

外文原文

Response of a reinforced concrete infilled-frame structure to removal of two adjacent columns

Mehrdad Sasani

Northeastern University, 400 Snell Engineering Center, Boston, MA 02115, United States

Received 27 June 2007; received in revised form 26 December 2007; accepted 24 January 2008

Available online 19 March 2008

Abstract

The response of Hotel San Diego, a six-story reinforced concrete infilled-frame structure, is evaluated following the simultaneous removal of two adjacent exterior columns. Analytical models of the structure using the Finite Element Method as well as the Applied Element Method are used to calculate global and local deformations. The analytical results show good agreement with experimental data. The structure resisted progressive collapse with a measured maximum vertical displacement of only one quarter of an inch (6.4 mm). Deformation propagation over the height of the structure and the dynamic load redistribution following the column removal are experimentally and analytically evaluated and described. The difference between axial and flexural wave propagations is discussed. Three-dimensional Vierendeel (frame) action of the transverse and longitudinal frames with the participation of infill walls is identified as the major mechanism for redistribution of loads in the structure. The effects of two potential brittle modes of failure (fracture of beam sections without tensile reinforcement and reinforcing bar pull out) are described. The response of the

structure due to additional gravity loads and in the absence of infill walls is analytically evaluated.

© 2008 Elsevier Ltd. All rights reserved.

Keywords : Progressive collapse; Load redistribution; Load resistance; Dynamic response; Nonlinear analysis; Brittle failure

1. Introduction

The principal scope of specifications is to provide general principles and computational methods in order to verify safety of structures.

The “ safety factor ” , which according to modern trends is independent of the nature and combination of the materials used, can usually be defined as the ratio between the conditions. This ratio is also proportional to the inverse of the probability (risk) of failure of the structure.

Failure has to be considered not only as overall collapse of the structure but also as unserviceability or, according to a more precise. Common definition. As the reaching of a “ limit state ” which causes the construction not to accomplish the task it was designed for. There are two categories of limit state :

(1)Ultimate limit state, which corresponds to the highest value of the load-bearing capacity. Examples include local buckling or global instability of the structure; failure of some sections and subsequent transformation of the structure into a mechanism; failure by fatigue; elastic or plastic deformation or creep that cause a substantial change of the geometry of the structure; and sensitivity of the structure to alternating loads, to fire and to explosions.

(2)Service limit states, which are functions of the use and durability

y of the structure. Examples include excessive deformations and displacements without instability; early or excessive cracks; large vibrations; and corrosion.

Computational methods used to verify structures with respect to the different safety conditions can be separated into:

(1) Deterministic methods, in which the main parameters are considered as nonrandom parameters.

(2) Probabilistic methods, in which the main parameters are considered as random parameters.

Alternatively, with respect to the different use of factors of safety, computational methods can be separated into:

(1) Allowable stress method, in which the stresses computed under maximum loads are compared with the strength of the material reduced by given safety factors.

(2) Limit states method, in which the structure may be proportioned on the basis of its maximum strength. This strength, as determined by rational analysis, shall not be less than that required to support a factored load equal to the sum of the factored live load and dead load (ultimate state).

The stresses corresponding to working (service) conditions with unfactored live and dead loads are compared with prescribed values (service limit state). From the four possible combinations of the first two and second two methods, we can obtain some useful computational methods. Generally, two combinations prevail:

(1) deterministic methods, which make use of allowable stresses. (2

) Probabilistic methods, which make use of limit states.

The main advantage of probabilistic approaches is that, at least in t

theory, it is possible to scientifically take into account all random factors of safety, which are then combined to define the safety factor. Probabilistic approaches depend upon :

(1) Random distribution of strength of materials with respect to the conditions of fabrication and erection (scatter of the values of mechanical properties through out the structure); (2) Uncertainty of the geometry of the cross-section and of the structure (faults and imperfections due to fabrication and erection of the structure); (3) Uncertainty of the predicted live loads and dead loads acting on the structure; (4) Uncertainty related to the approximation of the computational method used (deviation of the actual stresses from computed stresses). Furthermore, probabilistic theories mean that the allowable risk can be based on several factors, such as :

(1) Importance of the construction and gravity of the damage by its failure; (2) Number of human lives which can be threatened by this failure; (3) Possibility and/or likelihood of repairing the structure; (4) Predicted life of the structure. All these factors are related to economic and social considerations such as :

(1) Initial cost of the construction;
(2) Amortization funds for the duration of the construction;
(3) Cost of physical and material damage due to the failure of the construction;
(4) Adverse impact on society;
(5) Moral and psychological views.

