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REPAIR OF DAMAGED STEEL MOMENT FRAME CONNECTIONS WITH BOLTED BRACKETS

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ABSTRACT

Failures of welded steel moment frame connections in the Northridge earthquake have created an urgent need for a convenient, cost effective repair methodology in the Los Angeles area that restores steel moment frame connections to their design capacity. Currently, welded repairs are expensive, time consuming to install, inconvenient to tenants and the quality of the repair welds is difficult to quantify. Historically bolted connections have been relegated to providing only gravity or semi-rigid moment connections in steel frame buildings. However, recent tests of bolted connections by ICF Kaiser Engineers and Lehigh University as a joint industry study have shown that bolted connections are capable of providing rigid moment connections with cyclic plastic rotational capacities in excess of equivalent welded joints but with the same rigidity as welded connections. The cyclic testing conducted exceeded ATC-24 minimum standards on beams that ranged from 16 inches to 36 inches in depth. Field installation of bolted connections have proven fast and cost effective. Bolted repairs of welded moment frame connections are now underway in hundreds of connections.

KEYWORDS

Moment frame connection, steel, bolted connection

INTRODUCTION

The need to repair steel moment frame buildings in the Los Angeles region is widespread and growing. In 1995, the City of Los Angeles enacted an ordinance requiring inspection and repair of steel moment frame connections in geographic areas strongly affected by the 1993 Northridge earthquake. At least 600 buildings were directly effected by this ordinance. Likewise, adjacent cities have conducted their own inspections and uncovered earthquake damage. These cities are now in the process of enacting their own inspection and repair ordinances. Inspections for damage are not limited to buildings affected by ordinance requirements. The inspections of steel moment frame connections is also becoming a standard part of the "due diligence inspection", along with the typical inspections for asbestos and lead, required in most real estate transactions.

While few inspections have occurred outside the Los Angeles area, we can expect that damage also exists in steel moment frame buildings in the San Francisco-San Jose regions of Northern California due to the 1989 Loma Prieta Earthquake. Typical post-earthquake inspection procedures were not sufficient to uncover weld failures prior to the Northridge earthquake. However, Loma Prieta earthquake ground motions, particularly in the San Jose region, better known as "Silicon Valley", are similar to Northridge earthquake motions felt away from the epicenter that caused damage to steel moment frame connections.

Damage to moment frame connections from the Northridge earthquake generally occurred well below the yield capacity of the framing members and by all indications were brittle fractures rather than ductile yielding in the failure zone. Typically, the majority of these failures occurred in either the weld or the column flanges and, to a lessor extent, the beam flanges and web. Table 1 shows a connection damage classification chart in widespread use in the Los Angeles area. In accordance with the descriptions in Table 1, most of the column failures fall into classifications C1 to C4, and most of the weld failures fall into categories W1 to W2.

WELD REPAIR PROCEDURES

Welded repair of steel moment frame connections in occupied buildings is a complex process that presents a myriad of problems associated with the heat, fumes and accessibility of the welding. To repair a W2 failure (crack through the weld metal thickness) the occupied area must first be protected to prevent fires, contain sparks and provide for controlled venting of smoke and toxic fumes. This is best accomplished by installing a floor to ceiling gypsum wallboard enclosure. In most buildings the moment frames are on the perimeter of the building, making access to the beam flange adjacent to the perimeter window wall inaccessible for downhand welding. In such a case, the beam is shored and a large window cut in the beam web to provide access for welding the flange next to the window wall. Prior to welding, the beam and column flanges are pre-heated to 230°F. Preheating is accomplished by placing electrical resistance heating elements onto the beam and column flanges. The damaged weld is removed by thermal cutting or carbon air arc torch. The industry standard for SMAW and FCAW have changed to improve quality in the welds resulting in a more expensive welding processes. The slower, SMAW process, with small diameter E7018 electrodes is considered one of the best replacement weldments. After the backing bars are removed, the weld is back-gouged, rewelded and reinforced with a fillet weld. During the welding smoke and toxic fumes must be vented or filtered out. Continuous field inspection is required during the entire welding process.

