# Retrofit Methods to Mitigate Progressive Collapse

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## 1 INTRODUCTION

This paper describes retrofit methods suitable for mitigating progressive collapse for multistory buildings. Several different gravity support systems are considered; these are listed in Table 1-1 along with some of the risk factors associated with them.

## 1.1 PROGRESSIVE COLLAPSE DEFINITION

Progressive collapse due to a terrorist attack could occur to almost any conventionally designed building under sufficiently large and widespread loading. The goal is not so much collapse prevention as making collapse difficult for a terrorist to achieve. The intent is that any damage incurred should correspond with the magnitude of threat delivered.

## 1.2 PROGRESSIVE COLLAPSE RISK FACTORS

As the risk factors given in Table 1 imply, evaluating the risk of progressive collapse may be complicated. This complication is exacerbated by the uncertainty inherent in both the magnitude and nature of the loads and response engendered by a terrorist attack, where the attack may include thermal, impact, and blast (TIB) loads, which the structure must sustain both immediately and for some time after the attack to provide both time to evacuate occupants and treat the casualties. In essence, the retrofitting of a building to prevent progressive collapse in a terrorist attack involves consideration of a variety of factors, which will likely boil down to adding ductility, load paths, capacity, and continuity to the building's gravity support. Where, how, and what to add is the tricky part.

### *1.2.1 Structural Components*

Progressive collapse is related to loss of gravity load capacity in the structural frame, for example, resulting from the inability of the structural system to redistribute the load after the failure of a column. This may be caused by a lack of frame continuity and/or capacity, or other mechanisms, most of which fall into the category of load paths not considered in the original design, presumably because they were deemed to be of negligible importance—as in, this could never happen, or it would be too costly to consider, or we did not think of it. The loss of capacity of a member may result directly from the terrorist attack as in a blast load or as a consequence of the response of other members, for example, added unsupported length, loss of connection, lack of equilibration forces.

Identifying and preventing these conditions, thus, becomes the chief means of designing away from or mitigating progressive collapse, which in turn dictates the type of retrofit needed. Cost, instability, and the ability to predict response of the retrofit are other key factors in generating and evaluating retrofit concepts. Moreover, some members and connections are inherently more vulnerable to TIB effects than others. Identifying and understanding these differences is a key initial step in progressive collapse mitigation. Some examples include

- Bearing walls. Buildings using these walls as their primary structural support system are often viewed as weak with regards to resisting blast loads. However, this doesn't mean that progressive collapse cannot be prevented.
- RC Columns. Poorly confined RC columns (of almost any size) are inherently prone to lots of axial capacity due to damage.
- Steel connections.
- Loss of prismatic behavior of steel members.

### *1.2.2 Structural System*

Like structural components, some structural systems are inherently more vulnerable to TIB effects than others. Failure may happen in a variety of ways, but perhaps of more importance is identifying buildings and mechanisms that even though damaged, collapse did not occur. Figure 1 illustrates this, where

- **CTS-1.** CTS-1 is a four-story test article with 20-foot bays. It uses a reinforced concrete flat slab design with a perimeter spandrel to sustain the gravity loads. This design has proved to b e more rugged than initially believed since it has survived repeated close-in bomb loads.

- **Murrah Building.** The progressive collapse of the Murrah Building was primarily related to a framing system that could not sustain the initial column caused by the bombing. This may be contrasted (with) to the CTS-1 structure whose framing could sustain the needed redistribution of the gravity load. Both buildings were designed for regions where seismic activity is low.
- **WTC.** The WTC design peculiar in its reliance on bar joists as a key element in its framing design and the use of nearly continuous number perimeter columns

- **Khober Towers.** This building is composed of precast panels interconnected with embedded cabling. Unlike the above buildings, the lateral and vertical loads are carried by bearing walls.

These buildings illustrate that identifying both the mechanism of collapse or whether a collapse may occur under a particular TIB load is not going to be easy. The issue for retrofitting the buildings at risk of collapse is identifying the key mechanism(s) involved and finding ways to prevent their occurrence. The issue for buildings like the non-collapsed ones shown in Figures 1b/c is to be cleaver enough in predicting the outcome so that retrofitting is not performed where it is not needed.

## 1.3 DESIGN PARADIGM

The different levels of damage shown in Figure 1 imply that design can play a major role in the amount of damage incurred. In particular, the relatively small amount of damage occurring at Khobar Towers, where such a large bomb was used, illustrates how though some designs can be. Conversely, the damage incurred by the Murrah building seems to indicate a design with excessive vulnerability.

The difficulties likely to be encountered in developing retrofit designs for preventing progressive collapse, in the context of the vulnerabilities illustrated by the building responses shown, are likely to challenge our current capabilities in generating suitable retrofit design concepts and in predicting their response and impact related to preventing progressive collapse. To provide the engineering capability needed to adequately mitigate progressive collapse for a building to the thermal, blast, and impact loads that could be delivered in such attacks will require addressing the following five major areas of concern:

- The need for tools that can simulate the actual physics that occurs in the response of a building were it subjected to extraordinary load states like those shown.

- The desire of the owner/developer to pay the extra costs associated with the level of engineering sophistication likely to be needed to create cost effective, resilient framing systems for buildings.

- The leadership of academia and professional society communities in providing the needed training and professional standards.

- The Government for providing the research and development programs needed to generate performance data of sufficient detail and precision to demonstrate the effectiveness of the designs and to validate the analytic tools needed to support the design efforts.

- An atmosphere that fosters the development and use of new materials and design concepts.

Potential terrorist attacks are likely to involve increasingly outrageous insults. Improving our capability in the above key areas is paramount to reducing the uncertainties inherent in understanding and predicting such building responses and to mitigating the consequences of such attacks.

To effectively address the issue related to retrofitting structures to prevent progressive collapse will require changes not only in terms of upgrading engineering skills/tools, but also in the design paradigm itself. For example, what constitutes a design load; the notion of a safety factor for such an ephemeral load (as in uncharacterizable and rare) as is often related to the terrorist attacks against US buildings; how does an owner or building approval authority approve a design with such features; what constitutes an appropriate set of professional standards and practices.

The examples presented in Figure 1 illustrate the above points. In all the cases shown, the damage incurred was unexpected—that is, in the broadest sense of the term. Some did not expect such a load; some thought the actual damage incurred was far above expectation; and some thought the building performed extraordinarily well. Of paramount concern, however, is our capability to either have predicted the building's non-collapse and done nothing or to develop cost effective retrofit designs and design processes to knowingly address situations like those where collapse did occur.

### 1.3.1 Design Objectives

For terrorist attacks, we are looking at both preventing local failures and providing load redistribution so as to prevent progressive collapse should they occur. This requires definitively employing the concepts of continuity, ductility, and capacity for both the building's framing and its individual members and connections. This also means that the mechanics of the response must be better understood and predicted if we are to effectively address such futuristic designs.

### 1.3.2 Key Features of Retrofit Design

One key feature common among designs that perform well under blast loads is their reliance on ductile, plastic behavior and redundant load paths to achieve maximum protection. Making ductility a key feature of a design is a well trod approach for protecting buildings from seismic forces and in building hardened military structures; but it is not well understood by many structural engineers and blast consultants, who often seem focused on using brute strength to resist what are often very large forces. In contrast, strength concepts are often grossly inefficient and out of keeping with traditional architecture schemes, cost a lot to install, and are likely to increase the demand placed on the building by a blast or impact load. Moreover, the ability of structural frames to perform their job with a minimum amount of material often results in less disruption to the facility's operations and esthetics.

A second key feature of a good retrofit design is their consideration of shock related behaviors, that is, responses that are directly coupled to the intensity of the load. A study illustrating this point is shown in Figure 2, where a uniform aluminum beam is subjected to increasingly higher intensities of impulse. These results demonstrate that as the ratio of impulse to system mass (i.e., $I/m$) becomes large, particular attention must be paid to the design in the vicinity of the support. This is a general phenomenon that occurs in many kinds of conventional structures (e.g., in reinforced concrete columns and slabs where a shock load must be accommodated)—that is, the structural system in the vicinity of its anchorage/supports/etc. must accommodate large differences in early time motions. Otherwise, plastic response and ductility give way to tearing and a premature system failure. It is important to design away from this brittle response mode.

A third feature, missing from many designs, is that a design should not increase the risks. Unlike other structural loads, failure of a blast-resistant design is a realistic option because there is always a "bigger" bomb. Therefore, where such a failure occurs, it is important that whatever measures were taken to improve a building's resistance do not increase the risks to the building's occupants.

### 1.3.3 Retrofit Design Concepts for Preventing Progressive Collapse

Of key importance in the design process is determining the potential for progressive collapse for a particular building and any retrofit thereof. This may be addressed with the six-step process listed in Table 2, which illustrates the inherent difficulty—that is, of developing an integrated, balanced, reasonable, and fidelic process for what is, at best, a complex and difficult task. Determining the relative vulnerability to a specific set of terrorist attack loads for different classes of steel frame and reinforced concrete buildings for different retrofit design options is even more difficult.

#### 1.3.3.1 DESIGN TOOLS

Current design practice being promulgated by some US Government agencies [1-3] primarily relies on a "missing column" scenario to address the progressive collapse issue. This design guidance is apparently intended to be palliative rather than preventive, but regardless of intent, the likely outcome, at least in a de facto sense, is that such analyses will be taken as indicative of a strategy to reduce the risks associated with progressive collapse.

In addition, the current guidelines rely on a design strategy often employed in designing hardened structures, whereby member failure is related to support rotation—that is, the ductility, capacity, and continuity of a member are defined in terms of an allowable support rotation. Historically, the principle domain of applicability for these codes and the rotational failure criteria they employ was in performing BDA of conventional buildings and designing hardened structures to resist nuclear loads. These are fundamentally different applications than that of designing retrofits to prevent collapse of conventional buildings—that is, the efficacy of utilizing such codes for BDA or hardened structure design should not imbue them with sufficient credibility to allow their use in satisfying the professional responsibilities and liabilities implied by a set of stamped drawings for the retrofit of an existing building. If, on the other hand, these criteria are intended to provide only a ROM (rough order of magnitude) estimate of the adequacy of the design to meet a specific condition, then well and good; but then their use leads to a fundamentally different design process than used for the traditional loads on buildings—that is, a good enough standard. While ultimately the use of a "good enough" design process may be the most efficacious way to accommodate terrorist loads, it is imperative that the ramifications of such a standard be explored and overtly stated in the design process.

#### 1.3.3.2 DEFORMATION CRITERIA

Components of existing buildings are expected to have a broad range of ductilities and the degree of continuity and redundancy provided by the frame can be highly variable, which does not allow retrofit design to fit well with simplified design procedures and failure criteria like the rotation limit criteria. Moreover, the focus of these design limit criteria is the flexural response, where the simplified engineering tools (e.g., SDOF and FACEDAP [4], that is, models based on single degree-of-freedom methods or P-I curves) and these criteria are most compatible and most likely to work satisfactorily. Parenthetically, other component responses—such as shear response and those responses highly influenced by the gravity loads (e.g., column buckling)—are not readily addressed with such criteria. Nor are the response of many retrofit systems that may be composed of disparate materials likely to be well addressed either. System responses caused by TIB loads, like those related to progressive collapse, are not readily addressed at all.

With regard to these strength and ductility issues, of particular note are the recent static and blast effects tests conducted on full-scale square and rectangular RC columns [5]. These columns had both lower and upper levels of confinement and used a kinematic constraint similar to that occurring were they placed in a building. For the columns with little confinement, both static and dynamic tests resulted in severe damage due to shear failure. For columns with upper bound confinement, end rotations of above 20 degrees were measured (Figure 3). Moreover, at 20 degrees, the column was still carrying its full share of the building's gravity load along with a lateral load of 400 kips.

While the above indicates that measured values can be significantly better than the design criteria, there is also the problem of the wide variability of rotational capacities for actual situations. Stating, for example, as the DoD guidelines do, that "the designer is to ensure that the rotation criteria of the guidelines are met" is difficult with the uncertainty extant in connection performance and member behavior, as alluded to above. This is illustrated by the hypothetical connection behavior depicted in Figure 4, which is inherent in the joints of most steel framing relative to its behavior under TIB loads.

Moreover, the use of such criteria is not likely to achieve an optimal reduction in occupant risk, is likely to produce confusion among designers and others in trying to use them, and may create a false sense of security. If such criteria are to be used, their use should be based on a definitive and documented study of the use of rotational criteria in design of conventional buildings, especially as related to TIB loads. It is likely that such an approach can be used, but only in the context of defining these criteria in terms of more than just member type, for example, including the influence of member properties, and connection properties and geometry. This could be accomplished, at least, for a limited set of standard or widely used member and connection designs.

The use of allowable support rotations for RC or steel frames vis-à-vis determining the potential for progressive collapse too easily results in unwitting conclusions about the ductility and capacity of the structural system that are unwarranted and potentially dangerous. Criteria like these may be adequate for qualifying component response, but are a risky means to determine a building's potential for progressive collapse.

In particular, for a steel frame, the following issues would need to be investigated and resolved to achieve the joint rotational ductilities likely to be needed to prevent progressive collapse.

- The need for prescriptive detailing of moment, pinned, and brace connections so as to ensure their performance under TIB loads, for example, related to their providing sufficient girder-to-girder continuity across a failed column and sufficient post-attack reserve capacity, redundancy, and ductility. This could require a document similar to the prescriptions provided by FEMA for seismic connections (i.e., FEMA 350 [6]) but with the added feature of determining/computing allowable support rotation and support resistance functions.

- A prescriptive means to mitigate the effect of sustained high-intensity thermal loads on connections and members as a function of connection geometry, column and girder geometry, temperature, time, protection system, attack mode, steel type, and framing system.

#### 1.3.3.3 Need for Tools

Unlike new construction, which is typically of homogeneous materials and uses members homogeneous in their response behavior, retrofit construction often requires disparate materials to work together in ways that are not easily predicted. Retrofit systems are inherently more difficult to design and predict their outcome.

High-fidelity physics based finite element models are likely to be needed to both select the appropriate design parameters and ensure that they work together to provide the collapse mitigation needed. As will be shown in Section 2, some of the retrofit concepts suggested may require models well beyond the comfort zone for most engineering offices and budgets.

### *1.3.4 Terrorist Loads*

Terrorist loads may involve a combination of thermal, impact, and blast (i.e., TIB) loads. As compared to gravity and seismic loading, TIB loads related to a terrorist attack produce some structural behaviors uniquely related to themselves, such as:

**Blast Loads.** Close-in HE (high energy) detonations produce localized high magnitude pressures affecting only a relatively small part of the structure. They also produce widespread debris fields that may place at risk the occupants of many buildings even though structural damage is typically narrowly confined. The primary difference between blast (and impact) loadings and conventional loadings is their rapid load rate—the significant loading is typically over in less than 5 ms. The rapidity of load can excite higher structural modes or breaching failures that are usually neglected for other types of hazardous loads, such as earthquakes.

Detonations farther away produce a wider spread of damage, but likely less risk of collapse. Here, wall, glazing, and possibly roof damage are likely consequences. Pressures here may be in the hundreds of pounds per square inch, rather than the thousands for closer in HE, with durations substantially longer. Explosions from materials other than HE (e.g., hydro carbons) are also a possible source for blast loads.

**Thermal Loads.** Sustained high temperature (i.e., deflagration vs. detonation) loading of the structural frame, particularly steel ones, produces similar complexities in determining system performance as a blast load because of the need to postulate and characterize an event; then determine the degraded capacity of members and connections; and finally evaluate the framing systems continuity, capacity, and ductility to redistribute and sustain the gravity load. The key aspect of thermal loading is its ability to degrade the properties of the structural members. The effects are likely to be an on-going spatially dynamic process; but the end result, vis-à-vis, progressive collapse is similar to that related to blast and impact, in that, the consequence is related to the framing system's ability to redistribute the gravity loads to accommodate the degradation.

