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EARTHQUAKE RESISTANCE AND BLAST RESISTANCE: A STRUCTURAL COMPARISON

John R. HAYES, Jr.[1]; Stanley C. WOODSON[2]; Robert G. PEKELNICKY[3]; Chris D. POLAND[4]; W. Gene CORLEY[5]; Mete SOZEN[6]; Michael MAHONEY[7] and Robert D. HANSON[8]

SUMMARY

Several sources have suggested that current seismic design provisions can improve blast and progressive collapse resistance. To examine this suggestion, the Federal Emergency Management Agency (FEMA) of the U.S. Department of Homeland Security (DHS) sponsored a study at the U.S. Army Engineer Research and Development Center (ERDC). The Alfred P. Murrah Federal Building, which was severely damaged in a 1995 bombing, was hypothetically strengthened for high seismic demands. Three strengthening schemes were designed, and each strengthening scheme was then analyzed for its response to the 1995 bombing scenario. The blast and corresponding progressive collapse analyses showed that the pier-spandrel and special moment frame schemes would significantly reduce the amount of blast-induced damage and subsequent progressive collapse, compared with the response of the original building. The internal shear walls were less effective in reducing blast and progressive collapse damage. It was concluded that strengthening perimeter elements using current seismic detailing techniques improved the survivability of the building from blast loading.

INTRODUCTION

There is interest in knowing how effectively current seismic detailing provisions in model building codes improve resistance to other abnormal loading, such as blast loads and progressive collapse. The 1995 Oklahoma City Bombing severely damaged the Alfred P. Murrah Federal Building ("Murrah Building") and raised U.S. public awareness of blast hazards for public buildings. The FEMA Building Performance Assessment Team Report, FEMA 277 [1], documented the Murrah Building bombing, reported on the cause of the collapse of the building, and suggested how construction practices might be improved to

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[1] Structural Engineer, US Army Engineer Research and Development Center, Champaign, IL, USA.
[2] Structural Engineer, US Army Engineer Research and Development Center, Vicksburg, MS, USA.
[3] Structural Engineer, Degenkolb Engineers, San Francisco, CA, USA.
[4] Structural Engineer and CEO, Degenkolb Engineers, San Francisco, CA, USA.
[5] Senior Vice-President, Construction Technology Laboratories, Skokie, IL, USA.
[6] Kettelhut Distinguished Professor of Structural Engineering, School of Civil Engineering, Purdue University, West Lafayette, IN, USA.
[7] Senior Geophysicist, US Department of Homeland Security, Federal Emergency Management Agency, Washington, DC, USA
[8] Professor Emeritus, University of Michigan, Ann Arbor, MI, USA. Email: RDHanson2@aol.com

prevent a reoccurrence of the disaster. Among the suggestions was that constructing the building using modern seismic design details would have reduced damage and catastrophic collapse. To begin exploring this issue, FEMA sponsored a study with ERDC that is summarized in this paper to consider the potential benefits of seismic strengthening on blast and progressive collapse resistance.

This study used the Murrah Building because of its known performance in the 1995 bombing. The building was constructed of reinforced concrete, with a nine-story Ordinary Moment Frame (OMF) gravity-load system and a shear wall system for resisting lateral wind loads. Building plan dimensions were approximately 67 m, east-west, and 30.5 m, north-south. The OMF contained ten 6.1 m bays spanning east-west and two 10.7 m column bays spanning north-south. The building was designed in accordance with ACI 318-71 [2] for Oklahoma City, a nonseismic area.

Figure 1 Street face of Murrah Building before 1995 attack.

Figure 1 shows the street face of the Murrah Building prior to the 1995 attack. Figure 2 shows the 3rd floor plan. It typifies the floor plans of all upper floors in the building. The OMF was designed for gravity-load resistance. The elevator and stair cores of the building had 30 cm walls that resisted lateral loads, as did the air ventilation shaft walls at the four corners of the building. None of these elements is near the point on the street where the truck bomb was detonated (near Column G20 on the street face). The ground floor was a slab on grade.

On the street face, Column Line G, the OMF had widely-spaced columns, 12.2 m on-center, extending to the 3rd floor level. Above that, the column spacing was 6.1 m on center. Cross-sectional dimensions of the ground floor columns on Column Line G were 510 mm x 910 mm. Dimensions for columns above that level were typically 41 cm x 61 cm. Other cross-sectional dimensions were: 3rd floor level transfer girder, 910 mm x 1520 mm; girders on floors 4 through 8, 460 mm x 890 mm; and, roof girder, 460 mm x 1190 mm.

Overview of Observed 1995 Response

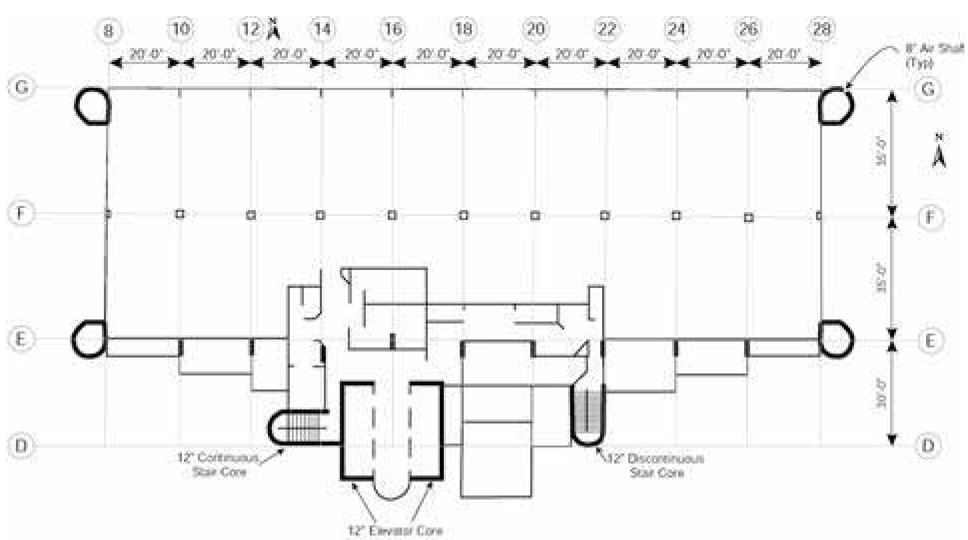

Figure 2 3rd Floor level of original building.

