Reconnaissance Report on Damage to Steel Building Structures Observed from the 1995 Hyogoken-Nanbu (Hanshin/Awaji) Earthquake

Abriged English Edition

May 1995

Steel Committee of Kinki Branch
the Architectural Institute of Japan (AIJ)

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Translated by

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Translator's Note:

This is an abridged English translation of "Reconnaissance Report on Damage to Steel Building Structures Observed from the 1995 Hyogoken-Nanbu (Hanshin/Awaji) Earthquake", which was originally published in Japanese in May 1995. This report was prepared by the ad-hoc team formed to investigate damage to steel buildings in the 1995 Hyogoken-Nanbu (Hanshin/Awaji) earthquake. The team consisted of selected members of the Steel Committee of Kinki Branch of the Architectural Institute of Japan. Professor Kazuo Inoue of Osaka University was the chairman of the Steel Committee and also served as leader of the investigation team.

This translation was made by Masayoshi Nakashima who served as technical secretary of the team. The translator wishes to acknowledge the assistance, comments, and suggestions provided by Prof. Michel Bruneau of University of Ottawa, who stays at Kyoto Univ. in his sabbatical, in conducting this translation.

The Steel Committee of Kinki Branch of the AIJ is planning to publish the unabridged English edition of the report within a few months. For availability of the unabridge English edition, please contact Masayoshi Nakashima after August 1, 1995 at:

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Reconnaissance Report on Damage to Steel Building Structures
Observed from the 1995 Hyogoken-Nanbu (Hanshin/Awaji) Earthquake

Table of Contents in Original Japanese Version

1. INTRODUCTION

A team was formed to closely investigate the damage to steel building structures that suffered from the 1995 Hyogoken-Nanbu earthquakes. The membership of the team is listed in Table 2.1. This team worked from the beginning of February 1995 for approximately one month and surveyed the damage to approximately 1,000 steel buildings, located from the western boundary of the city of Kobe (Suma-ward) to the eastern limit of the Hyogo Prefecture as shown in Fig.2.1.

Old steel buildings built with light-gauged columns and beams were not investigated, although many of them suffered significant damage including collapse. This decision was made for the two major reasons: 1) much deterioration in materials was observed, and these buildings apparently had a seismic resistance much smaller than what is required nowadays; and 2) building having this structural type most likely will not be constructed in the future.

Investigation was made from the exterior and also from the inside as much as possible. The standard data sheet used for the investigation appears in Table 2.3.

The scope of the investigation was to record as much as possible the extent and types of damage to steel buildings, and no effort was made to analyze buildings to identify the causes of damage. Such an effort is underway by the Steel Committee of the Architectural Institute of Japan, but it will naturally take more time before conclusive statements are made possible.

2. ORGANIZATION AND PROCEDURE FOR DAMAGE INVESTIGATION

2.1 Organization of Investigation

The membership of the special investigation team is listed in Table 2.1. The team was chaired by Kazuo Inoue of Osaka University, and Isao Kozu of Osaka Institute of Technology, Mototsugu Tabuchi of Kobe University, and Masayoshi Nakashima of Kyoto University, served as technical secretaries.

2.2 Areas of Investigation

The area covered by this investigation, hatched in Fig.2.1, is the same as that surveyed by the AIJ preliminary reconnaissance team but without Awaji Island. Details on the preliminary reconnaissance are provided in "The Preliminary Reconnaissance Report of the 1995 Hyogoken-Nanbu Earthquake", published by the Architectural Institute of Japan in April 1995.

2.3 Procedure for Investigation and Judgment of Damage Level

Approximately one month (starting February 10) was spent for this investigation. For each building examined, the investigator filled the form shown in Table 2.3 and took a few photos showing notable damage. The following general guideline was adopted to judge the damage level.

Minor Damage: No damage to major vertical force supporting members like columns and beams. Minor cracking and spalling of exterior finishes and/or buckling of rod or flat bar braces. No or nearly no permanent lateral deformations.

Moderate Damage: Buckling and rupture of bracing members and plastification of columns and beams. Small residual lateral deformations, approximately smaller than 1/100. Any damage not categorized as Minor, Severe or Collapse.
Severe Damage: Serious damage to columns, beams, and connections to an extent considered difficult to repair. Significant residual lateral deformations, more than 1/100.
Collapse: Collapse of the entire building or a story of the building.

3. GROUND MOTION AND DESIGN EARTHQUAKE FORCES

3.1 Previous Earthquakes in Kansai Region of Japan

Fig.3.1 shows the distribution of epicenters of previous earthquakes whose magnitude were 6 or greater. The ones shaded caused serious damage in Kansai Region. Table 3.1 is a list of these previous large earthquakes.

3.2 Design Earthquake Forces

The seismic design code adopted in 1981 provides two levels of design earthquake forces. The first level is for small to medium earthquakes whose maximum acceleration ranges approximately from 80 to 100 cm/sec^2; structures are required to remain elastic during such earthquakes, ensuring their serviceability. The second level is for large earthquakes whose maximum acceleration ranges approximately from 300 to 400 cm/sec^2, for which some degree of damage to structures is permitted. For buildings taller than 60 m, a special design procedure including dynamic response analysis is mandated. In the analysis, ground motion records with their maximum accelerations normalized to 200 to 300 cm/sec^2 are used for elastic behavior (to ensure serviceability), while enlarged ground motion records scaled to the maximum accelerations of 300 to 500 cm/sec^2 are used for inelastic behavior (to check ultimate resistance). These maximum accelerations approximately correspond to the maximum velocities of 20 to 25 cm/sec and 40 to 50 cm/sec respectively. The significant difference in deisgn force level for serviceability between low to medium rise buildings (80 to 100 cm/sec^2 in the maximum acceleration) and tall buildings (200 to 300 cm/sec^2 in the maximum acceleration) should be noted. Fig.3.2 shows these levels of earthquake design forces, for ordinary buildings (left) versus for tall buildings (right).

