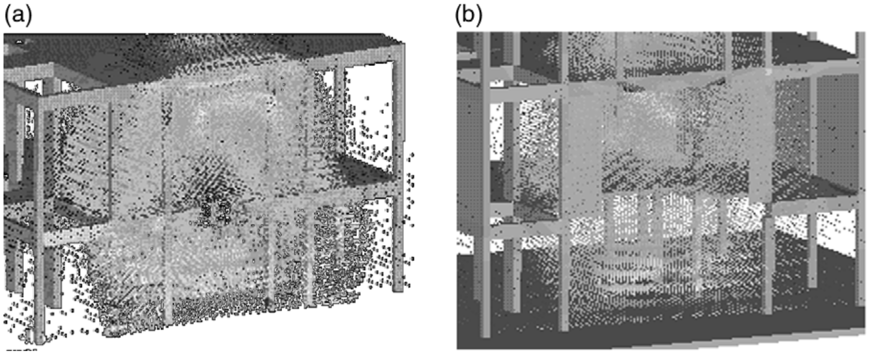

Fig. 6. Effect of the walls in the blast wave propagation. (a) Building without walls, (b) building with walls.

block can be seen in Fig. 7d. The final state of the collapsed structure can be observed in Fig. 7e.

The run time necessary for the analysis was approximately 310 h in a system with a Pentium IV processor and 500 MHz RIMM memory.

4.1. Collapse mechanism

The structural collapse was due to a gravitational mechanism produced by the failure of most of the load bearing columns of the first stage in the front block. See Fig. 7.

The four columns closer to the explosion focus failed due to the direct effect of the reflected pressure that produced the erosion of concrete by brisance effect (see Fig. 7a). Lateral columns in the front failed by a combination of shear and flexure. Columns at the back of the front block lost connection with the upper and lower beams (tension failure), due to the tension effect imparted by the first floor slabs that were pushed upwards and downwards by the pressure and then failed by lack of lateral support when the pressure wave reached them.

The pressure destroyed the ground floor just under the explosion (see Fig. 7a) allowing the blast wave to pass to the underground level and destroy some columns (see Fig. 7b).

The first three lines of columns of the underground level and the first level resulted almost completely destroyed (Fig. 7b) leaving without support the upper floors that began to fall down pulling from the back block (Fig. 7c). Beams and columns in the upper floors of the intermediate block have failed by a combination of tension, flexure and shear.

On the other side, the blast wave propagated upwards in the intermediate block, limiting the advance of damage to the back block. See Fig. 7b.

Both effects produced an inclined failure advancing backwards in height. See Fig. 7e. The front block lost connection with the rest of the structure along this inclined line and immediately began to fall down. The slabs impacted one over the other destroying themselves.

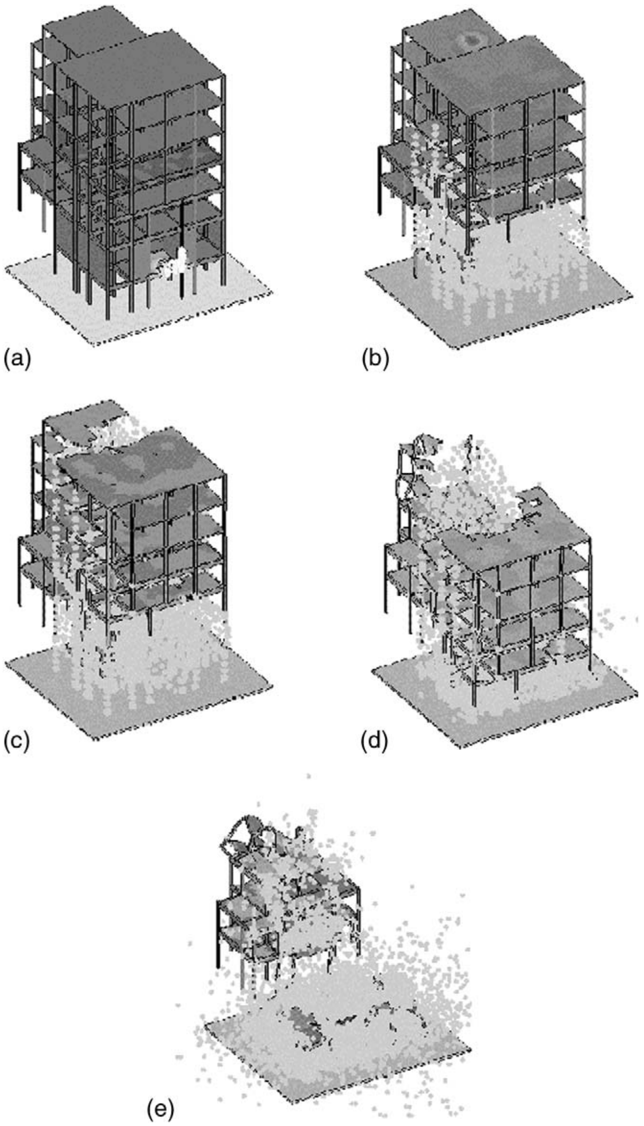

Fig. 7. Evolution of damage produced by the explosion. (a) 0.75 ms, (b) 254 ms, (c) 378 ms, (d) 1.35 ms and (e) 2.46 ms.