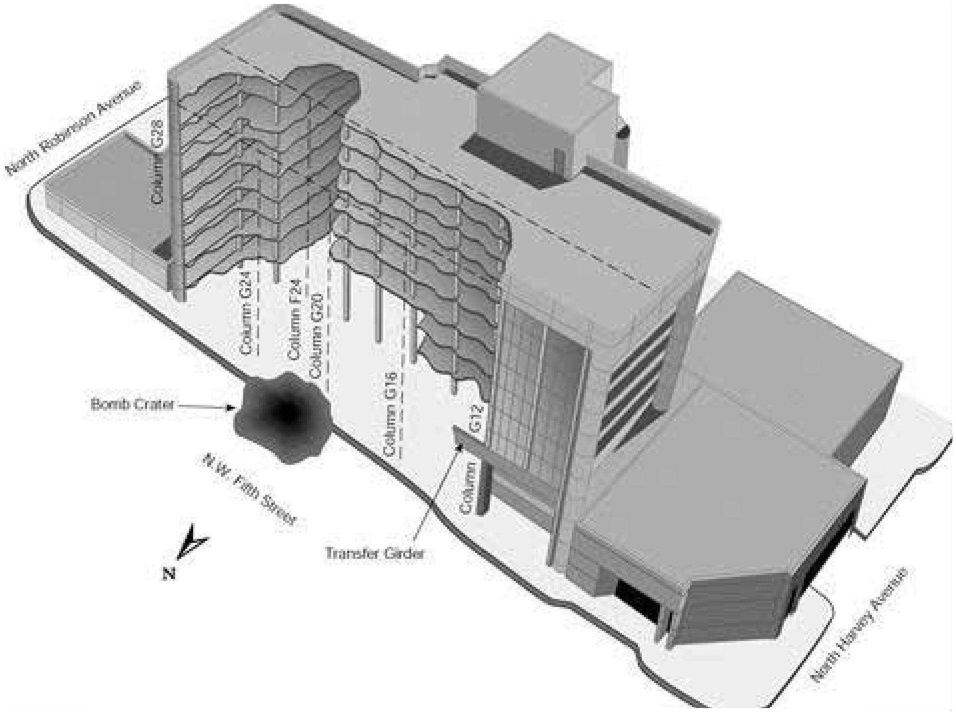

Figure 3 Post-bombing damage assessment, FEMA 277 [1].

The 1995 attack involved the detonation of explosives in a truck parked near Column G20 on the street running by the building. The bomb is estimated to have contained the explosive equivalent of 1,820 kg of TNT. The truck bed height was approximately 1.4 m above the street pavement level, and its center was less than 4.3 m from the building face, northeast of Column G20. Figure 3 shows structural failure boundaries that were noted in post-attack surveys.

The FEMA 277 [1] report describes the blast damage and ensuing progressive collapse in the actual bombing. Column G20 was destroyed. Column G24 underwent blast-induced shear failure. Column G16 was in a condition of incipient blast-induced shear failure. With no ground story columns at positions G18 and G22 and the failure of columns G16, G20, and G24, there was an unsupported length of 24.4 m (80 ft) of the large transfer girder at the 3rd floor level.

The 1995 investigation also determined that many of the floor slabs between Column Lines 18 and 24 on the 2nd through 5th floor levels were destroyed by blast effects. Because there was no top reinforcement at the slab mid-spans, they were vulnerable to uplift as the air blast propagated up and out. It is estimated that approximately 223 SM of the 2nd floor; 195 SM of the 3rd floor; 98 SM of the 4th floor; and, 28 SM of the 5th floor area were directly destroyed by the blast. The total area of each floor level was 1,412 SM. Floor slab losses severely weakened adjacent remaining primary structural elements, because of decreased lateral support and diaphragm capacity.

Even without blast-induced damage to the transfer girder near Column G20, the flexural capacity and axial tension capacities of the girder would have been insufficient for it to transfer the gravity-loads from Columns G16 through G24 to Columns G12 and G28. With collapse of the transfer girder, both the surviving floor slabs and the 3rd floor level beams that spanned from the Column Line F to the Column Line G between Columns G16 and G24 collapsed, pulling down slabs and columns from floors above the 3rd floor. All of the resulting debris fell onto the surviving slabs and the 2nd floor level.

Approximately 42% of the original building floor area was destroyed by blast-induced damage or the resulting progressive collapse. It is estimated that 10% of the damage was blast-induced, while 90% of the damage was progressive collapse-induced.