The definition of all these parameters, for a given safety factor, allows construction at the optimum cost. However, the difficulty of carrying out a complete probabilistic analysis has to be taken into account.

count. For such an analysis the laws of the distribution of the live load and its induced stresses, of the scatter of mechanical properties of materials, and of the geometry of the cross-sections and the structure have to be known. Furthermore, it is difficult to interpret the interaction between the law of distribution of strength and that of stresses because both depend upon the nature of the material, on the cross-sections and upon the load acting on the structure. These practical difficulties can be overcome in two ways. The first is to apply different safety factors to the material and to the loads, without necessarily adopting the probabilistic criterion. The second is an approximate probabilistic method which introduces some simplifying assumptions (semi-probabilistic methods). As part of mitigation programs to reduce the likelihood of mass casualties following local damage in structures, the General Services Administration [1] and the Department of Defense [2] developed regulations to evaluate progressive collapse resistance of structures. ASCE/SEI 7 [3] defines progressive collapse as the spread of an initial local failure from element to element eventually resulting in collapse of an entire structure or a disproportionately large part of it. Following the approaches proposed by Ellinwood and Leyendecker [4], ASCE/SEI 7 [3] defines two general methods for structural design of buildings to mitigate damage due to progressive collapse: indirect and direct design methods. General building codes and standards [3,5] use indirect design by increasing overall integrity of structures. Indirect design is also used in DOD [2]. Although the indirect design method can reduce the risk of progressive collapse [6,7] estimation of post-failure performance of structures designed

based on such a method is not readily possible. One approach based on direct design methods to evaluate progressive collapse of structures is to study the effects of instantaneous removal of load-bearing elements, such as columns. GSA [1] and DOD [2] regulations require removal of one load bearing element. These regulations are meant to evaluate general integrity of structures and their capacity of redistributing the loads following severe damage to only one element. While such an approach provides insight as to the extent to which the structures are susceptible to progressive collapse, in reality, the initial damage can affect more than just one column. In this study, using analytical results that are verified against experimental data, the progressive collapse resistance of the Hotel San Diego is evaluated, following the simultaneous explosion (sudden removal) of two adjacent columns, one of which was a corner column. In order to explode the columns, explosives were inserted into predrilled holes in the columns. The columns were then well wrapped with a few layers of protective materials. Therefore, neither air blast nor flying fragments affected the structure.



Fig. 1. A south view of hotel San Diego. Center structure is studied in this paper.



Fig. 2. Second floor of building (Looking south).

2. Building characteristics

Hotel San Diego was constructed in 1914 with a south annex added in 1924. The annex included two separate buildings. Fig. 1 shows a south view of the hotel. Note that in the picture, the first and third stories of the hotel are covered with black fabric. The six story hotel had a non-ductile reinforced concrete (RC) frame structure with hollow clay tile exterior infill walls. The infills in the annex consisted of two withes (layers) of clay tiles with a total thickness of about 8 in (203 mm). The height of the first floor was about 190 -800 (6.00 m). The height of other floors and that of the top floor were 100 -600 (3.20 m) and 160 -1000 (5.13 m), respectively. Fig. 2 shows the second floor of one of the annex

buildings. Fig. 3 shows a typical plan of this building, whose response following the simultaneous removal (explosion) of columns A2 and A3 in the first (ground) floor is evaluated in this paper. The floor system consisted of one-way joists running in the longitudinal direction (North-South), as shown in Fig. 3. Based on compression tests of two concrete samples, the average concrete compressive strength was estimated at about 4500 psi (31 MPa) for a standard concrete cylinder. The modulus of elasticity of concrete was estimated at 3820 ksi (26 300 MPa) [5]. Also, based on tension tests of two steel samples having 1/2 in (12.7 mm) square sections, the yield and ultimate tensile strengths were found to be 62 ksi (427 MPa) and 87 ksi (600 MPa), respectively. The steel ultimate tensile strain was measured at 0.17. The modulus of elasticity of steel was set equal to 29 000 ksi (200 000 MPa). The building was scheduled to be demolished by implosion. As part of the demolition process, the infill walls were removed from the first and third floors. There was no live load in the building. All nonstructural elements including partitions, plumbing, and furniture were removed prior to implosion. Only beams, columns, joist floor and infill walls on the peripheral beams were present.

