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AN INTEGRATED PROGRAM TO IMPROVE THE PERFORMANCE OF WELDED STEEL FRAME BUILDINGS

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ABSTRACT

This paper reviews the performance of welded steel moment frame buildings during the Northridge earthquake, and examines some of the studies being undertaken in the U.S. as part of the FEMA-funded SAC Steel Project to devise improved methods for designing new steel frame structures and for inspecting, evaluating and repairing seismic damage to these types of structures. The basis for some of the provisions is summarized.

KEYWORDS

Steel Buildings, Moment-Resisting Frames, Design Criteria, Inspection, Evaluation, Repair, Rehabilitation, Experimental Investigations, Dynamic Analyses

INTRODUCTION

One of the important lessons of the Northridge earthquake of January 17, 1994, was the widespread and unanticipated brittle fractures in welded steel beam to column connections. The economy, versatility and presupposed high plastic deformation capacity of welded steel moment-resisting frame (WSMF) buildings has resulted in their common usage in Los Angeles as well as in other regions of high and moderate seismicity. No casualties or collapses occurred as a result of these connection failures, and some welded steel moment frame (WSMF) buildings in areas of moderate shaking were not damaged at all. However, a wide spectrum of brittle connection damage did occur, ranging from minor cracking to completely severed columns. The most commonly observed damage occurred in or near the welded joint of a girder bottom flange to the supporting column flange; complete brittle fractures occurred in many of these joints. Damage was so severe in some buildings that all of the moment resisting connections at one or more floors fractured, or significant permanent lateral displacements occurred.

Thus far, more than 150 damaged steel buildings have been identified. Damage occurred in new as well as old buildings; in tall as well as in short structures. While inadequate workmanship played a role in the damage observed in some structures, most damaged buildings are believed to be constructed consistent with modern codes and standards of practice.

The effect of these observations has been a loss of confidence in the procedures used to design and construct welded connections in steel moment frames, and a concern that structures incorporating these connections may not be sufficiently safe. Current professional judgment is that historic practices used in the U.S. for the design and construction of WSMF connections do not provide adequate reliability and safety. As a consequence, pre-qualified connection details and design methods contained in the major U.S. building codes have been rescinded, and emergency code provisions generally stipulate that new designs must be substantiated by testing or test-backed calculations. To develop effective and economic design procedures and construction standards, and to restore public and professional confidence, several fundamental questions must be answered. These include:

- What happened to WSMF buildings during the Northridge earthquake?
- What caused the observed damage?
- How to identify WSMF buildings that may have sustained damage?
- How safe are damaged WSMF buildings and do they need to be repaired?

- How can damaged buildings be reliably repaired and/or upgraded?
- How to design and construct new buildings so they will not sustain similar damage in future earthquakes?
- Can the vulnerability of existing WSMF buildings to future earthquakes be reliably determined and mitigated through effective rehabilitation procedures?
- What are the economic, social and political impacts of any required changes in design and construction practices?

Answering these questions involves consideration of many complex technical, professional and economic issues including metallurgy, welding, fracture mechanics, connection behavior, system performance, and practices related to design, fabrication, erection and inspection. Unfortunately, current knowledge of many of these issues is inadequate. To resolve this situation, the U.S. Federal Emergency Management Agency (FEMA) initiated a four year program to develop reliable and cost-effective methods for the inspection, evaluation, repair, rehabilitation and construction of steel frame structures. The scope of these efforts is summarized in this paper, focusing primarily on issues related to evaluation and repair of existing WSMF . Additional information may be found in Ref. 1.

PROGRAM TO REDUCE EARTHQUAKE HAZARDS IN STEEL BUILDINGS

A coordinated, problem-focused program of research, investigation and professional development has begun under FEMA-sponsorship in order to develop and validate reliable and cost-effective seismic-resistant design and rehabilitation procedures for WSMF structures. This program is being managed and administered by the SAC Joint Venture. This joint venture consists of three not-for-profit professional and educational organizations: the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC) and the California Universities for Research in Earthquake Engineering (CUREe). The program actively involves engineers, researchers, construction experts and other specialists from throughout the U.S.