However, unlike blast loads, there are some readily available technologies for preventing thermal degradation, even indefinitely, which include high-temperature resistant exterior surface coatings and wrappings of structural members (e.g., carburet-containing polymers), as well as new connection technologies that provide more resistance to collapse in a fire.

**Impact Loads.** Vehicles and aircraft provide a ready means for a terrorist to apply a high-energy impact load to seriously damage a structure by slicing through building columns to precipitate structural failure mechanisms leading to progressive collapse and/or to gain access for delivery of HE or incendiary devices. Another source of impact loading that might cause the structural system to fail is debris or objects propelled by the airblast into the framing. This would include K-rails (i.e., Jersey barriers), heavy equipment, vehicles, cladding elements, etc. Fortunately, the effects of these types of secondary impact loads can be mitigated by careful planning and designs to limit their occurrence.

Impact loads may be complicated to define because they involve the compliance and kinetic energy of the delivery system (e.g., a vehicle), its shape, and other difficult to characterize factors. However, crashworthiness and other studies provide a ready source for developing the means to characterize these types of loads.

### *1.3.5 Developing an Effective Means for Threat Assessment*

The TIB loads must ultimately be quantified in such a way as to incorporate them into the design process. To do so effectively within the multi-jurisdictional environment likely to govern will require teamwork on an unprecedented scale, organizational structure, use of new products and methodologies, and recognition and procurement of the engineering skills and tools needed.

Ideally, specification of the terrorist threat, especially in terms of TIB loads, should be a process that not only reflects the national threat assessment but also accounts for local conditions and such factors as the marquee status of the building and its occupants, ease of access, etc. However, recent events and a lack of well established precedents and processes have added a marked ad hoc flavor to threat selection and the requirements placed on building owners, vis-à-vis, these threats. It would be more palatable if threat specifications could be a consensus effort between local authorities (e.g., local police agencies, building owners, etc.) who are on-scene and have the responsibility of maintaining a safe environment and getting the resources to do so (i.e., equipment, funds, etc.), and national agencies, such as the FBI, CIA, DIA, FAA, and GSA who look at the threats nationwide or worldwide.

Moreover, the current practice of separating the two functions of threat definition and blast effects analysis does not allow threat importance to be established as related to building damage, economic loss, number of casualties, etc., because the consequence of a particular threat is not realistically tied to the likelihood of the threat postulated. This is more important in retrofit design than new construction because so many of the important design parameters (e.g., standoff, type of construction, glazing, layout) are difficult to significantly change. Probably of most concern from a building owner's view is that threat selection needs to be a balanced process, such that the process distributes the mitigation measures in a way that minimizes the overall risk rather than focuses on specific concerns, such as window breakage.

A software tool for threat assessment on a building, neighborhood, and city basis is needed to properly define the risk and adequately develop the resources and solutions needed. Rudiments of such a tool already exist in such codes as AT Planner [7] but must be refined and extended to encompass modeling of an entire event. This would include modeling risks to all the occupants of the buildings and cityscapes affected, and require good estimates of casualties and deaths, and building and infrastructure damage over the affected zone.

### *1.3.6 Factors Inhibiting the Design of Retrofits*

Major contributing factors inhibiting development of a better retrofit designs for mitigating progressive collapse include (1) the difficulty of characterizing the actual threat (mainly because it's difficult to do), (2) tying structural response to actual occupant risk with sufficient specificity, and (3) lack of a broad set of innovative and accepted design concepts for structural components that perform effectively and predictably in a terrorist attack environment—that is, providing a high degree of certainty in preventing progressive collapse and preventing excessively hazardous debris will not easily be achieved. Moreover, the inherent difficulty of generating effective retrofit designs to address progressive collapse provides a challenge to both engineering skill and in finding appropriate materials and concepts to achieve the needed resilience—that is, because these systems are generally more complicated than new designs, especially in terms of getting the disparate behaviors and properties of the retrofit design to work together effectively, and as planned.

What is needed as the basis of design, but difficult to achieve, is a standard and practice based on sufficiency, as in good enough. There is always going to be a bigger bomb or a standoff too close for the risk to be mitigated or to overwhelm the design scenario employed. What terrorist-resistant design should achieve is a sense, perhaps even a consensus, that should a terrorist attack occur, the mitigation used was good enough. This of course is a difficult standard to define and probably impractical to implement.

Perhaps more achievable and a standard used in other similar design studies is a risk based design strategy that computes the risk of serious casualties and deaths. This strategy would require that risk be definable as a function of a series of plausible terrorist attack events and as a function of the particular design. The owner would specify the acceptable risk (e.g., $10^{-5}$ deaths per year). Unfortunately, this would require support calculations considerably more sophisticated than at present and a system for defining terrorist events and their probability.

In contrast, instead of a threat related performance criteria, design requirements are often mandated through policy decisions that, while likely to reduce risk at specific sites, are also so costly and broad based that they produce a far from optimal cost/benefit for the US populace or even a specific site. These requirements may even increase the risk because of inaction due to high costs or inadequate funds, or even a wasted effort because the mandated procedures are ineffective.

Inaction and wasted efforts are particularly troubling concerns relative to retrofitting existing buildings; in this situation, the issues are more likely to be misaddressed because they are more complex, primarily related to lack of technology or ignorance of newly available technologies. Existing buildings are best approached with a risk-based retrofit strategy to focus the efforts where they are really needed. Retrofit designs inherently require more skill from the designer, better design tools, and concepts that can accommodate a broad range of loads and site conditions.

### *1.3.7 Summary*

The ability to produce resilient designs to retrofit existing buildings is complicated by the lack of knowledge in the engineering, policy maker, and academic communities related to the effects of terrorist attack loads and the response of damaged structures. Much of this is due to the relatively recent need for these types of designs; the unique nature of civilian structures as compared to hardened military structures, where loads like these (e.g., blast loads) have always been a primary concern; and the security concerns related to much of the data and analytic models developed primarily within the DoD community.

As indicated by the material provided in Section 1, the development of retrofit designs relative to preventing progressive collapse is likely to be a complex process for many buildings. Uncertainty in the results will likely markedly add to the difficulties. Much in the way of basic engineering and research is still needed.

# 2 STRATEGIES FOR RETROFITTING STRUCTURAL SYSTEMS

## 2.1 APPROACH

Some approaches for retrofitting an existing building to prevent progressive collapse are given in Table 3. Retrofit methods are also listed in the table under the various approaches. Inherent in each approach is the need to evaluate the building's existing proclivities to progressive collapse and which proclivities need mitigation and which approach and retrofit method is most appropriate. This will require a high level of engineering skill as well as an extensive knowledge of behavior of complicated structures under TIB loads, and the analytic tools needed to support the development of the retrofit design.

### *2.1.1 Gravity Support Systems*

Strategies for improving the resilience of the gravity support systems that are listed in Table 3 are described below. Additional strategies could be included, but for the purposes of this discussion, these seemed sufficient. Moreover, each site would have its own peculiar needs relative to adding resilience, vis-à-vis, progressive collapse, which makes it likely that any concept described herein would need substantial tailoring to work at a specific location.

#### 2.1.1.1 FRAMES

Three approaches are identified in Table 3 for enhancing the resilience of a building's frame: redundant load path, strong column, and missing column. Each of these approaches to resilience has its pluses and minuses and likely combinations of all three would be used in an actual retrofit design. The addition of a redundant load path may allow load redistribution over a broad area of the framing. The strong column approach focuses on the benefits of preserving the column's axial capacity while the missing column approach recognizes the difficulty of keeping all columns intact.

#### 2.1.1.2 BEARING WALLS

Three approaches are identified in Table 3 for enhancing the resilience of a building's bearing walls: back-up wall, strong wall, and ductile wall. The strong wall approach would likely employ some form of fabric retrofit to control the breach area so that its size was kept below that that could cause collapse. This approach takes advantage of a bearing wall's ability to span a fairly large zone of failure. The backup wall is analogous to the redundant load path approach mentioned for frames. This wall would need to be hardened sufficiently to contain the debris from the existing wall as well as pick up the gravity load.

Bearing wall strengths have often been underestimated because compression-membrane behavior is ignored. This may be thought a reasonable conservative approach to assessment. However, it is often a major contributor to wall survival and it does the client a disservice to advise a retrofit when none is needed. Also largely unknown is the size of hole (i.e., zone of failure) that a wall can span, especially when for a multistory building the wall is likely to have a spandrel beam or some other form of support at each floor.

2.1.1.3 OTHERS

The WTC is not addressed herein. However, it is important to recognize that some gravity systems, like the WTC, are not explicitly covered under frames and bearing walls.

### *2.1.2 Load Environments Resulting from Terrorist Attacks*

Tests have demonstrated that the response of the building's primary framing system is highly influenced by the loading scenario (e.g., charge location and size) and the response of secondary and nonstructural components. For example, tests have shown a significant difference in column response depending on clear time of the blast around the column. Significantly more column damage occurred in tests that had infill walls between columns compared to tests without walls. Also, floor slab response is particularly dependent on blast wave propagation into the interior of the building. Both exterior and interior loads can include significant contributions from the debris generated, for example, by cladding failures or by the breakup of perimeter walls. Debris fragments produce both loading through momentum transfer and direct damage to structural members, especially walls.

For some frames in particular, cladding behavior is likely to strongly influence the loads transmitted to the building's frame. For example, consider the case of a typical precast cladding, such as that shown in Figure 5a that is damaged by a close-in blast. If the precast element fronting the columns remains intact, the column loads may be substantially larger than if these sections breakup early on and the column loading is more nearly related to the width of its flange.

In contrast, the cladding shown in Figure 5b is not well coupled to the steel columns and its motion will likely not add much to the column's response. Moreover, the steel columns are encased in masonry, which should add to their blast resistance.

Determining the appropriate load is of paramount importance in retrofit design. In the first instance, correctly assessing whether a retrofit is even needed and where is extremely important. Get it wrong, then as illustrated by the buildings in Figure 1, and either a costly retrofit project is performed which is not needed (Figure 1b/c) or is not performed when it might be appropriate (Figure 1a/d). Predicting the potential for a progressive collapse is both difficult and replete with uncertainties that has possibly enormous costs in lives/property or large costs in construction associated with it.

This theme of cost, uncertainty, and difficulty is repeated throughout every step of the process related to developing retrofits to mitigate progressive collapse.

## 2.2 RETROFIT METHODS FOR STEEL FRAMES

The capacities of a steel frame system to survive a terrorist attack will be predicated on three very different behaviors: (1) formation of plastic hinges, (2) local and/or global instabilities, and (3) fracture and structural discontinuity. In a particular building, all three behaviors or combinations of them are likely to occur and govern the capacity of a particular joint or member and the structural continuity and integrity of the system. Determining the capacity of columns will be particularly difficult since in many situations they are not designed for significant lateral loading. Finally, determining the capacity of the structural system to sustain the redistribution of gravity loads is especially complex because this must be done in the post-damage state—that is, the bomb blast, airplane strike, etc., has damaged a set of components, NOW (1) is the capacity of the remaining components combined with the residual capacity of the damaged ones sufficient to carry the gravity load, and (2) is there enough structural continuity in the framing system to distribute the gravity loads to the remaining components. In the event of a bombing, this redistribution must occur quickly; for a thermally driven scenario, the redistribution may be a continual and relatively slow process.

Determining the capacity of a steel frame in terms of support rotation, as is done in simplified assessment codes [4, 7] and some design guidelines [1, 2] does not provide a realistic or appropriate means to judge a steel frame's capacity to prevent progressive collapse or design a retrofit to do so. As found in recent seismic studies, frame capacity is related to two different aspects of frame behavior:

- **Member Response.** For example, as controlled by plastic rotational strength and deformation characteristics, or local and global buckling.

- **Connection Response.** For example, as controlled by bolt fracture, premature brittle weld failure, and panel zone failure.

Most of the specific behaviors related to frame capacity are not readily addressed by support rotation limit criteria. For example, forces and geometry govern the various buckling failures associated with steel frames; premature brittle fracture of connections is related to connection geometry; and section twisting and subsequent loss of member capacity is related to torsional rigidity.

Member capacity criteria for compact sections may be usefully prescribed in terms of maximum allowable support rotations; however, connection capacity and member capacity for non-compact sections or sections with insufficient torsional rigidity cannot easily be cast in terms of allowable support rotations. As alluded to in Figure 7, joint region behaviors and capacity are governed by a combination of member and connection behaviors, which can be very complex and difficult to predict, especially under a blast load or aircraft impact scenario followed by the demand of redistributing the gravity loading.

Some member response modes are shown in Figure 6. In the 1,000-pound bomb response, the response node is nearly flexural; some local failures are also occurring—for example, the folding of the flange loaded by the blast. For the larger bomb, panel zone shearing at the base causes the column's collapse.

In addition, as illustrated in Figure 4, the forces and mechanics of joint rotation may mean different things for different sections and connection types. Response at a connection may be a combination of plastic, fracture, and buckling phenomena. For example, because of hardening, an initially plastic hinge-like response at the connection can transform to a local buckling response that will still provide plastic resistance to rotation but with a softening aspect. This is further illustrated in Figure 8, where the complex nature of stress/strain at a connection that is just under gravity load is shown. Some of the consequences of these complex stress states, in terms of unexpected connection failure, were observed in the Northridge Earthquake. Examples of observed Northridge failures are shown in Figures 9 and 10. Further complicating the retrofitting process is the variety of connection types that may be encountered, some of which are shown in Figure 11.

Some specific retrofit methods for steel frame buildings are described below. The most classical one of strengthening a member is mentioned first.

### *2.2.1 Strengthening Members*

Strengthening members, especially columns, by adding plates or encasing/filling them with concrete provides a low tech, effective means to add resilience, particularly for a few columns—for example, those on the ground floor. An example of a filled column is shown in Figure 12.

### *2.2.2 Redundant Load Path*

Addition of a redundant load path to a building's frame may be accomplished in a variety of ways. Two examples are shown in Figure 13: one interior to the frame constructed floor by floor, and another constructed outside the frame spanning multiple floors.

### *2.2.3 Cabling*

Cabling has been employed in new construction to aid in developing a missing column strategy, as illustrated in Figure 14. This concept may also be used in a retrofit capacity.

Placement of the cables is important, as illustrated in Figure 15, where the original placement in the floor slab would likely have caused cable failure were the slab damaged. Forces in the cable lines are also important to ensure no overload and confine the loss of capacity should a cable break.

### *2.2.4 Megatruss*

Using a megatruss to create a strong floor at intervals may provide a useful concept for some high-rise buildings. It could suspend/support damaged portions of the building and provide an alternate load path around the damage zone. An illustration of such a concept is presented in Figure 16. Columns or cables could be added on the building's exterior to aid in supporting the trusses.

### *2.2.5 Steel Connections*

A retrofit for upgrading an existing traditional moment frame connection is shown in Figure 17. This concept uses a SidePlate™ retrofit system, where the physical separation between the face of the column flange and the end of the beam mitigates the triaxial stress concentrations shown earlier in Figure 8. Physical separation is achieved by means of parallel full-depth side plates that eliminate reliance on through-thickness properties and act as discrete continuity elements to sandwich and connect the beam and the column. The increased stiffness of the side plates inherently stiffens the global frame structure and eliminates reliance on panel zone deformation by providing three panel zones [i.e., the two side plates plus the column's own web]. Top and bottom beam flange cover plates are used, when dimensionally necessary, to bridge the difference between the flange widths of the beam and the column.

This connection system uses all fillet-welded fabrication. All fillet welds are made in either the flat or horizontal position using column tree construction. For new construction, shop fabricated column trees and link beams are erected and joined in the field using one of four link beam splice options to complete the moment-resisting frame. Link beam splice options include a fully welded CJP butt joint, bolted matching end plates, fillet-welded flange plates, and bolted flange plates.