3.3 Recorded Ground Motions and Their Response Spectra

Fig.3.3 shows some maximum accelerations recorded from the 1995 Hyogoken-Nanbu earthquake. Among the selected five sets of records, digitized data have been available for Kobe University (NS:270 cm/sec^2, EW:305 cm/sec^2) and the Japan Meteorological Agency at Kobe (NS: 818 cm/sec^2, EW: 617 cm/sec^2, UD (vertical): 332 cm/sec^2). Fig.3.4 shows the recorded accelerations obtained at the Japan Meteorological Agency of Kobe (simply designated as Kobe in Fig.3.5). For comparison, Fig.3.5 shows the acceleration and pseudo velocity response spectra (S_A and S_V) for Kobe, Elcentro and Taft ground motion records.

4. OVERVIEW OF DAMAGE

4.1 Distribution and Occupancy of Buildings Investigated

A total of 988 steel buildings were examined, and 90 were rated as Collapsed, 332 as Severely damaged, 266 as Moderately damaged, and 300 as Minorly damaged. The ratios of these numbers are approximately 1 to 3 to 3 to 3. Table 4.1 shows the number of buildings with respect to the damage level (vertically, from top to bottom, collapse, severe, moderate, minor, and total) and the area (horizontally, from left to right, Suma, Nagata, Hyogo, Chuo, Nada, Higashinada, Ashiya, Nishinomiya, and total). Table 4.2 shows the number of buildings with respect to the number of stories (vertically, from top to bottom, 1 to ≥10 stories, unknown, and total). Table 4.3 shows the number of stories (vertically, same as Table 4.2) with respect to the damage level (horizontally, from left to right, collapse, severe, moderate, minor and total). Fig.4.1 is a graphical representation of Table 4.3. The numbers along the horizontal axis show the number of buildings, and the number of buildings rated as collapse, severe, moderate, and minor are identified by shading from the darkest to the lightest. Most of the collapsed buildings are 2 to 5 stories tall, and no building with seven stories or more collapsed. Table 4.4 shows the number of damaged buildings with respect to occupancy (vertically, from top to bottom, house, shop, office, hospital, school, industrial facility, warehouse, parking, others, and unknown). The horizontal distribution is identical to Table 4.1. The number of damaged buildings is the largest for houses, followed by shops and offices. Note that buildings used for multiple occpancy have been counted as many time as needed, while the numbers in brackets are the totals for the dominant occupancy and match the true total number of damaged buildings.

4.2 Structural Properties of Buildings Investigated

Buildings were classified as having one of three possible types of structural system, namely:

R-R: unbraced frames in two horizontal directions
R-B: unbraced frame in one horizontal direction and braced frame in the other direction
B-B: braced frames in two horizontal directions

Table 4.5 shows the number of buildings with respect to the structural type, with 388 instances rated as "unknown" (in many cases, it was difficult to identify the structural type, sometimes due to the entire collapse of the building, other times because the building was completely covered by its architectural finishes). The table shows that about 70 % of buildings whose structural type could be identified were unbraced in both directions (R-R). Table 4.6 shows the cross-sectional types used for columns (table on the left), for beams (table in the middle), and for braces (table on the right). For columns (table on the left), square-tube (almost exclusively cold-formed, which has been common practice in Japan for the past fifteen years), round-tube, wide-flange (H), wide-flange covered with exterior plates (*1), other built-up sections, and unknown situations are listed (from top to bottom). For beams (table in the middle), wide-flange (H), built-up, and unknown sections are listed from top to bottom, and for braces (table on the right), rod, angle, flat bar, round-tube, wide-flange, square-tube, channel, and unknown sections are listed (from top to bottom). For columns, wide-flange cross-sections are most widely used, followed by square-tube cross-sections. For beams, wide-flanges are used in most cases.

*Translater's Note (*1):*
In old practices prior to the introduction of cold-formed square-tubes in Japan, a pair of steel plates were often welded to the flanges of a wide-flange cross-section to form a tube (see FigT1). Such built-up column shapes were abandoned approximately twenty years ago.

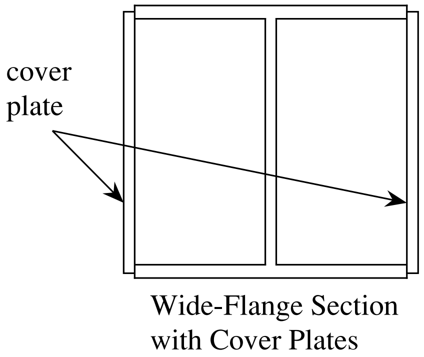


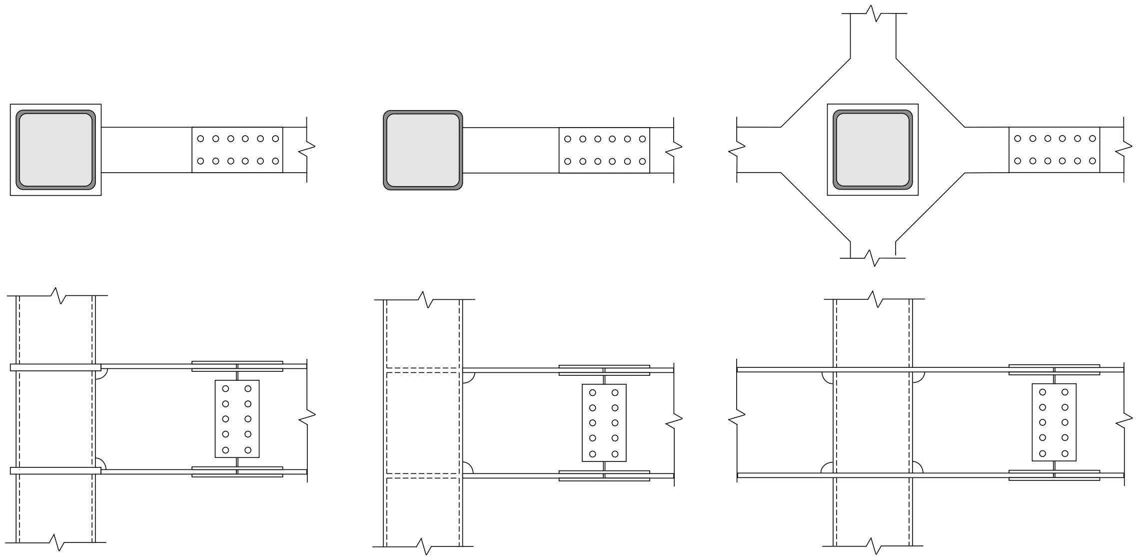
Fig.T1 Wide-Flange Section with Cover Plates

