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# A connection model for the seismic analysis of welded steel moment frames

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An analytical approach is described for performing seismic evaluation of welded steel moment frame buildings constructed prior to the 1994 Northridge earthquake. The approach is based on an analytical model that captures the possibility of connection fracture and the subsequent loss of strength and stiffness. This connection model was developed and calibrated on the basis of laboratory experiments. It was employed in the seismic analysis of a 13-story building which experienced fractured connections in the Northridge earthquake and for which there are seismograph records and a damage survey. Results from several analyses indicate that the inelastic and fracture models are capable of predicting the behavior of steel moment frame buildings and of estimating the level of damage. © 1997 Elsevier Science Ltd.

**Keywords:** brittle fracture, inelastic analysis, seismic performance, steel structures, welded connections

## 1. Introduction

The Northridge earthquake of 17 January 1994, caused significant damage to welded steel moment frame (WSMF) construction. The unexpected damage typically involved fracture of the steel beam-to-column connections. The welded beam-to-column connection in widespread use was that 'prescribed' in the Uniform Building Code[1]. This connection was presumed to have sufficient strength to permit either the beam to yield in flexure or the column panel zone to yield in shear. The brittle manner in which the welded frame connections failed in the Northridge earthquake has called into question the ability of welded steel moment frames, built since the early 1970s, to dissipate energy through ductile inelastic behavior.

The buildings damaged in the earthquake will be repaired as required and new construction will proceed employing connections verified by test to provide the strength and ductility as required by an emergency code change to the 1994 edition of the Uniform Building Code. Yet there are literally thousands of existing welded steel moment frame buildings in service which have not experienced a significant earthquake, and which remain vulnerable to future earthquakes. It is important to be able to analyze the seismic response of WSMF structures in order to (1) identify potentially deficient buildings; (2) assess the vulnerability of both damaged and undamaged buildings; and (3) evaluate repair and rehabilitation strategies.

### 1.1. Background

Seismic design of WSMF construction is based on the assumption that, in a significant earthquake, frame members will be stressed beyond the elastic limit. Inelastic action is permitted in frame members, since it is presumed that they will behave in a ductile manner thereby dissipating energy. Welds and bolts, being considerably less ductile, are not permitted to fracture. Thus, the design philosophy requires sufficient strength be provided in the connection to allow beam and or column panel zones to yield and deform inelastically[2]. The beam-to-column connections are required to be designed, therefore, for either the nominal strength of the beam in flexure or the moment corresponding to the joint panel zone shear strength.

The Uniform Building Code (UBC), which is adopted by nearly all California jurisdictions as the standard for seismic design, incorporates this philosophy for the design of WSMF connections[1]. Since the 1988 edition, the UBC has prescribed a beam-to-column connection that is deemed to satisfy the above strength requirements. This 'prescribed' detail requires the beam flanges to be welded to the column using complete-penetration (CP) groove welds. The beam web connection may be made by either welding the web directly to the column or by bolting the web to a shear tab, which in turn is welded to the column. A version of this detail is shown in *Figure 1*.

The prescribed or conventional detail was justified by

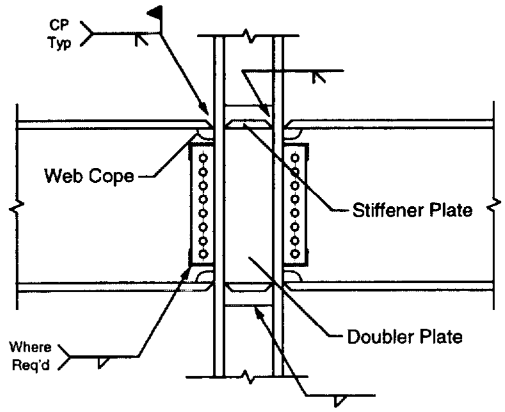



*Figure 1* Welded flange beam-to-column moment connection

tests from the early 1970s[2]. These tests confirmed adequate strength and plastic rotation capacity for specific beam sizes and loading patterns. However, while most test programs on conventional connections were able to show impressive results with some specimens, all experienced some unacceptable behavior limited by non-ductile connection failures[3-7].

While connection reliability can be questioned on the basis of historical test results, the performance of steel frames in earthquakes prior to Northridge has been considered to be excellent. In design practice, the WSMF has long been considered to be among the most reliable structural system for resisting seismic loads[8]. Confidence in the prescribed connection detail has led to its widespread use with a variety of member sizes, frame dimensions, shear connectors, flange weld processes, and lateral force resisting system configurations.