A type C2, or flange tear-out repair is more complex than the W2 repair described above. In the C2 repair, in addition to replacing the beam to column full penetration weld, the damaged portion of the column flange must be removed, surfaces ground smooth, inspected and replaced with new weld material. Welded repair of C2 damage is difficult because the new beam flange full penetration weld is fused to the replacement weldment of the column flange and not to the parent material of the column flange, complicating the normal shrinkage problems associated with thick weldments. After the welding is completed members must be post-heated to 450 °F for at least 2 hours and then wrapped in insulation for another 8 hours.

Welded repair is clearly a complex process, with many potential hazards to the building contents. Fires may start when flames and sparks are not adequately contained. If sprinkler systems are set off due to smoke or fire, then water from sprinklers cause severe damage to the contents of the building. Proper venting of the fumes is vital to protect the occupants from toxic fumes and alleviate odors. Typical repairs can take from 2 to 5 days, and range in cost from $5,000 for W2 repairs to $10,000-$15,000 for C2 to C4 repairs.

BOLT REPAIR PROCEDURES

Testing of Bolted Connections

The bolted connections being installed to repair steel moment frame buildings in Los Angeles were designed and tested through a joint industry study by ICF Kaiser Engineers and Lehigh University, is now called the "Kaiser/Lehigh Bolted Connection". The design is the result of an intensive collaborative effort between industry experts and university researches. Testing of the bolted rigid connections was completed in September of 1995 at the National Science Foundation Center for Advance Technology for Large Structure Systems Laboratory, Lehigh University.

A total of eight full scale connection tests were performed in two phases. In phase 1, four tests on light beam-column subassemblies were conducted using W12x65 columns and W16x40 beams, similar to Anderson's welded connection test (Anderson and Linderman 1991). In Phase 2, four tests on heavy beam-column subassemblies were conducted using W14x426 columns and W36x150 beams, similar to Englehardt's welded connection test (Engelhardt and Sabol 1994).

Two types of bolted connection devices were developed and used. The first type is called a "Haunched Bracket (Fig. 1 left), and the second is called a "Pipe Bracket" (Fig. 1 right).These brackets were place in four specimens as summarized below:

HH-1 Specimen:	Haunched brackets on top and bottom flanges
PP-1 Specimen:	Pipe Brackets on top and bottom flanges
WH-1 Specimen:	Welded top flange and haunch bracket on the bottom flange
WP-1 Specimen:	Welded top flange and pipe bracket on the bottom flange

The Haunched bracket is a very stiff "seat angle" constructed from thick high strength plates, ASTM A572, Grade 50, shop welded together and bolted to the beam and column using large diameter, ASTM A490, high strength bolts. The pipe bracket utilizes double extra strong pipes (ASTM A500, Grade C) welded to high strength plates (ASTM A572, Grade 50). The beam flange is bolted to the plate and columns flanges are connected to the plate through bolts in the pipes. Details of the Phase 1 and 2 testing are described in greater detail by Kasai and Bleiman (1995 and 1996).

The results of testing verified that the Kaiser/Lehigh Bolted Connections provide rigid moment frame connections with high ductility and without changing the elastic stiffness of the joint. Plastic rotational ductility, measured at the hinge, varied from 0.05 to 0.08 radians. The maximum story drifts were between 4% and 6% for the beam-column subassembly. In all cases plastic hinging through ductile yielding occurred in the beam while the connections remained largely elastic. In no cases did any bolts, plates, or welds fail in the bolted connections. The hysteretic curves for phases 1 and 2 are shown in figures 2 and 3.

Field Installation of Bolted Connections

The Kaiser/Lehigh Bolted Connection arrives as a finished product to the building site. No welding is required to install the bracket. The brackets weigh between 200 and 300 pounds. They are lifted into place with lightweight lifting platforms more commonly used to install ductwork in ceiling spaces. These lifting platforms have a narrow base and can be wheeled through doorways and down narrow aisles. The brackets come to the field with all the holes pre-drilled in the bracket. The holes in the column flange are drilled in the field using a lightweight drill which clamps itself to the column with its own magnetic base. The column flange holes are up to 1 3/4" in diameter. However, because the drill cores a round slug out of the metal rather than chipping out the metal like a traditional drill, each hole can be drilled in under 5 minutes through steel flanges up to 3 inches thick. The column flange bolts act in tension so the column flange holes are standard holes, oversized from the nominal bolt diameter by 1/8".