All connection fillet welds are loaded principally in shear along their length. Moment transfer from the beam to the side plates, and from the side plates to the column, is accomplished with plates and fillet welds using equivalent force couples. Beam shear transfer from the beam's web to the side plates is achieved with vertical shear plates and fillet welds. The side plates are designed with adequate strength and stiffness to force all significant plastic behavior of the connection system into the beam, in the form of flange and web local buckling centered at a distance of approximately one-third the depth of the beam away from the edge of the side plates.

The SidePlate™ connection for upgrade construction differs from its configuration for new construction by featuring an initial opening in each side plate to permit welding access, saving the cut-out pieces of plate for use as closure plates to close the access window after welding is completed. All new welds are fillet welds loaded principally in shear along their length. The existing Complete Joint Penetration (CJP) welds joining the beam flanges to the column flange are removed by air arcing to eliminate reliance on through-thickness properties and triaxial stress concentrations. The existing shear tab of the steel moment-frame beam(s) is left in place to provide gravity support. Existing continuity plates may be left in place to act as horizontal shear plates as depicted in Figure 17.

Additional information about this connection can be gotten from FEMA 351 [8]. Stress strain fringe plots for this type of connection are shown in Figure 18.

## 2.3 RETROFIT METHODS FOR REINFORCED CONCRETE FRAMES

### *2.3.1 Cast-in-Place*

The CTS-1 test structure (Figure 19) was built to provide a venue to test retrofits for improving the resilience to blast loads of a cast-in-place building typical of those found on the East Coast. These buildings were thought to be more vulnerable than those designed for active seismic zones, providing a lower bound on vulnerability. The building was designed [13] using a flat slab, 20-foot bays, and a spandrel along the perimeter of the slab. The spandrel was required because of the heavy cladding specified, which was to represent a heavy brick in-fill wall. Because of recent (since 1989) ACI code revisions, continuity steel was added to the spandrel.

### 2.3.1.1 JACKETING/WRAP OF REINFORCED CONCRETE COLUMNS

Typically, RC columns lack sufficient confinement to develop their full plastic moment, especially when account for their compression-membrane response. Thus, the first step in prevention of progressive collapse of an RC structural system is to ensure that the columns can perform ductility and reach their full moment capacity. Wrap is often the most effective means for accomplishing this because with its use the column can realized its full moment capacity with increasing the direct shear demand. In contrast, the increased moment capacity of metal jackets causes an increased risk of a direct shear failure.

Composite wrap has the added advantages over steel jacketing of being less expensive, not increasing the size of the column's cross section, can conform the columns shape, and can be installed without the need of welding and the handling of heavy plates.

**Design.** Designing column wrap for retrofitting an RC column for surviving a blast load is not nearly as straightforward as designing just a plain RC column using the ACI code. This is indicative of retrofit design in general, which is harder, requires more experience and judgment, and is difficult to codify as compared with new construction design. Also, the general lack of tests and test data suitable for judging the efficacy of the retrofit designs, especially given that there are so many design parameters, tends to produce a design environment that is not easily mastered.

Column wrap design involves calculating the number of wraps needed to prevent shear failure, checking whether direct shear can occur (which it usually cannot), and determining the number of wraps needed to reach the level of ductility. The physics involved in this relatively straightforward design problem is both complex and problematic [5, 10, 19]. At K&C, we have performed [5] many hundreds of these calculations using HFPB finite element models that have been validated with full-scale tests we have conducted on more than 20 blast and statically loaded wrapped and unwrapped column specimens.

This work has been boiled down into a simplified design procedure that uses an SDOF model for its analytic engine and a resistance function specially derived for wrap columns. The code uses a simple GUI to facilitate input/output; it is to be released soon. Unfortunately, like other retrofit design procedures, this code has its flaws, which are not readily apparent to the user. Thus, vetting of such a code for a broad based user community seems unlikely in the near future.

For more advanced designs, like placing extra wrap at the column supports where the shear is the highest, and adding flexure capacity, even more complicated behaviors are involved, which are even more difficult to put in a design procedure.

**Behavior.** The effectiveness at resisting nearby blast loads of both jacketed and wrapped RC columns was tested using the CTS-1 columns. The column's section and properties are shown in Figure 20 along with the response when subjected to the same blast for the existing column (Figure 20c), which was damaged severely, and a CFRP wrapped column, which remained nearly elastic. Some of the physics and data associated with the column's response is presented in Figures 21 to 23.

The deformed mesh plot shown in Figure 21 illustrates the important response modes. Early on, a direct shear response mode is seen. This mode is important because the wrap/jacket retrofit is not effective in resisting this mode; however, this mode rarely causes failure. Later on, the diagonal shear mode is dominate; and since many columns are under-reinforced in shear so that they cannot develop their ultimate moment capacity, this is likely to be a catastrophic failure mode because it leads to complete loss of axial capacity. CTS-1 had this type of failure (Figure 20c). This point about under-reinforcement may be missed if the added flexure resistance due to the compression-membrane forces is neglected; another reason to be extremely careful about the notion of being conservative. The presence of compression membrane is illustrated in Figure 23 by the marked increase in the axial force measured in the test.

Wrapping the column prevents this and allows the column to use its full moment capacity and remain ductile out to very large support rotations, as shown in Figure 22.

### *2.3.2 Precast*

Khobar Towers (Figure 1c) certainly demonstrates the robustness that can be provided in a design using precast panels. Here, the provision of continuity and ductilities is achieved through using cabling to tie the panels together making for a robust and resilient design. However, the precast panels themselves were not very strong, which lead to a lot of casualties. While not strictly needed to prevent progressive collapse, use of one of the wall retrofits described in Section 2.4 would seem advisable.

### *2.3.3 Post Tension*

The retrofit technology described herein focuses primarily on the need to strengthen post tension parking garages. The strategy envisioned involves developing a retrofit design based on preventing multiple column failures either because the spacing of the columns is sufficiently large to preclude multiple failures or the columns are strengthened sufficiently to preclude all but the column closest to the bomb from failing. To prevent the failure of the one column to cause collapse of the garage, a secondary cabling system (similar to the one shown in Figure 15b) is strung along the column line that is attached to the top of the columns, such that if a column below is destroyed the cable will ensure that the column line above this location will be carried by the cabling and the load transferred to the adjacent column lines. Cables are run only in a single direction along the short span between columns. Cables would employ inline fuses to ensure against breakage. Paired fuse systems would be through-bolted together on the outer face of the column. The fuses would also act as a tie should a cable line break, ensuring that the rest of the cable lines remain intact.

If the BEA indicates that more than a single column is failed by the blast load, then the adjacent columns must be retrofit to preclude this. The retrofit would use a composite wrap to provide the necessary added strength and ductility.

## 2.4 RETROFIT METHODS FOR BEARING WALLS

The response of Khobar Towers provides a good example of outstanding performance in the face of a large blast load that can be achieved with buildings where the principal gravity support is provided by exterior bearing walls. In this case, there was sufficient load path redundancy that the whole front of the building could be destroyed and (the building) still stand up.

Retrofit strategies for a bearing walls need to minimize the zone over which the gravity support is lost as well as prevent entry of debris into the occupied spaces. One retrofit system for masonry that meets these requirements uses a Kevlar laminate bonded to the inside surface of the masonry. This material, which was recently produced by DuPont for lining tornado shelters, is shown in Figure 24a. The laminate comes in sheets with a stiffness similar to cardboard; it is attached to the wall with a polyurethane adhesive. The sheets come in different number of plies with strengths from around 3,400 pounds per inch to 17,000 pounds per inch and higher. In tests, this retrofit has proved it ability to limit wall deflections sufficiently to prevent loss of bearing capacity even though the wall is heavily damaged.

Another form of retrofit using a composite panel (Figure 24b) also provides a strong/stiff response suitable for minimizing bearing wall failure.

The high-strength metal stud wall shown in Figure 25 provides a ready means to construct a secondary bearing wall immediately adjacent to the existing wall. This secondary wall provides an alternate load path around the damaged section of the existing bearing wall.

## 2.5 PRACTICALITY

The costs and installability (capability for installation) of a retrofit concepts must be a major concern. Installation in terms of getting the materials into the building; handling once inside and during installation; and the amount of site preparation, safety precaution, demolition and the necessity to excavate the space are also major factors in measuring the efficacy of a retrofit design.

In addition, many retrofits must be attached to the building's diaphragm for support, which is an important design consideration. Under blast loads, shear forces at anchorages can be very large, and may be directly related to the blast load as well as related to the flexure capacity of the structure supported. Out-of-plane loads are also likely since many retrofit designs (as described in several K&C references (12, 10, 20]) employ large deformations to achieve their resilience. Careful attention to details at anchorages, use of fuse elements to accommodate peak forces, and attenuation of force peaks are useful ingredients for a successful anchorage design.

Finally, since these are retrofit design, the effects of the uncertainties must be factored in. These might be due to not knowing what properties to use for the materials of the existing building, poor quality construction, missing drawings, and the host of problems that crop up in most retrofit projects, which are likely to be more problematic for retrofit designs for preventing progressive collapse.

# 3 RESEARCH NEEDS

As mentioned in Section 1.3.6 concerning factors inhibiting retrofit design, inaction and wasted efforts are particularly troubling concerns relative to retrofitting existing buildings. In the retrofit situation, important issues are more likely to be misaddressed because they are more complex, primarily related to a lack of technology, ignorance of available technologies, and uncertainties about the design of the building involved.. Existing buildings are best approached with a risk-based retrofit strategy to focus the efforts where they are really needed. Retrofit designs inherently require more skill from the designer, better design tools, and concepts that can accommodate a broad range of loads and site conditions—for example, very high blast loads because of a lack of standoff.

I believe that a good blast-resistant retrofit design cannot be achieved without consideration and quantification of the risk of casualties based on realistic and encompassing terrorist attack scenarios. Moreover, the blast design can only minimize risks not preclude them. New capabilities and revised policies are needed in several areas to improve the infrastructure of design, for example:

- Policies that define realistic and efficacious threat definitions in probabilistic terms.

- The capability to translate the threat into structural loads suitable for predicting the response of the building and the actual risks to its occupants.

- The capability to define casualty probabilities as a function of structural design.

- Legislation that provides protection from liability given that an accepted risk base design scenario is followed. A sure safe design paradigm is inappropriate for terrorist attack loads. A 95% solution for the structural design combined with a "200%" estimate of the threat is far better (than the reverse) because resources can then be focused more effectively.

- Capabilities to adequately determine structural response and related risks, where "adequately" means with sufficient accuracy to enable trade off decisions to be made based on achieving an acceptable risk rather than some response standard (e.g., a response limit).

- A program to foster the development of innovative concepts for blast protection and certify their use. The reliance on industry to self-sponsor the needed product/capability development is likely to preclude a broad array of mitigating devices and new structural concepts because, at least initially, the market is too small and uncertain to attract the level of industrial participants needed. The unintended consequence of not fostering such a program is having an array of products that work well for lower level threats (where the market is bigger) that in turn convinces or lures the design criteria authors to lower their criteria to accommodate the existing product base—essentially a tail wagging the dog scenario that has greatly impeded bringing to market the needed capabilities.

- More extensive availability of blast testing opportunities is needed. It is impractical for industrial or commercial concerns to field the large blast load tests needed to evaluate/develop promising new products and concepts for protecting occupants from terrorist attacks. The government needs to provide more tests, and test venues and fixtures to foster product and design concept development.

- Design concepts that rely on ductility, structural continuity, and residual capacity to mitigate a threat should be preferred over those that are based primarily on strength and have little in the way of reserve post-attack capacity. For retrofitting, the goal should be that even if it fails, the consequence should be no worse than if nothing were done in the first place—a standard that is all too often not practiced.

## 3.1 FUTURE DESIGN PRACTICE

While it may be impractical to include the level of detail and effort in design practice implied by the above discussion in specific building designs, this level of detail and effort could be used in conjunction with development of standards and codes so that procedures and generic practices can be related to actual risk estimates and to ensure a sufficiency of cost effective and beneficial blast protection products and design concepts. Perhaps the future design practice could follow a two-pronged approach:

- One with short term, doable goals like improving the prescriptive rotational limits for members given in codes and manuals, for example, like P-397 [17] and FACEDAP, where more sophisticated criteria are added and caveats are made—especially to those provisions that are overly optimistic about the ductility and capacity of common steel frames and connections, and excessively pessimistic about well confined RC columns. For example, steel connection recipes like those given in FEMA 350 [6] could be generated to specifically identify rotational criteria and the effects of confinement on RC column behavior could be included in the rotational criteria.

- The other approach could have the long-term goal of developing some sort of risk based design procedure. This would foster terrorist-resistant designs where risk was the guiding principle in screening designs rather than an arbitrary set of rules having no definable connection to risk.

Both of these approaches require that we significantly improve on present practice and that more research and development are performed.

## 3.2 ANALYTIC TOOLS NEEDED

Developing adequate analytic models for predicting the response of building components and structural frames for use in designing and assessing the resistance of buildings to terrorist attack is an important adjunct to the design process. These tools will differ depending on whether they are for the design of new buildings, for the retrofit of existing buildings, for assessing building vulnerability, or for determining occupant risk. Additional factors related to design tools are the experience and capabilities of the analyst needed to operate them, and the level of effort and time available. With regard to terrorist-resistant designs, the design community cannot perform responsibly without using the best of the available technology. This will require pushing hard for better-educated engineers and more R&D funds to make improvements in the available technology base.

Unfortunately, predicting the potential for progressive collapse after a bombing or airplane impact is well beyond the capability of most engineering firms. Adequate prediction tools could be developed within the current technology base, but funds are lacking. Currently, trillions of dollars in assets and the safety of many are governed by overly simplistic analyses and wishful thinking that, at best, only poorly address the progressive collapse issue. Moreover, the lack of analysis tools and those who can run them has contributed to the promulgation of the simplified approaches widely used for today’s designs, but whose predictions may be extraordinarily inaccurate.

# 4 SUMMARY AND CONCLUSIONS

## 4.1 SUMMARY

Certainly much more is needed in terms of retrofit technology and methods to address the varied needs presented by the progressive collapse problem related to existing structures, especially in those structures constructed with relatively weak lateral system with little continuity and capacity to redistribute load. However, maybe the biggest need for research and development effort is in the area of design infrastructure—the tools, methodologies, engineers, and professional standards by which designs are performed. Infrastructure is particularly important to retrofit design where determining the behavior of complex structures under complex conditions is the norm. Several suggestions for infrastructure improvements were provided in Sections 1.3 and 3.

In summary with respect to retrofitting buildings to prevent progressive collapse, much research is still needed, especially in the area of predicting the capability of damaged building systems to redistribute the load from failed columns.

## 4.2 CONCLUSIONS

Of paramount concern to developing retrofit methods to mitigate progressive collapse is having a design process to support the effort. The ability to produce a design approach for retrofitting existing buildings for preventing progressive collapse is complicated by the lack of knowledge in the engineering, policy maker, and academic communities related to the response of building systems, the performance of retrofit concepts under TIB loads, and characterizing specific terrorist attack loads suitable for design. Much of this is due to the relatively recent need for these types of designs; the unique nature of civilian structures as compared to hardened military structures, where loads like these (e.g., blast loads) have always been a primary concern; and the security concerns related to much of the data and analytic models developed primarily within the DoD community.

Recent efforts by ASCE, ACI, and AISC to discuss and incorporate blast effects related design issues are a necessary initial step. However, the process is still very fragmented and often based on opinions or specific viewpoints versus broad based experience, consensus, and factual based notions. Much of the problem may be attributed to the dearth of applicable tests, the difficulty of modeling responses caused by terrorist attacks, and the sparse number of structural engineers having sufficient experience and capability in blast-resistant design. It is critically important that terrorist-resistant design policy-makers benefit from the lessons learned following the now infamous Northridge earthquake of 1994 concerning the unwittingly prescriptive and extraordinarily inaccurate pre-Northridge design practices used in the seismic design of steel frame structures.