### 1.2. *Observed failures*

Many initial inspections of steel frame buildings following the Northridge earthquake found only minor non-structural damage. Based on prior earthquake experience, engineers had no reason to suspect cracked welds or fractured columns concealed by soffits, ceilings and fireproofing. Only after a few reports of steel damage began to circulate did engineers and owners revisit buildings to perform more complete inspections. In time, these inspections revealed serious cracking that had not been observed in past earthquakes. By September 1994, eight months after the earthquake, over 100 buildings in the Los Angeles area were reported to have experienced cracking in beam-to-column connections. At that time, however, there was no uniformity in the inspection and reporting of damage to the buildings. For the WSMF population as a whole, many issues remained unresolved such as the extent of different damage types and the correlation between damage and such factors as ground motion, frame configuration, and weld quality. To address these issues, the National Institute of Standards and Technology (NIST) contracted Nabih Youssef and Associates (NYA) to compile and analyze available data on WSMF buildings inspected since the Northridge earthquake. The goal of the survey was to identify the nature and extent of observed damage, providing an accurate assessment of the situation as of November 1994. The survey report[9] presents data from a total of 51 surveyed buildings.

While the sample of damaged buildings was small, several observations could still be made regarding the population of buildings under study:

- Of 2066 inspected 'floor-frames', that is, each set of connections in a single frame at a single floor:
  - 50% had damage to the bottom beam flange weld;
  - 20% had damage to the top beam flange weld;
  - 15% had damage to the column flange at the bottom beam flange joint;
  - 5% had fracture through the column flange and into the column web.
- In low-rise buildings (less than 5 stories) more damage was found in the lower floors.
- In mid-rise buildings (greater than 5 and less than 20 stories) there was no significant damage pattern, but damage tended to be found around mid-height.
- No statistical correlations were found between observed damage and building size or regularity, age of building, nominal material strength or member sizes, or redundancy of lateral force resisting system.

### 1.3. *Characteristics of connection failures*

For the purposes of the survey, observed damage was classified as to type and location. Survey results and more complete failure analyses of individual buildings, revealed that the fractures often initiated at the root of the bottom beam flange-to-column groove weld, near the weld mid-length[10]. The fracture, once initiated, often propagated along the heat affected zone (HAZ) of the column base metal. The fracture sometimes surfaced at the face of the column, excavating a portion of the column material. In other instances the fracture propagated either along or through the column flange, sometimes extending well into the column web. A schematic showing typical fracture of a bottom flange is shown in *Figure 2*.

Fractographic inspection of the fracture surface indicated the steel failed in brittle fracture with no evidence of stable crack growth from prior load cycles. When a flange fractured completely, the remaining flange in combination with the web provided a reduced flexural resistance. Tearing along the bolt line or web-to-column weld due to additional loading further softened the connection. Reverse loading resulted in the crack closing and connection behavior approximating that of an uncracked connection.

## 2. Computer modeling of welded steel moment frames

Computer programs in use today are generally not capable of modeling the effect of sudden weld failures at beam-to-column connections. To provide such an analytical capa-

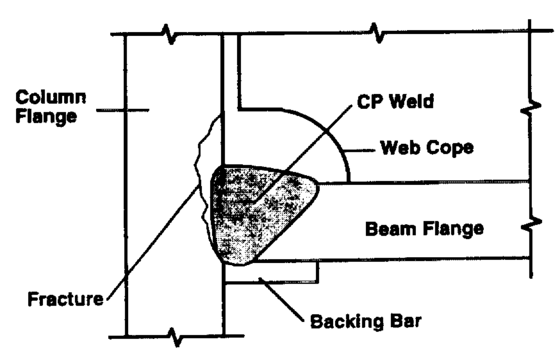



*Figure 2* Typical bottom flange weld fracture

bility, NIST contracted to have enhancements made to the IDARC computer program[11]. The program, renamed IDASS for Inelastic Damage Analysis of Structural Systems now includes the following features:

- Steel flexural member based on finite plastic hinge length at yielding sections.
- Column panel zone model to account for the elastic and inelastic deformations of the beam-to-column joint region.
- Hysteresis model for the steel element accounting for flange fracture and post-fracture behavior.