The SAC Steel Project is divided into two major phases. The first phase focused on the development of *Interim Guidelines* [1] for the inspection, evaluation, repair, modification and construction of steel structures. This phase included limited amounts of laboratory and field testing, as well as other topical investigations. Major efforts to identify long-term solutions and to develop and verify reliable and cost-effective seismic design criteria for steel frame structures are contained in the second phase. Currently, Phase 2 efforts are just beginning.

The focus of the SAC Steel Project is on the development of specific design advisories, guidelines and other criteria for design, inspection, evaluation, repair, modification and rehabilitation of WSMF structures. The *Interim Guidelines* [1] developed in Phase 1 were written by a committee of ten experts from a variety of disciplines. The *Guidelines* were subjected to extensive review by engineers, researchers, building regulators and other public officials, and representatives from the steel and construction industries. The scope of the *Interim Guidelines* includes welding procedures, quality assurance, post-earthquake actions, and new construction. Specific chapters cover: (a) welding and metallurgy; (b) quality control and assurance; (c) visual inspection; (d) non-destructive testing; (e) classification and implications of damage; (f) post-earthquake evaluation; (g) post-earthquake inspection; (h) post-earthquake repair and modification; and (i) new construction.

As shown in Fig. 1, various preliminary investigations and tests were carried out to support the development of the Interim Guidelines and the planning of Phase 2. The following sections describe some of these studies as they relate to the inspection, evaluation and repair of existing WSMF buildings.

SURVEY OF NORTHRIDGE BUILDING DAMAGES

A systematic effort has been undertaken to assess the actual performance of steel buildings the Northridge earthquake. Three levels of survey were conducted. In the first, a brief questionnaire was sent to more than two hundred, randomly-selected owners of steel buildings. This preliminary survey was used to estimate the overall scope of damage to steel buildings and to help identify geographic areas where steel buildings were damaged. Based on this and other more detailed, the *Interim Guidelines* recommended detailed inspection of steel buildings where ground motions exceeded 0.2g.

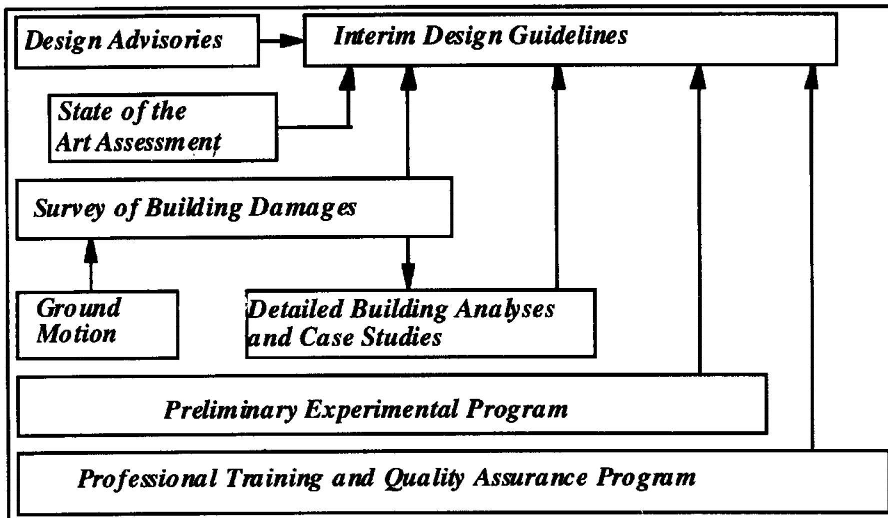

Figure 1.- Task flow chart for Phase 1 of the SAC Steel Project.

A second level of survey was carried out by engineers on damaged steel frames [3,4]. Detailed information was obtained on 89 buildings regarding the types and locations of damage observed and their structural configuration, materials and detailing. This second survey has been supplemented by a detailed survey of damage in 8 buildings selected for dynamic analyses. Results of these surveys were used to further identify methods that can be used to select buildings for inspection, and the joints within a suspect buildings that should be inspected. For example, it was found that on average 70% of the floors of the buildings surveyed had serious damage to at least one welded joint. Only 25% of the connections were found with no damage. About 20% of the building frames included in the study had more than 40% of their connections damaged; in a few instances, all of the connections at one or more floors were damaged.