The bracket is then lifted into place and several column bolts are set to support the weight of the bracket. Using the bracket as a template the beam flange holes are drilled into the beam in the overhead position, again using lightweight magnetic based drills. The beam flange bolts are bearing bolts so the holes for the beam bolts are less than 1/16" oversized and are tightened in with load indicator washer.

The column bolts require a special tightening procedure using highly accurate ultrasound technology to verify bolt force. The action of the bracket depends, in part, on the pretension force in the column flange. In the laboratory, the bracket performance was unaffected by bolts that were below AISC preload specifications by as much as ten percent. However, traditional ways to verify bolt pretension load: the "turn of the nut" method, load indicator washers and calibrated torque wrenches are not accurate to 10% of the load. In the laboratory we found the "turn of the nut method" can results in pretension forces in the bolts far below AISC specifications. This may be due to the unusually large A490 bolts used in the connections. Load indicator washer are not reliable for large diameter bolts closely spaced together. Calibrated torque wrenches require a ten fold multiplier to work for the large diameter A490 bolts, and slight errors are magnified by high multipliers.

Bolt force measuring devices that use ultrasound technology have been used in the manufacturing industry for many years, including aerospace and engine manufacturers. They work by sending a microwave pulse down the length of the bolt, measuring its un-stretched length with high accuracy. As the bolt is tightened, the bolt force can be read directly from a hand held device which uses the microwaves to measure the change in reference length, converting the change in length to force readings automatically. The column flange bolts are tightened twice. First they are all tightened 50%, then they are all tensioned fully to 100% of load. The tightening is sequenced to compensate for the changes in bolts force that happens as one bolts is tightened close to another. Once the column bolts are tightened the bracket is fully installed and ready for re-fireproofing.

In occupied areas bolting requires only modest protection to prevent injury to the finishes. Since no welding takes place, only minor precautions are necessary to prevent grease and dirt from spoiling finishes. The drill process produces very little odor or heat. Typically, a bracket can be installed overnight in under eight hours, with minimum impact on the use of the space in occupied areas. Brackets can be installed for $2,000-$3000 per connection.

The Kaiser/Lehigh Bolted Connection can be used to replace damaged welds by simply installing the bracket in lieu of repairing the connection. When column flanges are damaged the bolted brackets can still be used to advantage. The column flange will still require welded repair, which consists of filling in the crack or divot with new weld material. Instead of rewelding the beam flange, a bracket is installed to provide the rigid flange connection. The bracket is simpler to install than a new full penetration flange weld. In addition, unlike a new flange weld, the bracket has does not put tension stresses on the face of the column flange built from weldments.

Conclusion

Bolting is an economically viable alternative to welded repair of damaged moment frame connections that addresses the problems inherent in welded repairs describe above, namely the convenience, cost and quality. It is apparent, that by avoiding the use of torches in occupied areas the bolting process poses fewer risks to the contents of the building. Bolting also differs from welding in that a weld is a crafted product, whose quality is highly dependent on the skill of the welder. However much inspection and however much thought is given to the welding procedure specifications, in the end the quality of the weld is completely dependent on the ability of the operator to perform dozens of passes of fused weldment to form a single weld without excessive porosity, voids or stress concentrations. In bolted connections all of the materials are fabricated or manufactured under controlled conditions in factories or fabrication shops. The installation of the bolted connection is routine and the quality of the connection is not dependent on the skill of the installer. Full scale testing has shown, bolted connections out perform conventional welded connections for plastic rotational ductility.

