7th Canadian Conference on Earthquake Engineering / Montreal / 1995
7ième Conférence canadienne sur le génie paraséismique / Montréal / 1995

The Northridge Earthquake Steel Moment Frame Failures: Results of Phase One of the S.A.C. Joint Venture Research Program

A.E. Ross[1]

INTRODUCTION

The Northridge earthquake has shaken the structural engineering community's confidence in the seismic performance and safety of steel moment frame buildings. Before January 17, 1994, steel moment-resisting frame buildings were considered to be the best form and material for seismic resistant construction. Now, with observations of extensive fractures in beam-to-column connections, steel structures are understood to be susceptible to severe seismic damage. While none of the buildings damaged in the Northridge earthquake collapsed or caused serious injury to their occupants, their damaged condition may leave them especially vulnerable to future earthquakes and represents a major economic cost for repair.

In response to these observations the SAC Joint Venture initiated the SAC Steel Program: Reducing Earthquake Hazards in Steel Moment Frame Structures. The SAC Steel Program is one of professional practice development and problem-focused investigations. SAC is a joint venture of the Structural Engineers Association of California (SEAOC), of the Applied Technology Council (ATC) and the California Universities for Earthquake Engineering (CUREe). The objective of the SAC Steel Program is:

Develop professional practices and recommend standards for the repair, retrofit and design of steel moment frame buildings so that they provide reliable, cost-effective seismic performance in future earthquakes.

A comprehensive program was formulated by the SAC Joint Venture to carry out the technical studies and analyses necessary to achieve the above stated goal. The overall program has a time frame consistent with the urgency of the problem.

The four specific efforts addressed by the overall SAC program are to:

1. Characterize and understand what happened to steel moment frame buildings during the Northridge earthquake;
2. Prepare interim procedures for professional practices and standards applicable to:
   - identification of buildings that may have been damaged and that require detailed inspection and investigation;
   - evaluation on seismically vulnerable buildings, including the characterization of the safety of inspected or damaged buildings;
   - rehabilitation of damaged buildings; and
   - design of new steel frame structures.

[1] Arthur E. Ross, Chairman of SAC Joint Venture Management Committee

3. Conduct directed and coordinated technical investigations, analyses and research, as necessary, to develop new knowledge necessary to develop reliable and cost effective design guidelines and standards of practice related to steel moment frame structures; and
4. Prepare recommendations for the repair, retrofit and design of buildings based on rational understanding of seismic behavior.

THE WORKPLAN

Upon initial funding by the State of California in September of 1994, a workplan for Phase I was developed which outlined by task, the work to be done and budget amount for each task. The workplan is outlined as follows:

Table 1 -- Phase 1 Tasks

Task 1	Organize Phase I Program and Engage Key Participants	
Task 2	Inspect and Assess How Buildings Performed During the Northridge Earthquake to Understand the Damage	
	Sub-Task 2.1	Surveys of performance of steel frame buildings in the Los Angeles vicinity
	Sub-Task 2.2	Development of Damage Maps
	Sub-Task 2.3	Interviews to gather and synthesize field experience related to inspection, evaluation, repair and construction.
	Sub-Task 2.4	Testing in support of Task 2
Task 3	Perform Detailed Assessment of the Performance of Selected Buildings	
	Sub-Task 3.1	Detailed investigations of particular buildings
	Sub-Task 3.2	Field testing of selected buildings
	Sub-Task 3.3	Detailed case study interviews
	Sub-Task 3.4	Identification of preliminary implications of weld fractures
	Sub-Task 3.5	Sensitivity studies
Task 4	Characterize Ground Motions at the Sites of Subject Buildings	
Task 5	Develop Design Advisories	
	Sub-Task 5.1	Prepare Design Advisory No. 1 (for distribution)
	Sub-Task 5.2	Prepare Design Advisory No. 2 (internal working document)
	Sub-Task 5.3	Prepare for Design Advisory Workshop
	Sub-Task 5.4	Hold Design Advisory Workshop
	Sub-Task 5.5	Prepare Advisory #3 from Workshop
	Sub-Task 5.6	Disseminate Advisory No.3
Task 6	Assess Current Knowledge	
	Sub-Task 6.1	State of art papers
	Sub-Task 6.2	Literature survey database

