Analytical Verification of Blast Testing of Steel Frame Moment Connection Assemblies

Authors:

Jesse E. Karns, Myers, Houghton & Partners, Inc. (MHP), 4500 East Pacific Coast Highway, Suite 100, Long Beach, California 90804, Phone: 562-985-3200, jkarns@mhpse.com
David L. Houghton, Myers, Houghton & Partners, Inc. (MHP), dhoughton@mhpse.com)
Bruce E. Hall, General Services Administration, Washington, D.C.
Joonghwan Kim, Myers, Houghton & Partners, Inc. (MHP)
Kyungkoo Lee, Ph.D., Myers, Houghton & Partners, Inc. (MHP)

INTRODUCTION

Attacks on buildings in the United States over the past decade by terrorists have heightened concerns over the availability of proven technologies to economically design and construct multi-story buildings capable of mitigating progressive collapse when subjected to terrorist bomb blast. To resolve these concerns, and in recognition of the potential favorable relative economics of steel frame construction over reinforced concrete, the General Services Administration (GSA), Office of the Chief Architect, elected in 2004 to fund and conduct a first-ever, high-level blast and progressive collapse steel frame test program (the "GSA Test Program") to investigate the performance of conventional welded steel frame construction. Prior to 2004, GSA's progressive collapse guidelines for the design of steel frame buildings [1] were based on analytical research only. The primary objectives of the GSA Test Program are five-fold:

1. To either validate or dispel, through actual testing and corroborative state-of-the-art analysis, the notion that reinforced concrete is preeminent over structural steel in mitigating the effects of bomb blast attack on conventional public structures.
2. To verify and validate analysis tools needed to conduct reliable blast and progressive collapse analyses in order to minimize the need for future costly and time-consuming full-scale testing.
3. To investigate the relative performance of contrasting steel frame beam-to-column *connection types* when subjected to the high strain rate effects of blast attack and subsequent impending progressive collapse conditions.
4. To identify steel frame beam-to-column connection systems that are reliable solutions for the mitigation of the effects of bomb blast, and the positive arrest of impending progressive collapse, for use by the GSA and other U.S. Federal government agencies.
5. To gain the knowledge necessary to modify, as necessary, a variety of steel frame connections types being used in the construction industry today, in order to be able to reliably and economically provide GSA's prescribed security expectations.

GSA TEST PROGRAM SYNOPSIS

The GSA Blast Test Program is strategically configured to systematically acquire critical knowledge related to conventional steel frame design and construction, while shortening the release time of pertinent and vital information to practicing structural engineers who must design buildings that can achieve viable mitigation of blast effects, and the effective arrest of structural mechanisms known to trigger progressive collapse of buildings. The design, fabrication and erection of GSA test articles replicate steel frame configurations found in typical building construction in *low seismic* applications, however, the metrics of member size properties used are directly scalable to *high seismic* applications. The large-scale arena blast tests were conducted at the Chesapeake range, Kirtland Air Force Base, New Mexico, and use explosives in quantities that are slightly higher than, yet are compatible with, the threat philosophy represented in the Interagency Security Committee (ISC) Security Criteria [2].

Fundamental to achieving GSA's program objectives is the ability to judiciously select the appropriate size of the explosive charge, and the stand-off distance (i.e., the *range*) of that charge, for a given test article. This is admittedly and inherently a balancing act, requiring a charge size and range that will challenge the test article with enough blast energy to produce both high-order strain rates and inelastic deformations, and/or cause rupture in critical components of the test article, while not destroying either the test article or its reaction structure in the process. Implicit to judging whether or not the appropriate test parameters have in fact been selected is the successful and repeatable creation of a 'missing column' scenario using those selected parameters which achieve a violent removal of the column's gravity load carrying capability, including localized web buckling, localized flange buckling, lateral torsional buckling of beams (i.e., permanent twist), and minor axis bending of both beams and column. This achievement provides ample validation that a) this threat scenario is indeed a credible event, and b) the ISC's prescriptive 'alternate path' methodology is a credible design approach.

All four progressive collapse test articles consisted of double-span steel frame assemblies, spanning approximately 35 feet (Figure 1), and were configured to simulate representative beam spans and boundary conditions encountered in typical building construction (Figure 2). This double-span configuration provides an opportunity to a) evaluate the relative ductility characteristics of critical connection welds which are oriented in both parallel and perpendicular directions to the applied strains; and b) assess and/or validate the *post*-blast structural integrity of *beam-to-beam* continuity. The term 'discrete beam-to-beam continuity' is defined in Section 2.1 of the GSA Progressive Collapse Guidelines [1], and in Section 5.1.1.1 *Local Considerations* of those Guidelines it is designated as the first of four (4) fundamental and essential connection attributes in mitigating progressive collapse.

There are three reasons for using the double-span test article configuration in the GSA Test Program:

1. To create a 'missing column' scenario with the removal of the column's gravity load carrying capability when subjected to high-level air blast attack and debris impingement from the disintegration of an immediately-adjacent concrete cladding wall; and to determine the blast integrity of beam-to-column connections, including the effects of girder twist and weak-axis bending, albeit absent gravity loads.

2. To subject, in situ, blast-damaged steel frame assemblies to the monotonic application of a simulated sustained gravity load, including post-yield interaction of beam bending moment and axial tension, in order to assess the post-blast integrity of the primary gravity load-carrying beam-to-beam structural continuity across a compromised column. The loading is increased incrementally until the double-span condition of the spandrel beam experiences large vertical displacement (i.e., large tension loads are developed in the beam when it wants to act more like a cable than a beam), which is needed to capture post-yield interaction of applied moment and axial tension in the beam, and/or until the beam-to-beam connection (across the compromised column) experiences rupture and/or critical inelastic deformation.
3. To subject *non*-blast-damaged steel frame assemblies to the monotonic application of a simulated sustained gravity load to:
   a) Assess the integrity of primary gravity load-carrying beam-to-beam structural continuity when subjected to a double-span condition simulating an undamaged upper-story level following the removal of a column at a lower story level. These *non*-blast-damaged test articles are identical to the configuration of blast-damaged test articles, with the only exception being that only a stub column is provided. The column extends from two feet above the beam, to a few feet below, thus replicating a 'missing column'.
   b) Provide a *Base Line* for assessing the relative *non*-blast-damaged performance between each of the two connection types selected for investigation, for an *upper*-story condition.
   c) Create a *Benchmark* for assessing the relative performance and post-blast integrity of similarly configured blast-damaged test articles subjected to a similar post-facto application of gravity load simulating a sustained gravity load condition.

All four GSA test articles were constructed using 'real world' fabrication and erection means and methods, including simulated 'field welding' conditions for a particular beam-to-column connection type, as applicable, to replicate standard industry practice. The first two progressive collapse test articles were subjected to a high-level bomb blast prior to being tested for progressive collapse potential. A full-height four-inch thick reinforced concrete wall was installed immediately adjacent to each test article to simulate concrete cladding and to deliver increased blast energy to the steel frame assembly (Figure 3). The remaining two test articles were intentionally tested as *non*-blast-damaged specimens for reasons explained earlier herein. Each of the two connection types selected was tested under the exact same conditions for all test scenarios. All steel frame assemblies were constructed using identical material properties, and identical beam and column sections sized for both *East Coast* (low seismic) and *West Coast* (high seismic) design applications. While all test articles utilize 'seismic sections' (as defined by AISC [3], which practice is typically representative of west-coast design applications, the column size selected is representative of the size and weight of members commonly found in east-coast design applications.

Member Size Selection Criteria

Member sizes for both the column and beams were chosen using the following *Member Size Selection Criteria*:

1. Potential for achieving actual blast damage when taking under consideration the limitations imposed by available charge size and the finite capacity of the reaction structure.
2. Scalability to larger and deeper sections typically used in West Coast design applications.
3. Sections must be classified as 'seismic sections' per AISC requirements and be applicable in both East and West Coast design applications.