Damage in low rise structures appears to be more or less uniformly distributed over height (with slightly more severe damage occurring near the bottom of the structure), whereas tall buildings exhibited greatest damage in the upper half. Survey results also show that damage tends to congregate; thus, finding a severely damaged connection as part of an inspection process could serve as a trigger for inspection of other nearby connections.

DETAILED ANALYSES OF NORTHRIDGE STEEL

Twelve buildings subjected to the Northridge earthquake were selected for detailed analysis by consultants using elastic and nonlinear analysis programs. Buildings were selected with heights ranging from 2 to 17 stories, and in locations varying from Santa Clarita on the north of the epicenter to Santa Monica on the south. The analyses were intended to help identify the causes of the damage, as well as the reliability of various analytical methods and modeling assumptions in predicting the observed damages. In two cases, buildings with out any apparent damage were included; these buildings were immediately adjacent to other damaged buildings included in the study. In four of the buildings, recordings of response during the Northridge earthquake were available and in three other cases, ambient vibration tests were performed.

The analysis results indicate that the buildings were very strong in comparison with the design forces incorporated in current U.S. building codes. For many of the building sites the estimated response spectra were nearly double those considered in building codes (assuming elastic response, $R_w = 1$). Elastic analysis results showed that the most heavily damaged buildings were only stressed 2 to 3 times their capacities; suggesting that the buildings may be 4 to 8 times stronger than required. The main reason for this over-strength appears to be the use of large members to satisfy stringent drift requirements.

Analysis results have limited to poor correlation with observed damage. The most heavily stressed joints are the most likely to be damaged; however, the precise location and severity of damage is not reliably predicted by analysis. The 60% most highly stressed connections have roughly equal chance of being damaged. Similarly, areas of low computed stress were subject to damage. The reasons for these differences include the effect of initial defects and poor workmanship, and the limitations of the analytical methods. More detailed elastic and inelastic analyses shed some light on this last possibility. First, both types of analyses indicate that higher mode effects were *very* important. Thus, static lateral force methods in the elastic range, and nonlinear push-over analysis are of limited value for longer period structures. The predicted distribution of damage in is highly sensitive to small perturbations in the spectral characteristics of the ground motions considered.

Another aspect of this difficulty is the traditional models ignore or only crudely treat deformations in the beam to column panel zones. More refined analyses, especially those performed in the inelastic range, indicated that panel zone deformations were significant. Review of experimental data and finite element results suggests that in addition to anticipated axial deformations, high local bending and shear distortions are induced in the beam (and column) flanges near the welded joint due to panel zone yielding and other causes. Composite slab effects were also found to have an important effect on local deformations, and overall structural stiffness and periods. Current methods for modeling these phenomenon are quite simple and may be inadequate in many situations.

PRELIMINARY TEST PROGRAM

A total of 37 tests of full size beam-to-column connection subassemblies were done as part of this preliminary investigation. Twelve test specimens (like those shown schematically in Fig. 2) were constructed utilizing pre-Northridge details, half of the specimens had W36x150 beams and half had W30x99 beams. Fourteen-inch wide-flange sections were used as columns in both cases. Dual certified ($f_y > 50$ ksi) steel was used in all cases. Slabs were not included in any of the specimens. These specimens exhibited brittle appearing fractures; some specimens had fractured without any plastic deformation, while a few were able to deform to a plastic rotation of about 0.02 prior to fracturing.

These damaged specimens were repaired or upgraded. Repair consisted of simply rewelding the connections using high notch toughness FCAW procedures; backing bars were removed, the root pass of the CJP weld on the beam flange to column flange connection was air-arc gouged and repaired with a fillet weld. This is the prevalent practice in repairs of damaged buildings in the Los Angeles area. Test results indicated that the repaired specimens, constructed with careful quality control, were able to retain their pre-damage strength and stiffness. Plastic rotation capacities were not significantly different from those achieved in the first tests.