These features are described further in the following sections.

### 2.1. *Flexibility-based member model for steel sections*

Existing formulations for the nonlinear analysis of steel frame structures are commonly based on the parallel-member model introduced by Clough[12] in which one member is elastic-perfectly plastic, while the second member remains elastic. This formulation accounts, in an approximate way, for the kinematic strain hardening of the yielding member. Additionally, it is common to use a concentrated plasticity model whereby all plastic rotations occur at a 'zero-length' hinge at the member ends. The DRAIN-2D computer program[13] employs these concepts and is used in verifying results of the flexibility-based member model presented herein.

The notion of a finite hinge length to model the spread of plasticity along a member length was first proposed by Soleimani *et al.*[14] In this approach, the curvature distribution along the member length is approximated by considering inelastic zones at the member ends which exhibit a reduced effective stiffness. A similar model was later used by Meyer *et al.*[15] More recently, a beam 'superelement' consisting of Soleimani's spreading plasticity element, a joint element to account for fixed-end rotations at the beam–column interface, and an elastic element to characterize the flexural behavior prior to yielding was developed by Filippou and Issa[16].

In the current study, a flexural element with offsets to represent the 'rigid' beam-to-column joint region, as shown in *Figures 3a* and *3b*, has been developed. Inelastic panel zone deformations are accounted for using a bi-linear panel joint model described later. For an assumed flexibility distribution, the coefficients of the $2 \times 2$ flexibility matrix may be obtained from the following relationship:

$$f_{ij} = \int_0^L m_i(x) m_j(x) \frac{1}{EI(x)}\,\mathrm{d}x \tag{1}$$

where $m_i(x)$ is the moment distribution corresponding to a unit moment at end $I$, $m_j(x)$ is the moment distribution corresponding to a unit moment at end $j$, and $EI(x)$ is the flexural rigidity. The element stiffness matrix is, in turn, constructed from the flexibility matrix using static equilibrium relationships[17].

The notion of concentrating plasticity at a 'hinge' of finite length involves an approximation of the flexibility distribution which, when used in equation (1), produces satisfactory results for the limiting cases. The distribution shown in *Figure 3c* was determined through trial to produce a correct stiffness matrix for the case in which both ends

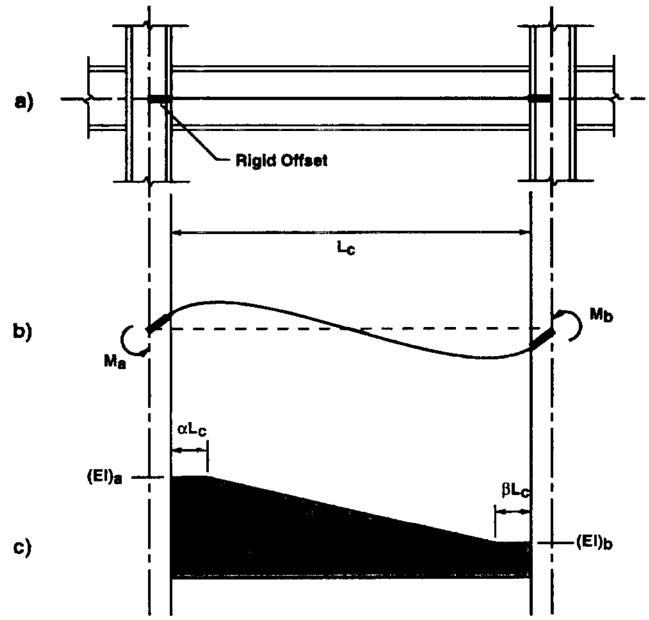



*Figure 3* Steel flexural element with rigid offsets

of a member remain elastic or both ends yield. When one end of a member yields, the distribution shown, using a fixed hinge length of 5% of the clear span and 5% strain hardening, produces a flexural stiffness slightly greater than that resulting from the parallel member approach. The variation in the element flexibility shown in *Figure 3c*, when substituted in equation (1), results in the following flexibility coefficients:

$$\begin{aligned} f_{11} = {} & (3 + 3\alpha - 3\alpha^2 + \alpha^3 - \beta - \beta^2 - \beta^3 \\ & + 2\alpha\beta - \alpha^2\beta + \alpha\beta^2)/12(EI)_a + (1 - 3\alpha \\ & + 3\alpha^2 - \alpha^3 + \beta + \beta^2 + \beta^3 - 2\alpha\beta + \alpha^2\beta \\ & - \alpha\beta^2)/12(EI)_b \end{aligned} \tag{2a}$$