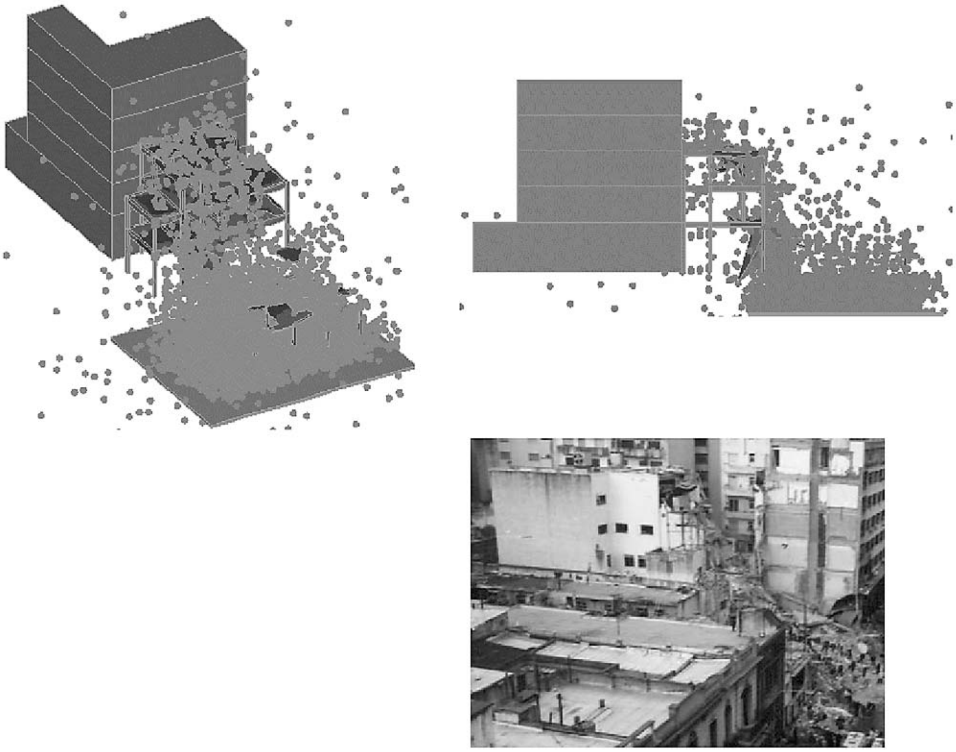

Fig. 8. Comparison with actual damage.

4.2. Comparison with actual damage

Comparisons of the numerical simulation results with photographs of the actual damage produced by the explosion are presented in Figs. 8–11. Fig. 8 shows similarity between the final state of model and the distribution of the remains of the demolition with those registered in the actual building. Fig. 9 shows that the numerical simulation reproduces the fall of the front slabs that resulted hanging from the back part of the building. Fig. 10 presents a view of the reinforced concrete frames that remained after the explosion. It may be noted the coincidence with the photograph taken after removing the rests of the demolition. The line limiting the destroyed part of the actual building is drawn on the picture obtained from the numerical simulation of the collapse in Fig. 11. It can be observed that the numerical analysis reproduces the limit of the destruction zone in accordance with the actual damage observed.

5. Conclusions

The analysis of the structural failure of a reinforced concrete building caused by a blast load is presented in

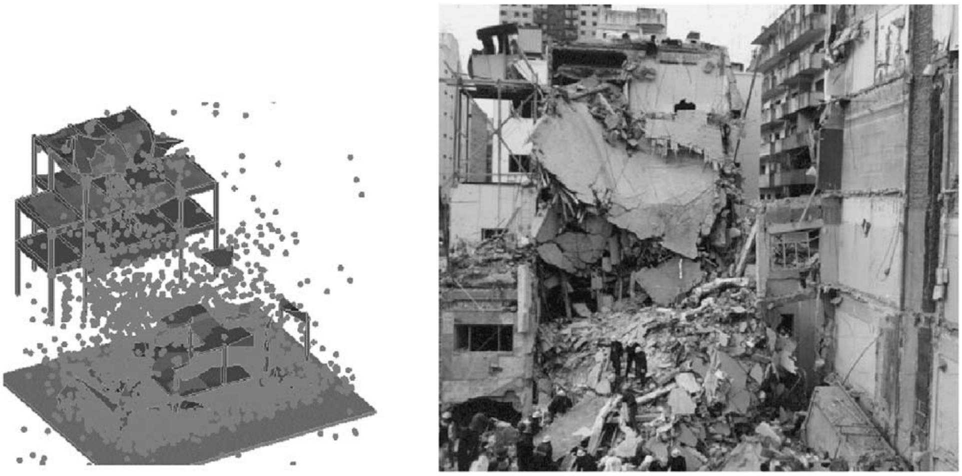

Fig. 9. Slabs hanging from the higher stages.

Fig. 10. Remaining frames.

Fig. 11. Limit of the destruction zone.

this paper. All the process from the detonation of the explosive charge to the complete demolition, including the propagation of the blast wave and its interaction with the structure is reproduced.

The comparison of numerical results with photographs shows that the numerical analysis accurately reproduces the collapse of the building under the blast load confirming the location and magnitude of the explosion previously established based on other analysis.

The good agreement between actual damage and that one numerically obtained proves that the simplifying assumptions made for the structure and materials are allowable for this type of analysis and nowadays represent the only way to successfully run a complete collapse analysis of an entire building.

The collapse was due to a gravitational mechanism originated by the destruction of the lower columns. In this case, the explosive charge was determined based on other data, but the demolition of the front block of the building analysed could have been produced with a smaller charge.

The type of analysis presented can be used for structure vulnerability assessment in order to choose structural configurations that prevent damage to extend damage beyond that caused directly by the blast.

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