Some of the specimens were upgraded in an attempt to improve their plastic deformation capacity. In the cases investigated, inclined haunches were applied to one or both sides of the beam at its connection to the column. This detail attempted to move the plastic hinge in the beam away from the face of the column to the end of the haunch. These tests supplemented earlier tests by others which utilized trapezoidal and rectangular shaped cover plates, vertical fins, or side plates. Test results on the triangular haunches indicate that they are generally able to increase the plastic deformation capacity of the connection to at least a plastic rotation of 0.03.

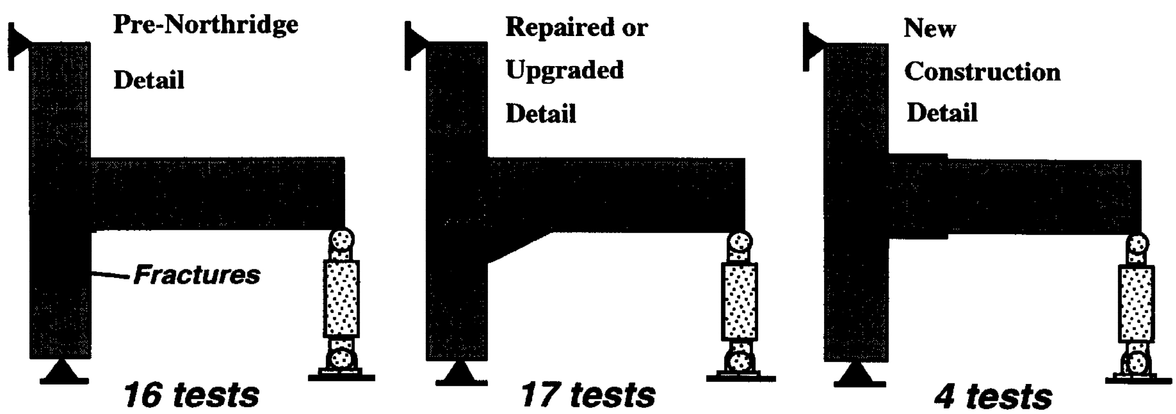

Figure 2 - Preliminary Test Program

Four beam-column assemblages were removed during the demolition of a heavily damaged building. These specimens were tested in their damaged condition as well as following repairs. Four tests have also been conducted on details believed appropriate for new construction. Simple weldment specimens were used to assess effects of various weld procedures, initial defects, materials, repair methods and loading rates.

RECOMMENDATIONS FOR EVALUATION, INSPECTION AND REPAIR

Based on the results of the studies described above, additional studies undertaken by others, practical considerations and expert opinion, a preliminary methodology was developed as part of the SAC Steel Project for the inspection, evaluation and repair of WSMF buildings subject to damaging earthquakes. The overall process is illustrated in Table 1. This consists of a preliminary evaluation in which key parameters are examined to identify the need to conduct a detailed inspection and evaluation of a particular building. A building believed to have sustained structural damage is then subjected to additional analysis and physical inspection to assess its likely state of damage, represented by two parameters: D_i and P, where D_i is the likely damage index at a floor "i" and P which represent the probability that the damage index for any floor in the building exceeds 1/3. Recovery actions to be taken depend on the values of D_i and P, and range from doing nothing to completely modifying the structural system. Detailed procedures for repairing or modifying damaged connections are included in the *Interim Guidelines* [1].

Preliminary Evaluation	Detailed Evaluation	Recovery Actions
• Estimate ground motion at site • Proximity to fault rupture • Preliminary inspection of damage to architectural and exposed structural elements • Observe damage in neighboring buildings	• Analyze building and/or • Inspect sample of connections in building - random selection - deterministic selection - analytic selection • Evaluate damage state - assign damage intensity factor, d_g, to each connection - estimate damage index D for structure -probability P that damage on at least one floor exceeds 1/3	• Building occupancy - continued, or - interrupted • Repairs - not required, - some damaged connections, or - all damaged connections • Modification of connections or system

Table 1 - Basic Steps in Evaluation Process

Selection Of Buildings For Detailed Inspection

The widespread damage to welded and other connections in WSMF buildings during the Northridge earthquake supports the need for rigorous post-earthquake evaluation of buildings incorporating vulnerable welded moment-resisting connections. The *Interim Guidelines* identify several triggers that would be appropriate to initiate a detailed evaluation as indicated in Table 1. The *Interim Guidelines* indicate that a structural analysis is not necessary, but may be performed.