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Table 1: Connection Damage Classification Table

TYPE	LOCATION	DESCRIPTION
G1	GIRDER	Top or bottom flange fracture in the heat affected zone
G2	GIRDER	Top or bottom flange fracture outside heat affected zone
G3	GIRDER	Fracture of the web
C1	COLUMN	Incipient flange crack (detectable by UT)
C2	COLUMN	Flange tear-out or divot
C3	COLUMN	Full or partial flange crack
C4	COLUMN	Full or partial flange crack extends into web
C5	COLUMN	Lamellar flange tearing
C6	COLUMN	Fully severed column
W1	C/P WELD	Root indication, thickness > 3/16" or $t_f/4$: width $>b_f/4$
W2	C/P WELD	Crack through weld metal thickness
W3	C/P WELD	Fracture at Girder Interface
W4	C/P WELD	Fracture at column interface
W5	C/P WELD	UT detectable indication - AWS non rejectable

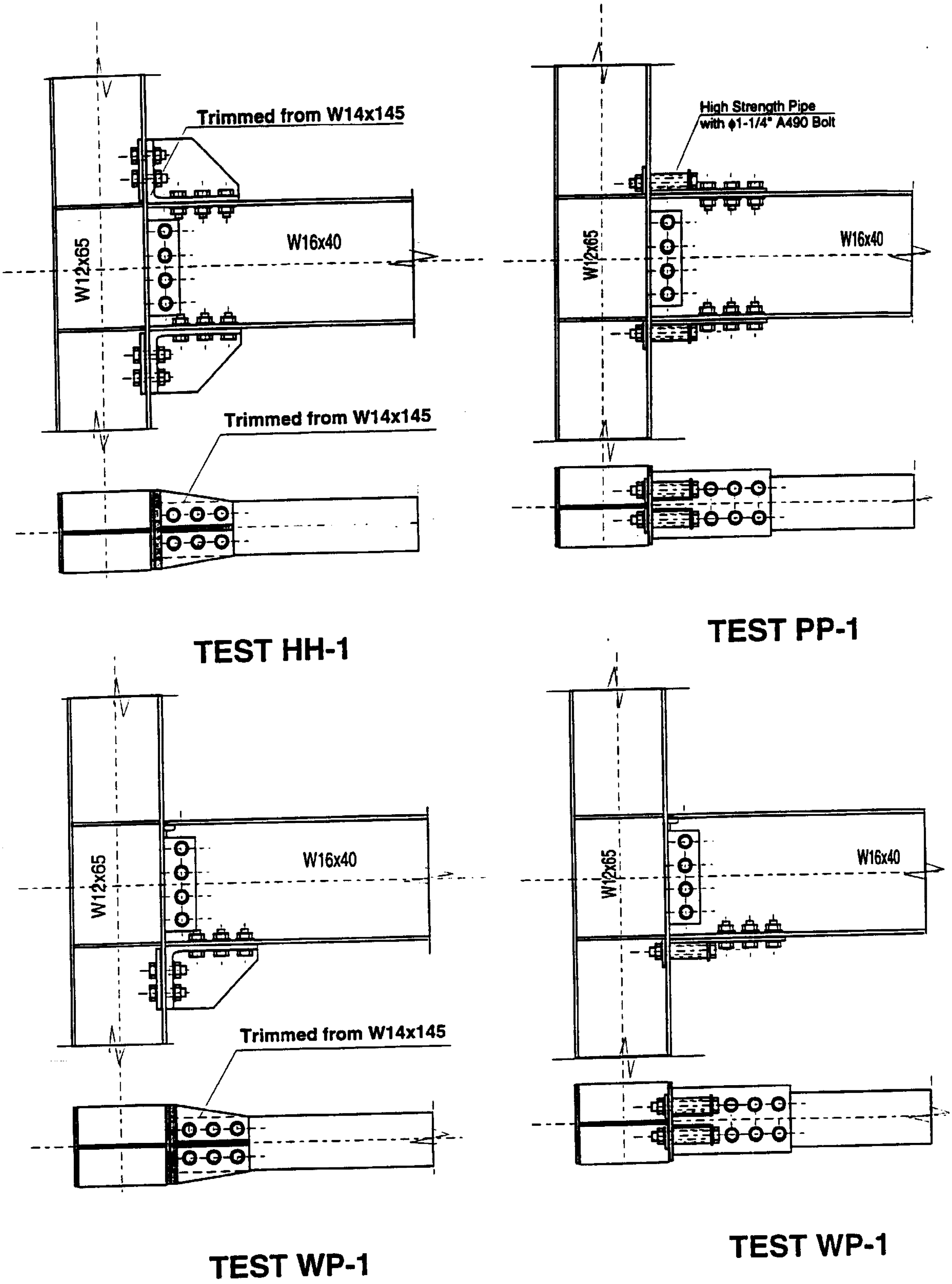

Fig. 1 Phase 1 Test Specimens.