Table 4.7 shows types of connection details. From the top left table in the clockwise direction, data are presented for column-to-column splices, beam-to-beam splices, beam-to-column connections, brace connections, and column bases. For column-to-column splices, 186 were welded, and 19 bolted (514 unknown). For beam-to-beam splices, welding was used in 12 buildings, and bolting in 397 buildings (457 unknown). For beam-to-column connections, field welding was used in 40 buildings, and shop welding in 271. With respect to the diaphragm (stiffener plate) arrangement, “through-diaphragm” type (2*) was used in 144 buildings, “exterior diaphragm” in 6, and “interior diaphragm” in 8, while all of them combined with square-tube columns. Stiffener plates (called continuity plates in the U.S.) were used in 161 buildings having wide-flange (H) columns. For brace connections, welding was used in 43 buildings, bolting in 135 (283 unknown). For column bases, standard base plates were used in 270 buildings, concrete encased column bases in 70, and embedded column bases in 86 (569 unknown) (*3). What can be observed from these tables is:

* Column-to-column splices are mostly accomplised by welding.
* Beam-to-beam splices are almost always accomplished using high tension bolts.
* Through-diaphragms are used in most square-tube columns, and stiffener (continuity) plates are used in wide-flange columns.
* Based plate connetion to footing is most common.
* Braces were connected mostly by bolting, except for small rod and flat bar braces, which are generally welded.

*Translator's Note (*2):*

In Japan, following three types of detail are commonly used for beam-to-column connections when combined with sqaure-tube columns, namely: "through-diaphragm" type, "interior diaphragm" type, and "exterior diaphragm" type (see Fig.T2). Among those, the "through-diaphragm" type is most pupular as indicated in Table 4.7. In the through-diaphragm connection, a long square-tube is cut into three pieces: one used for the column of the lower story, one for the connection's panel zone (a short piece, often called a dice in Japan), and one for the column of the upper story. Two diaphragm plates are inserted between the three separated pieces and shop-welded all around. The "interior diaphragm" and "exterior diaphragm" types are self-explanatory.

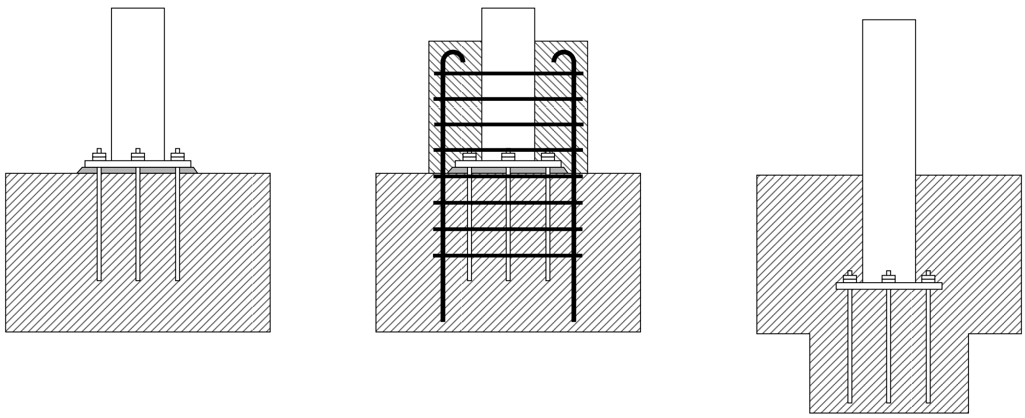


(a) through diaphragm (b) interior diaphragm (c) exterior diaphragm

Fig.T2 Types of Beam-to-Column Connections

*Translator's Note (*3):*

In Japan, three types of column base connection are commonly adopted (Fig.T3), that is: standard base plate connecction, concrete encased column base connection, and embedded column base connection.



(a) Base Plate Connection (b) Concrete Encased Column Base (c) Embedded Column Base

Fig.T3 Types of Column Base Connections

4.3 Structural Types and Damage

Table 4.8 shows the correlation between the structural type and damage level. The left column (enclosed by bold lines) shows the structural type (R-R, R-B, and B-B), with sub-divided as a function of column cross-section, namely: square-tube (cold-formed), wide-flange (H), and wide-flange with cover plates. The middle section of the table contains data on the damage level, reading as collapse, severe, moderate, and minor going from left to right. Totals area in the rightmost column. Fig.4.2 to 4.5 are nearly self-explanatory graphical presentations of Table 4.8. These show the number of damaged buildings with the four damage levels being proportional to the darkness of shading (black for collapse). The rectangular symbol having a horizontal bar in its middle is used to represent the wide-flange cross-section with cover plates. The following is observed.

* There exists no significant difference in damage levels with respect to the structural type (R-R, R-B,and B-B).
* Buildings with wide-flange columns have suffered more serious damage (as seen in Fig.4.5).

Table 4.9 shows the location of damage as a function of structural type. The left column (enclosed by bold lines) shows the structural type as done in Table 4.8. Right of the bold line are statistics (going left to right) for columns, beams, beam-to-column connections, braces, and column bases. For example, among buildings of the R-R type, 153 sustained column damage. Figs. 4.6 and 4.7 are the graphical presentations of Table 4.9. The horizontal axis shows the number of buildings that sustained damage to columns, beams, beam-to-column connections, braces, and column bases as expressed by different shading going from left to right. Major observation is as follows:

* In unbraced frames, columns suffered the most damage (in terms of the number of buildings), while in braced frame, braces are naturally the most frequently damaged structural element.
* In unbraced frames, damage to beam-to-column connections and column bases were the next most freuently damaged again in terms of the number of buildings for which this form of damage was observed. For columns in unbraced frames, damage to beam-to-column connections was most significant (in terms of the number of buildings) for unbraced frames having square-tube columns, and that to columns was most significant for unbraced frames having wide-flange columns.