As a class, it was believed that existing undamaged WSMF buildings appear to have a lower risk of collapse than many other types of buildings with known seismic vulnerabilities, the performance of which is currently implicitly accepted. Consequently, mandated or emergency programs to upgrade the performance of these buildings does not appear necessary to achieve levels of life safety protection currently tolerated in the U.S. However, the risk of collapse is definitely greater than previously thought. Thus, the *Interim Guidelines* recommended that individual owners should be made aware of the increased level of seismic risk and encouraged to perform modifications to provide more reliable seismic performance, particularly in buildings housing many persons, or in critical occupancies.

Evaluation of Damage State, Safety and Recovery Actions

Once a building is suspected of having sustained earthquake damage, the damage state and safety of the building must be assessed. The *Interim Guidelines* recommend an 8 step procedure, as outlined below:

1. Assign connections to groups
2. Determine sample of connections to be initially inspected in each group
3. Make inspections and find connection damage factor, d_j $(0 < d_j < 10)$
4. Perform additional inspections near badly damaged connections
5. Estimate average damage index D for connections in group
6. Determine probable D_{max} at any floor and probability P that one floor has $D_{max} > 1/3$
7. Determine appropriate actions from D_{max} and P
8. Prepare report to owner

The welded steel moment connections in a structure are divided into groups expected to have similar mechanical characteristics. An initial sampling of connections from each group is made. Visual and non-destructive methods are used to identify structural damage in these connections. Three methods have been recommended in the *Interim Guidelines* to select the inspected connections: random sampling; a deterministic pattern selection process; and an analytically-based process. Because of the weak correlation between analytical predictions and observed damage, it was felt adequate to randomly select the locations of the connections to be inspected. Typically, 10 to 30% of the connections are inspected as a minimum, with a larger percentage required when smaller numbers of connections form the group. Detection of severe damage triggers the need for additional inspections of adjacent connections. This and other requirements (Table 3) can substantially enlarge the number of inspections.

Observed Damage	**Connector damage index, d_j**
Buckled flange	4
Yielded flange	1
Flange fracture (top or bottom)	8
Flange fractures (top and bottom)	10
Web yielding/buckling	4
Web fracture	10

Table 2 - Typical Damage Indices d_j Corresponding to Girder Damage, Excluding Damage To Welds

It is desirable to rationally estimate the effect of any detected damage has on the stiffness, strength and deformability of the connections. Given the difficulties in making this assessment, a simple connection damage index d_j is suggested in the *Interim Guidelines*. The connection damage indices vary between 0 (for no damage) and 10 for total loss of effective capacity. Various categories of damage are considered, including damage to welds, girders, columns, shear tabs, and panel zones. Examples of damage indices associated with girders attached to a connection are shown in Table 2.

It is also recognized that the structural deformations causing the damage in the welded beam to column connections may cause damage in other types of welded or bolted connections. Thus, the *Interim Guidelines* recommend that inspection be extended to these types of connections under certain circumstances.

To assess the safety and need for specific recovery actions, a floor level damage index D_i is computed. This index ranges from 0 (no damage) to 1 (complete loss of connection capacity at a floor). First order, second moment statistical procedures (assuming normal distributions of damage) are used along with the d_j values determined for the inspected connections to estimate the maximum likely value of D_i at a floor and the likelihood (P) that any D_i exceeds 1/3. The values of P and the maximum D_i value for all floors (D_{max})are used to determine appropriate recovery actions. Recommended recovery actions are shown in Table 3.

Damage State	**Recommended Recovery Action**
All cases	Repair connections with $d_j > 5$
$D_{max} > 0.1$	Repair connections with $d_j > 2$
$D_{max} > 0.2$	Inspect <u>all</u> connections.
$D_{max} > 0.33$	Assess building safety. Repair connections with $d_j > 1$.
$D_{max} > 0.5$	**Unsafe condition exists.** Repair all damage and modify connections or system for improved performance.