The following member sizes were selected for the GSA test program and satisfy the above Member Size Selection Criteria:

- Beams W18x35, ASTM A572 Grade 50 steel
- Column W16x57, ASTM A572 Grade 50 steel

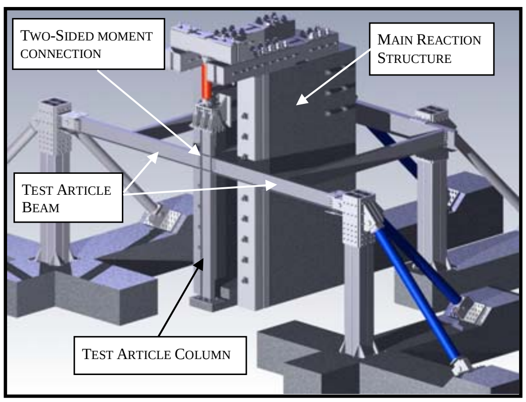

FIGURE 1
AIR BLAST/PROGRESSIVE COLLAPSE TEST ARTICLE AND REACTION STRUCTURE (DEPICTED WITHOUT THE CONCRETE CLADDING WALL FOR CLARITY).

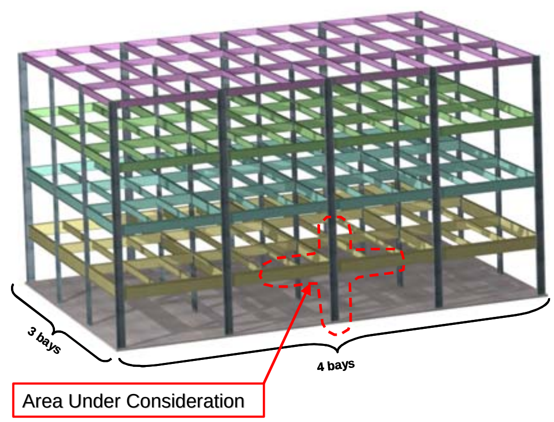

FIGURE 2
MODEL STEEL FRAME BUILDING.

FIGURE 3
TYPICAL CONCRETE CLADDING WALL INSTALLED ADJACENT TO AIR BLAST/PROGRESSIVE COLLAPSE TEST ARTICLE.

Connection Type Selection Criteria

The welded steel frame moment connection types selected for investigation were chosen using the following *Connection Type Selection Criteria*:

1. Must be cost competitive relative to each other [14].
2. Must exhibit contrasting connection attributes, including distinctly different connection geometries; weld orientation vs. direction of applied load; available redundancy in load paths; and available robustness in the beam-to-column joint [8, 9].
3. Must be currently in use by Engineers and Contractors in design and construction of conventional steel frame buildings throughout the United States, covering all seismic regions.

The two welded steel frame connection types selected for investigation are:

- *'Traditional'* moment connection system (Figure 4)
- *SidePlate*® moment connection system(1) (Figure 5)

(1) U.S. Patent Nos. 5,660,017, 6,138,427, 6,516,583, 6,591,573

Both of these connection types are in current use nationally and are categorized as post-Northridge moment connection systems, developed after the 1994 Northridge Earthquake in California. Both fully satisfy the above Connection Type Selection Criteria, and therefore were selected by the GSA for the test program.

For the 'Traditional' moment-resisting beam-to-column connection, a Welded Unreinforced Flange – Bolted Web (WUF-B) moment connection system, as defined in FEMA 350 [4] was selected. The beam or girder flanges are directly attached to the face of a column flange using *field*-welded complete joint penetration (CJP) groove welds in a T-joint configuration. All improvements outlined in FEMA 350 for the 'Traditional' connection were incorporated into the design and construction of the test article. These *post*-1994 Northridge earthquake recommendations include the following: 1) weld metal with appropriate Charpy V-notch toughness; 2) removal of weld backing from the beam's bottom flange-to-column flange weld, back-gouging the weld's root pass, and the addition of a reinforcing fillet weld; and 3) use of improved weld access hole shape and surface finish.

As observed following the Northridge earthquake and documented by FEMA [4], failure modes for the 'Traditional' connections can occur in one of several ways. In some cases, the fractures rupture through the column flange base metal just behind the weld in the form of a "divot" pull-out, that remains joined to the connecting weld, but pulls free from the remainder of the column flange. In other cases, the fractures rupture completely through the thickness of the weld. Alternately, if the weld at the face of column does not fail, the fracture may rupture completely through the beam flange, which is what occurred in both of GSA's 'Traditional' progressive collapse tests. When any of these types of failures occurs, the flexural demand on the welded beam flange-to-column flange juncture must now be resisted by the beam's bolted web, which in turn quickly deteriorates because of its incapacity to resist such forces.

The beam-to-column connection geometry of the SidePlate® moment connection uses continuous structural steel plates, sandwiching the steel beams and column together. The SidePlate® moment connection geometry also exhibits a physical separation between the face of the column flange and the end of the beam (commonly referred to as a 'gap'), as the beam is connected to the full-depth side plates and not directly to the face of column. All bending moment, axial tension and vertical shear load transfer from the beam to the column is provided exclusively by these two parallel side plates. The load transfer is accomplished with simple plates and fillet welds. The connection is designed with adequate strength and stiffness to force all significant plastic behavior into the beam adjacent to the connection. Construction of the SidePlate® moment connection beam-to-column joint uses all *shop*-welded 'column tree' fabrication for improved quality control. A fully welded link beam-to-column tree splice is provided in the field using CJP welds for both flanges and web to complete the steel frame erection.

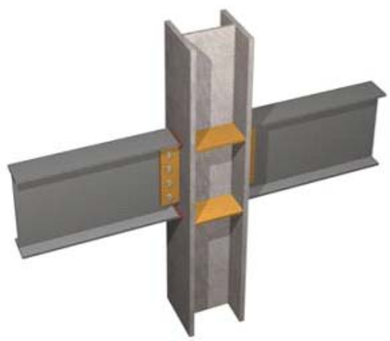

FIGURE 4
'TRADITIONAL' MOMENT CONNECTION GEOMETRY.

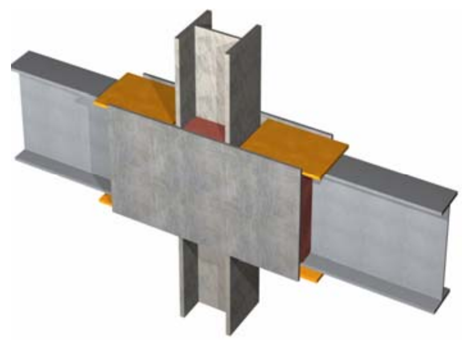

FIGURE 5
PATENTED SIDEPLATE® MOMENT CONNECTION GEOMETRY (COURTESY OF SIDEPLATE SYSTEMS, INC., LAGUNA HILLS, CA).

PREDICTIVE ANALYSIS

High-fidelic physics-based (HFPB) finite-element nonlinear modeling, coupled with time history and monotonic analyses of each test article, were conducted using parametric models to capture certain relevant nonlinear behaviors enumerated earlier herein, which are needed to accurately predict their performance. These analyses were performed for blast response, and for double-span gravity capacity testing with or without blast damage. The same HFPB model used for the blast analysis of each connection type was used to conduct the corresponding predictive progressive collapse analysis, thereby providing for the superposition of results from one analysis to the next. The models include both clad and unclad test articles. A combination of three-dimensional continuum elements (i.e., *solid* elements), shell elements and nonlinear material models are used to build each model to replicate the actual plastic behaviors occurring, with a level of fidelity suitable to the complexity of a steel frame connection (Figure 6).