$$\begin{aligned} f_{12} = f_{21} = {} & (1 + \alpha + \alpha^2 - \alpha^3 - \beta - \beta^2 \\ & + \beta^3 + \alpha^2\beta - \alpha\beta^2)/12(EI)_a + (1 - \alpha \\ & - \alpha^2 + \alpha^3 + \beta + \beta^2 - \beta^3 - \alpha^2\beta \\ & + \alpha\beta^2)/12(EI)_b \end{aligned} \tag{2b}$$

$$\begin{aligned} f_{22} = {} & (1 + \alpha + \alpha^2 + \alpha^3 - 3\beta + 3\beta^2 \\ & - \beta^3 - 2\alpha\beta - \alpha^2\beta + \alpha\beta^2)/12(EI)_a \\ & + (3 - \alpha - \alpha^2 - \alpha^3 + 3\beta - 3\beta^2 + \beta^3 + 2\alpha\beta \\ & + \alpha^2\beta - \alpha\beta^2)/12(EI)_b \end{aligned} \tag{2c}$$

The hinge lengths defined by $\alpha$ and $\beta$ may be specified either as a fixed value or allowed to vary as a function of the beam end moments.

### 2.2. *Joint panel model*

Deformations in panel zones of steel frame beam-to-column joints may contribute significantly to lateral drift in a building. To model the behavior of the panel zone, the shear distortion, which is the relative rotation between the beam and column framing into a joint, is related to the panel zone shear force. A two rotational degree-of-freedom element has been added to IDASS to model the panel zone. The elastic shear force vs shear distortion behavior is speci-

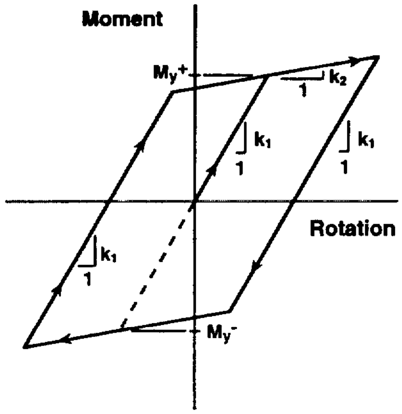



*Figure 4* Bi-linear hysteresis model

fied using geometry of the panel and shear modulus of the column web. The post-yield behavior is accounted for by means of a bilinear non-degrading hysteresis envelope as shown in *Figure 4*. The stiffness of the elastic and post-yield regions as well as shear force at general yield may be calculated using any of several models (e.g. that proposed by Krawinkler[18]). It should be noted that there are differences among response predictions provided by the various models and a lack of experimental evidence to support one model over another.

### 2.3. *Degrading hysteresis model to account for flange fracture*

A hysteresis model was developed for the specific purpose of capturing the effects of weld fracture and subsequent nonlinear response of the connection. Since limited data are available on the post-fracture response of moment connections, a conceptual model was developed based on results obtained from tests conducted at the University of California, Berkeley. The features of the hysteresis behavior and parameters used to define the behavior are shown in *Figure 5*.

The hysteresis response prior to weld fracture is characterized by a bilinear envelope with yield capacity specified by $M_y$. Weld fracture is presumed to occur at a moment denoted by $M_{cr}$. At present, this critical moment is specified as a function of the yield moment. However, when additional experimental data becomes available, it may be possible to base the critical moment on, for example, cumulative damage. Subsequent to weld fracture, a reduced stiffness and strength, bilinear regime is followed. Unloading from this regime results in a further reduced stiffness. The

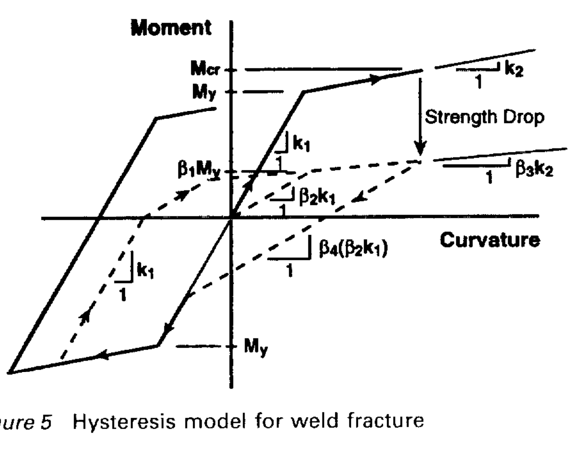



*Figure 5* Hysteresis model for weld fracture

unloading path intersects the initial stiffness simulating crack closing, as seen in *Figure 5*. With the model described here, failure of only one beam flange is permitted.