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**Table 1. Structural systems and the risk factors associated with them pertaining to progressive collapse.**

| Type of Gravity Support System | Sub-type | Risk Factor |
|---|---|---|
| Steel frame | Moment frame<br>• Zone 1<br>• Zone 4 | <br>• Performance of typical connection uncertain<br>• Non-moment joints likely to be weak |
| | Shear wall | Weak connection |
| | Braced frame | • Braced panels at only some locations<br>• Panels without bracing weak |
| Reinforced concrete frame | Moment frame<br>• Zone 1<br>• Zone 4 | <br>• Column may be weak in shear<br>• Column may shatter at midspan |
| | Shear wall<br>Flat slab | |
| Pre-stressed, post-tensioned girders on steel or reinforced concrete columns | Lacking in continuity | • Lack of ductile frame behavior |
| Bearing wall | • Reinforced<br>• Unreinforced<br>• Thick (as in heavy and deep) standard | • Lacks ductility<br>• May shatter or fragment (i.e., breach)<br>• Uncertainty about allowable zone of failure |

**Table 2. Process for determining a building's potential for progressive collapse.**

| Step | | Description | Complexity |
|---|---|---|---|
| 1 | Threat | Determine threat condition. | Easy in the sense it's likely to be specified. Complex in the sense it's likely to be arbitrary and overreaching |
| 2 | Properties | Identify the framing elements of the building and their properties | Easy |
| 3 | Load | Determine frame airblast/debris/impact loading for a specific threat. Adjacent structural properties where fire is a factor. | Moderate to difficult |
| 4 | Current Damage | Determine the damage caused by terrorist attack loads for particular members and connections. | Straightforward, dependent on level of engineering skill. Easy to moderate |
| 5 | System Damage | Determine capacity/ductility of damaged connections and members to sustain gravity load. Determine structural beam-to-beam continuity in vicinity of damaged columns where capacity/ductility has been significantly reduced by the terrorist loading. | Easy to difficult |
| 6 | Collapse Prediction | Evaluate progressive collapse by performing a gravity load analysis. | Straightforward, dependent on level of engineering skill |

**Table 3. Retrofit technologies.**

| Application | Approach | Technology |
| --- | --- | --- |
| Frame | Redundant load path | • Add redundant gravity system<br>▪ Add brace frame to moment frame<br>▪ Mega truss |
| Frame | Strong column | • Jacketing, wrap for reinforced concrete<br>• For steel, side plates, concrete encased/filled<br>• For all columns, add crush zone for contact charge |
| Frame | Missing column | • Reinforced concrete<br>▪ Joint strengthening<br>▪ Additional continuity reinforcement<br>• Steel<br>▪ Cabling<br>▪ SidePlate™ connection<br>• Post-tensioned<br>▪ Cabling short span columns<br>▪ Preclude multiple column failures by wrapping columns (if necessary) to prevent failure |
| Bearing wall | Strong wall | Limit failure zone, strengthen wall to control size of failure zone to what can be spanned |
| Bearing wall | Back-up wall | Build second wall or gravity carrying frame just inboard existing wall |
| Thick bearing wall | Ductile wall | Polyurethane spray to prevent punching shear failure |

(a) Murrah building.

(b) CTS-1 test structure, a flat slab design for an area of low seismicity; center first-floor column (shown to right) was destroyed by blast.

**Figure 1. Examples of building responses to thermal/impact/blast loads.**

(c) Khobar Towers.

(d) WTC.

**Figure 1. Examples of building responses to thermal/impact/blast loads (Continued).**

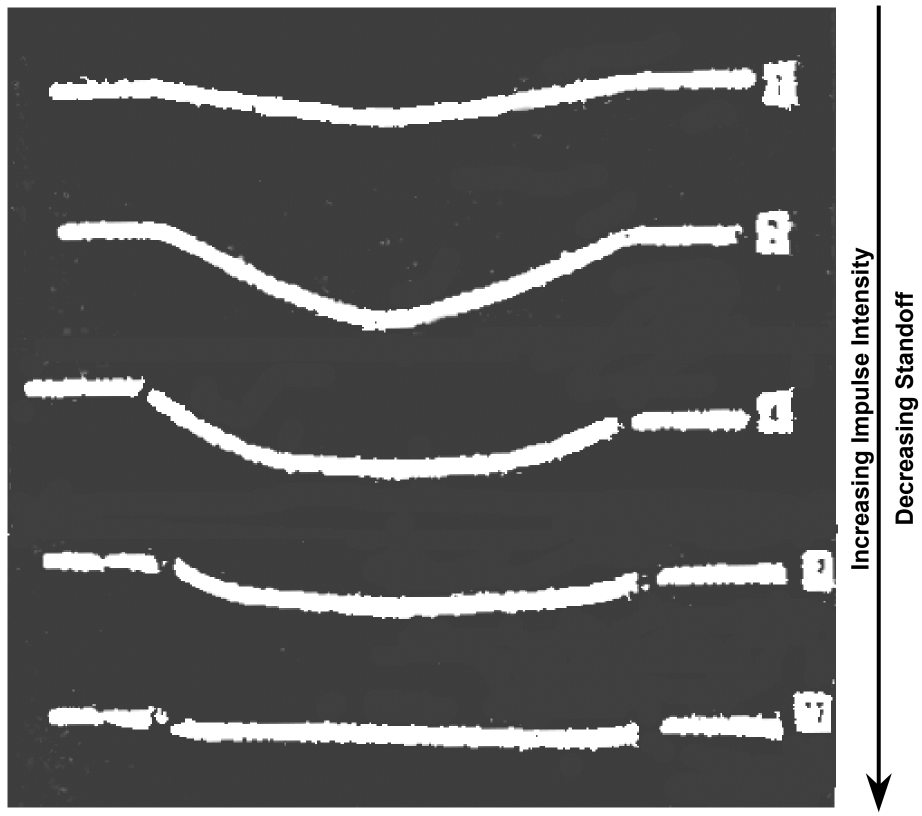



**Figure 2.** **Illustration of diminution of member ductility as load intensity becomes much higher than member resistance [16]; the brittle response indicated in the bottom picture should be designed out of force protection systems.**

**Figure 3. Column with good confinement having support rotation, around 10° rotation.**

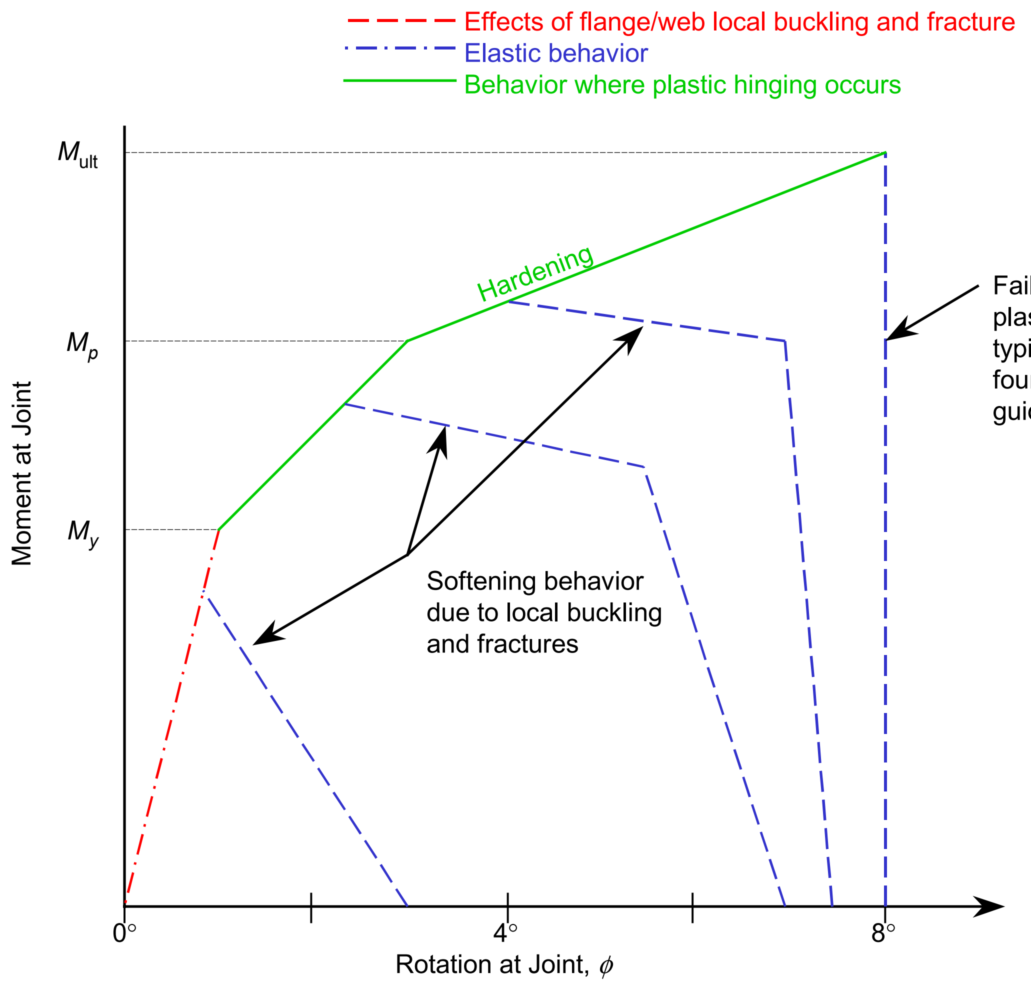
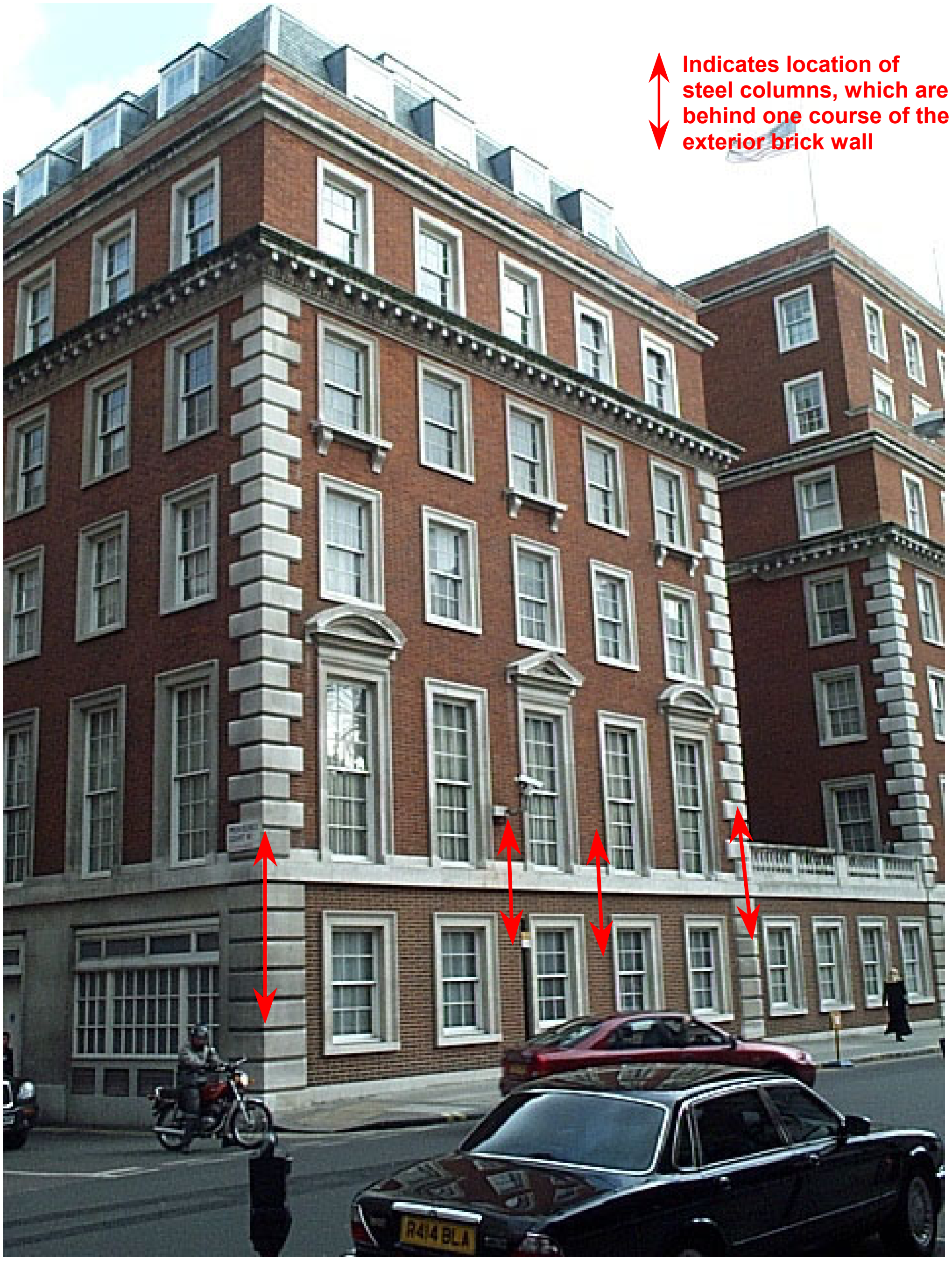



**Figure 4. Illustration of possible moment-rotation behaviors at steel joints; the softening behavior shown would be due to fracture or buckling. The ideal plastic hinge behavior shown (as the solid line) would at some $\phi$ change to a softening because of the initiation of local fracture or buckling.**

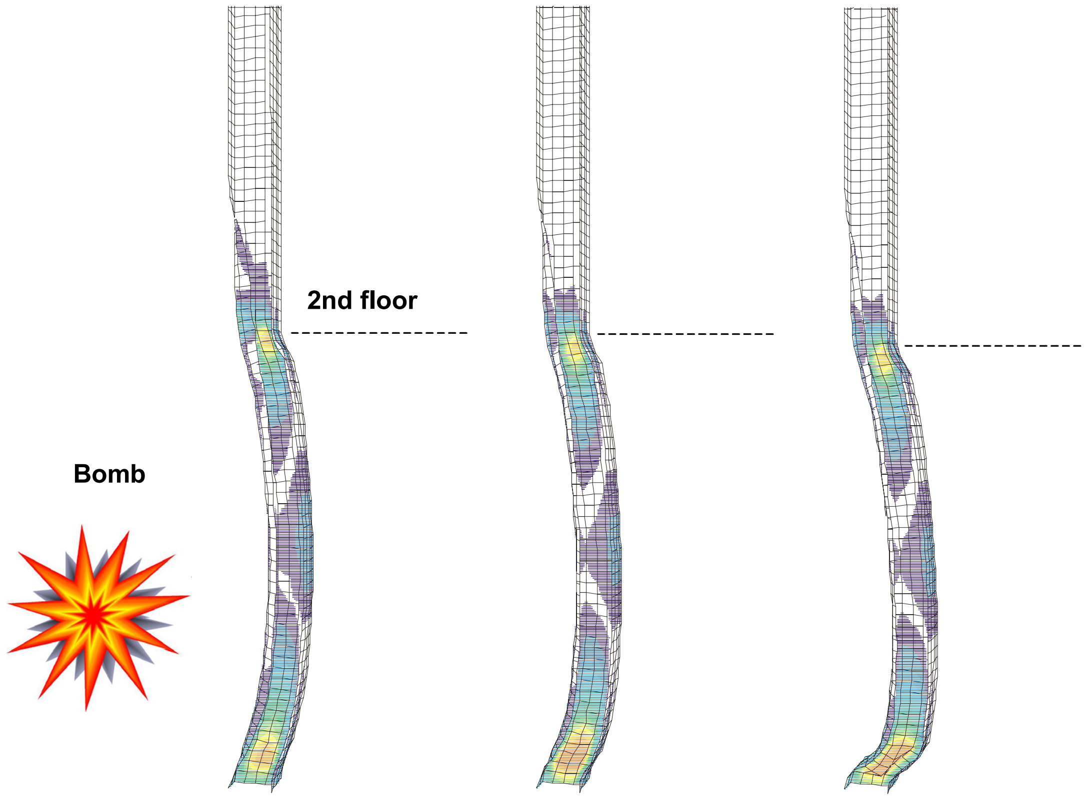

(a) Precast cladding.