Task 7 Conduct Laboratory Tests of Steel Assemblies

Sub-Task 7.1 Establish details of the test program

Sub-Task 7.2 Testing of details for new and repaired beam to column connections

- 24 total tests on two different sizes (W36x150 and W30x99 girders)
- 12 tests of pre-Northridge connections
- 4 tests of re-weld only repair
- 4 tests of T-haunch
- 4 tests of sloped T-haunch
- ATC-24 loading protocol

Sub-Task 7.3 Special tests to assess factors influencing the performance of weldments

Sub-Task 7.4 Testing of material properties used in test assemblies and evaluation of specimen failure modes

Task 8 Develop Draft Interim Guidelines

Sub-Task 8.1 Identify issues for Interim Guidelines and develop outline for draft Interim Guidelines

Sub-Task 8.2 Develop draft Interim Guidelines

Task 9 Conduct Users Workshop and Finalize Interim Guidelines

Sub-Task 9.1 Conduct Users Workshop

Sub-Task 9.2 Revise Interim Guidelines

Task 10 Publish the Interim Guideline Documents and conduct a Seminar in Southern California

Sub-Task 10.1 Publish and distribute Interim Guidelines

Sub-Task 10.2 Publish and distribute other resource material

Sub-Task 10.3 Identify conclusions and recommendations

Sub-Task 10.4 Conduct Seminar(s)

At the time of this writing, all of Tasks 1, 2, 4 and 5, and most of Task 3 have been completed. Task 7 is approximately 60% complete, and Task 8 is under way.

PHYSICAL TESTING

The intent of the testing program is to justify the proposed repair and strengthening details which may be included in the guidelines. It also will serve to be a pilot program for the more comprehensive testing program proposed for Phase II. The testing program was not envisioned to look into new innovative ideas or solve all of the research needs of the program.

The basic thrust of the testing program is to build and test to failure several pre-Northridge joints. These joints will then be rewelded or rewelded with strengthening added and then retested.

POST-EARTHQUAKE INSPECTION/EVALUATION OF BUILDINGS

Inspection is required when an earthquake of magnitude greater than or equal to 6.5 has caused ground shaking with peak ground acceleration exceeding 0.20g, or peak ground acceleration greater than or equal to 0.30g for earthquakes with magnitude 6.0 or greater.

In addition, inspection is required, when any of the following exists:

1. Significant structural damage is observed in a welded steel moment frame structure within 1 kilometer of the building's site;
2. Significant structural damage is observed to modern, apparently well-designed structures (of any material) within 1 kilometer of the building's site;
3. The structure is, for an earthquake having a Moment Magnitude of 6.5 or greater, either within the projection of the thrust block of a buried thrust earthquake or within 5 kilometers of a surface rupture.
4. Significant architectural or structural damage is observed.
5. Permanent out-of-plumbness of greater than 0.5% is observed.
6. Significantly excessive drift, or significant and anomalous period lengthening are observed in aftershocks.
7. Whenever the use or occupancy of the building has been limited because of earthquake damage regardless of the type or nature of the damage.

REPAIR OF JOINTS

Repair constitutes any measure or measures taken to restore a damaged joint, connection or element to its basic original design configuration. This refers to reconditioning the joint, connection or element in a manner such that both the strength and stiffness of the pre-damaged structure are restored. Work that increases structural strength or stiffness by more than 5% shall be classified as modification.

Based on the nature and extent of damage several alternative approaches to repair should be considered. Repair approaches may include, but should not be limited to:

a) replacement of portions of base metal (i.e. column and beam section),
b) replacement of connections elements,
c) replacement of connection weld, or
d) repairs to portions of any of the aforementioned components.

Where base material is to be removed and replaced with plates, clear direction should be given to orient the plates with the direction of rolling of the plate parallel to the direction of application of major axial loads to be resisted by the plate.