The new hysteresis model was compared to results from an experimental study conducted at the University of California, Berkeley (UCB) as part of the SAC Joint Venture effort. In the UCB series of tests of typical beam-to-column connections, loading of one specimen was continued beyond the load which caused fracture of the beam flange thus providing data for comparison with an analytical simulation. The test specimen was modeled using the IDASS steel flexural element and degrading hysteresis. Beam tip deflection vs applied force for both the UCB experiment and IDASS analytical response are shown in *Figure 6*. For the IDASS analysis, fracture of the weld was specified on the basis of observed response. It is evident, however, that the IDASS hysteresis model is capable of simulating flange fracture and post-fracture behavior.

## 3. Case study building

The building selected for study is the Blue Cross Headquarters facility, located in the San Fernando Valley, approximately 4.8 km (3 miles) from the epicenter of the 17 January 1994 Northridge earthquake. This building was selected because it was instrumented at the basement, 6th floor, and roof levels, thus providing an excellent opportunity to compare analytical predictions with actual building performance.

### 3.1. *Lateral force resisting system*

A typical floor plan of the Blue Cross Building is 48.77 m (160 ft) square with the lateral force resisting system comprising the perimeter frames. The corner columns are box sections welded from plate while the remaining columns are heavy wide flange sections oriented in their strong direction. *Figure 7* shows a typical elevation of the building. Beam and column sections, box column sizes, and doubler plate thicknesses are given in *Table 1*. Cumulative weights used in gravity column load and floor mass calculations included the steel frame, concrete slab, partitions, exterior wall etc., but live loads were not considered.

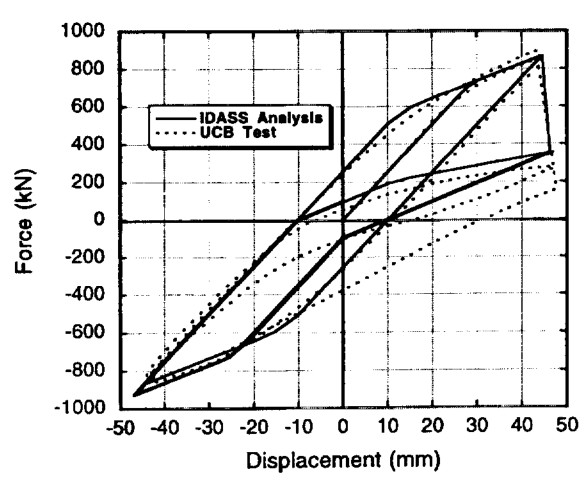



*Figure 6* Observed and simulated hysteresis response of a beam-to-column connection

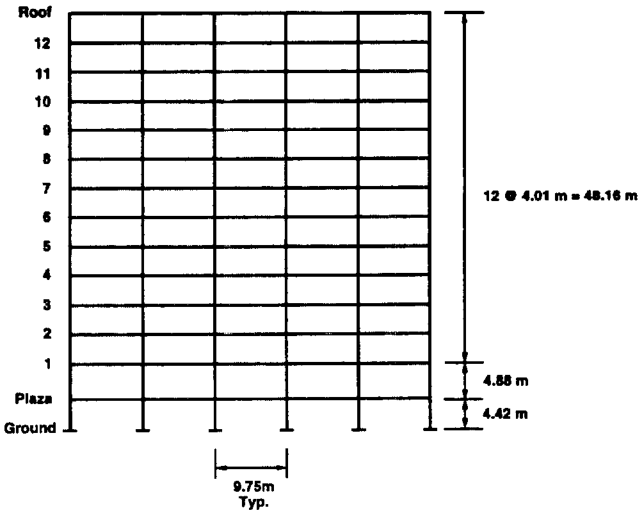



*Figure 7* Lateral force-resisting structural system

### 3.2. *Material properties*

Mechanical properties of the beams, columns and plate material were not available, although all material is reported to be ASTM A36 steel. While some researchers have used an expected value of yield strength of 326 MPa (47.3 ksi) for A36 steel, a value of 276 MPa (40 ksi) was used for this study. This value was chosen on the basis of mechanical tests of similar sections of A36 material removed from five buildings damaged in the Northridge earthquake and tested by Lehigh University[10]. Fracture analyses conducted by Lehigh University revealed that fracture would occur without significant yielding in the beam flanges.