For some of the models, both the air and the steel test article were modeled, including fidelic modeling of all boundary conditions of the test configuration, and the concrete cladding (as applicable), using a *Eulerian* mesh and *Lagrangian* mesh, respectively, to model state-of-the-art shock wave propagation in air, and the interaction of the blast wave (through air) with the solid structure; and then employing the use of the explicit LS-DYNA code [5] for explosive modeling. The use of the LS-DYNA code and this *air fluid/structure* modeling process [10, 11] to replicate the actual explosive material and the shock wave pressures (Figure 7) on a coupled model is a complex, but verifiable procedure that is independent from other simplified blast load codes. Many of the analyses used blast pressure loading time histories derived from the SHAMRC code [13]. Where SHAMRC loads were used, the time histories were applied directly to the face of the structural mesh.

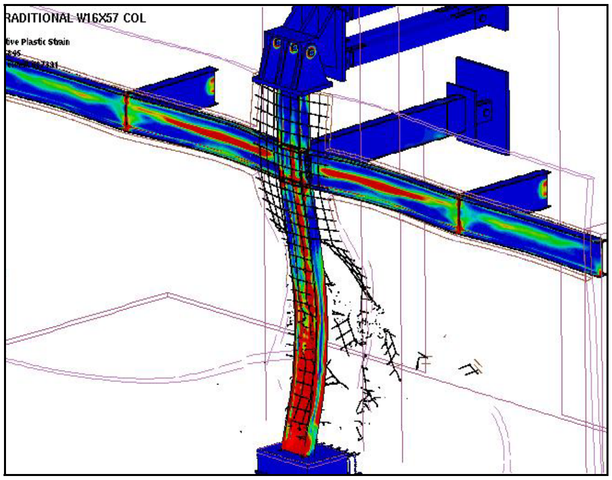

FIGURE 6
BLAST/PROGRESSIVE COLLAPSE ANALYTICAL MODEL

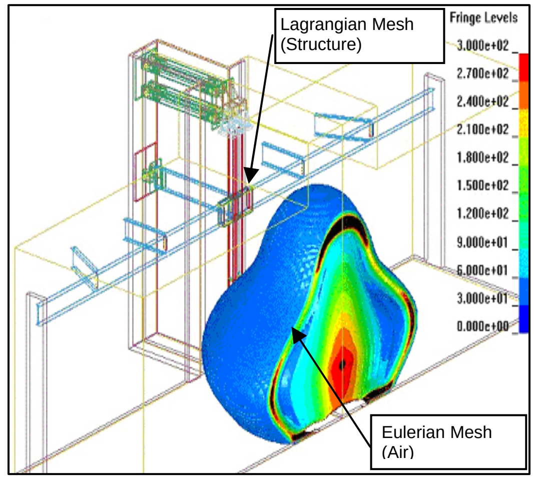

FIGURE 7
AIR FLUID/STRUCTURE ANALYSIS.

The predictive analysis of the blast testing, as defined and characterized herein, is very complex. To capture adequate fidelity of the HFPB models, including the stress and strain distribution through the flanges, welds and bolts, models composed of up to 2.5 million finite elements were used. This complexity was warranted to simulate the subtle but critical nuances of non-linear behavior in such a dynamic state. For blast loading (dynamic), LSDYNA's *explicit* solver was used, while LSDYNA's *implicit* solver was used to replicate the progressive collapse portions of the test (static and monotonic). Steel material was modeled using the 'Piecewise Linear Plasticity' model (MAT_024), and concrete was modeled using the 'Brittle Damage' model (MAT_096). For the material properties of steel, both dynamic strain rate and softening effects were considered. Selection of material models and material limits used in the analyses becomes critical in the ability to accurately predict observed global and local behaviors. Ultimately, determining fragility (fracture) thresholds of welds, bolts and plates was an iterative process. Fracture strain (true strain) limits of approximately 60% (0.60 in/in) and 40% were used for steel and weld materials, respectively. These values were determined by calibrating the material models with an earlier series of blast tests conducted by the Defense Threat Reduction Agency (DTRA) and GSA, and are consistent with the blast test behavior of A572-50 steel with E70 weld electrodes.

One of these earlier tests, funded by GSA, consisted of a wide flange column blast test and associated predictive analysis. The test article consisted of a single W27x217 column, with the weak axis turned perpendicular to the air blast. The W27x217 is considered a 'seismic section' per AISC requirements and is directly scalable to the W16x57 column that was used in the double-span test articles presented herein. Although classified as a 'seismic section', the depth-to-thickness ratio of the test column's web (i.e., d/tw = 34.3) is higher than most columns used in seismic design.

The predictive HFPB finite element non-linear time history analysis using LS DYNA appeared to accurately predict the global deformations of the W27x217 test column (Figure 8), as well as the observed damage at the concrete base. Recorded test results show that the permanent midpoint displacement of the test article was slightly over 15 inches with no fracture initiation anywhere, and with the center of web having a permanent displacement of 7.5 inches relative to the flange. The analysis, however, was not completely accurate because it predicted failure along the fillet transition between the column flange and connecting web (i.e., the inherently weak 'k-line'), extending from the top of the concrete base to an approximate height of 24 inches. A *post*-test refinement of the analysis and non-linear material properties resulted in a more realistic representation of the test results, i.e., actual true strains in the column at the k-line ranged between 0.60 in/in and 0.70 in/in (60% to 70% strain). The results of these parametric analyses provided the needed calibration of fracture initiation strain levels for subsequent predictive analyses of blast test articles.

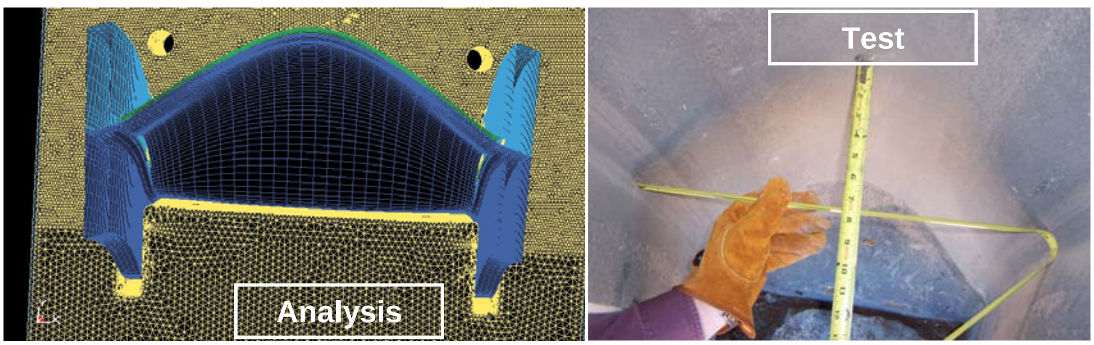

FIGURE 8
DUCTILE BEHAVIOR OF DEEP COLUMN SUBJECTED TO DIRECT AIR BLAST.

PRINCIPAL FINDINGS AND SUPPORTING FACTS

The following three (3) principal findings and supporting facts are validated by actual test results and are corroborated by finite element non-linear time history analyses using LS DYNA:

1. Conventional steel frame construction can behave in a very ductile manner when subjected to high-strain rates associated with blast loading.
   a) Based on the observed performance of both the GSA blast-tested conventionally-constructed steel frame test articles and similar tests conducted at the site by DTRA, the notion that structural steel is a poor choice for blast resistant structures is unfounded. In fact, the GSA Blast Test Program has demonstrated that conventional welded steel frame construction can be a surprisingly good choice for resisting the effects of high-level bomb blast and debris impingement due to its inherent overall ductility and strength. Notwithstanding the intentional violent removal of a typical building column, the strength and ductility of the remaining framing system was virtually uncompromised in each of the two tests conducted. This behavior was corroborated and predicted by finite element non-linear time history analysis using LS DYNA.
   b) To maximize the relative cost effectiveness of selecting structural steel over

reinforced concrete, structural engineers typically rely on the use of *deep* (i.e., high slenderness ratios for both flanges and web) rolled wide-flange steel shapes as columns and beams in building design and construction. Earlier wide flange column blast tests and analyses have demonstrated that when deep rolled shapes are subjected to direct high-level bomb blast attack, they can respond in a highly ductile fashion, despite the typically slender characteristics of the member components (i.e., webs and flanges).