Table 3 - Recommended Recovery Actions as a function of D_{max} [Each action includes actions required for previous states.]

When a WSMF experiences damage to a significant percentage of its moment-resisting connections (on the order of 25% in any direction of resistance), in addition to repair, consideration should be given to modifying the configuration of the individual damaged connections and possibly some or all of the undamaged connections to provide improved performance in the future. However, partial modifications to the structural system should be made with due consideration of the effect on overall system behavior. When a WSMF building has had many seriously damaged connections (on the order of 50% in any direction of resistance), owners should be informed that this damage may have highlighted basic deficiencies in the existing structural system, or a geologic feature which unusually amplifies site motion. In such cases the existing system should be both repaired and modified to provide an acceptably reliable structural system. Modifications may consist either of local reinforcement of individual connections and/or alteration of the structure's basic lateral-force-resisting system.

Inspection Procedures

The *Interim Guidelines* contain precise recommendations for the conduct of the inspections. These include methods for exposing, visual inspections and nondestructive testing of connections, qualifications of inspectors, and a standardized classification and reporting system. Emphasis is placed on the need for careful visual inspection, as supplemented by the use of magnetic particle testing and liquid dye penetrant testing. While the use of ultrasonic testing is encouraged, difficulties exist in detecting fractures in joints, especially at the beam to column connections where backing, runout tabs and other conditions may lead to spurious images

Repair of Earthquake Damages

Currently, the vast majority of connections damaged by the Northridge earthquake are being simply repaired to their pre-earthquake condition. As used in the *Interim Guidelines*, the term repair means restoration of the strength, stiffness and deformation capacity of structural elements that have been damaged or have construction defects. Modification means actions taken to enhance the capacity of either damaged or undamaged elements.

Currently, there is no clear evidence that one welding procedure can always produce better welds and structural performance. However, it appears that poor workmanship and inadequate structural details can produce poor performance regardless of the welding procedure. Thus, it is imperative that repair techniques fully account for proper welding procedures and incorporate rigorous quality control and assurance programs. A written Welding Procedure Specification is required to provide specific instruction to the welder and inspector for each weld condition encountered. In particular, care must be exercised in using proper pre-heat and post-heat, electrode diameters and deposition rates, and inspection procedures. Special reference is made in the *Interim Guidelines* to the desirability of using weld filler material with high impact toughness (>20 ft-lbs at 0° F).

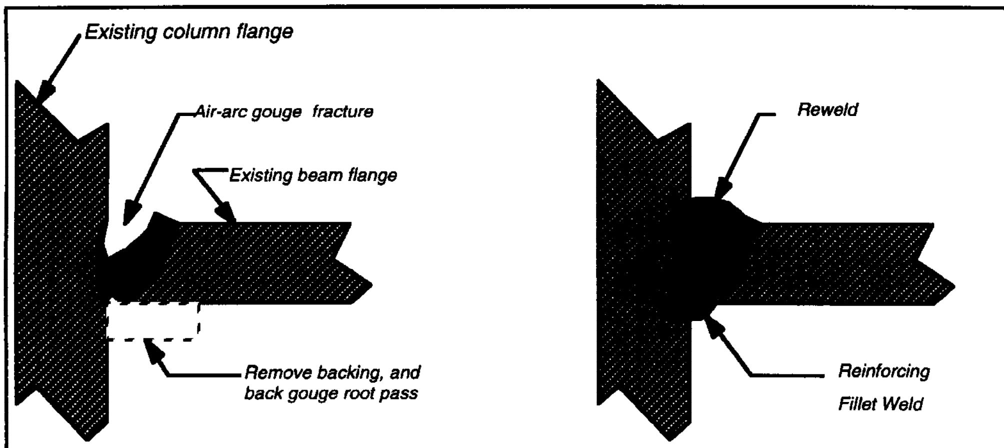

Figure 3 - Back-gouge and re-weld of typical beam to column flange connection

Several examples of repairs are cited in the *Interim Guidelines*. Two of these are shown in Figs. 3 and 4. In situations where a fracture has occurred in or near the weld joining the beam and column flanges, it is recommended that the backing bar be removed and the root pass be air-arc back gouged and subsequently repaired with a fillet weld. Any fractures in the weld and base metal should be air-arc gouged, ground to achieve proper fit up, rewelded, and inspected. In some circumstances the fracture may extend into the column (e.g., a divot of column base material may be removed by the fracture) and it may be necessary to rebuild the column section by filling the divot with weld material, grinding the repair weld surface smooth, and inspecting the repaired column flange. In all welding situations, it is recommended that the engineer should consult with knowledgeable experts in the fields of welding and inspection.