Table 4.10 provides a more in depth look than Table 4.9, but for unbraced frames (R-R) only. It shows the damage level (vertically, from top to bottom: collapse, severe, moderate, minor, and total) with respect to the location of damage (horizontally, from left to right: column, beam, beam-to-column connection, and column base). Tables 4.11 and 4.12 are identical to Table 4.10, but for unbraced frames having square-tube columns, and for unbraced frames having wide-flange columns, respectively. Major observation is as follows:

* In unbraced frames, the damage level tends to be severe for buildings that sustained damage to their columns and beam-to-column connections.
* In unbraced frames having square-tube columns, the damage level is higher when beam-to-column connections are damaged. In unbraced frames having wide-flange columns, the damage level is higher when columns sustained damage. It is notable that damage to wide-flange columns consisted of plastification, excessive bending in the weak axis, and local buckling.

4.4 Damage to Structural members and Elements

From the observations described up to now, important damage locations that had caused serious damage to buildings were found to be columns, beam-to-column connections, and column bases for unbraced frames, and braces, columns, and column bases for braced frames. In this section, damage to beam-to-column connections, braces, and column bases is examined in more details.

4.4.1 Beam-to-Column Connections

Fig.4.8 is used to describe the location and type of damage to beam-to-column connections more accurately. In this sketch of a beam-to-column connection baving through-diaphragms (a most extensively used detail as shown in Table 4.7), five potential locations of damage are represented by hatched zones; these are, from top to bottom, column side of B-th floor's column bottom, panel side of B-th floor's column bottom, panel side of A-th floor's column top, and column side of A-th floor's column top, and beam side of B-th floor on the right.

Table 4.13 shows the number of buildings in which damage to beam-to-column connections was observed. The leftmost column (enclosed by bold lines) shows the structural types. The second group (enclosed by bold lines) reports all cases where fillet welding was used and is sub-divided into damage to beam side, column side, and panel side (going from left to right). The third group (enclosed by bold lines) is for cases where full penetration welding was used, also sub-divided into beam, column and panel sides. The fourth group similarly reports the cases for which welding type was unkown. This table shows very few cases of beam-to-column connection damage in braced frames. Table 4.14 is identical to Table

4.13, but for unbraced frames only. The leftmost column shows the cross-section type of the columns, divided into square-tube (cold-formed) and wide-flange (H). Major observation is as follows:

* When fillet welding was used, damage (meaning cracking and fracture in the regions specified in Fig.4.8) almost equally occurred in beam, column, and panel sides, whereas when full penetration welding was used, damage was observed to have occurred mostly on the beam side. However, when wide-flange columns were used, damage almost exclusively developed on the beam side, primarily because columns are commnly continuous through the connections in that case.

Tables 4.15 and 4.16 show the correlation between damage level and location of damage in beam-to-column connections for unbraced frames having square-tube columns and unbraced frames having wide-flange columns, respectively. The leftmost column shows the damage level (collapse, severe, moderate, minor, and total, from top to bottom). The second column presents the number of buildings, the third column the type of welding (fillet, full penetration, unknown welding going from left to right), and the fourth column location of beam-to-column connection damage (beam side, column side, and panel side, from left to right). Major observation is as follows:

* 59 unbraced frames having square-tube columns sustained damage to the beam-to-column connections. About 70% of those are rated as either collapse or severe in their damage level.
* 29 cases with fillet welding were surveyed, half of which collapsed.
* 17 cases with full penetration welding were in entry, out of which 3 collapsed.
* A majority of those that sustained damage on the column or panel side collapsed or suffered serious damage.
* Among those that sustained damage to the beam side, only 4 collapsed.

Table 4.17 shows the correlation between the number of building stories as a function of the damage to beam-to-column connections. The leftmost column is the number of stories, the first row being for buildings of 5 stories or smaller, and the second row being for buildings taller than 5 stories. The description for the second to fourth columns are identical to that presented for Tables 4.13 and 4.14. Except for a single case, damage to fillet welded connections was observed only in buildings of 5 stories or smaller.

4.4.2 Braces

Table 4.18 shows the types of braced frames with respect to the damage level. The convention adopted is R-B for frames braced in one direction only, B-B for frames braced in both directions, diagonally arranged bracing (X), and chevron-type bracing (K). As usual, the numbers of buildings rated as collapse, severe, moderate, and minor are tabulated from left to right, with totals in the last column. Fig.4.9 shows the damage distribution for the R-B and B-B cases, revealing that there is no significant difference in the extent of damage in these two building types. Damage level was generally higher when diagonal bracing (X) was used as compared to chevron-type bracing (K). It is notable that smaller cross-sections were used in diagonal bracing, and that buildings with chevron-type bracing were generally larger (both in terms of space and height).

Table 4.19 shows the correlation between the cross-section used for braces and the damage level. The first column lists the cross-section, from top to bottom: rods, angles, flat bars, sub-total of the three (classified as small cross-sections), round-tubes, wide-flanges, square-tubes, channels, sub-total of the four (classified as large cross-sections), unknown, and grand total. The second column shows the damage level, with collapse, severe, moderate, and minor, and total from left to right. Fig.4.10 shows the damage level distribution when braces having small cross-sections were used (top) and when braces having large cross-sections were used (bottom). Damage was clearly more severe in the smaller cross-sections. Although this was not quantified, the size of cross-sections used is strongly correlated with the age of buildings, with the small cross-sections used more frequently in older buildings.