**Figure 5. Examples of cladding fronting steel frame, which complicates determining the loading of the framing member.**



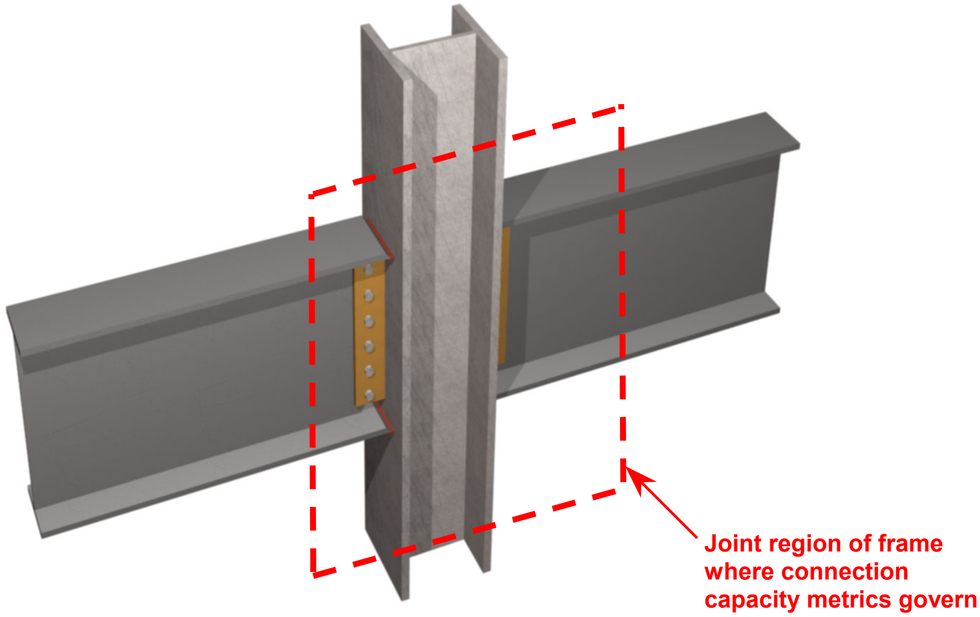

(b) Example of masonry fronting frame.

**Figure 5. Examples of cladding fronting steel frame, which complicates determining the loading of the framing member (Continued).**

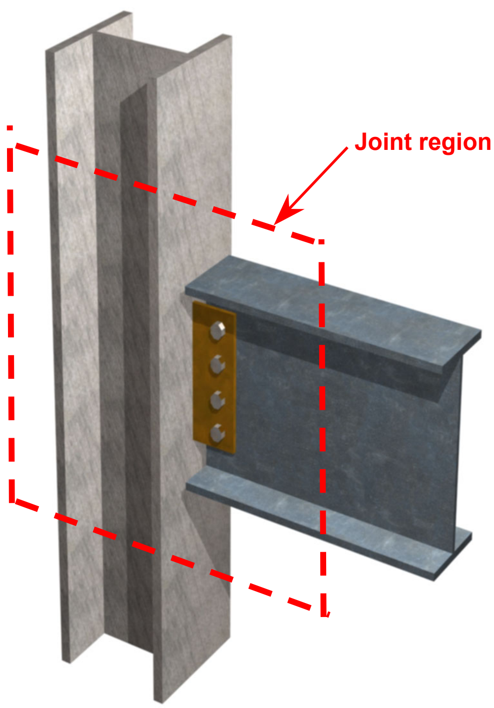



(a) 1,000 pounds at 10 feet just before collapse.
(b) 10,000 pounds at 30 feet just before collapse.
(c) 10,000 pounds at 30 feet just after collapse.

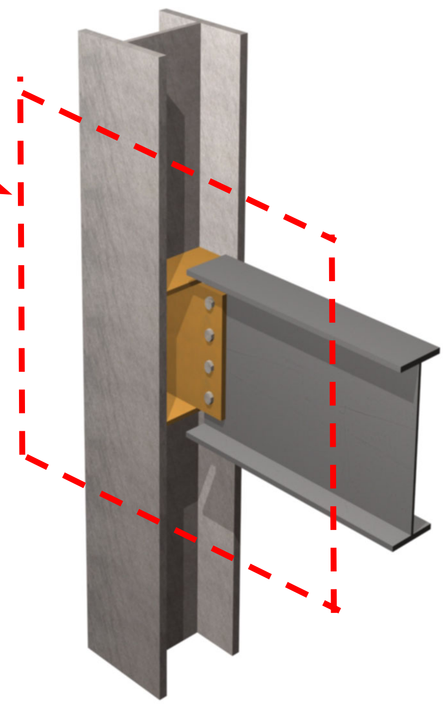
**Figure 6. Failure modes, as demonstrated by the deformed meshes computed by DYNA3D for a W12×87, fringes of plastic strain are shown. Only the first floor is loaded.**




(a) Double sided strong axis connection.

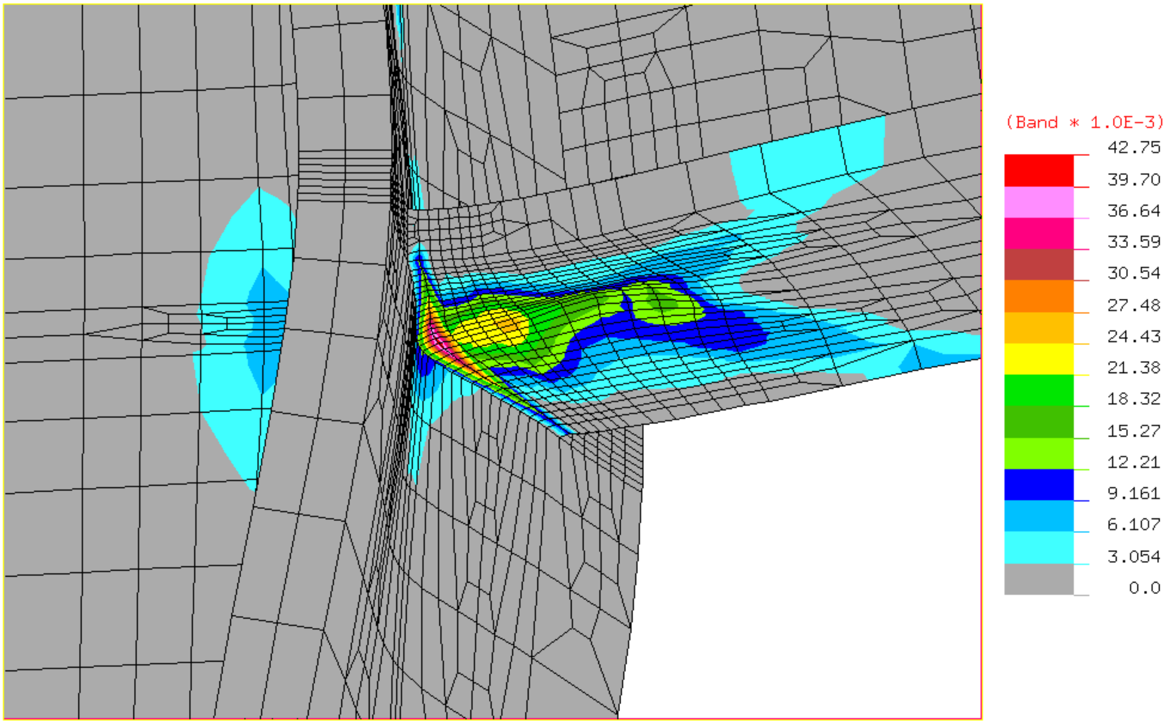



(b) Single sided strong axis connection.

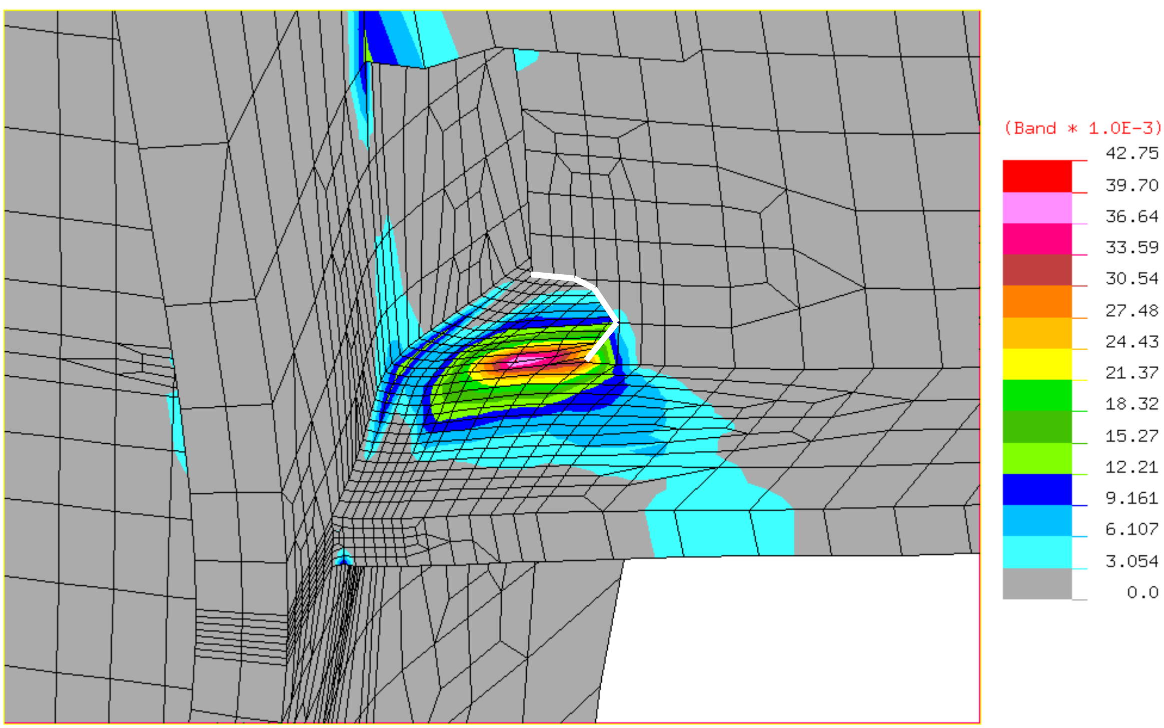


(c) Single sided weak axis connection.

**Figure 7. Illustrations of regions where system capacity is governed by the connection/member effects inherent in joints; outside those joint regions, capacity and behaviors are governed by member properties. There are several other joint configurations that are also important, including two girders connected to a corner column, three girders into an exterior column, and four girders into an interior column.**

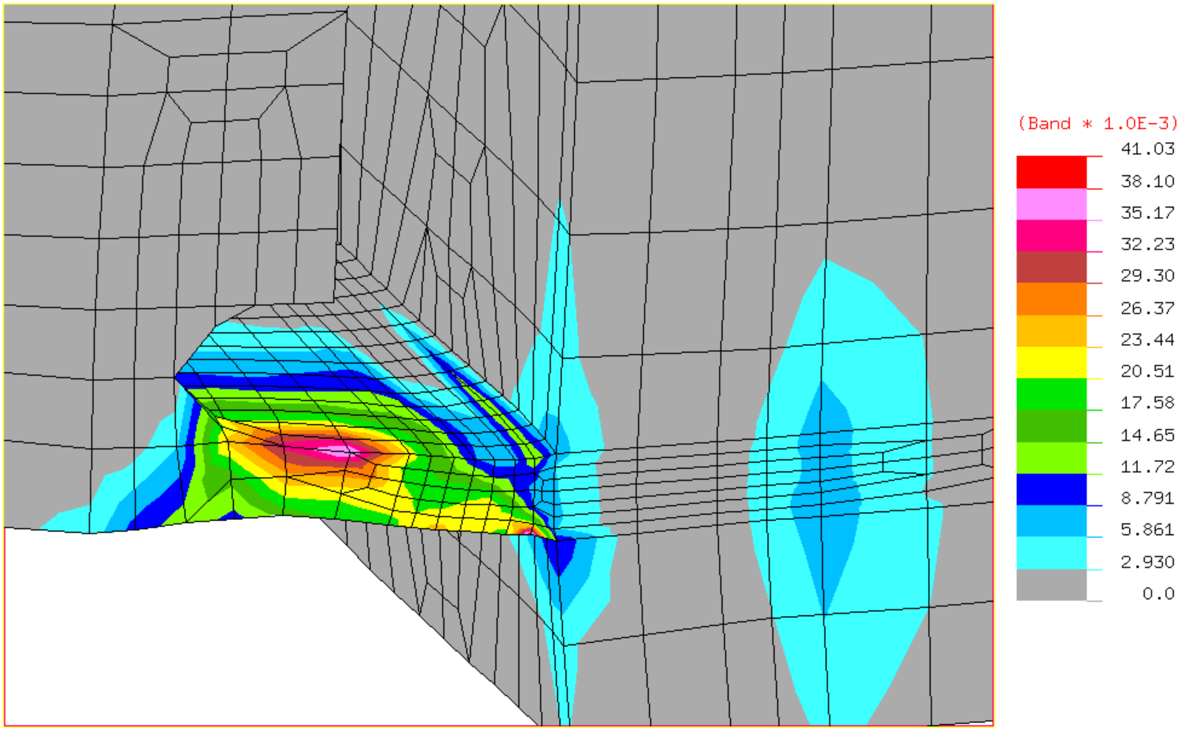



(a) Bottom face of bottom flange.



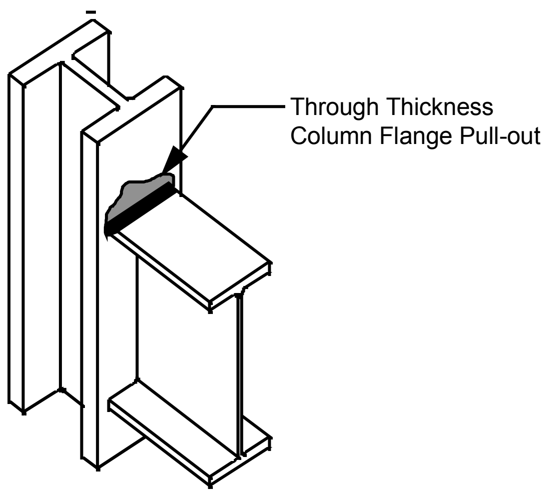

(b) Top face of bottom flange, weld access hole is outlined.

**Figure 8. High-order bi-directional localized plastic strains at weld root of critical juncture between beam and column [9].**

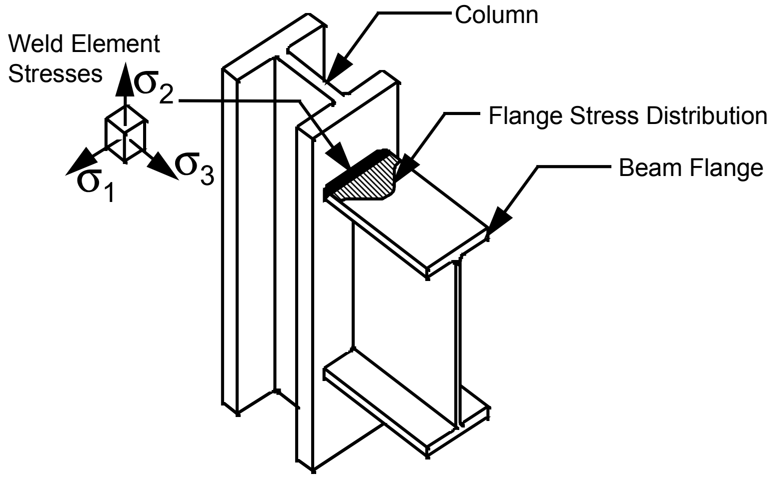



(c) Equivalent plastic strain in slice along plane of beam web.