All fractures and rejectable defects found in weld material, either between beam and column or between connection element and structural member, shall have sufficient material removed to completely eliminate any discontinuity or defect. Simple fillet welds may be repaired by backgouging to eliminate unsound weld material and replacing the damaged weld with sound material. Complete joint penetration (CJP) welds fractured through the full thickness shall be replaced with sound material deposited in strict accordance with the Welding Procedure Specification (WPS) and project specifications. The use of weld dams on new welds is prohibited. Backup bars, existing dams, run-off table shall be removed, with the root backgouged to sound material. After backgouging, the root is to be re-welded utilizing the same welding electrodes as the balance of weld material being restored and repaired, with a reinforcing fillet added over the final weld pass. The structural engineer is cautioned to avoid mixing different weld metals in the process of replacing weld metal in a partially backgouged weld, e.g. E7018 stick electrodes should not be used to weld over flux core metal welding. Care should be taken in selecting weld procedures and electrodes to limit the extent of the heat affected zone (HAZ). Removed weld material from fractures not penetrating the full weld thickness shall be replaced in the same manner as full thickness fractures.

Any column fracture observable with the naked eye or found to be rejectable by nondestructive testing shall be repaired. Repairs shall include removing the fracture such that no sign of discontinuity or defect within a six (6) inch radius around the fracture remains. Removal shall include eliminating any zones of fracture propagation, with a minimum of heat used in the removal process. Repairs of removed material may consist of replacement of portions of column section, build-up of weld material where small sections of column were removed, or local replacement of removed base metal with weld material. Procedures of weld fracture repair should be applied to limit the heat affected area and to provide adequate ductility to the repaired joint.

Special attention should be given to conditions where more than 20% of the column cross section has been damaged or that are within the heat affected area of repair, as special temporary shoring may be warranted. In addition, care should be taken when applying heat to a flange or web containing a fracture, as fractures have been observed to propagate with the application of heat.

Where the top or bottom flange of a beam has buckled, and the rotation between the flange and web is less than or equal to the mill rolling tolerance given in the AISC Manual of Steel Construction, ninth edition, the flange need not be repaired. Where the angle is greater than mill rolling tolerance but less than 10 degrees, repair is required and shall consist of at least adding full height stiffener plates on the web over each portion of buckled flange, contacting the flange at the center of the buckle. For buckled flanges with angles greater than or equal to 10 degrees, the entire portion of flange that has buckled shall be removed and replaced.

Should flange buckling occur on only one side of the web, and the buckle repair consists of adding stiffener plates, only the side that has buckled need be stiffened. In case of partial flange replacement, special shoring requirements should be considered by the design engineer.

At bottom beam flange repairs, if the damage is such that backgouging removes sufficient material such that a backing bar is required for the repair, the back up bar is to be removed from the beam after welding. The weld root is to be inspected and tested for imperfections, which if found, are to be removed by backgouging to sound material. The size of the reinforcing fillet weld shall be equal to one-quarter of the beam flange thickness, but not less than one-quarter inch nor more than three-eight inch (see Note J, Figure 2.4 of AWS D1.1.)

If the bottom flange weld requires repair, the following procedure may be considered:

1. The root pass should begin in the center of the joint, in the area of the web access hole. After the arc is initiated, travel shall progress toward the end of the joint (outboard beam flange edge), and the weld shall be terminated on the weld tab.
2. The half length root pass shall be thoroughly slagged and cleaned.
3. The start of the weld in the web access hole shall be visually inspected to ensure fusion, soundness, freedom from slag inclusions, and excessive porosity. The resulting bead profile shall be suitable for obtaining good fusion by the subsequent pass to be initiated on the opposite side of the beam web. If the profile is not conductive to good fusion, the start of the first root pass shall be ground, gouged, chipped or otherwise prepared to ensure adequate fusion.
4. The second half of the weld joint shall have the root pass applied before any other weld passes are performed. The arc shall be initiated in the area of the start of the first root pass, and travel shall progress to the outboard end of the joint, terminating on the weld tab.
5. Each weld layer shall be completed on both sides of the joint before a new layer is deposited.
6. Weld tabs should be removed and ground flush to the beam flange with minimal disturbance.

WELDING CONSIDERATIONS

Repairs involving complete or partial joint penetration welding at or around joints should be reviewed by a structural engineer having special expertise in welding or a Welding Engineer. The welding engineer should review the repair for proper selection of welding electrodes, appropriate welding procedures (including preheat, postheat and cool down requirements), the effects of welding on the metallurgy of the parent materials, and limitation of the HAZ.