2. Conventional steel frame construction can be an excellent and cost-effective solution for both blast resistance and progressive collapse mitigation if, and only if, beam-to-column connections are properly configured and detailed.
   a) Connection detailing is of critical importance to achieving adequate performance in resisting progressive collapse. The test results demonstrate the relative performance differences in connection geometry, ductility and robustness as a function of a) weld orientation versus direction of applied load; b) the ability of the connection's geometry to minimize the danger of brittle fracture by allowing free movement of the base material, minimizing the effects of weld shrinkage, and by avoiding strain concentrations and tri-axial strains; and c) the degree of reserve capacity available in the beam-to-column joint to accommodate the demand from inelastic levels of bending moment and axial tension interaction. These observations validate many of the same concerns highlighted in TM5-1300, Section 5-18.3 [6].

      There was a significant difference in performance of the two contrasting moment connection geometries tested when subjected to virtually identical progressive collapse conditions (i.e., a real 'missing column' scenario). In particular, the relative difference in energy input needed to fail each of the two connection types tested was approximately five (5) times (Table 1), at failure of the beam's bottom flange. The SidePlate® connection achieved the higher capacity for both the blast-damaged ground story level and the *non*-blast-damaged upper story level simulations (Figures 9 and 10). The predictive analyses that were conducted corroborate this important and distinctive difference in performance. While the gravity load-carrying-capacity of the two tested assemblies should have been similar due to the fact that the beam sizes and spans were identical and the column's compromised condition was virtually identical, there was a large, marked difference in the recorded capacities, and this large difference was repeated for each of the two test protocols conducted. The SidePlate® moment connection was able to maintain stability under much higher applied loading than the 'Traditional' moment connection.

Moment Connection Type	Condition at Start of Test	Energy Input at <u>First failure</u>	Performance Ratio (SidePlate®/'Traditional')	Energy Input at <u>Complete failure</u>	Performance Ratio (SidePlate®/'Traditional')
'Traditional'	Blast Damaged	374 in-kips.	**4.6**	630 in-kips	**3.5**
SidePlate®		1,721 in-kips.		2,213 in-kips	
'Traditional'	*non*-Blast Damaged	433 in-kips	**5.0**	818 in-kips	**3.4**
SidePlate®		2,177 in-kips.		2,730 in-kips	

TABLE 1

PROGRESSIVE COLLAPSE TESTS PERFORMANCE - COMPARISON OF EXTERNAL ENERGY BY CONNECTION TYPE

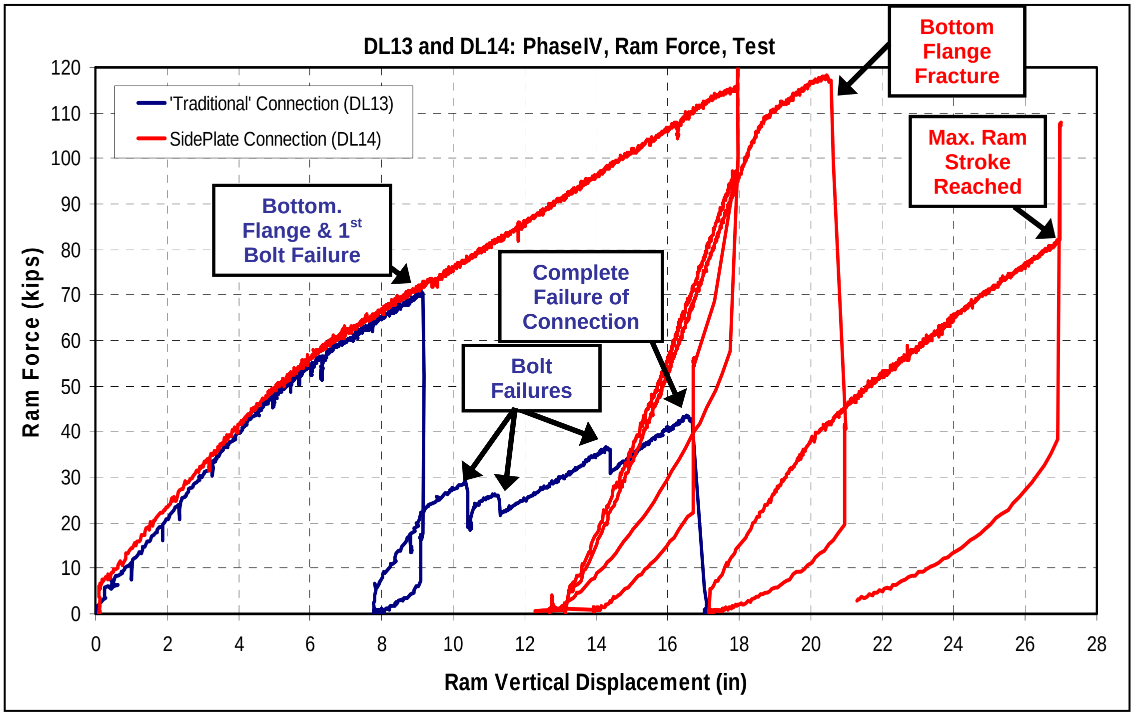

FIGURE 9
FORCE VS. DISPLACEMENT CURVES FOR BLAST-DAMAGED PROGRESSIVE COLLAPSE TESTS.

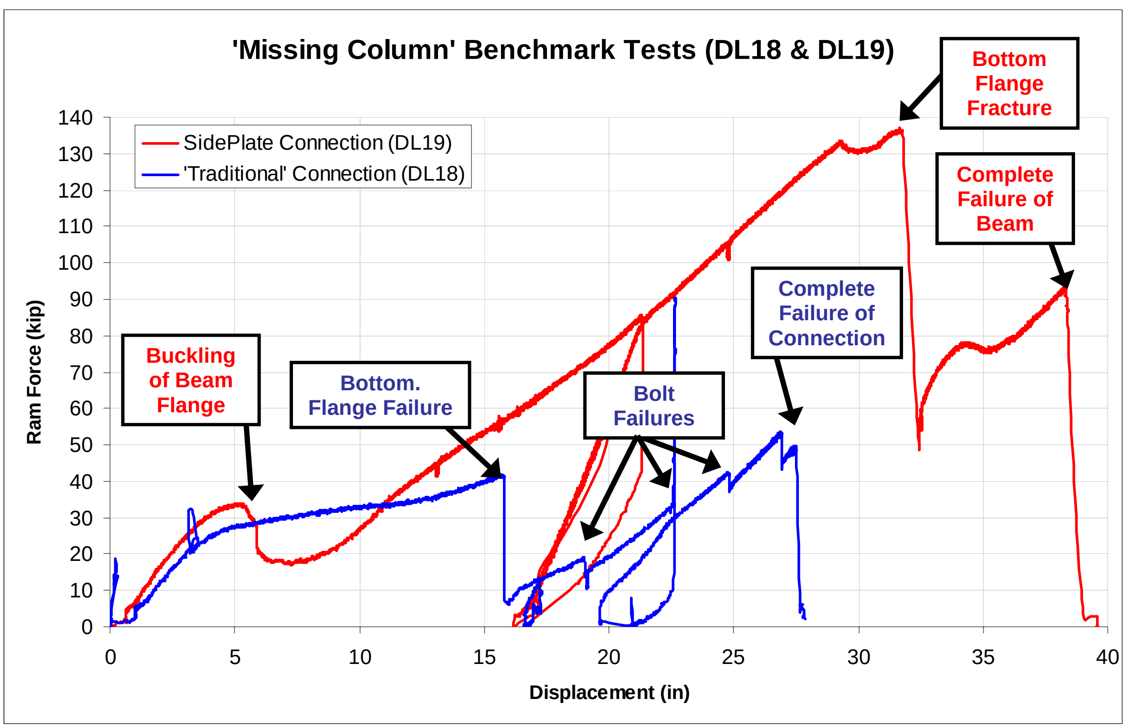

FIGURE 10
FORCE VS. DISPLACEMENT CURVES FOR NON-BLAST-DAMAGED PROGRESSIVE COLLAPSE TESTS.