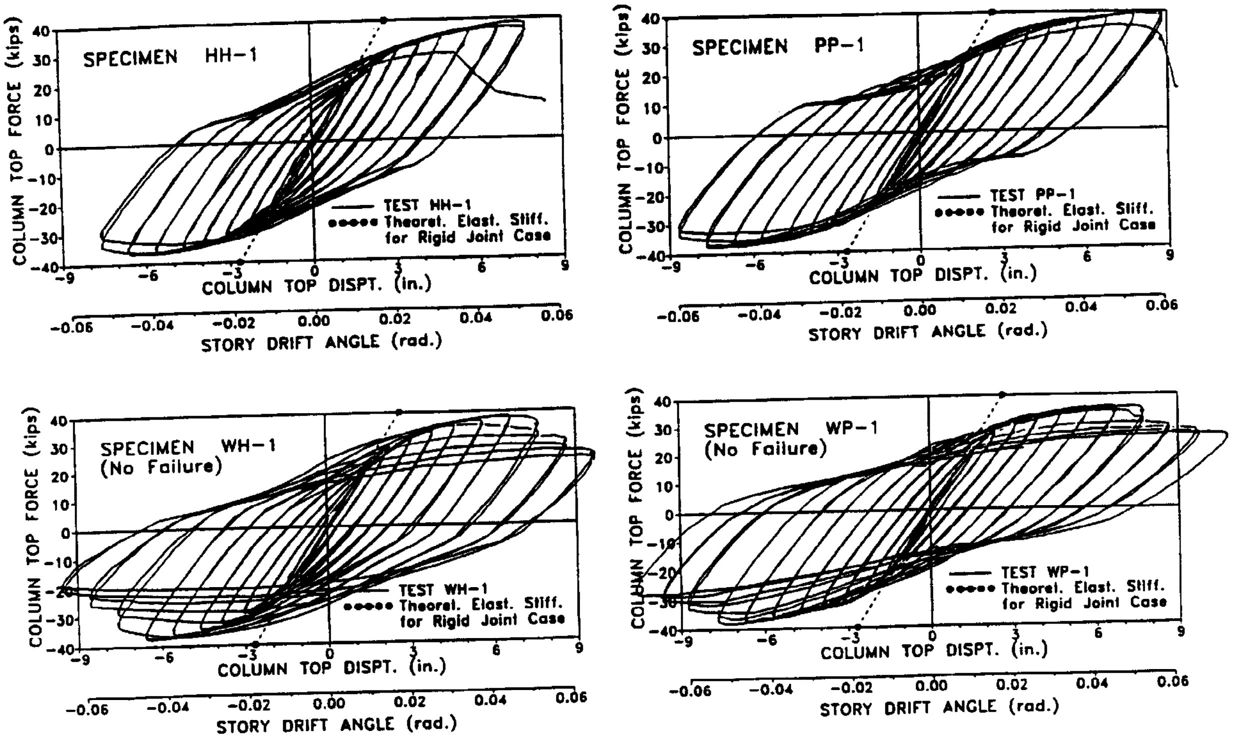

Fig.2 (a) Column Top Defection vs. Column Top Force Relationship for Phase 1 Tests.