Approach Adopted in Study

The study analyzed the possible improvement in blast and progressive collapse resistance that would be provided by applying details used in areas of high seismicity. The original Murrah Building was artificially "re-sited" to an area of high seismicity. Its original structural configuration was evaluated for earthquake resistance, and strengthening measures to improve its earthquake resistance were designed. The strengthening measures were then analyzed for their blast and progressive collapse resistance. Strengthening measures that were included in this study and the analyses of the blast and progressive response mechanisms that would have developed in the 1995 bombing scenario are summarized. The paper concludes with observations about the effectiveness of the strengthening systems in reducing the degree of structural collapse from that which occurred in the 1995 bombing.

SEISMIC EVALUATION

A Life Safety seismic evaluation was performed based on the assumption that the Murrah Building was located in a region of very high seismicity. The evaluation procedures of ASCE 31-02 [3], which is based on FEMA 310 [4], were used. The building was assumed to be at a location in downtown San Francisco, CA, with soil conditions similar to those at the original Oklahoma City site, but with the higher potential ground motions found in CA. Site-specific response spectra were developed for the evaluation and subsequent design work, using *ST-Risk*™ software [5] for the BSE-1 (10% probability of being exceeded in 50 years) and BSE-2 (2% probability of being exceeded in 50 years) earthquakes stipulated in FEMA 356 [6]. The evaluation included a Tier 1 checklist screening, a Tier 2 evaluation using linear static analyses of the building, and a Tier 3 evaluation using lateral load limit (nonlinear static "pushover") analyses of the building. The seismic evaluation showed that the building was deficient primarily because of poor column reinforcement lap splice details, negative post-yield stiffness due to the absence of seismic detailing, and torsional irregularities caused by the asymmetric shear wall layout. A lack of seismic detailing resulted in poor confinement of structural elements and reduced member shear capacity.

STRENGTHENING TO IMPROVE EARTHQUAKE PERFORMANCE

Both longitudinal and transverse deficiencies were addressed. One transverse strengthening scheme was developed, adding lightly reinforced 300 mm reinforced concrete shear walls at the east and west ends of the building, Column Lines 8 and 28. While only about ¾ of the total end wall length between the air shaft walls was needed for required strength and stiffness, each wall was extended the full length between the air shafts, so that existing foundation elements could be utilized, reducing foundation construction cost.

Three longitudinal strengthening schemes were developed, following FEMA 356 [6] guidelines. All schemes used new reinforced concrete elements designed in accordance with ACI 318-02 [7]. Two schemes focused on strengthening the OMF by adding new structural systems to the street face (Column Line G). The first scheme was a large pier-spandrel system. The second scheme was a new ductile special moment frame (SMF) system. The third scheme added a shear wall system within the building interior. The strengthening schemes were developed without consideration for blast resistance. In each design, limiting base shears were determined for both modal response spectrum-based and uniform lateral story force distributions, using nonlinear static analysis procedures.

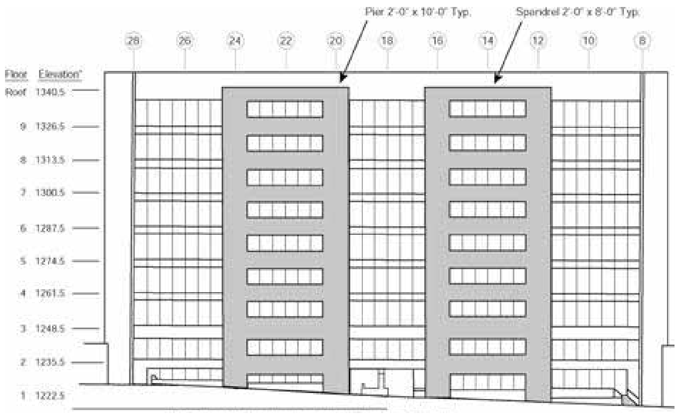

Figure 5 Elevation, pier-spandrel strengthening scheme.

The pier-spandrel longitudinal strengthening scheme added two new shear walls to the street side of Column Line G. These walls have large openings at each floor level, preserving most of the original window openings. The pier-spandrel system is shown in elevation in Figure 5. The new elements are 610 mm thick. Piers are 3.05 m wide and spandrel elements are 2.44 m tall. The pier-spandrel system would be connected to the original Column Line G OMF using dowels, ensuring interaction for seismic response. The pier-spandrel system would be founded on existing column caissons.

The SMF longitudinal strengthening scheme added a new reinforced concrete frame to the street face of Column Line G between Columns G10 and G26, extending the full building height. The SMF system is shown in elevation in Figure 6. The new SMF is also 610 mm thick. Columns in this frame are typically 1.22 m wide. For the 2nd through 8th floor levels, girders are 1.22 m tall; at the 9th floor and roof levels, girders are 910 mm tall. The SMF would be connected to the original Column Line G OMF using dowels, again ensuring interaction for seismic response. It would be founded on existing column caissons.

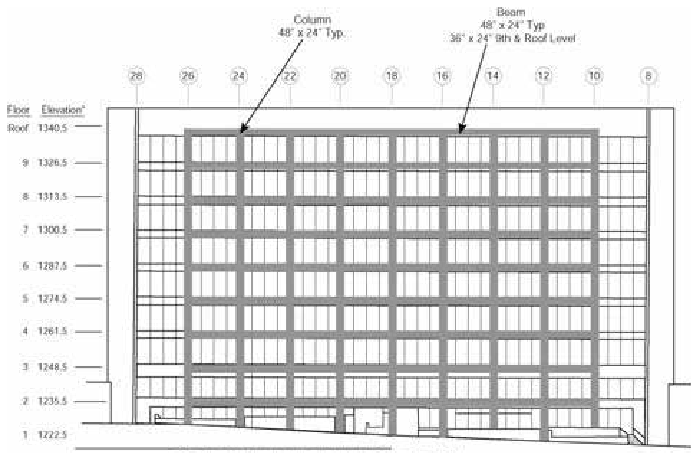

Figure 6 Elevation, special moment frame strengthening scheme.