4.4.3 Column Bases

Among the 218 buildings for which footing system was identified, 127 used standard column base plates. Out of these, 101 sustained serious permanent deformation or collapse. Table 4.20 shows the damage level with respect to the type of damage in those base plate connections. The first column shows the damage level (collapse, severe, moderate, minor, and total, going from top to bottom), the second column lists cases of fracture of the anchor bolts, the third column welding failure of base plates, and the fourth column significant deformation to base plates. Fig.4.11 is a graphical presentation of Table 4.20. The horizontal axis shows the number of buildings from collapse (darkest shading) to minor (white).

5. DAMAGE TO STRUCTURAL COMPONENTS

5.1 Beam-to-Column Connections

Two types of failure pattern were observed: (i) cracking and fracture accompanied by significant plastification of beams, and (ii) cracking and fracture at welds without any sign of plastification.

Cracking and fracture with significant plastification was observed mostly in unbraced frames having square-tube columns. In the majority of these cases, no sign of plastification was observed in the joining columns. Most of the fractures occurred in the lower flange of the beams, and the beams exhibited clear signs of plastification as well as local buckling in many cases. Typical locations of fractures are shown in Fig.5.1. Fracture initiated either from: (i) the corner of a weld access hole, (ii) near a run-off tab or a weld toe, (iii) heat affected zones on the beam flange side, and (iv) heat affected zones on the diaphragm side. In two cases, fracture progressed into the column flanges. Such fractures occurred in a brittle manner after the flange experienced significant yielding. In some cases, ductile failure involving partial necking was observed. Such brittle failure after significant yielding was attributed to material's hardening by plastic hysteresis and is not regarded as equivalent to brittle failures occuring under low stress. Most of buildings that sustained such cracking and fracture did not exhibit large permanent deformations or significant damage to their exterior and interior finishes.

On the contrary, most of the beam-to-column connections that cracked and fractured without signs of plastification of the joining members were fillet welded. These fillet weld apparently were of a size insufficient to develop the capacity of the connected members, and many of the buildings having such connections suffered serious damage.

Damage to beam-to-column connections when wide-flange beams were used was seen in older buildings. Many of those connections had no stiffening plates (continuity plates), had their web connected only by bolting, or were connected only to a seat angle. In all of these cases, little plastification was observed in the framing beams and columns, and damage was localized to the connection.

SELECTED PHOTOS (5.1)

5.1.1 Damage to Square-Tube Column to Wide-Flange Beam Connections

<u>Page 32 (No.611):</u>

9 story, 4 by 3 bays, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB1.1.1 fracture of bottom flange after full plastification
Photo CB1.1.2 fracture of bottom flange after full plastification
Photo CB1.1.3 fracture of bottom flange after local buckling
Photo CB1.1.4 fracture of bottom flange initiated at near run-off tab

<u>Page 33 (No.671):</u>

14 story, cold-formed square-tube columns, wide-flange beams through-diaphragms, field welding at beam ends.

Photo CB1.2.1 fracture of bottom flange
Photo CB1.2.2 fracture at a haunched part of bottom flange
Photo CB1.2.3 view from bottom of beam in Photo CB.1.2.2
Photo CB1.2.4 fracture initiated at a backup plate and extended into the diaphragm
Photo CB1.2.5 local buckling and fracture of bottom flange
Photo CB1.2.6 fracture of web's high tension bolts

<u>Page 34 (No.673):</u>

8 story, 3 by 2 bays, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB1.3.1 fracture of bottom flange in the heat affected zone on panel side
Photo CB1.3.2 fracture of bottom flange in the heat affected zone on panel side
Photo CB1.3.3 local buckling and fracture of bottom flange
Photo CB1.3.4 local buckling at column top
Photo CB1.3.5 local buckling at column base
Photo CB1.3.6 column locally buckled at its top

<u>Page 35 (No.672):</u>

8 story, 5 by 2 bays, cold-formed square-tube columns, wide-flange beams, through-diaphragms, field welding at beam ends.

Photo CB1.4.1 beam exhibiting lateral buckling
Photo CB1.4.2 fracture of high tension bolts connecting beam web
Photo CB1.4.3 fracture of bottom flange after full plastification
Photo CB1.4.4 fracture initiated near the run-off tab

Page 36 (No.676):

5 story, 4 by 2 bays, unbraced frames, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB1.6.1 local buckling and fracture of beam's bottom flange
Photo CB1.6.2 fracture of bottom flange after plastification
Photo CB1.6.3 fracture of bottom flange and part of the web
Photo CB1.6.4 ductile fracture of bottom flange
Photo CB1.6.5 fracture of top and bottom flanges and part of the web

Page 37 (No.657):

7 story, 2 by 2 bays, unbraced frame, cold-formed square-tube columns, wide-flange beams, through diaphgrams, shop welding at beam ends.

Photo CB1.7.1 fracture of bottom flange and part of the web
Photo CB1.7.2 chevron pattern of tears along fractured surface
Photo CB1.7.3 local buckling of bottom flange and fracture of diaphragm
Photo CB1.7.4 fracture in heat affected zone of diaphragm
Photo CB1.7.5 fracture extending from diaphragm into column

Page 38 (No.680):

14 story, unbraced frame, cold-formed square-tube column, wide-flange beams, through-diaphragms, field welding at beam ends.

Photo CB1.8.1 fracture of lower flange initiated at toe of weld access hole
Photo CB1.8.2 crack in heat affected zone of lower flange

Page 38 (No.677):

7 story, 7 by 4 bays, unbraced frame, cold-formed square-tube columns, wide-flange beams, through diaphgrams, shop welding at beam ends.

Photo CB1.9.1 local buckling and fracture of bottom flange
Photo CB1.9.2 different view of details in Photo CB1.9.1

Page 39 (No.678):

7 story, 2 by 1 bays, unbraced frame, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB1.10.1 fracture of bottom flange
Photo CB1.10.2 closer view of fracture of Photo CB1.10.1
Photo CB1.10.3 fracture of bottom flange

Page 39 (No.278):

7 story, 3 by 1 bays, cold-formed square-tube columns, wide-flange beams, through diaphgrams, shop welding at beam ends.