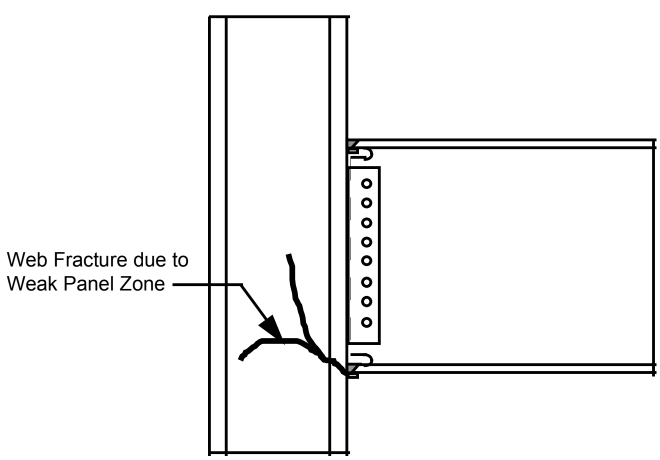
**Figure 8. High-order bi-directional localized plastic strains at weld root of critical juncture between beam and column (Continued).**




(a) Abrupt “divot” pull-out column flange base metal.



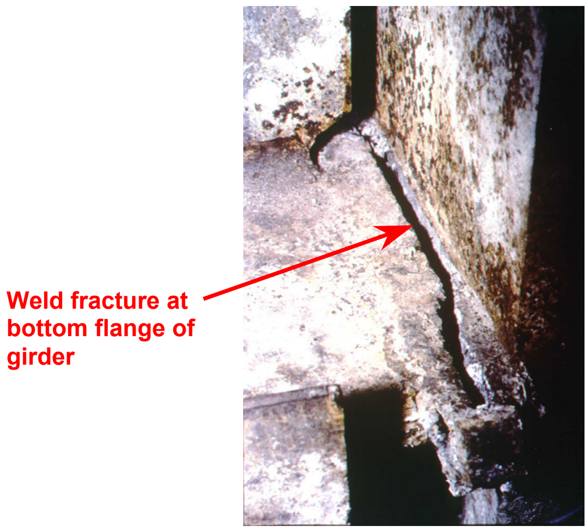

(b) Brittle weld fracture due to peaked triaxial strains.



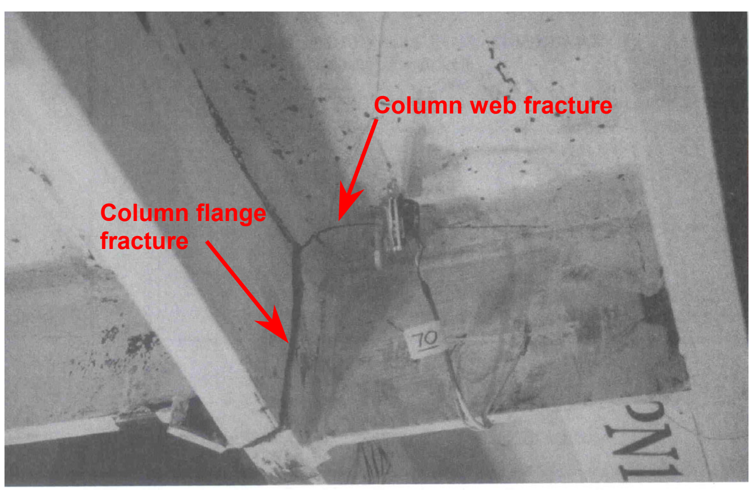

(c) Sudden column web fracture due to inherently weak panel zone.

**Figure 9. Illustrations of the various brittle failure modes.**


(a) “Divot” pull-out of column flange base metal.

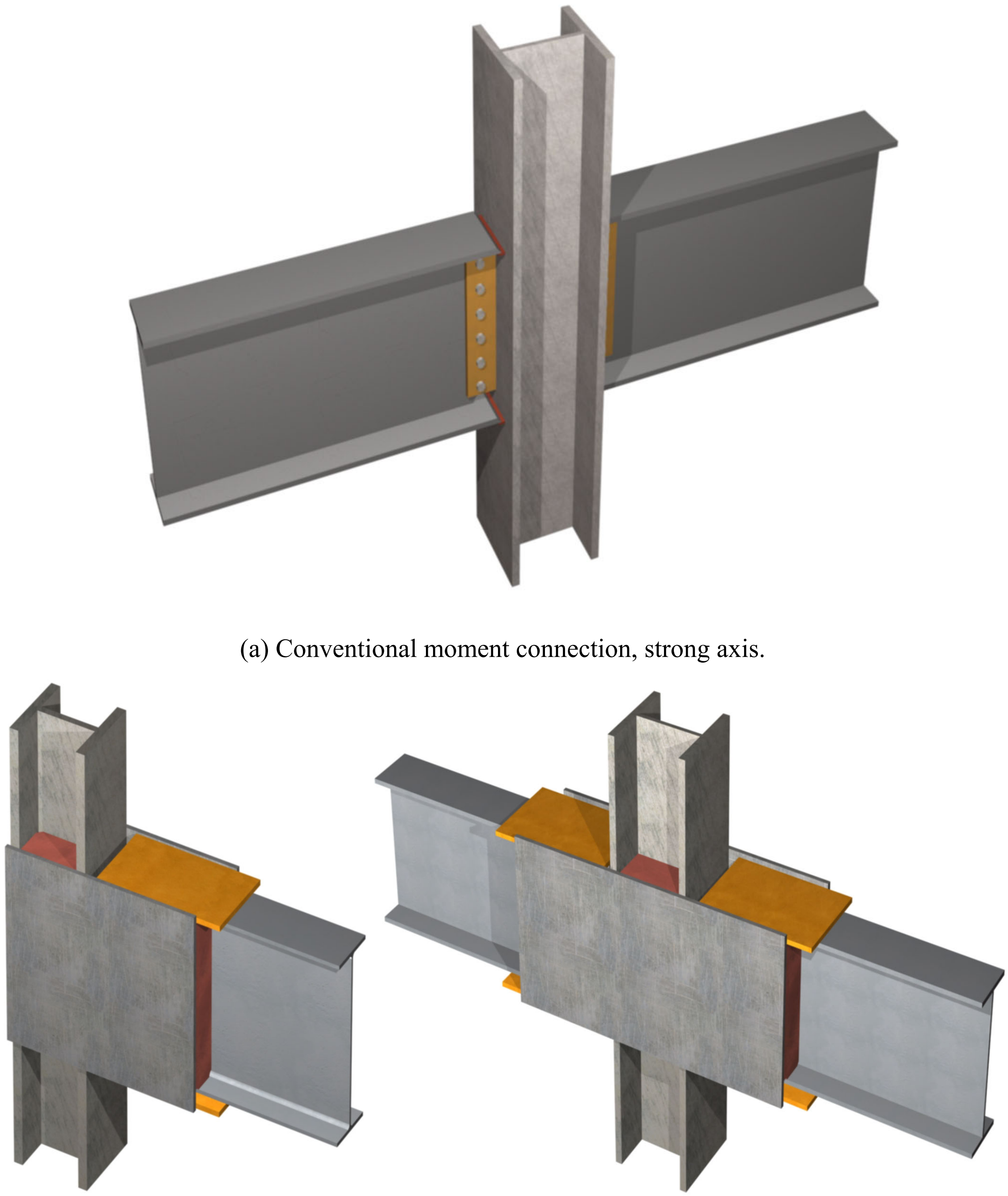



(b) Brittle failure of girder flange weld of girder-to-column weld connection.




(c) Failure of bolted girder web connection to column flange.

**Figure 10. Photos of the various brittle failure modes.**

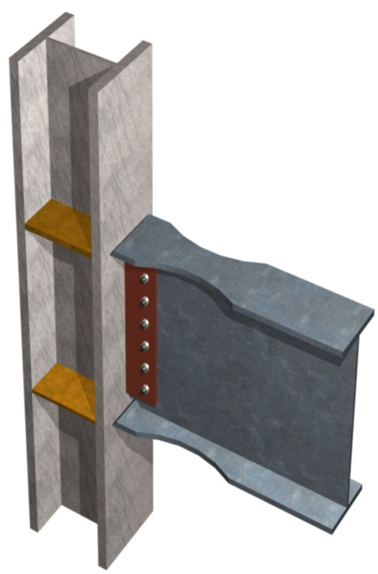


(a) Conventional moment connection, strong axis.

(b) One sided and two sided SidePlate™ moment connection, strong axis.

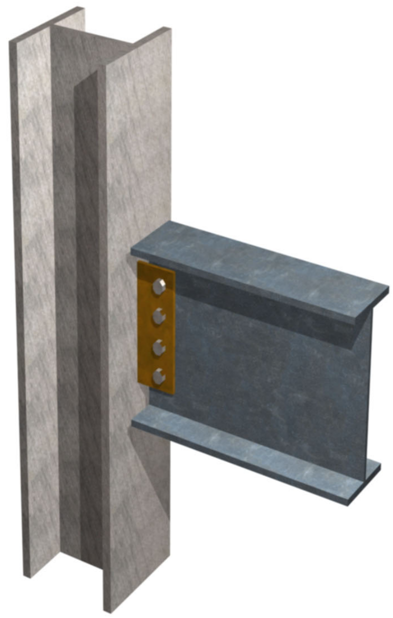
**Figure 11. Typical of connection types used in steel frame buildings.**

(c) Conventional bolted shear connection, weak axis.



(d) Dogbone connection:  post-Northridge, fully-rigid moment resisting connection; seismic zone 4.

**Figure 11.  Typical of connection types used in steel frame buildings (Continued).**



(e) Bolted shear plate connection:  gravity only, strong axis connection.

**Figure 11.  Typical of connection types used in steel frame buildings (Continued).**

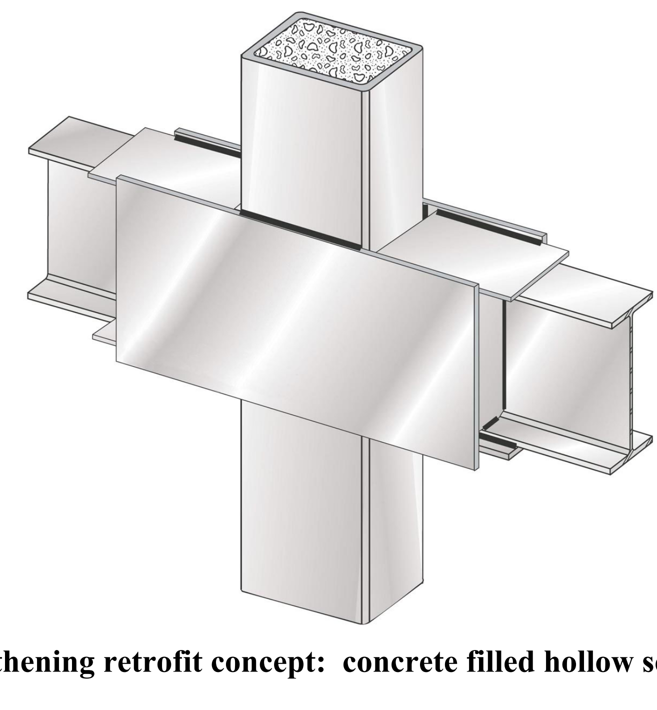
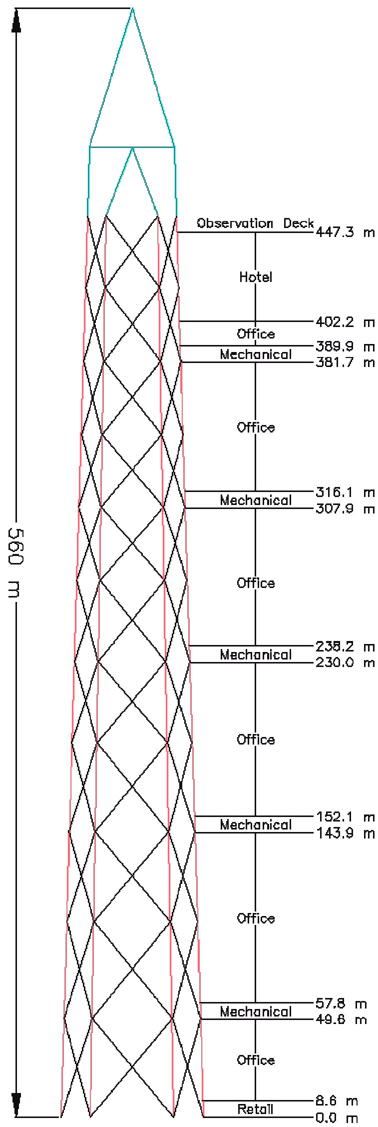


**Figure 12.  Strengthening retrofit concept:  concrete filled hollow section of column.**

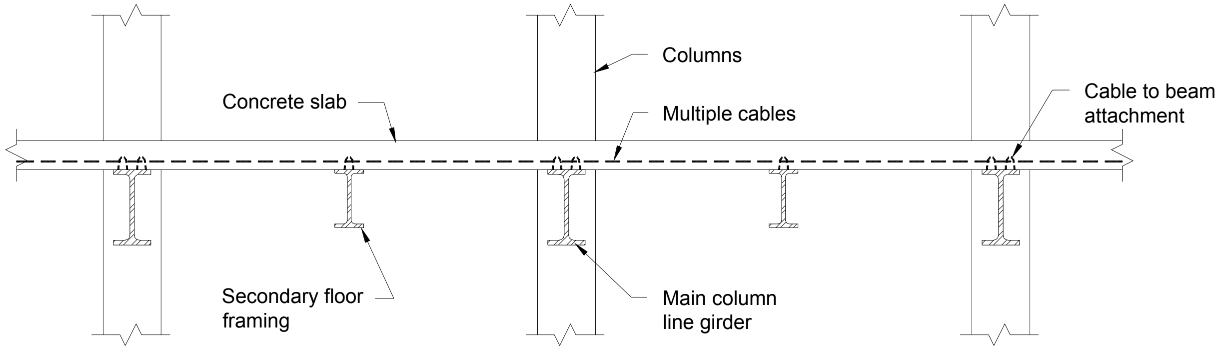
(a) Super-x bracing subsystem added to moment frame floor by floor,
possibly bay by bay too, or maybe just perimeter bays.

**Figure 13. Added load path retrofit concept.**

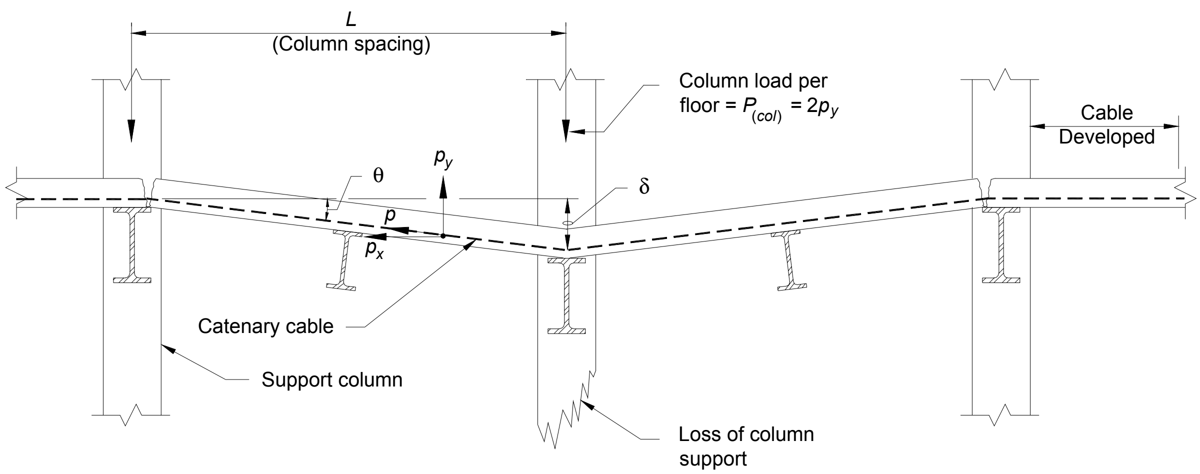



(b) Mega X-bracing added external of the building, covering multiple floors.