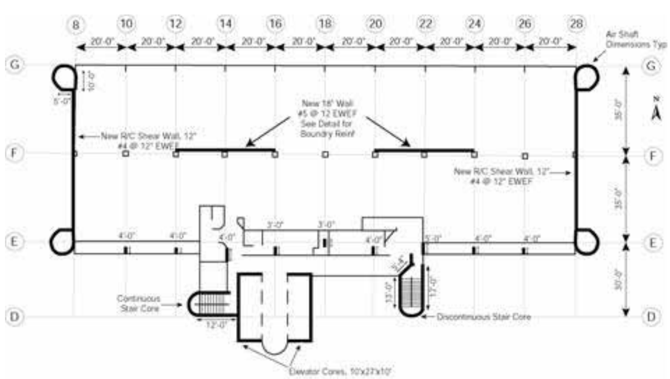

Figure 7 Plan, internal shear walls, Line F.

The last longitudinal strengthening scheme used two shear walls in the building interior. The walls are lightly reinforced, 460 mm thick walls that extend the full building height between Columns F14 and F16 and Columns F20 and F24. Each wall is 12.2 m long. An interior location for the shear walls was selected because solid walls on the street face of the building were believed to be architecturally undesirable. Figure 7 shows a potential Column Line F location for the interior shear wall strengthening scheme. Relatively light wall reinforcement is augmented by special boundary reinforcement elements at the ends of each wall, to improve post-yield response in earthquakes. The walls could be founded on existing column caissons.

An alternate location for the walls, using the same thickness and reinforcement, half way between Column Lines F and G, was also analyzed. This line was coined Line "F.5" in the study. This option was analyzed to assist in determining whether a conscious decision to move such a wall closer to the exposed street face, to assist with blast protection, would be cost-effective. A wall on Line F.5 would require the construction of new pier foundations.

All earthquake response analyses were performed using *SAP2000*® [8]. Figure 8 provides a comparison of calculated roof drift vs. base shear curves for the longitudinal strengthening schemes. The curve for the pier-spandrel option is essentially the same as that for the SMF option, because the pier-spandrel stiffness was emulated in the SMF option.

BLAST RESPONSE

Blast Analysis Programs

Blast response analysis software like that used in the development of FEMA 277 was used. The software incorporated post-1995 updates. Software used for blast analyses includes *ConWep*, *SPAN32*, and *WAC*. They were developed by the U.S. Army Corps of Engineers and have been calibrated against a large experimental database, to support protective structure design.

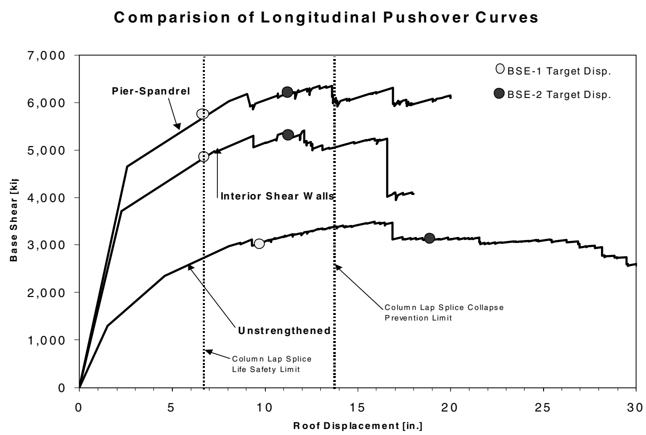

Figure 8 Longitudinal pushover analysis curves.

ConWep was developed as a computerized version of Army Technical Manual 5-855-1 [9]. *ConWep* was used to predict blast loads on structural elements. *ConWep* forms a grid on reflecting surfaces and, for each grid point, calculates the angle of incidence; incident, or side-on, pressure and impulse; and reflected pressure and impulse, which is a function of the angle of incidence and incident pressure. For a finite reflecting surface, *ConWep* accounts for air blast clearing time. The analysis target area may be smaller than the total reflecting surface. *ConWep* also develops an equivalent uniform load for the target based on its assumed deformed shape, using flexural yield lines. Equivalent uniform pressure and impulse values are useful for developing load-time functions for single-degree-of-freedom (SDOF) dynamic response analysis.

In addition, *ConWep* also provides breaching predictions. “Breaching” is a term for the shattering or crushing effects from the sudden release of energy in an explosion. If breaching occurs, a portion of the affected element is severely damaged and reduced to rubble. *ConWep* determines breaching failure using empirical procedures presented in ESL-TR-87-57 [10] and calculates the minimum distance from an explosive source to a target that is required to avoid shattering of concrete elements.

SPAN32 [11] and *WAC* [12] are similar SDOF system response programs. *WAC* analyzes masonry walls, reinforced concrete elements, and elements with user-defined resistance functions. Arching action (compressive membrane enhancement of capacity) may be included in calculating the resistance function for unreinforced walls that do not contain windows. *WAC* calculates the flexural resistance function (load-deformation behavior) of an element, given its construction details, then transforms the multiple-degree-of-freedom (MDOF) resistance into an “equivalent” SDOF model by the use of transformation factors based on assumed deformed shapes for the structure. Transformation factors are available for beams, one-way slabs, and two-way slabs. Actual and SDOF-equivalent (transformed) loads are calculated, given the weight of explosive and range to from explosive to target. A user-defined loading may also be used.