b) Adequate connection rotational capacity is fundamental to arresting progressive collapse. Accordingly, until requirements specific to progressive collapse are developed, as a minimum requirement, moment connections used to mitigate progressive collapse should be pre-qualified as a *Special Moment Frame* (SMF) connection, in accordance with the provisions of Appendix S and/or P of ANSI/AISC 341 [3].

c) Moment connections that have been prequalified for rotational capacity due to bending alone may not be capable of concurrently resisting the interaction of axial tension and bending moment, which is an essential performance attribute for preventing progressive collapse. While tension stiffness ('cable-like' action) can significantly increase the load carrying capacity of the system over bending moment alone, the beam-to-column connection must be able to transfer the large flange tension forces developed from the combination of bending moment and axial tension. The test results highlight the critical nature of this interaction between rotational (bending moment) demand and concurrent axial tension demand on a beam-to-beam connection across a compromised column (Figure 11).

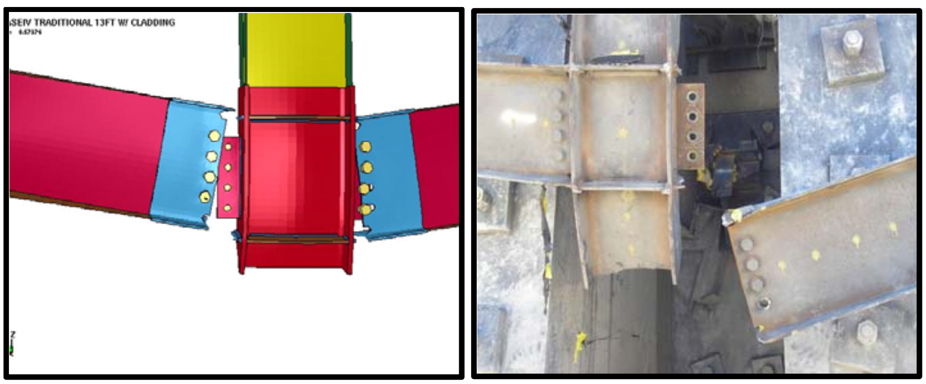

FIGURE 11
FAILURE OF 'TRADITIONAL' MOMENT CONNECTION DUE TO INTERACTION OF AXIAL TENSION AND BENDING MOMENT, ANALYSIS VS. TEST.

d) The 'Traditional' moment connection survived the blast test without any observable material compromise to any of the connection welds and components while undergoing the effects of complete column demise and severe global deformation, including a significant inelastic twist of 30 degrees off vertical, albeit in the absence of sustained gravity load. The column, however, was critically damaged as the column fractured and pulled away from its base (Figure 12) and laterally displaced approximately 75 inches. Similar failure mechanisms and behaviors were observed in finite element non-linear time history analysis (Figure 18). For each of the two progressive collapse test articles employing the 'Traditional' moment connection, the mode of failure appeared to be controlled by brittle failure of the beam's bottom flange, at the toe of the CJP groove weld connecting beam flange to face of column flange (Figure 14), followed in rapid succession by failure of the beam's web connection bolts, and ultimately by the failure of the CJP groove weld connecting the beam's top flange to the face of column flange (Figure 15). A comparison of the recorded progressive collapse performance between the connection types tested is presented in Table 2.

Even with significant twist to the beam and damage to the column, the blast-damaged progressive collapse specimen was able to resist 70 kips of applied gravity load while the non-blast damaged specimen never exceeded 40 kips (Figures 9 and 10). The large difference in load capacity appears to be a function of the axial tension in the system. While the axial load in the blast-damaged test article quickly increased, thus providing additional strength and stiffness to resist the applied ram force, the axial load in the *non*-blast damaged test article initially increased gradually until the beam began to form a bending plastic hinge. The difference in the axial load histories of the two tests appears to be attributed to the initial sag in the blast-damaged system, and the associated inelastic bending of the beam prior to the start of the progressive collapse test.

FIGURE 12
'TRADITIONAL' MOMENT CONNECTION: COMPLETE COLUMN DEMISE AND SEVERE GLOBAL DEFORMATION OF TEST ARTICLE WITHOUT COMPROMISING THE CONNECTION, ABSENT FLOOR GRAVITY LOADS.

FIGURE 13
SIDEPLATE® MOMENT CONNECTION: COMPLETE COLUMN DEMISE AND SEVERE GLOBAL DEFORMATION WITHOUT COMPROMISING THE CONNECTION, ABSENT FLOOR GRAVITY LOADS.

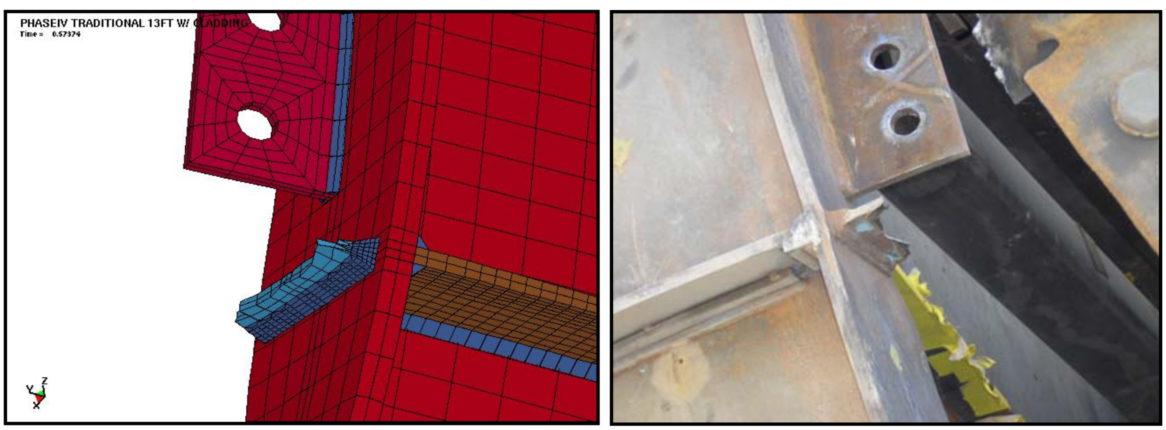

FIGURE 14
BRITTLE FRACTURE OUTLINE AND LOCATION IN BEAM'S BOTTOM FLANGE, PREDICTION VS. TEST FOR THE 'TRADITIONAL' CONNECTION TEST ARTICLE.

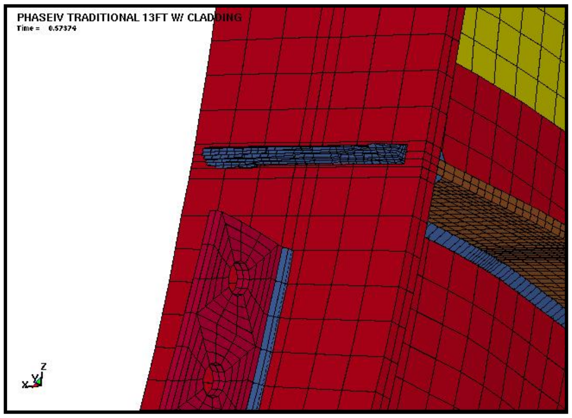

FIGURE 15
BRITTLE FRACTURE OUTLINE AND LOCATION IN BEAM'S TOP FLANGE GROOVE WELD, PREDICTION VS. TEST FOR 'TRADITIONAL' CONNECTION TEST ARTICLE .