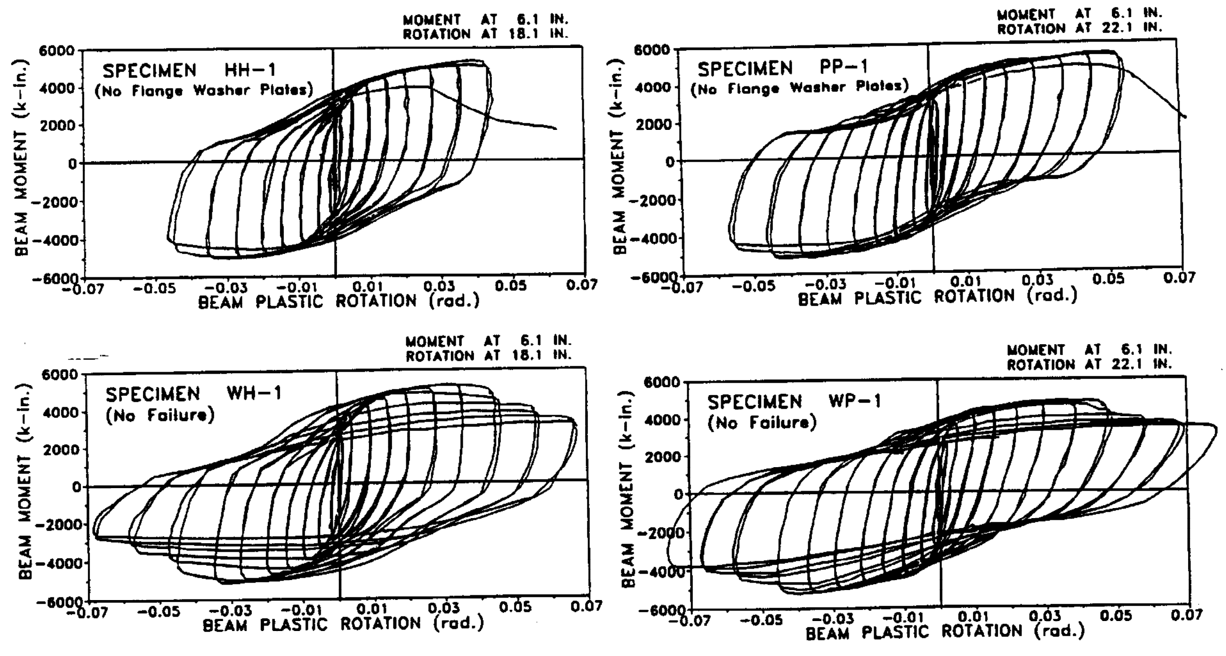

Fig.2 (b) Beam Plastic Rotation vs. Beam End Moment Relationship for Phase 1 Tests.