SPAN32 analyzes both steel and reinforced concrete elements, allows user definition of member properties, and computes member resistance functions. *SPAN32* allows input of the loading function and performs an SDOF analysis; it does not compute loads. In both SPAN32 and WAC, the equation of motion is solved by numerical integration (central difference technique) to determine the dynamic response of a critical point on a wall element (usually at mid-height and mid-width).

Pier-Spandrel System Response

Table 1 summarizes reflected air blast loads, corresponding maximum mid-span deflections, and damage mechanisms for all severely impacted elements in the pier-spandrel system. In Tables 1-3, where ranges of blast load magnitude are listed, the range indicates the variation in peak pressure over the element surface, from a point nearest the bomb to a point furthest from the bomb. The listed displacements are averages for assumed fixed and simple end moment restraint conditions. It is likely that actual moment restraints at member supports lie somewhere between total fixity and simple support.

The rear edge of pier G20 is only 4.3 m from the bomb. *ConWep* predicts shattering of the 600 mm pier thickness at a range of 5.1 m or less from the bomb. The pier G20 will therefore be shattered, extending from the ground into the new 2nd floor spandrel beam that spans between Columns G20 and G24.

The ground level portion of pier G24 is far enough from the bomb to avoid being shattered. Its flexural response to the blast pressure pulse was analyzed. The original 3rd floor transfer girder is thicker than the original columns, protruding 150 mm out from the plane of the original column faces. The gap between the pier and original column would be filled by new concrete and dowels. Original Column G24 thus provides lateral support for the new pier, so the pier was modeled as a wall, with each half of it considered to be fixed at the original column line and cantilevering horizontally from the line. That half of the pier close to the bomb was analyzed, with a 6.4 m clear height, spanning from the ground level to the 3rd floor transfer girder. Peak reflected pressures range from approximately 2.1 MPa to 3.2 MPa. The reflected pressure distribution is shown in Figure 9 (units shown in psi).

Table 1 Significant blast loading response, pier-spandrel system.

PIERS				
Story Level	**Column Line**	**Max. Reflected Pressure (MPa)**	**Displacement (mm)**	**Description**
Ground (1-2)	G20	48-117	N/A	Shattered
Ground (1-2)	G24	2-3	10	Mid-Height Flexural Displacement
3	G20	2-4	10	Mid-Height Flexural Displacement
SPANDRELS				
Floor Level	**Between Columns**	**Max. Reflected Pressure (MPa)**	**Displacement (mm)**	**Description**
2	G20-G24	2-61	N/A	Shattered @ G20

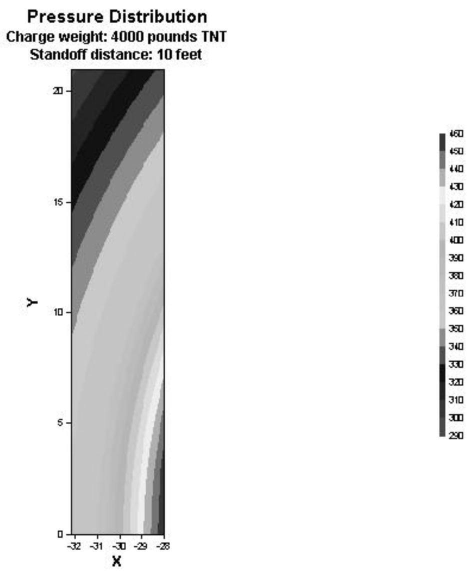

Figure 9 Reflected pressure (psi) distribution on half-width of pier G24 closest to bomb.

Figures 10 and 11 show examples of the assumed pressure histories and calculated responses. Figure 10 shows the approximate equivalent uniform reflected pressure function for ground level pier G24 calculated in *ConWep*. The roughly exponential decay of the pressure with time is approximated as a linear decay, resulting in an idealized "triangular" pressure-time loading function. Using *WAC* to compute the dynamic response of pier G24 results in the response shown in Figure 11, with the top and bottom edges fixed. Figure 11 shows that no damping was used for the SDOF modeling, because only maximum and permanent responses were needed. Maximum predicted displacements for pier G24 are small. This small deflection (0.2% of column height) will not reduce pier gravity-load capacity.

Pier G20 spanning from the 3rd to 4th floors was analyzed in the same manner as the lower piers. The distance from the bomb to the bottom of this pier section is approximately 7.5 m, beyond the shattering range. The story height is conservatively modeled as 4.0 m, with the top and bottom edges modeled as fixed. Equivalent uniform pressures and impulses are similar to quantities computed for the lower levels of pier G24, so the response would be similar to that of the lower levels of pier G24, not significantly reducing gravity-load capacity.

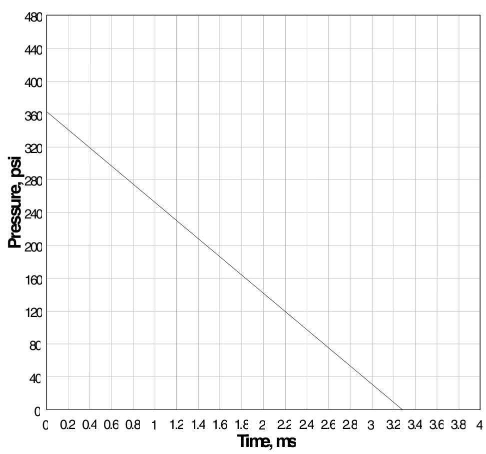

Figure 10 Idealized "triangular" pressure-time history for pier G24, ground story.