Moment Connection Type	Condition of Test Article before Start of Test	Vertical Displacement/Ram Force at First Fracture	Maximum Vertical Displacement/ Ram Force at End of Test	Joint Rotation at First Fracture (elastic plus plastic)	Joint Rotation at End of Test	Mode of Failure
'Traditional'	Blast Damaged	9" / 70,000 lbs.	17" / 44,000 lbs.	4.2% radians	7.8% radians	Connection Failure
SidePlate®		21" / 118,000 lbs.	27" / 82,000 lbs.	9.8% radians	12.6% radians	Beam Failure
'Traditional'	*non*-Blast Damaged	16" / 42,000 lbs.	27" / 52,000 lbs.	7.4% radians	12.6% radians	Connection Failure
SidePlate®		32" / 138,000 lbs.	38" / 91,000 lbs.	14.9% radians	17.7% radians	Beam Failure

TABLE 2
PROGRESSIVE COLLAPSE TESTS - SUMMARY COMPARISON OF CONNECTION TYPES

e) As with the 'Traditional' moment connection, the SidePlate® moment connection, including its column tree-to-link beam splice connection, survived the blast test without any observable material compromise to any of the connection welds and components while undergoing the effects of complete column demise and severe global deformation, including a significant inelastic twist of 35 degrees off vertical. (Figure 13). Again, as with the 'Traditional' connection test, the column was significantly damaged. The connection of the column to the base plate failed, allowing the base of the column to move laterally. Overall, the column laterally displaced at its midpoint approximately 42 inches. Although the magnitude of the column displacements is different, due to the timing of the base plate failure, similar behaviors and magnitudes of damage were observed in the finite element analysis predictions for the SidePlate® moment connection using LS DYNA (Figure 16 and 18).

For each of the two progressive collapse test articles employing the SidePlate® moment connection, the mode of failure was controlled by exceeding the strain-hardened capacity of the spandrel beam (Table 2). The beam's failure was initiated by a gradual tear along the end of one of the two bottom horizontal fillet welds connecting the connection's cover plate to the beam flange tips, before turning diagonally into the beam's bottom flange wherein additional tearing occurred over approximately a quarter of the full flange width before complete abrupt fracture occurred across the balance of the flange, progressing a short distance into the web. Following complete fracture of the beam's bottom flange, the beam's web experienced gradual vertical tearing across its depth to a height just below the fillet transition between the web and top flange of the beam (Figure 17), culminating with failure in tension of the spandrel beam's top flange for the *non*-blast-damaged test article. Note that for the blast-damaged test article, axial tension failure of the spandrel beam's top flange was not reached because the test was prematurely stopped due to lack of available ram stroke before complete beam failure was achieved. Subsequently, the test apparatus used for the *non*-blast-damaged test article was modified to accommodate the anticipated need for additional stroke. A comparison of the recorded progressive collapse performance between connection types tested is presented in Table 2.

It is noted that, early into the *non*-blast-damaged progressive collapse test the beam experienced an acute buckle of the top flange, due to compression forces and the flange's unsupported length across the mouth of the weld access hole at the link beam splice, which in turn caused an abrupt drop in the ram force history (Figure 10). This occurred at approximately 5" of displacement, before reversing itself as axial tension in the beam began to build.

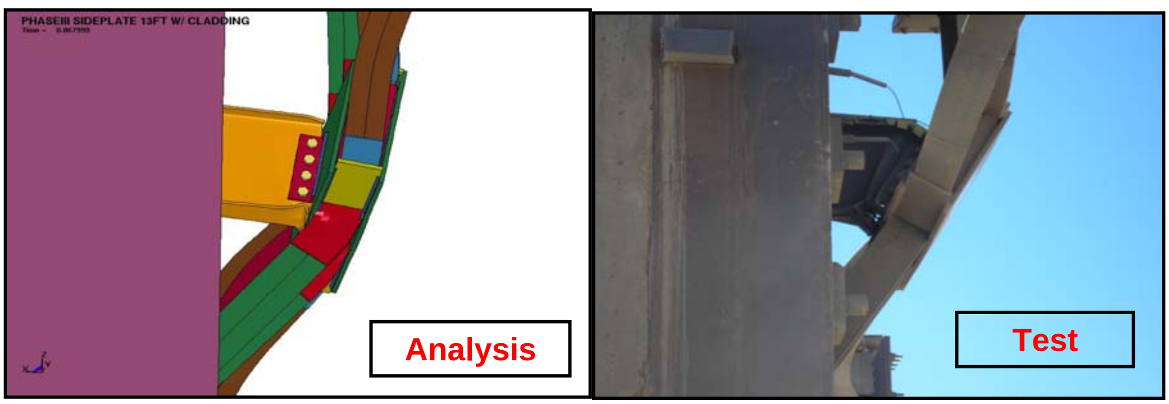

FIGURE 16

SIDEPLATE® MOMENT CONNECTION: PREDICTIVE BLAST ANALYSIS CORROBORATES INELASTIC TWIST AND VERTICAL DISPLACEMENT OF DOUBLE-SPAN BEAM.

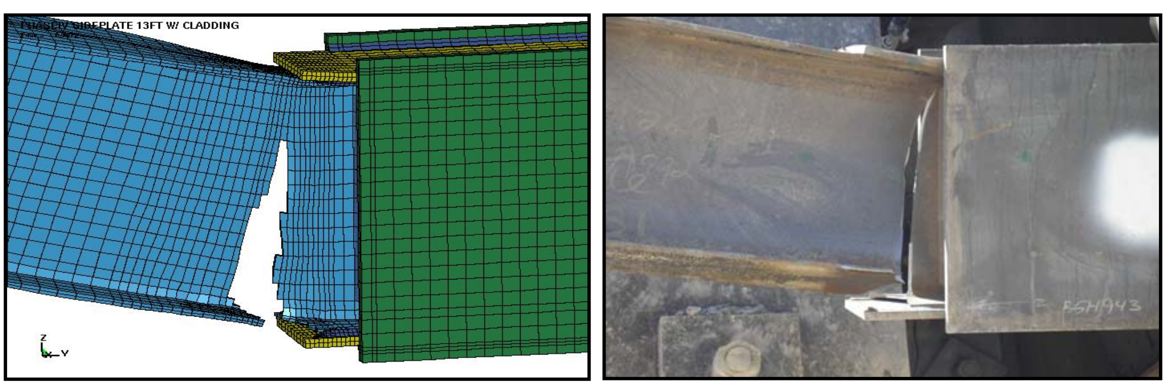

FIGURE 17

DUCTILE FRACTURE PROPAGATION THROUGH BEAM'S WEB, PREDICTION VS. TEST FOR SIDEPLATE® CONNECTION TEST ARTICLE.

3. Predictive blast and progressive collapse analyses tools, and alternate path design methodologies, have been developed and validated.
   a) Qualified structural engineers, using advanced structural engineering analysis methods and commercially available analytical tools, can adequately predict direct air blast damage and progressive collapse potential of blast-damaged steel frame construction and associated connections, thereby minimizing the need for future costly and time-consuming full-scale blast testing. Furthermore, standoff distances can be competently determined by analysis on a project-specific basis for a given threat, thereby minimizing the need for prescribed 'minimum standoff' distances [2]. The use of air fluid/structure modeling techniques, coupled with parametric HFPB simulations of critical connection components and welds, and the selection of credible non-linear material models, can be used to reliably and cost-effectively predict steel frame and connection behavior when subjected to direct air blast attack. Such codes and analysis techniques can be used to develop simplified tools and criteria that can be used by design professionals.

      A comparison of analysis results to recorded test displacements is presented in Figures 18, 19, 20 and 21 for the two blast tests, and Figures 22 and 23 for two of the progressive collapse tests.