Photo CB1.11.1 fracture of diaphragms and crack extending to web
Photo CB1.11.2 fracture through welds

Page 40 (No.687):

8 story, 7 by 4 bays, unbraced frame, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB1.12.1 fracture of bottom flange
Photo CB1.12.2 crack at toe of weld access hole
Photo CB1.12.3 brittle fracture of diaphragm
Photo CB1.12.4 crack at corner of column welds
Photo CB1.12.5 fracture of welds at beam-to-beam filed splices

Page 41 (No.688):

8 story, 6 by 5 bays, unbraced frames, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB1.13.1 brittle fracture of a bottom flange
Photo CB1.13.2 fracture of bottom flange
Photo CB1.13.3 crack running vertically on column flange
Photo CB1.13.4 crack on column flange
Photo CB1.13.5 vertical crack on column flange

Page 42 (No.421):

6 story, 2 by 1 bays, unbraced frames, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB1.14.1 fracture at welds of bottom flange
Photo CB1.14.2 local buckling of column
Photo CB1.14.3 crack at corner of column

Page 43 (No.411):

3 story, 7 by 3 bays, unbraced frame, round-tube columns, wide-flange beams, shop welding at beam ends.

Photo CB1.15.1 fracture of bottom flange
Photo CB1.15.2 fracture of bottom flange and web

Page 44 (No.218):

2 story, 3 by 1 bays, unbraced frame, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB.1.17.3 repair of crack by rewelding

Page 45 (No.334):

3 story, 2 by 1 bays, unbraced frame, cold-formed square-tube columns.

Photo CB1.19.1 partial collapse as a result of beam-to-column conneton failure
Photo CB1.19.2 fracture of beam-to-column fillet weld connections

Page 46 (No.004):

6 story, 4 by 2 bays, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB1.21.1 fracture surface at column corner (internal buck-up bars visible)

Photo CB1.21.2 fracture of welded column (fracture through welds, partially extending into base metal)

Photo CB1.21.3 through-diaphragm visible after rupture of welded connection to column

Page 47 (No.061):

4 story, 3 by 2 bays, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB1.22.1 beam-to-column joint after rupture of welded connection to column

Photo CB1.22.2 closer view of joint in Photo CB1.22.1

Photo CB1.22.3 column after rupture of welded connection to through-diaphragm

Page 48 (No.284):

7 story, 2 by 1 bays, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB.1.23.1 fracture of column top on panel side

Page 49 (No.094):

3 story, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends

Photo CB1.25.1 leaning column after fracture of column top (fillet welded)

Page 49 (No.002):

5 story, cold-formed square-tube column, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB.1.26.1 fractured diaphragm (fillet welded)

Page 50 (No.280):

5 story, cold-formed square-tube column, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB1.27.1 fracture at column top (fillet welded)

Photo CB1.27.2 fracture at beam end (fillet welded)

Page 51 (No.378):

4 story, 1 by 1 bays, unbraced frame, cold-formed square-tube column, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB1.30.1 fracture at column top and separation of column from panel

Photo CB1.30.2 panel separated from column

Page 53 (No.053):

4 story, 2 by 1 bays, unbraced frame, cold-formed square-tube column, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CB.1.34.1 first story collapse

Photo CB1.34.2 separation of diaphragm from panel (fillet welded)

5.1.2 Damage to Wide-Flange Column to Wide-Flange Beam Connections

Page 57 (No.319):

4 story, wide-flange columns, wide-flange beams, shop welding at beam ends.

Photo CB2.1.1 through fracture at beam ends (fillet welded)

Page 57 (No.322):

6 story, 2 by 2 bays, wide-flange columns, wide-flange beams, shop welding at beam ends.

Photo CB2.2.1 local buckling of column and through fracture at beam end.

Page 58 (No.116):

3 story, 4 by 1 bays, wide-flange columns, wide-flange beams.

Photo CB2.4.1 fracture of shear plate and buckling of seat angle

Page 59 (No.013):

3 story, 2 by 1 bays, wide-flange columns, wide-flange beams, shop welding at beam ends.

Photo CB2.5.1 collapse by flexure about column's weak axis

Photo CB2.5.2 severely distorted column

Page 61 (No.254):

3 story, 3 by 1 bays, wide-flange columns, wide-flange beams, flat bar braces in weak axis, shop welding at beam ends.

Photo CB2.9.1 panel's shear distortion

Photo CB2.9.2 local buckling of column flange

Photo CB2.9.3 bending of corner column (including slight torsion)

Page 63 (No.087):

parking, 3 by 2 bays, wide-flange columns, wide-flange beams, rod braces.

Photo CB2.13.1 crushing of column web (no continuity plate)

Photo CB2.13.2 plastification and local buckling of column flange

5.3 Damage to Wide-Flange Column with Cover Plates to Wide-Flange Beam Connections

Page 65 (No.414):

4 story, 6 by 1 bays, wide-flange columns with cover plates, wide-flange beams.

Photo CB3.2.1 fracture at beam ends (fillet welded as seen from footprint)

Page 67 (No.395):

4 story, 4 by 1 bays, unbraced frames, wide-flange columns with cover plates, wide-flange beams, shop welding at beam ends.

Photo CB3.6.1 separation of cover plate from wide-flange column

Photo CB3.6.2 cracking of stiffening plate

5.2 Beams

Many beams sustained damage to their beam-to-column connections. Such damage was described in 5.1, and therefore, the number of damage cases presented in this section is limited.

SELECTED PHOTOS (5.2)

5.2.1 Damage to Wide-Flange Beams

Page 69 (No.987):

6 story, 22 by 6 bays, unbraced frames, cold-formed square-tube columns, wide-flange beams.

Photo BM1.2.1 local buckling of bottom flange

Page 70 (No.405):

3 story, 4 by 1 bays, unbraced frame, wide-flange column with stiffening plate, wide-flange beams, shop welding at beam ends.