In some cases it is necessary to replace portions of a member. For instance, it may be necessary to remove the bottom flange of a beam due to severe damage or to gain access to make repairs to the column flange. In such cases, the beam flange may be replaced by a plate. Care needs to be exercised in orienting the plate with the rolling direction in the proper direction. In some cases, the plate can be welded in the same location as the original flange (welding the plate to the beam web along the K line as well as to the column and beam flange). In other cases it may be acceptable to weld a plate to the surface of the existing flange (see Fig. 4). In both cases, care must be taken regarding local details, welding procedures and avoidance of local or lateral buckling of the flange and/or web.

It may be desirable to improve behavior of a connection through modification. Currently, most approaches involve moving the plastic hinge away from the face of the column where welds join the beam to the column. This may be accomplished by locally strengthening the beam near the column face, or locally weakening the beam at a location where the plastic hinge is desired. Various details have been suggested (see SAC (1995) for a more complete discussion). Local strengthening can be accomplished by addition of cover plates, side plates, vertical fins, haunches and knee braces. Weakening of the beam away from the face of the column can be achieved by reducing the area of the flanges by drilling holes in the beam flanges or by trimming the width of the flanges.

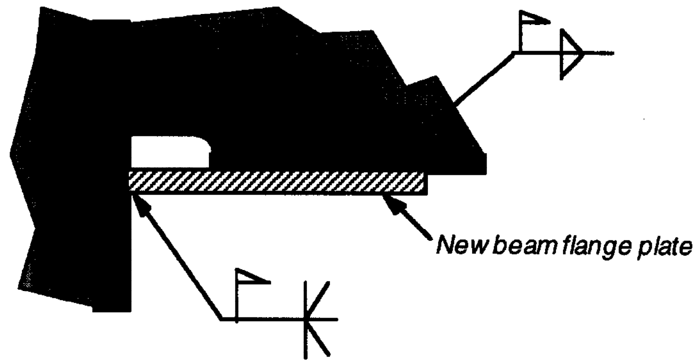

Figure 4- Example of beam flange repair by replacement with steel plate

Because behavior of modified details have not worked satisfactorily in all cases, it is necessary to confirm designs using tests or test-validated calculations. Recommendations are made in the *Interim Guidelines* regarding the loading and instrumentation program, as well as for situations where new tests are required. Plastic rotation capacities recommended for modified details for use in existing buildings are 0.025 radians. This is lower than recommended values for new buildings (0.03 radians) since it is not expected that existing buildings will be designed for the same level of reliability as new structures.

Design recommendations for modified details are based on realistic estimates of actual material properties including strain hardening effects. Similarly, moving the plastic hinge towards the midspan of a beam will increase the plastic rotation demands and, when the beam is strengthened at its end, increase the moment at the face of the column . This increase in moment will require careful evaluation of demands on the panel zone and column, in order to avoid unwanted plastic deformations in these locations.

CONCLUDING REMARKS

While the *Interim Guidelines* represent current U.S. thinking on the proper evaluation, inspection and repair of existing WSMF buildings as well as for the design and construction of new ones, there are clearly many uncertainties and unresolved questions. In Phase 2 of the FEMA/SAC Steel Project additional research and testing will be conducted to more clearly define the parameters controlling the performance of connections and systems, and to develop and verify procedures for design and rehabilitation of moment frame structures. Efforts will be made to cooperate with related activities worldwide.

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REFERENCES

SAC Joint Venture (1995), Interim guidelines: evaluation, repair, modification and design of welded steel moment frame structures, Report FEMA 267, Federal Emergency Management Agency, Washington DC.