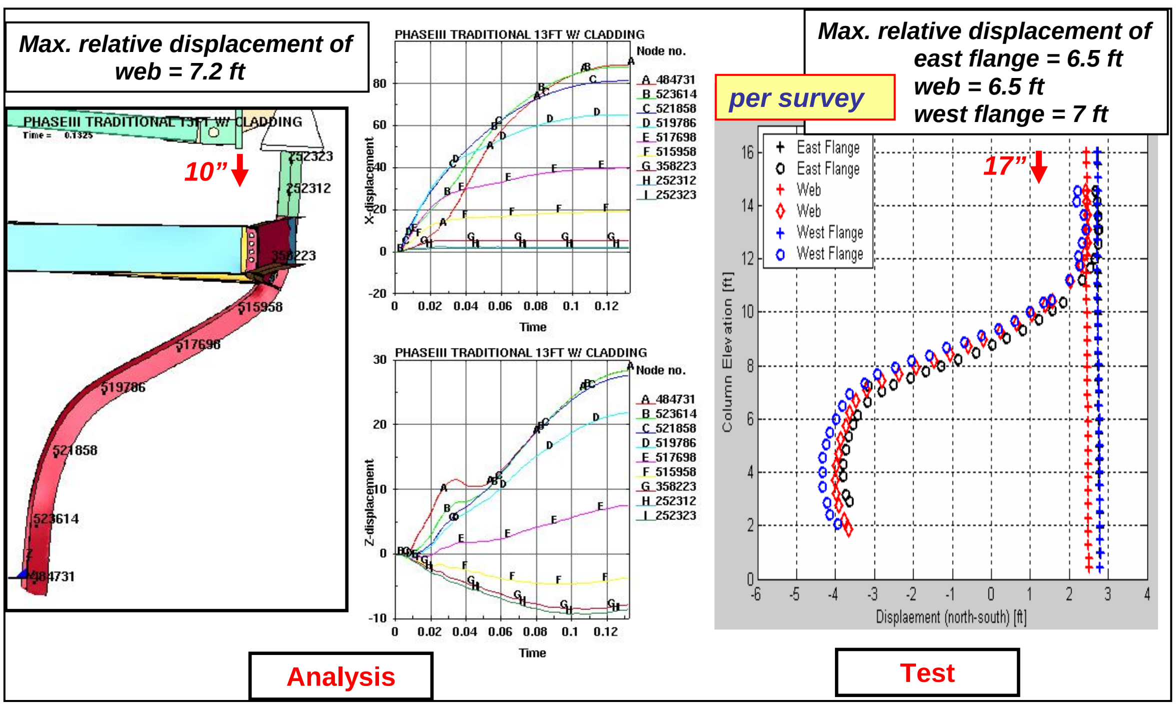

FIGURE 18

'TRADITIONAL' MOMENT CONNECTION: COMPARISON BETWEEN PREDICTED VS. FINAL TEST ARTICLE COLUMN DISPLACEMENTS.

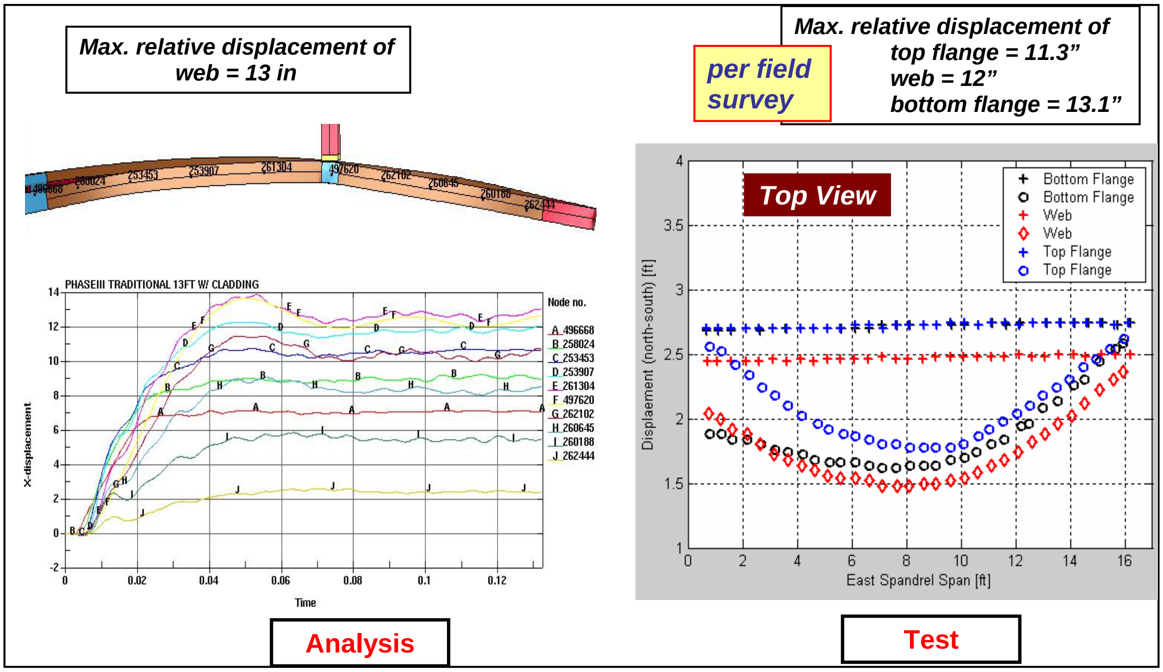

FIGURE 19

‘TRADITIONAL’ MOMENT CONNECTION: COMPARISON BETWEEN PREDICTED VS. FINAL TEST ARTICLE BEAM DISPLACEMENTS.

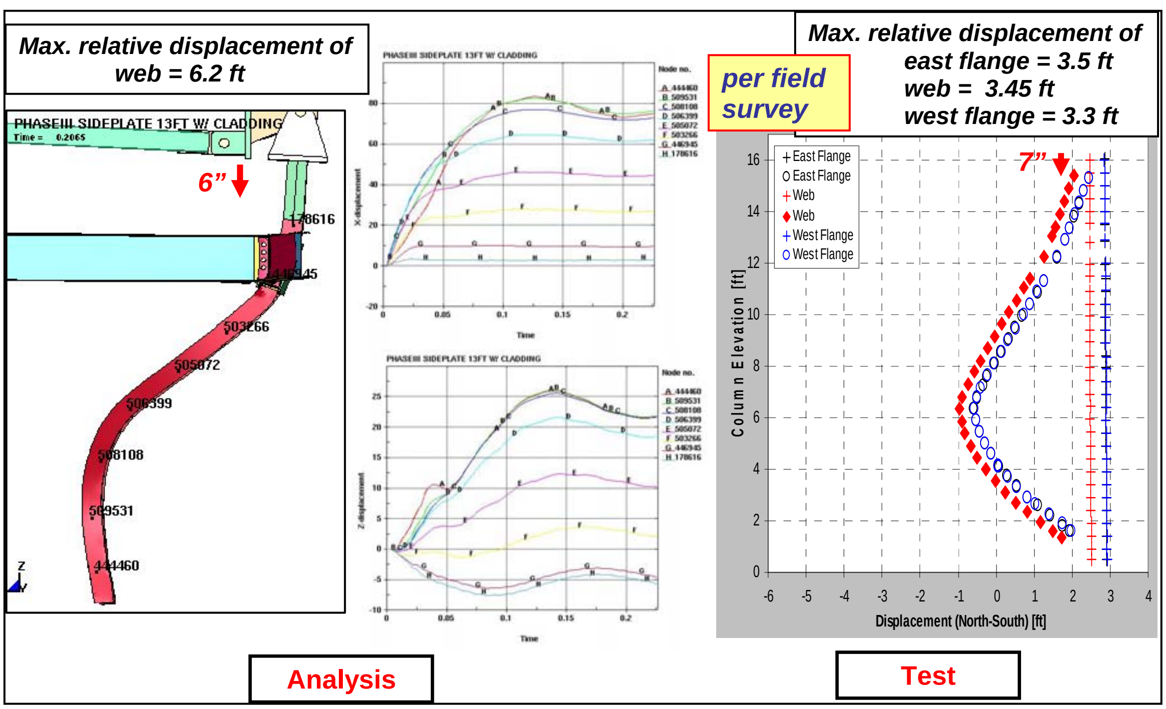

FIGURE 20

SIDEPLATE® MOMENT CONNECTION: COMPARISON BETWEEN PREDICTED VS. FINAL TEST ARTICLE COLUMN DISPLACEMENTS.