Photo BM1.3.1 plastification and cracking of splice plate

Photo BM1.3.2 fracture of high strength bolts used for splice

Page 71 (No.679):

12, 9 by 4 bays, braces arranged eccentrically as shown in the inset figure. Portion of beam from column flange to brace connection sustained damage.

Photo BM1.4.1 out-of-plane buckling of beam web

Photo BM1.4.2 fracture of beam top flange and part of web

5.3 Columns

Damage occurred in numerous columns, most of which sustained it in the vicinity of beam-to-column connections, as have been presented in Section 5.1. Column damage introduced in this section includes plastifiation near column ends, excessive bending, and local buckling. In particular, many wide-flange columns sustained excessive bending in their weak axis.

SELECTED PHOTOS (5.3)

5.3.1 Damage to Square-Tube Columns

Page 73 (No.373):

3 story, 2 by 2 bays, unbraced frame, cold-formed square-tube columns, through-diaphragms, shop welding at beam ends.

Photo CL1.2.1 local buckling at column bottom

Page 74 (No.674):

8 story, 1 by 1 bays, unbraced frames, cold-formed square-tube columns, wide-flange columns, through-diaphragms, shop welding at beam ends. First story columns were filled with concrete up to a height of 1000 mm from their base. Fracture ran horizontally on the column at the level where the concrete fill terminated.

Photo CL1.3.1 column repaired using wide-flange studs

Photo CL1.3.3 fractured column, repaired using plates

Photo CL1.3.4 fracture running vertically

Page 75 (No.275):

8 story, braced frame, cold-formed square-tube columns, wide-flange columns. The entire section of a corner column (as shown in the inset figure) was fractured near its column base.

Photo CL1.4.1 flexural buckling of column

Photo CL1.4.2 fracture of column

Photo CL1.4.3 column exposed after removal of base concrete

Photo CL1.4.4 fracture of high strength bolts

5.3.2 Damage to Welded Column Splices

Page 77 (No.097):

5 story, 2 by1 bays, unbraced frame, cold-formed square-tube columns.

Photo CL2.1.1 fracture of column at fully welded (full penetration) column splice

Photo CL2.1.2 close-up view of Photo CL2.1.1

Photo CL2.1.3 fracture surface and backup plate inside of column

Photo CL2.1.4 repair using welded vertical ribs.

Photo CL2.1.5 fracture of beam bottom flange and part of web

Page 79 (No.282):

10 story, 1 by 1 bays, unbraced frame, cold-formed square-tube columns, through-diaphragms.

Photo CL2.3.1 fracture surface of column at welded splice

Photo CL2.3.2 fractured column, repaired using a wide-flange stud

Page 80 (No.332):

7 story, 2 by 2 bays, unbraced frame, cold-formed square-tube columns, wide-flange beams, through-diaphragms.

Photo CL2.4.1 fracture of column at welded splice

Photo CL2.4.3 fracture of column, crack extending into base metal

Page 81 (No.073):

5 story, 2 by 1 bays, unbraced frame, cold-formed square-tube columns, wide-flange beams, through-diaphragms, shop welding at beam ends.

Photo CL2.6.2 and CL2.6.3 large deformation at column top

5.3.3 Damage to Columns in Mega-Frame Structures

Page 82-83 (No.104-108, 448-467):

Mega-frame high-rise structures that have sustained numerous column fractures. A typical plan of these frames is shown in Fig. CL3.1.

Photo CL3.1.1 fracture in base metal above column splice

Photo CL3.1.2 fracture in base metal accompanied by tearing of cover plate

Photo CL3.1.3 Evidence of plastification near fractured section

Photo CL3.1.4 fracture of column extending into diagonal brace

Photo CL3.1.5 fracture of column extending into diagonal brace

Photo CL3.1.7 fracture of column at wedled column splice

Photo CL3.1.8 fracture of column at wedled column splice

Three types of fracture pattern were observed in those mega-frames: (A) fracture in the base metal, 200 to 300 mm above a column splice (e.g. Photo CL3.1.1); (B) fracture at the column to brace connection, with extension of the fracture into the brace (e.g. Photo CL3.1.4); (C) fracture of the column at the welded column splice (e.g. Photo CL3.1.7). Table CL3.1 summarizes all the observed fractured sections as a function of the story where they occurred. The first column of the table indicates the story (going from top to bottom, starting at 14 or more, down to 1), the second column lists the numbers of fractured sections according to the type of failure: (A), (B), and (C), and the totals are presented in the last column. Among all these buildings, the tallest (29 stories) building had no column fracture, shortest (14 stories) buildings had only a few fractures. Most of fractures were observed in buildings having19 or 24 stories. Type (A) failure was observed mostly in the first story (13 fractures reported), Type (B) failure in the 2nd to 9th stories (7 fractures), and Type (C) failure in the lower 13th stories (37 fractures). No fracture was observed in or above the 14th stories. Also, it is worthwhile to mention that:

* Plastification and local buckling was observed in diagonal members, and some degree of plastification was sometimes witnessed in columns (Photo CL3.1.3).
* Column fracture was brittle, exhibiting a fracture surface typical of brittle crack propagation. In those failures, surfaces were rather rough, involving shear lips and tear ridges, which confirmed that fracture was brittle involving a small amount of plastification.

5.3.4 Damage to Wide-Flange Columns

Page 84 (No.428):

3 story, unbraced frame in strong axis, braced frame in weak axis, wide-flange columns, wide-flange beams, angle braces, shop welding at beam ends.

Photo CL4.1.1 local buckling of column

Page 85 (No.008):

3 story, 5 by 2 bays, unbraced frame in strong axis, braced frame in weak axis, wide-flange columns, wide-flange beams, flat bar braces.

Photo CL4.5.1 first story failure, collapsed by flexutre about strong axis

Photo CL4.5.2 diagonally arranged bar braces placed in columns's weak axis

5.4 Braces

SELECTED PHOTOS (5.4)

5.4.1 Damage to Braces

Page 89 (No.590):

7 story, chevron bracings, cold-formed square-tube columns, wide-flange beams, wide-flange braces.