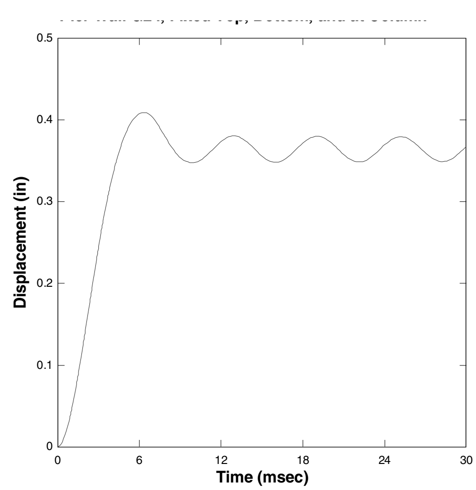

Figure 11 Response of pier G24, fixed supports, ground story.

Ground story piers G12 and G16 are further from the bomb than pier G24, so they are expected to develop smaller maximum deflections and less resulting damage.

The spandrel beam at the 2nd floor level that spans between piers G20 and G24 would be shattered in the vicinity of pier G20. The minor levels of estimated damage for the piers above the 2nd floor level indicate that the spandrels extending between piers G20 and G24 above the 2nd floor level would not be severely damaged, since spandrel and pier thickness are the same.

Figure 12 Estimated damage to pier-spandrel system in 1995 bombing scenario.

Estimated pier-spandrel damage is illustrated in Figure 12.

Special Moment Frame (SMF) Response

Table 2 summarizes the reflected air blast loads, corresponding maximum mid-span deflections, and damage mechanisms for all severely impacted elements in the SMF system.

The SMF columns are located at each column line in all stories. The area near Column G20 is again the most severely loaded section of the structure. Because the SMF column thickness at Column G20 is the same as the pier-spandrel system, 610 mm, destruction of SMF Column G20 is expected from the ground level to the vicinity of the beam-column joint at the 2nd floor level. Estimated SMF damage is shown in Figure 13.

Details of the blast response will be published in 2004.

Table 2 Significant blast loading response, exterior special moment frame system.

COLUMNS				
Story Level	**Column Line**	**Max. Reflected Pressure (MPa)**	**Displacement (mm)**	**Description**
Ground (1-2)	G18	2-4	25	Mid-Height Flexural Displacement
Ground (1-2)	G20	48-117	N/A	Shattered
Ground (1-2)	G22	4-56	N/A	Shattered
Ground (1-2)	G24	~ 2.5	10	Mid-Height Flexural Displacement
2	G20	3-30	N/A	Severe Damage – Limited Capacity
3	G20	2-4	10	Mid-Height Flexural Displacement
BEAMS				
Floor Level	**Between Columns**	**Max. Reflected Pressure (MPa)**	**Displacement (mm)**	**Description**
2	G18-G24	2-73	N/A	Shattered @ G20
3	G20-G22	3-9	70	Mid-Span Flexural Displacement

Interior Shear Wall (ISW) Response

The blast analysis was performed for the Line F.5 wall location, because its blast environment would be more severe than that of the Line F location. The ISW concept uses two shear wall segments. Shear Wall A is centered on Column Line 22 and Shear Wall B is centered on Column Line 14. Each wall is 460 mm thick and 12.2 m long. Table 3 summarizes the reflected air blast loads, corresponding maximum deflections, and damage mechanisms for all severely impacted elements in the Line F.5 shear wall system. Details of the blast response will be published in 2004.

Table 3 Significant blast loading response, interior shear wall system at Line "F.5."

Story Level	Shear Wall	Max. Reflected Pressure (MPa)	Displacement (mm)	Description
1	A	4-15	550	Flexural Failure
2	A	6-11	460	Flexural Failure
3	A	3-6	170	Mid-Height Flexural Displacement
1	B	~ 1	80	Mid-Height Flexural Displacement

PROGRESSIVE COLLAPSE ANALYSES

Figure 14 Estimated damage to internal shear wall on Line F in 1995 bombing scenario.

Figure 13 Estimated damage to special moment frame in 1995 bombing scenario.

Approach

FEMA 277 stated that most of the structural damage that occurred in the 1995 bombing was due to the "progressive collapse" of the structure that propagated from the blast-induced damage to key gravity-load elements. Progressive collapse denotes a chain-reaction event that follows damage to a portion of a structure, in which total damage propagating from initial damage is disproportionately larger than the initial damage. A localized structural failure may occur because of an abnormal loading on an element or elements, overloading, and possibly collapsing, surrounding structural elements. The process is repeated until either the entire structure collapses or some failure-limiting mechanism is reached. The capacity of a structure to continue carrying its gravity-loads without collapse, through redistributing the internal forces resulting from the loads, defines its progressive collapse resistance.

In this study, progressive collapse assessments are based on gravity-load analyses of each strengthening scheme, with those elements that are deemed to have failed because of blast effects having been removed from the structural models. The analyses are combined with engineering evaluations of the building response. Progressive collapse calculations combine a series of basic hand calculations, linear structural analyses, and judgment. For each strengthening scheme, members representing those estimated to be destroyed due to blast load effects were removed from the analytical model. Calculations were then performed to determine if the "damaged" structure could support its gravity-loads. Gravity-loads were assumed to be self weight, 8 KPa for the floors and 7.5 KPa for the roof, plus 25% of the estimated 2 KPa live load for office space and roof live load. If the gravity-load capacity was insufficient to resist applied loads in an elastic analysis, virtual work analyses were performed to determine whether the capacity to resist the loads is sufficient when the structure yields and redistributes forces.