3. Sensors

Concrete and steel strain gages were used to measure changes in strains of beams and columns. Linear potentiometers were used to measure global and local deformations. The concrete strain gages were 3.5 in (90 mm) long having a maximum strain limit of ± 0.02 . The steel strain gages could measure up to a strain of ± 0.20 . The

strain gages could operate up to a several hundred kHz sampling rate. The sampling rate used in the experiment was 1000 Hz. Potentiometers were used to capture rotation (integral of curvature over a length) of the beam end regions and global displacement in the building, as described later. The potentiometers had a resolution of about 0.0004 in (0.01 mm) and a maximum operational speed of about 40 in/s (1.0 m/s), while the maximum recorded speed in the experiment was about 14 in/s (0.35 m/s).

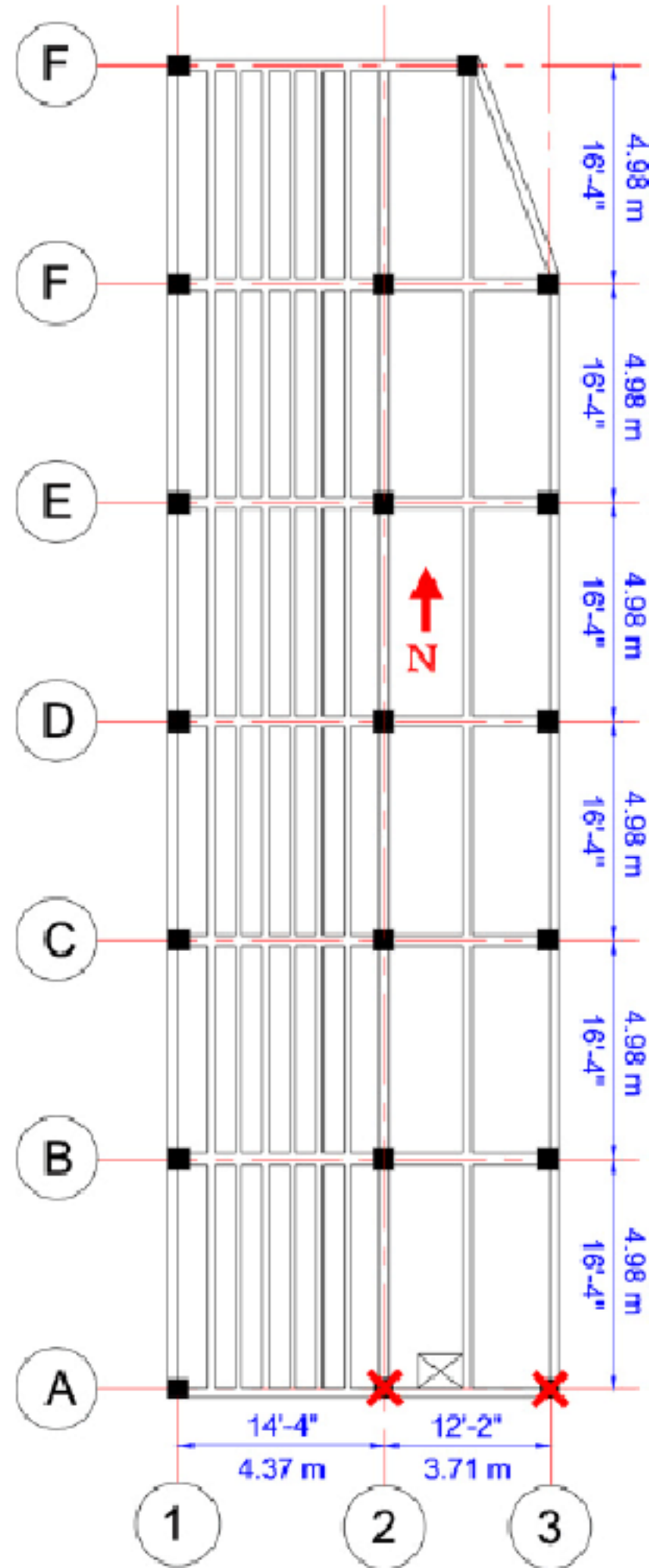


Fig. 3. Typical plan of Hotel San Diego (South Annex). First floor removed columns are crossed.

以上内容仅为本文档的试下载部分，为可阅读页数的一半内容。如要下载或阅读全文，请访问：<https://d.book118.com/058034120067006120>