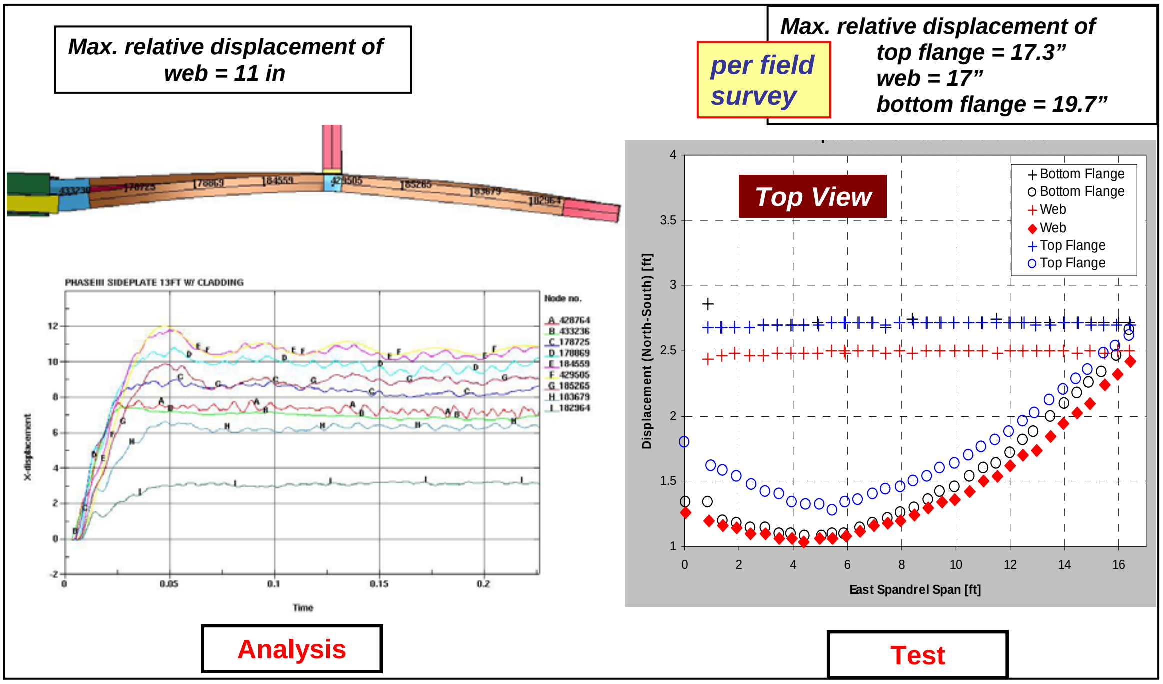

FIGURE 21
SIDEPLATE® MOMENT CONNECTION: COMPARISON BETWEEN PREDICTED VS. FINAL TEST ARTICLE BEAM DISPLACEMENTS.

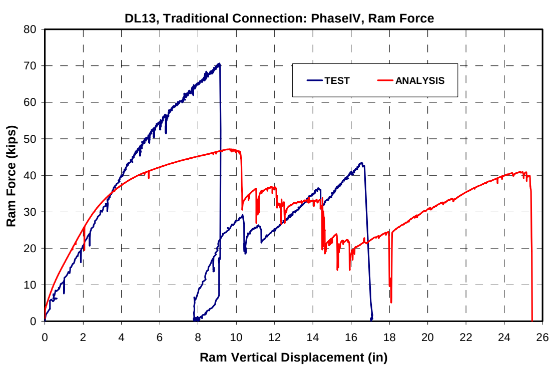

FIGURE 22
‘TRADITIONAL’ MOMENT CONNECTION (BLAST-DAMAGED): COMPARISON BETWEEN PREDICTED VS. RECORDED TEST PERFORMANCE HISTORIES.

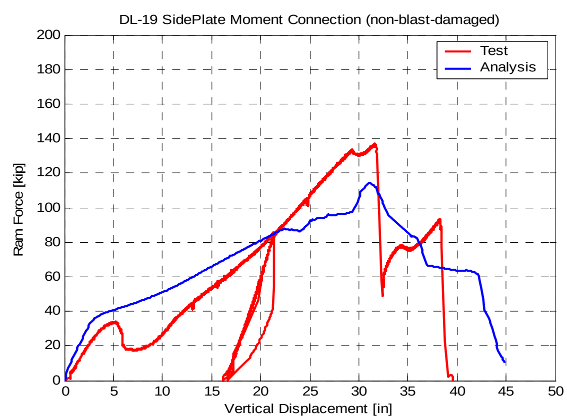

FIGURE 23
SIDEPLATE® MOMENT CONNECTION (NON-BLAST-DAMAGED): COMPARISON BETWEEN PREDICTED VS. RECORDED TEST PERFORMANCE HISTORIES.

b) ‘Missing Column’ scenarios, as prescribed in ISC’s ‘alternate path’ design methodology to mitigate progressive collapse, are clearly credible events, as evidenced in each of the blast tests, thereby underscoring the credibility of this design approach to mitigate progressive collapse.

SUMMARY & CONCLUSIONS

The behavior of steel frame test articles subjected to a specified blast event can be reasonably predicted with the use of HFPB computer simulations. While further calibration of material models is anticipated and recommended, collectively, the analytical and modeling approaches used resulted the ability to a) obtain the desired and/or expected critical deformations and high-order strains that are needed to challenge the test article, as evidenced in the actual tests conducted; and b) identify promising solutions for the mitigation of bomb blast effects and progressive collapse.

One of the primary aims, if not the ultimate objective, of the GSA Test Program is to identify steel frame beam-to-column connection systems that constitute effective solutions, capable of mitigating the effects of blast and arresting progressive collapse. An effective solution for achieving this performance expectation must not only be economically viable, but must also be able to satisfy technical requirements, physical attributes, and multi-hazard capabilities, which include the following:

1. Ability to resist the effects of blast and debris impingement, including the effects of extreme global inelastic deformation, and do so without significantly comprising the residual ductility and strength of the framing system.
2. Ability to resist sustained *post*-blast gravity load demands, when subjected to a '*missing column(s)*' condition resulting from an actual blast-damaged vertical support member, including the interaction of applied bending moment and axial tension in the beam.
3. Ability to achieve adequate rotational capacity in resisting sustained gravity loads.
4. Ability to develop the full inelastic capacity of the connecting beam.
5. Ability to *minimize* the danger of brittle fracture by using welds and component geometries in their most ductile orientations.
6. Earthquake performance prequalification by an independent, nationally-recognized jurisdictional authority.

Based on the results to date of actual blast and progressive collapse tests of full-scale steel frame assemblies, with adequate design and careful selection of connection geometries that exhibit all of these attributes and performance qualifications, steel framed structures can provide protection from blast induced progressive collapse. Although both connection types survived the initial blast load intact and subsequently displayed varying amounts of post-blast capacity to resist gravity loads, the SidePlate® moment connection system achieved significantly higher load and rotational capacities than the 'Traditional' WUF-B moment connection configuration, with up to 5 times the external energy at first failure. The results were consistent for both the physical testing and the corroborative predictive analysis. Based on tested performance, the SidePlate® connection system was more effective, and thus a more robust solution for combating the effects of a terrorist bomb blast, including progressive collapse, on steel framed multi-story buildings.

Additional information and documentation supporting the principal findings of the GSA Test Program, as summarized herein, were presented by the GSA/OCA to the Executive Director of the ISC, U.S. Department of Homeland Security (DHS), on May 25, 2006, and are available to design professionals and general contractors who are directly involved in Federal government projects, upon request [7].

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