Because blast-damaged elements are removed almost instantaneously, loading will be increased above the nominal gravity-load by impact effects. Recommended progressive collapse analysis procedures often double gravity-load values to account for impact. Assuming that impact effects double gravity loads, surviving structural elements must be twice as strong as the calculated gravity load demands to prevent collapse. This approach was used here. After removal of blast-destroyed elements, gravity load capacity-to-demand (C/D) ratios were calculated for shear and flexure for each system of remaining elements.

Using the assumed doubling of gravity-load forces implies that a C/D ratio of 2.0 or larger for gravity-loads will prevent progressive collapse. If the C/D ratio exceeded 2.0, then progressive collapse was not anticipated. If the C/D ratio was less than 1.0 for either flexure or shear, failure was anticipated. If the C/D ratio was less than 2.0, but more than 1.0, collapse was deemed possible and the potential failure mechanism was examined more closely. If the collapse mechanism was ductile (e.g., flexural yielding of beams), then team members assessed whether a progressive collapse was likely to occur. If the failure mechanism was brittle (e.g., shear failure), then a collapse was assumed to occur. If local collapse was deemed to be likely, the potential for collapse propagation was assessed.

Pier-Spandrel System Performance

Pier G20 would be shattered by blast below the 2nd floor level, as would a portion of the spandrel beam spanning between Columns G20 and G24. The ground story of the pier-spandrel system at Column G20 would not support gravity-load. Piers G12, G16, and G24 would still support gravity-load.

The ground story level of the pier G20 was removed from the computer model, and an elastic gravity-load analysis was performed. The only overstressed members were the 3rd floor and roof beams of the original building, where they frame into Column G20. Positive yield moments would occur at the joint faces as the frame sags down at column G20. Other original frame elements and the new elements had a C/D ratio of 1.4 before first member yielding, indicating that the 3rd floor beam will yield where it frames into Column G20 first, after which loads will redistribute. Next, the roof beam will yield where it frames into Column G20, followed by other floor beams. The resulting load redistribution increases moments in the floor beams and new spandrels.

A two-dimensional (2-D) virtual work analysis of the G-line frame, including the new pier-spandrel walls, confirmed that there was sufficient residual capacity in the system to accommodate load redistributions. Conservatively neglecting the strength of transverse beams framing into the G-line frame, the 2-D analysis showed the frame had a maximum C/D ratio of 1.9. A three-dimensional (3-D) virtual work analysis including the transverse beam capacities increased the C/D ratio to 2.3.

No progressive collapse of the remaining elements was estimated to occur. While the pier-spandrel elements may shield some of the floor slabs from the bomb blast effects, it was assumed that all slabs lost due blast in the actual bombing would again be lost. It was estimated that approximately 543 SM of the total floor area in the building would be lost due to combined blast and progressive collapse damage, approximately 10% of that observed in the original building (5,400 SM).

Special Moment Frame Performance

Column G20 from the ground level through the 3rd story and Column G22 at the 1st story level would shatter due to blast. The 2nd story beam spanning from G18 to G24 fails due to a combination of lost column support and blast-induced damage. The 2nd story beam-column joint G20 would probably be shattered. Third story beams survive with light to moderate damage.

Hand calculations were used to perform linear analysis of the SMF on Column Line G. Two sets of 2-D analyses were performed, to provide upper bounds on the positive moment at Column G20 and the negative moments at Columns G16 and G24. First, after removing shattered elements, each beam from the 3rd floor level to the roof was modeled as a simply supported beam loaded at its ⅓ points. Second, after removing the failed elements, each beam from the 3rd floor level to the roof was modeled as a fixed-ended support beam loaded at its ⅓ points. The calculations showed that the roof beam will yield (1.4 - 2.0 times yield capacity), and the 9th floor beam may yield (1.1 times yield). The reserve flexural capacity in the beams at the 3rd through 8th floor levels not accounted for in this approach could accommodate some load redistribution. There was enough flexural capacity in the 3rd floor beam to support its load and the load from the 2nd floor slabs and beam, which must hang from this beam, because the supporting column would be destroyed.

A 2-D virtual work analysis was also performed on the SMF. The frame was found to have a C/D ratio of 2.3. Progressive collapse would not occur.

With no progressive collapse of frame elements, the only floor area lost would be blast-induced. It was again estimated that approximately 543 SM of the total floor area in the building would be lost due to combined blast and progressive collapse damage, approximately 10% of that observed in the original building.

Internal Shear Wall Performance

The Line F.5 shear wall scheme is first addressed. Shear Wall B would be undamaged in the bombing. Shear Wall A would be severely damaged in the lower two stories, eliminating its gravity-load capacity.

Since no new structural elements are added between Line F.5 and line G, the original structure would be unprotected from the blast in this area. The G-line frame and floor slabs between line F.5 and line G would experience the same direct blast and progressive collapse damage that occurred in the actual bombing.

The walls would provide some protection behind Line F.5 not seen in the original building. Shear Wall B survives intact, protecting that region of the structure from blast effects and providing added gravity-load capacity. Shear Wall A provides added blast protection and gravity-load capacity above the 3rd floor level.

Shear Walls A and B also reduce unsupported span lengths of the transverse beams on Column Lines 12, 14, 16, 20, 22, and 24 that are normal to the G-line frame, from 10.7 m to 6.1 m. This span reduction might enable the beams to survive as cantilevers that extend from the shear walls, along Column Lines 12-16 and 20-24, toward Column Line G. The possible cantilever action was examined. Since there was no steel reinforcement in the tops of the transverse beam mid-spans, the only beam negative moment reinforcement, which would be mobilized in cantilever action, was the slab temperature steel. That was minimal and placed at slab mid-depth, offering little resistance to negative moment. Since the bottom two stories of Shear Wall A failed, portions of the wall not removed by the blast would add large point loads to the transverse beams. It was concluded that cantilever action would not add significant new collapse resistance in the structure following the blast-induced damage.