**Figure 13. Added load path retrofit concept. (Continued).**

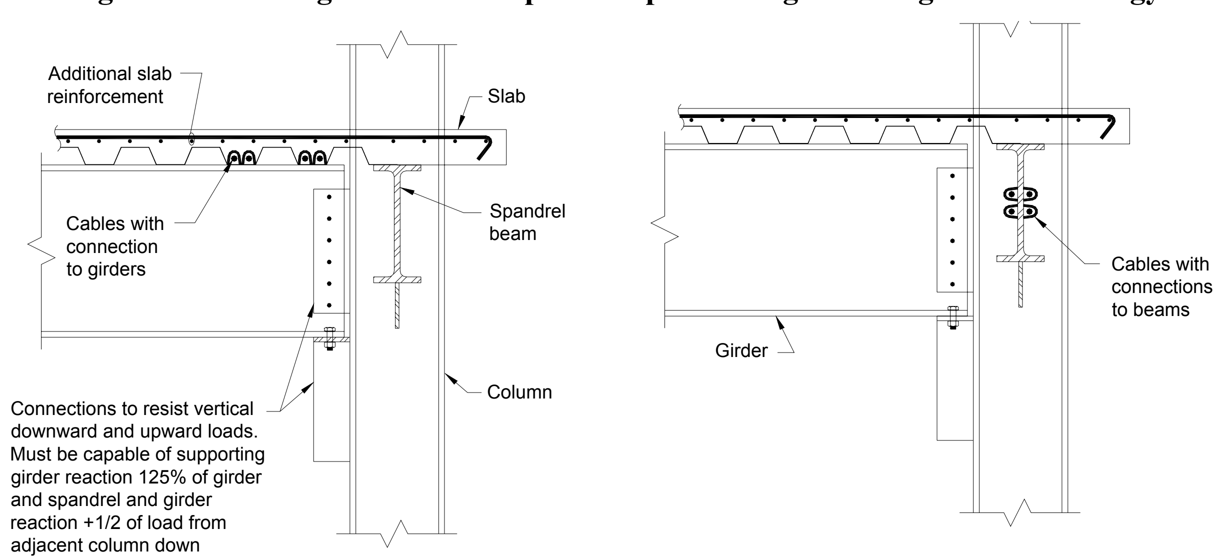



(a) Undeflected shape.



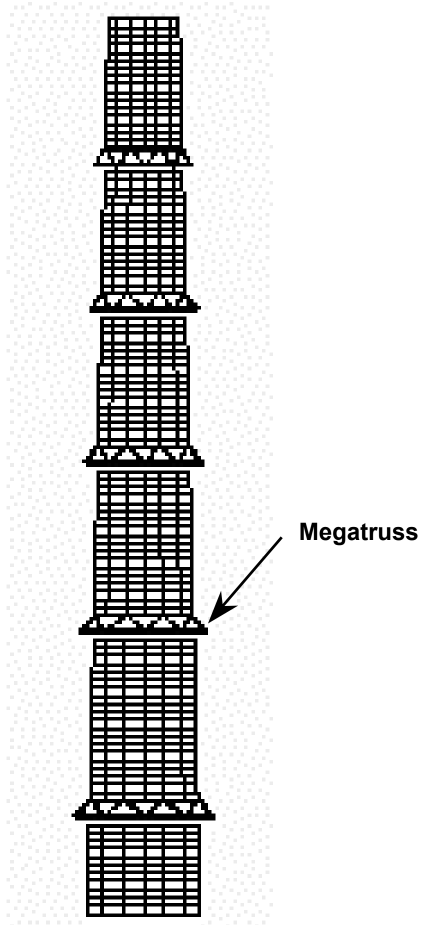

(b) Deflected shape.

**Figure 14. Cabling retrofit concept for implementing a missing column strategy.**

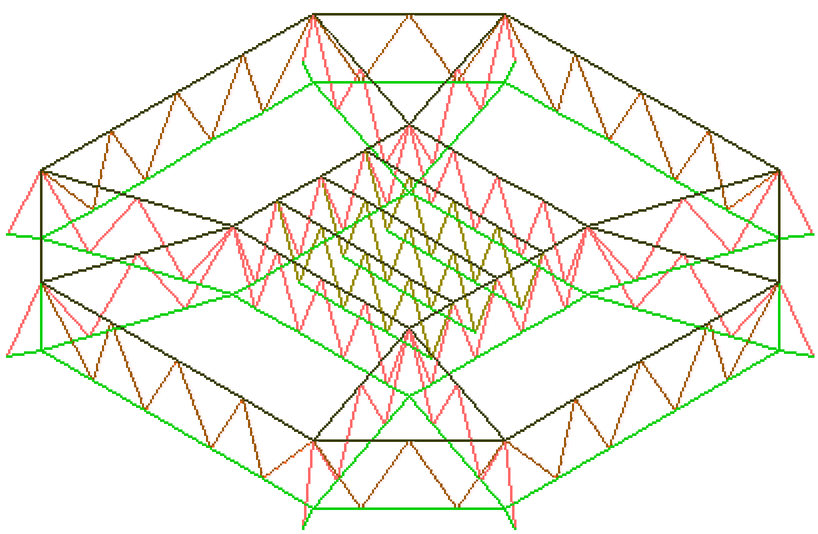



(a) Design 1 (as proposed). (b) Design 2 (suggested alternative).

**Figure 15. Section showing alternative cable placement for cabling retrofit.**




(a) Megatrusses used at various story heights as a barrier against further collapse.



(b) Megatruss detail.

**Figure 16. Megatruss retrofit concept.**

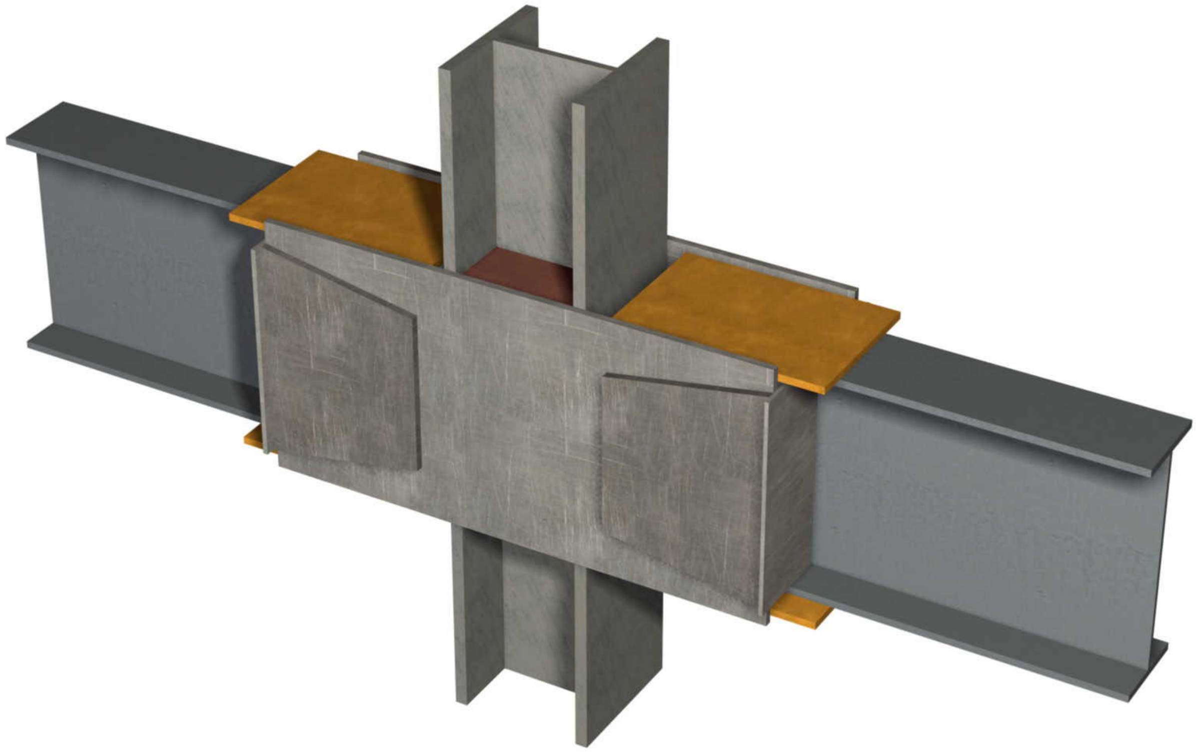
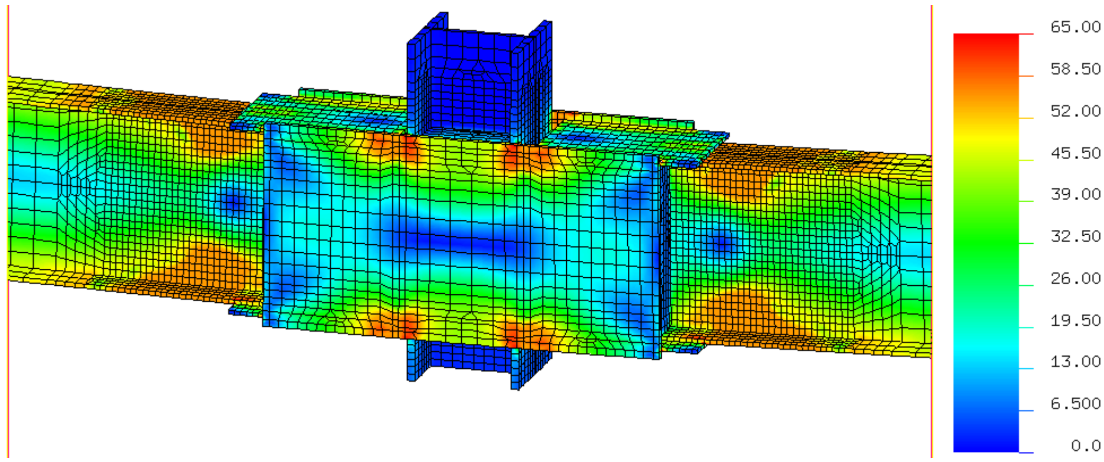
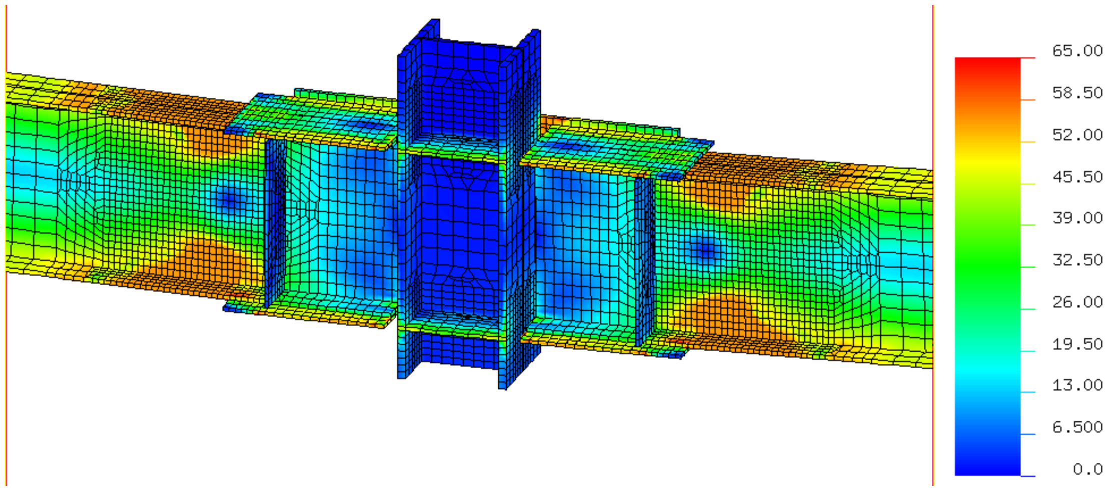


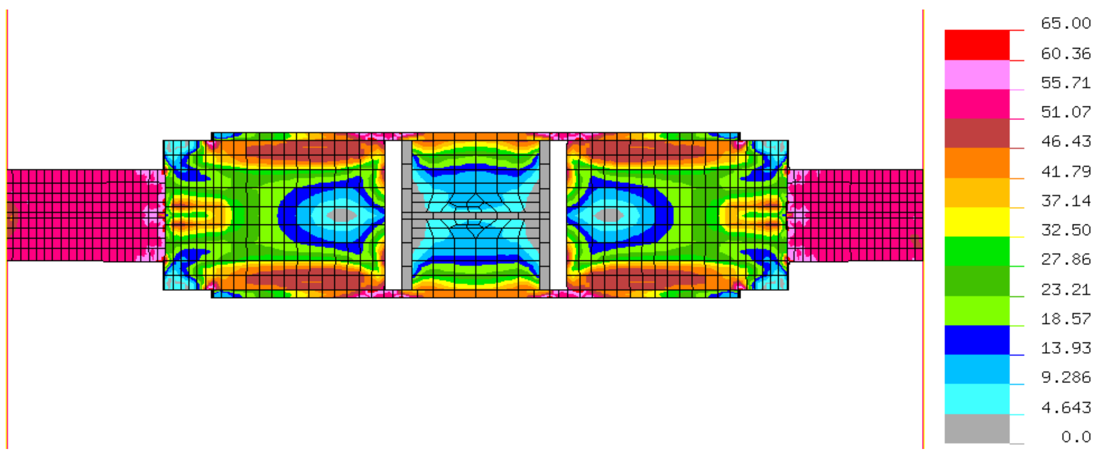
**Figure 17. SidePlate™ retrofit connection.**




(a) Close-up of von Mises stress, ksi.




(b) Von Mises stress, with side plate removed, ksi



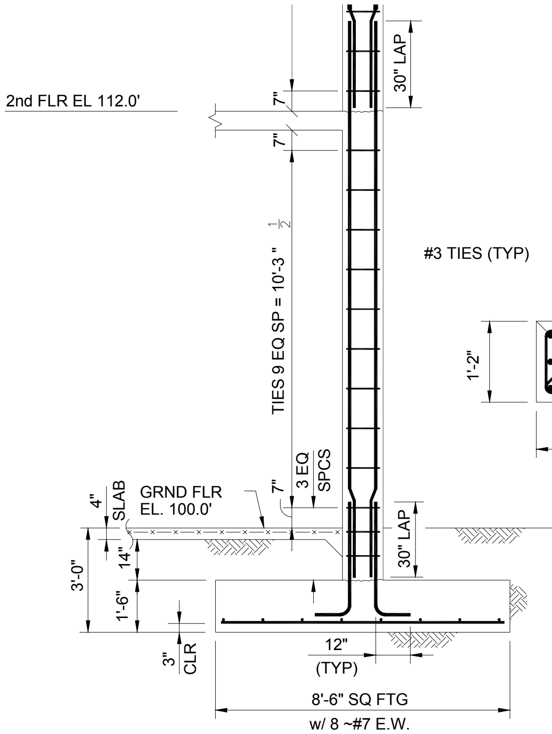

(c) Von Mises stress looking down on top flange of girder, ksi.

**Figure 18. Two-sided, SidePlate™ moment connection.**

(a) Pretest.

(b) Posttest, DB 6.