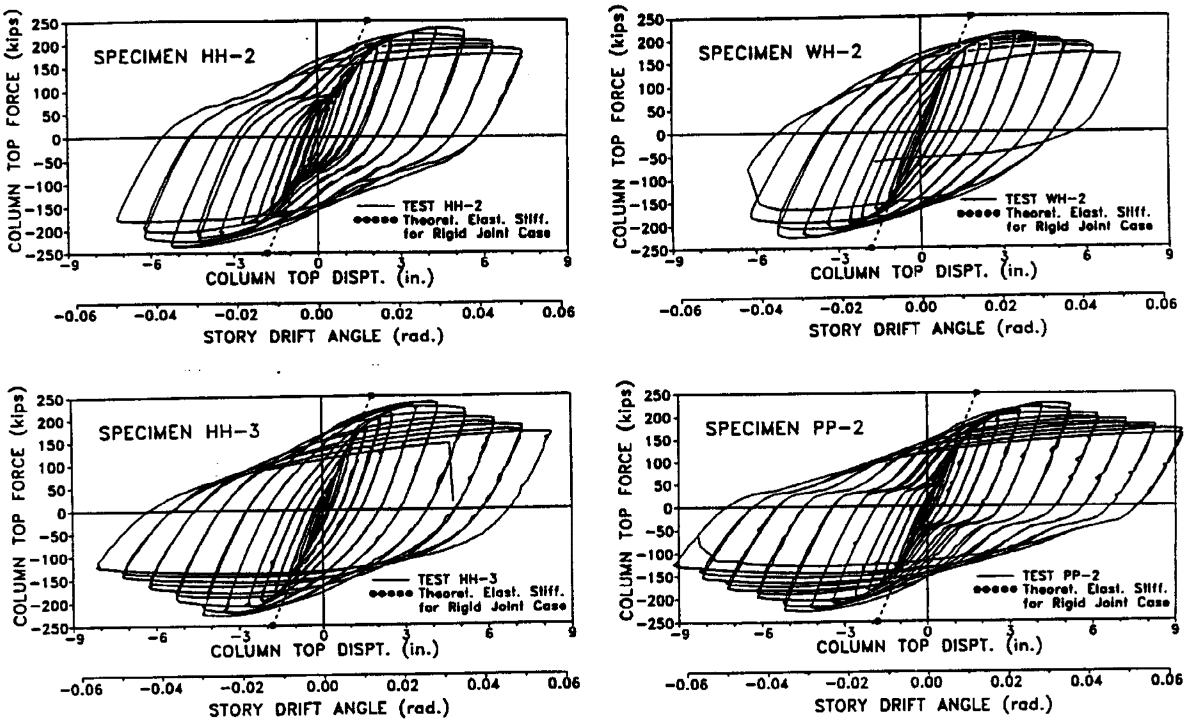

Fig.3 (a) Column Top Defection vs. Column Top Force Relationship for Phase 2 Tests.

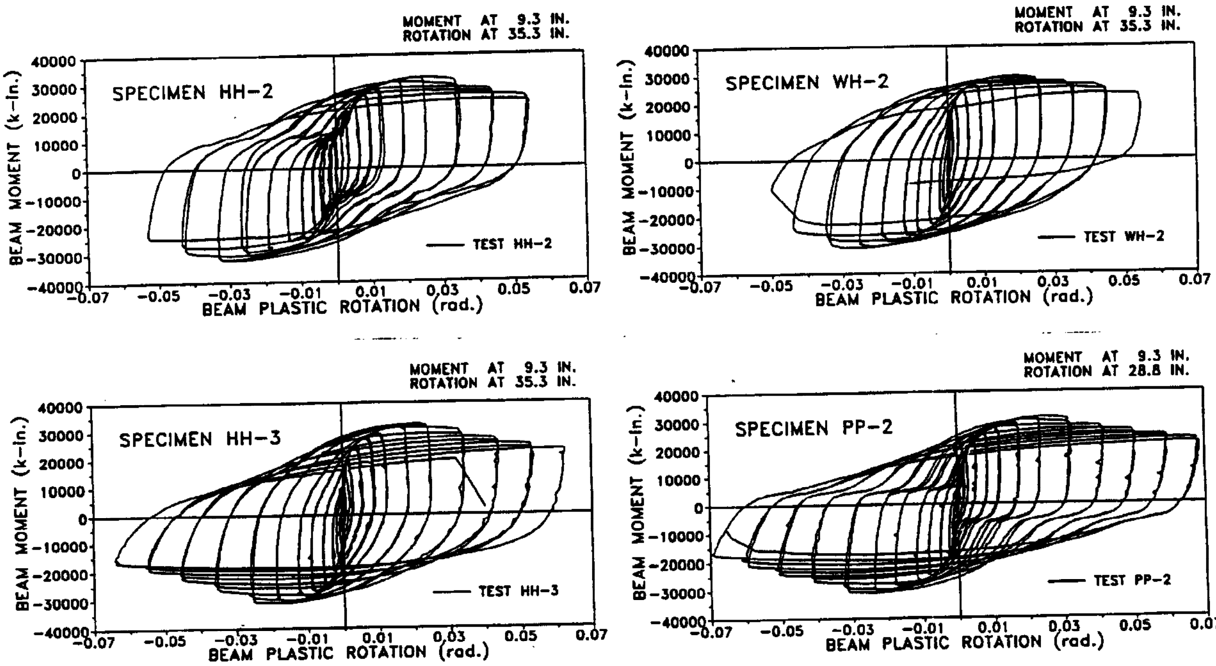

Fig.3 (b) Beam Plastic Rotation vs. Beam End Moment Relationship for Phase 2 Tests.