A WPS shall be established for every different weld configuration, welder position, material, type and location. The WPS shall be reviewed by the structural engineer responsible for repairs. The WPS is a set of focused instructions to the welders and Inspectors stating how the welding is to be accomplished. Each class of weld is to have its own WPS solely for the purpose of that weld. WPS should include instructions for joint preparation based on material property and thickness, as well as welding parameters. Weld process, electrode type, diameter, stick-out, voltage, current, deposition rate, and interpass temperature should be clearly defined. In addition, joint preheat and post-heat requirements should be established, including insulation guidelines if applicable. A copy of the WPS is to be located on site, accessible to all parties involved in the repair.

The structural engineer or designated Welding engineer shall perform a complete review of the Fabricator/Erector's quality control program, equipment condition, and availability of equipment and qualified personnel. In addition, the structural engineer or designated Welding

Engineer shall also perform a complete review of the QA Agency. This review should encompass personnel qualification, written procedures manual, NDE equipment condition, and availability of equipment and qualified personnel. The Agency shall employ an ASNT Level III qualified person who oversees equipment calibration and personnel certification and training for the project on a full time basis.

STRUCTURAL MODIFICATION

Within the context of damage to WSFM's, the team "structural modification" refers to alterations to the joint to improve its earthquake performance and that involves substantial changes to the joint's geometry, capacity, or relevant limit states (e.g. flexural or shear strength or stiffness.

Structural modifications to the connections in a WSMF can be made at connections that have sustained damage as well as those that are undamaged but nevertheless are to be modified. It is recommended that the modification of connections follow a rational spatial distribution that considers the effect of those modifications on the performance of the lateral system as well as on the performance of individual components of the frames. Such a rational distribution should be developed as a result of an appropriate analysis to ensure that undesirable stiffness irregularities are not introduced or made more severe, and that excessive demand is not concentrated in joints unable to resist the applied loads or deformations.

Demand on modified connections should reflect realistic estimates of the required level of plastic rotation. In the absence of project-specific information to the contrary and unless rational demands are specifically calculated and shown to be lower, connections should be shown to be capable of developing a minimum plastic rotation of 0.2025 to 0.030 radian. The demand may be lower when supplemental damping or braced frames are introduced into the moment frame system. The effect of these alternative structural systems on existing components should be considered.

Modified beam-column connections can be qualified using two basic procedures: testing and calculations.

The testing program should replicate as closely as practical the anticipated conditions in the field, including such things as:

1. Member sizes and material specifications.
2. Welding process, details and construction conditions.
3. Cover-plates, continuity plates, web tabs, bolts, and doubler plates.
4. Connection configuration (e.g. beams on both sides)
5. Induced stresses because of restraint conditions on the welds and connection members.

Calculations should be correlated to tested materials properties for base metals and welds. The properties should be those corresponding to the axes of loading of the base metal or weld in the connections and to the welding processes and materials intended for use. The tested properties may be specific to the materials and processes to be used in the project, or based on a statically-based testing program. Use of properties inferred from other testing programs must be done with appropriate care and, where such inferred properties are used, designs should reflect the uncertainty inherent in such an indirect approach.

Design of the connection shall account for the amplification in the moment at the plastic hinge to the column. The following equation relates the connection moment (M_c) and column moment to the specified plastic moment (M_p):

$$M_c = a\, B\, M_p \qquad (1\text{-}1)$$

Where:

a coefficient that amplifies $BM_p = M_{pr}$, to account for the connection moment and column moment being greater than the plastic hinge beam moment.

B amplifies M_p, to account for over-strength because of actual vs. specified yield strength and strain hardening as well as to account for uncertainty in analytical models of joint behavior. In the absence of adequate testing of the type described above, *B* shall be taken as 1.7y for A36 steel and 1.4 for A572, Grade 50 steel.

It should be noted that joint strengthening methods that involve shifting the plastic hingeaway from the column face through the use of cover-plates, haunches, side plates or other means will magnify the demand at the beam-column joint. These effects shall be considered in any analytical model of joint demand at both the connection and within the column. This effect is represented by the coefficient "*a*" in equation (1-1). The problem may be of particular significance in frames with narrow bays. In addition, other limit states, such as shear, may govern the connection design.