Photo BR1.1.1 flexural buckling of brace

Photo BR1.1.2 plate buckling of short beam at top of brace

Page 89 (No.602):

3 story, braced frames in both directions, diagonal bracing, wide-flange columns, wide-flange beams, channel braces.

Photo BR1.2.1 flextural buckling of channel brace

Page 90 (No.281);

7 story, 2 by 1 bays, unbraced frame in strong axis, braced frame in weak axis, diagonal bracing, wide-flange columns, wide-flange beams, flat bar braces.

Photo BR1.3.1 buckling and fracture of flat bar braces

Page 92 (No.286):

7 story, 5 by 5 bays, chevron bracing in both directions, wide-flange beams, wide-flange braces.

Photo BR1.9.1 fracture at junction between brace and beam

Page 93 (No.337):

3 story, 5 by 4 bays, chevron bracing, wide-flange columns, wide-flange beams, double-angle braces.

Photo BR1.10.1 fracture at brace-to-beam connection

Photo BR1.10.2 fracture at gusset plate

Page 94 (No.596):

6 story, 6 by 3 bays, braced frames in both directions, wide-flange column with stiffening plates, wide-flange beams, square-tube braces.

Photo BR1.12.1 fracture of high strength bolts at connection

Photo BR1.12.2 out-of-plane distortion of gusset plate

Photo BR1.12.4 plate buckling of beam web

Page 95 (No.887):

11 story, 8 by 6 bays, unbraced frame in strong axis, eccentric bracing in weak axis, wide-flange columns, wide-flange beams, square-tube braces (partially wrapped by concrete for fire protection).

Photo BR1.14.2 out-of-plane bending of brace

Page 97 (No.422):

8 story, 4 by 3 bays, diagonal bracing in both directions, cold-formed square-tube columns, wide-flange beams, T-shaped braces, through-diaphragms, shop welding at beam ends.

Photo BR1.17.1 buckling of brace

Photo BR1.17.2 fracture of high strength bolts

Photo BR1.17.3 fracture of beam bottom flange initiated at toe of weld access hole

Photo BR1.17.4 close view of fracture of lower flange

5.5 Column Bases

SELECTED PHOTOS (5.5)

5.5.1 Damage to Anchor Bolts

<u>Page 99 (No.005):</u>

4 story, 6 by 2 bays, unbraced frame, wide-flange columns, wide-flange beams.

Photo BS1.1.1 fracture of anchor bolts

Photo BS1.1.2 pull out of anchor bolts

5.5.2 Damage to Base Plates

<u>Page 100 (No.308):</u>

6 story, 2 by 1 bays, unbraced frame, cold-formed square-tube columns, wide-flange beams.

Photo BS1.4.1 fracture of anchor bolts and lateral displacement of base plate

<u>Page 104 (No.288):</u>

3 story, 3 by 2 bays, unbraced frame, wide-flange columns, wide-flange beams.

Photo BS2.2.1 flexural deformation of base plate and cracking at weld

5.5.3 Damage to Concrete Encased Column Bases

<u>Page 107 (No.614):</u>

6 story, 3 by 1 bays, unbraced frame in strong axis, diagonal bracing in weak axis, wide-flange columns, wide-flange beams, flat bar braces.

Photo BS3.1.1 failure of column encased in concrete

<u>Page 107 (No.598):</u>

9 story, 2 by 2 bays, cold-formed square-tube columns.

Photo BS3.2.1 failure of column base encased in concrete

Photo BS3.2.2 pull out of anchor bolts

<u>Page 108 (No.361):</u>

6 story, 4 by 2 bays, unbraces frame, cold-formed square-tube columns, wide-flange beams, through-diaphragm, shop-welding at beam ends.

Photo BS3.4.1 failure of column base encased in concrete

Photo BS3.4.2 failure of column base encased in concrete

5.6 Large Span Structures

SELECTED PHOTOS (5.6)

5.6.1 Damage to Truss Systems

Page 110 (No.495):

diagonal bracing in both directions, truss roof, wide-flange columns, wide-flange beams, angle braces.

Photo SP1.1.1 flexural buckling of column

Photo SP1.1.3 buckling of lattice members in roof

Page 111 (No.433):

large-span truss roof.

Photo SP1.2.1 leaning of supporting column

Photo SP1.2.2 dislocation of lattice members from a connection node

5.7 Finishes

Many types of exterior finishes sustained damage. Table 5.1 shows the type of exterior finishes agaist the number of buildings sustaining damage to finishes. The left column lists the type of finishes, going from top to bottom as: mortar, curtain wall, precast concrete panel, ALC (autoclaved light weight concrete) panel, folded plate, others, and unknown. Damage to nonstructural elements is summarized in Table 5.2. The left column lists the type of nonstructural elements, going from top to bottom: exterior finishes, interior (partition) walls, glass windows, emergency staircase, balconies, and others.

SELECTED PHOTOS (5.7)

Photo FS1.1.1 falling of exterior mortar

Photo FS2.1.1 fracture of bolts at metal angle connection of precast concrete wall

Photo FS2.1.2 failure of precast concrete panel near connection

Photo FS3.1.1 vertical cracks in ALC panels

Photo FS3.1.2 horizontal cracks in ALC panels

Photo FS3.1.4 cracks at corner of ALC panels

APPENDIX: LIST OF BUILDINGS INVESTIGATED

The long table (from Page 123 to 167) lists the outline of the 988 buildings investigated in the survey, and reads as follows:

* first column: ID number of buildings
  [The ID number appears in the upper right corner of the text in Chapter 5, in which damage to individual buildings is described, and photos showing the damage provided.]
* second column: number of stories
* third column: location of damage, subdivided into (from left to right): beam-to-column connection, beam, column, brace, column base, and others.
  [The cirle indicates the location of dominant damage. The alphanumeric code denotes the photo number that appears in the text (Chapter 5).]
* fourth column: damage level
* fifth column: short description of damage

(End of Translation)

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