**Figure 19. CTS-1 test article.**




(a) Configuration of RC column modeled and tested.

(b) Response of wrapped column.

(c) Response observed in DB 6 blast test.

**Figure 20. Behavior typical of reinforced concrete columns subjected to blast loads, with/without CFRP wrap.**

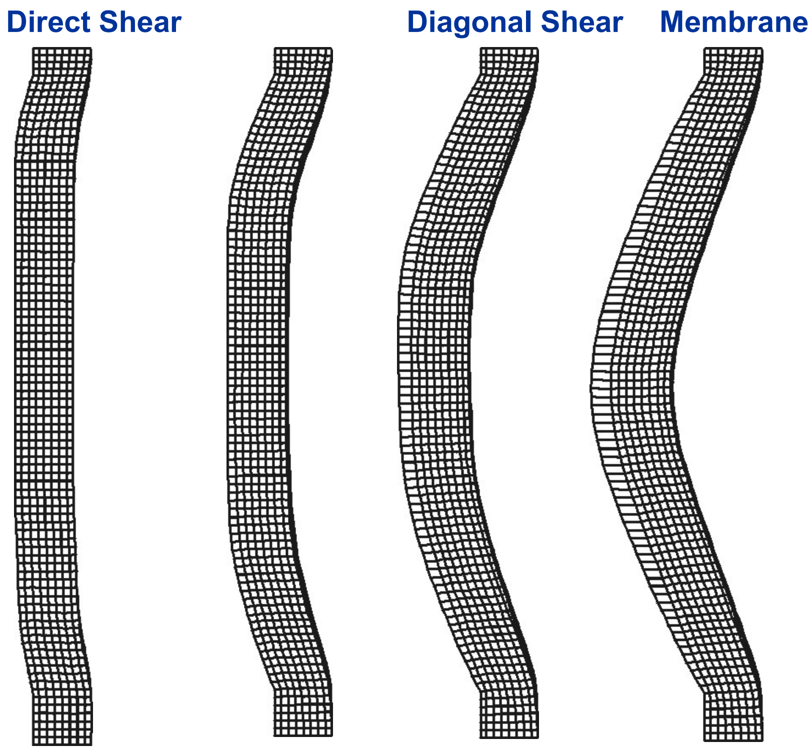
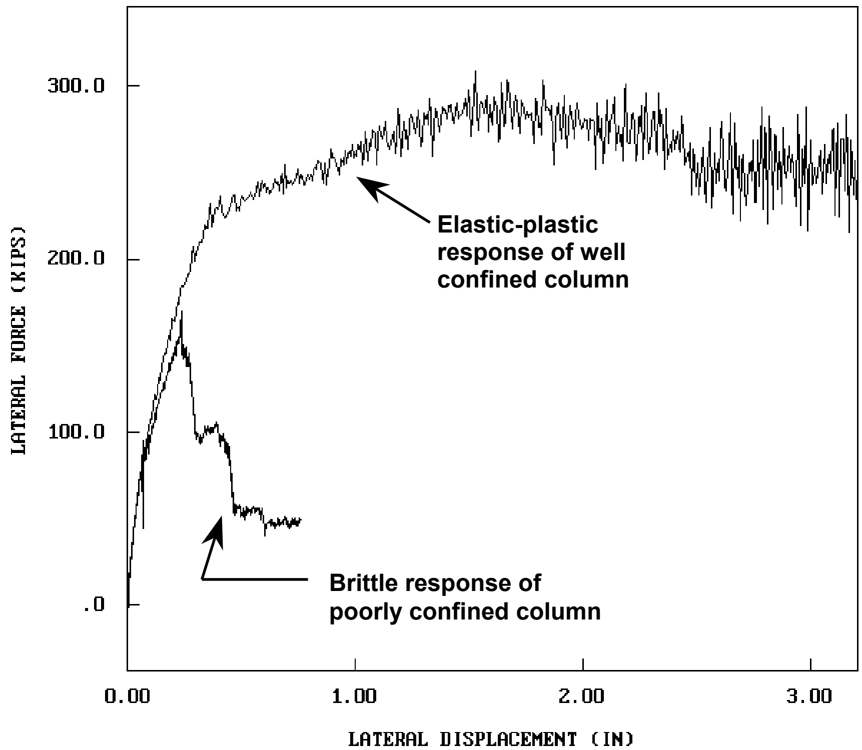


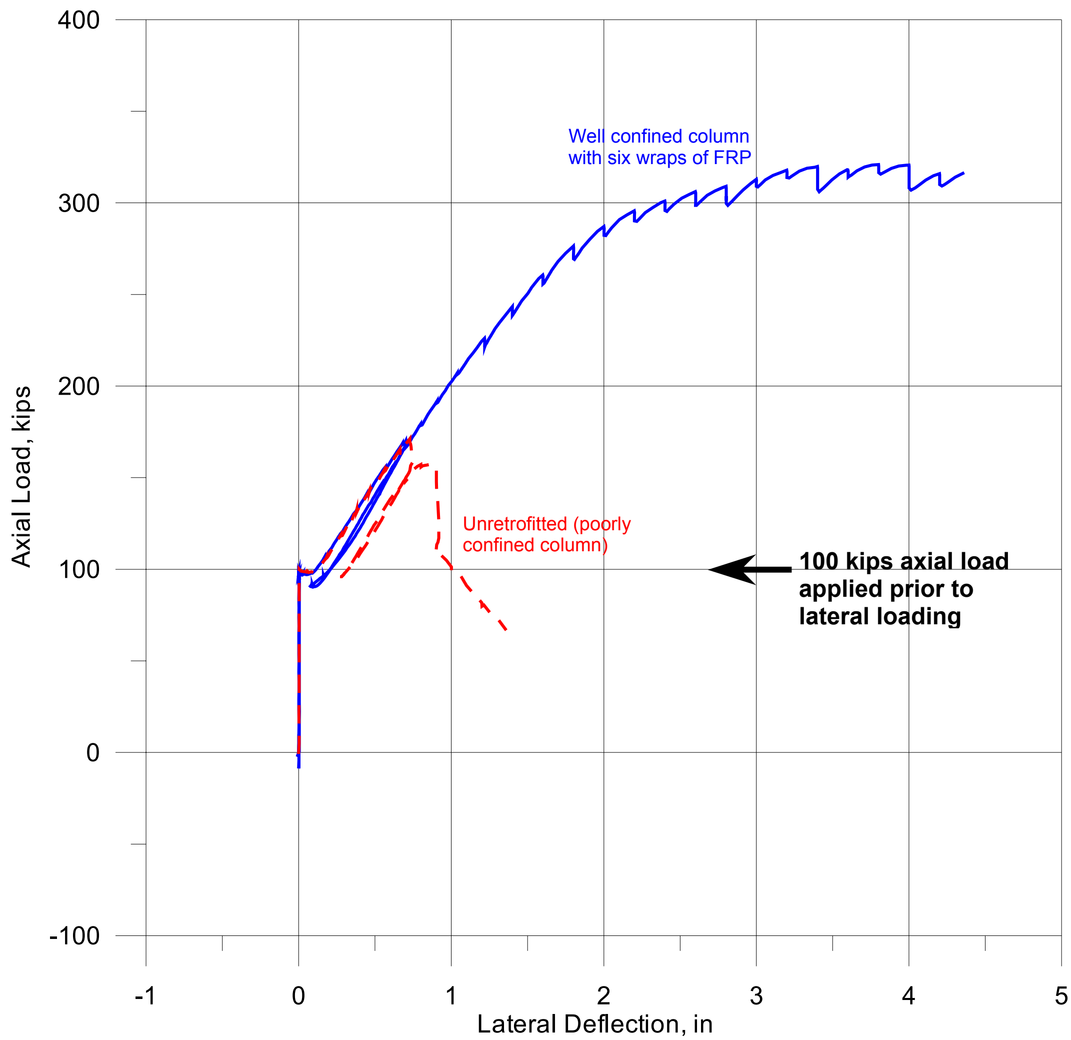

**Figure 21. Behavior typical of reinforced concrete columns subjected to blast loads, as predicted by DYNA3D from early time to peak deflection.**

**Figure 22. Composite wrap for reinforced concrete column, converting brittle blast response of column to highly ductile one.**



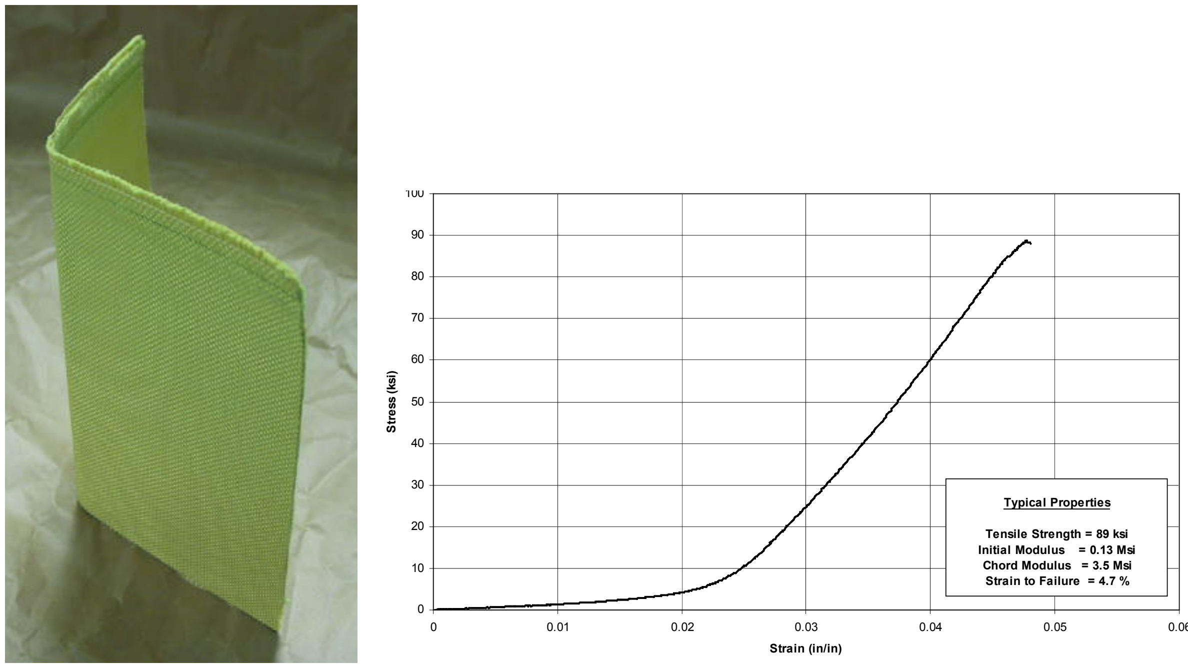

(a) Calculated lateral load history of well and poorly confined columns (nearly exact match of test data).

**Figure 23. Lateral and axial load history of well- and poorly-confined RC columns.**

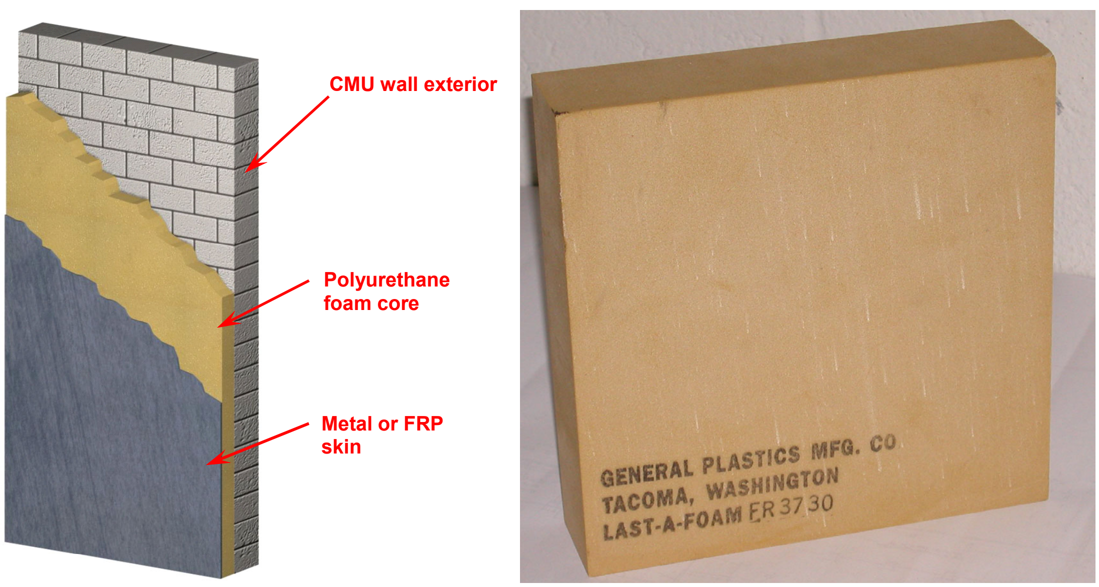



(b) Measured axial load history of well- and poorly-confined columns (calculation matches measured data); for well-confined column, axial capacity at 9.5 inches of lateral deflection reduces back to the initial 100 kips applied at the start of the test.

**Figure 23. Lateral and axial load history of well- and poorly-confined RC column (Continued).**




(a) Kevlar laminate, provided in multiple plies (10 plies = 17,000 lb/in capacity).




(b) Polyurethane panel.

**Figure 24. Concepts for strengthening bearing walls.**

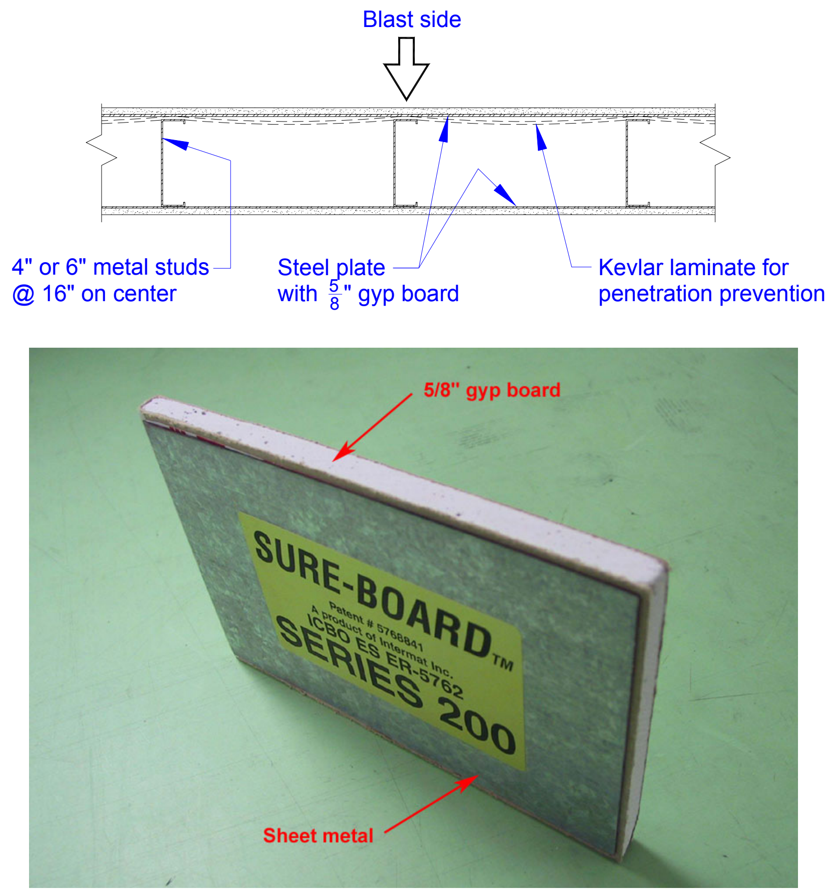



**Figure 25. Secondary bearing wall to be blast-resistant, use Sure-Board™, for constructing stud walls.**