A progressive collapse would likely occur following the blast for the Line F.5 scheme. The floor area that would be lost due to the collapse is estimated to be approximately 75% of that observed in the original building.

Blast response analyses were not performed for the walls placed on Line F. This is the more likely location for a shear wall system if blast and progressive collapse threats are neglected. The Line F walls were far enough from the blast source to survive intact. Portions of the floor slabs and transverse beams between Lines F.5 and F would be more vulnerable to blast effects than they would be in the Line F.5 scheme, and progressive collapse resistance in the vicinity of Shear Wall B would be somewhat lower than for the Line F.5 scheme. The only potential significant reduction in the degree of progressive collapse when compared to the original building would be in the portions of slabs that collapsed between Lines E, F, 22 and 26 in the actual bombing. They would probably survive in this strengthened configuration. The floor area that would be lost due to the collapse is estimated to be approximately 81% of that observed in the original building.

CONCLUSIONS

Table 4 summarizes the damage estimates from the progressive collapse analyses. The blast and progressive collapse analyses show a significant reduction in the overall severity of collapse for the schemes that strengthened the perimeter of the building – the pier-spandrel and SMF. In those systems, the collapsed area was reduced from approximately 42% of the total floor area in the 1995 bombing to an estimated 4%. The shear wall schemes more modestly improved the building response. The estimated collapsed area was reduced to an estimated 31% - 34% of the total floor area, depending on the location of the shear walls. In the original building, the blast was located away from the primary lateral load-resisting structural elements (elevator and stair core walls) and close to weaker framing elements, resulting in the greatest amount of damage. Strengthening the front wall frame resulted in the greatest blast resistance improvement. Since most fatalities in the 1995 bombing were caused by crushing injuries, any reduction in collapsed area can be expected to result in an approximately proportional reduction in casualties.

The improved response for the pier-spandrel and the SMF systems results from larger structural cross-sections, increased longitudinal and transverse reinforcement, and enhanced longitudinal reinforcement continuity. The added thicknesses of the various structural elements at the lower level in the building and their increased transverse reinforcement (ties in columns, stirrups in beams) act to improve blast resistance through increasing shear and diagonal tension strength and through increasing the amount of energy that can be absorbed by the structure before it breaks up. The additional size and mass of the new members offer more inertial resistance, also permitting the elements to resist larger direct blast loads. All of the member size and reinforcement increases act to improve the flexural and shear strength of the building. The improved continuity of the longitudinal reinforcement enables the structure to be tougher and redistribute loads away from the locations where gravity support has been lost.

The shear wall schemes strengthen the building for earthquake loads, but their locations within the interior of the building leave many of the elements of the original exterior frame exposed to the bomb blast. This leaves the building to respond much as it did during the 1995 bombing. The primary gain from the shear wall schemes is the protection of the floor area behind the walls from debris generated by the blast and the ensuing collapse.

Two major conclusions can be drawn from the study. First, strengthening an existing reinforced concrete building to provide high seismicity ("Zone 4") earthquake resistance will improve its resistance to blast and progressive collapse. Second, in constructing a new building or in strengthening an existing building, it is more efficient for external blast and impact resistance to place the elements that are proportioned and detailed to resist lateral forces on the building perimeter.

Table 4 Summary of estimated blast and progressive collapse damage.

Floor Level	Floor Area (SM)	Original Direct Blast Damage (SM)	Total Floor Area Lost (SM)				
			Original Building	Pier-Spandrel Scheme	SMF Scheme & Re-detailed Frame	Shear Wall Scheme Line F.5	Shear Wall Scheme Line F
Roof	1,412	0	585	0	0	432	488
9th	1,412	0	585	0	0	432	488
8th	1,412	0	585	0	0	432	488
7th	1,412	0	585	0	0	432	488
6th	1,412	0	585	0	0	432	488
5th	1,412	28	585	28	28	432	488
4th	1,412	98	585	98	98	432	488
3rd	1,412	195	650	195	195	432	488
2nd	1,412	223	650	223	223	478	488
Total	12,708	544	5,395	544	544	3,934	4,392
% of Total Floor Area Damaged		4%	42%	4%	4%	31%	34%
% of Damaged Area Due to Blast		-	10%	100%	100%	12%	12%
% of Damaged Area Due to Progressive Collapse		-	90%	0%	0%	88%	88%

The study focused on the effect of strengthening an older reinforced concrete building that contained ordinary moment frames to support gravity-loads and shear walls to resist lateral load. Strengthening was for high seismicity demand, so the study cannot be directly extrapolated to seismic strengthening using lesser seismic demands. The conclusions cannot be directly transferred to other structural systems (e.g., steel moment frames). While the benefit of strengthening exterior elements on reducing blast resistance for external threats and preventing the onset of progressive collapse is likely to be generally applicable, the degree to which such strengthening contributes needs further study.

While improvements in blast and progressive collapse resistance can result from well-placed seismic strengthening measures, it is not implied that seismic design details alone replace specific measures to mitigate blast and progressive collapse vulnerabilities; there will be instances in which known threats justify specific design for blast and progressive collapse. However, the study does suggest that proper application of current-practice seismic detailing can reduce vulnerability to blast and progressive collapse. Knowledge of this benefit may convince an existing building owner in a high seismic area to see what might otherwise be viewed as only an incremental step in seismic strengthening as added protection against blast and progressive collapse.

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