### 3.3. *Strong motion records*

Acceleration records for three component directions, available from the Office of Strong Motion Studies, California Division of Mines and Geology (CDMG), were used in the analysis. *Figure 8* shows the E–W and N–S acceleration records. The peak ground acceleration (PGA) for the east–west record is recorded to be 0.319 $g$, while that for the north–south record is 0.411 $g$.

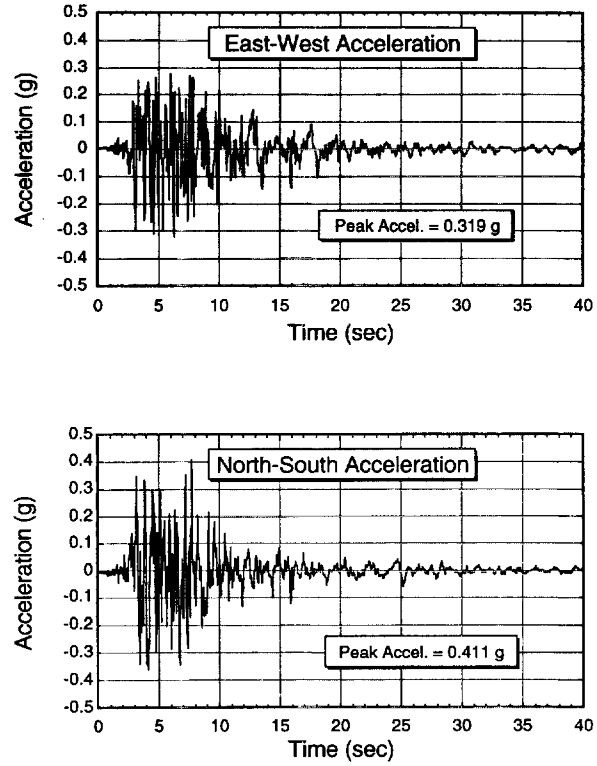


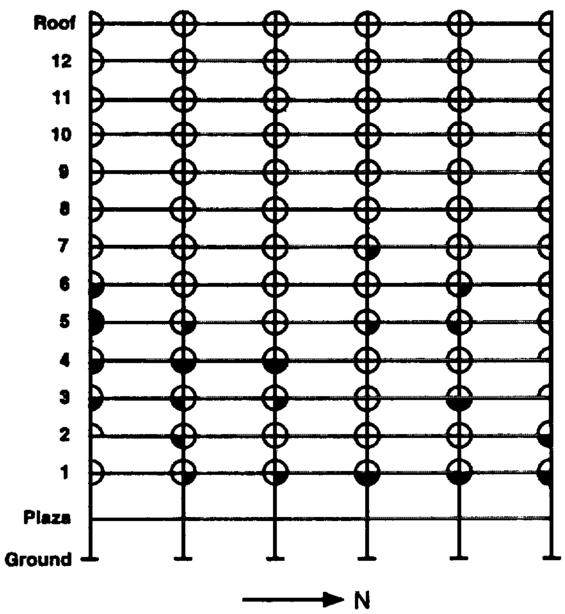

*Figure 8* East–west and north–south base acceleration records

### 3.4. *Observed damage*

Almost every connection in the lateral force resisting frames of the Blue Cross Building was inspected. In a few instances, the bottom flange was not inspected due to limited access. *Figure 9* presents results of the damage survey for the north–south oriented frames[19]. A circle is used to indicate that the connection was inspected, while the shaded circle indicates that damage was detected. Damage found in the Blue Cross Building included fracture of the column or beam flanges and fracture of the beam-to-column weld. Uang[19] reported that full or partial fracture of the column flange occurred in many instances. It can be seen that damage was, in general, limited to levels 1–6. Damage to the east–west lateral force resisting frames was considerably less significant.

*Table 1* Beam, column and doubler plate sizes

| Level | Beam section | Interior column wide-flange sections | Exterior column box sections Size (mm) | Exterior column box sections Plate thickness (mm) | Doubler plate thickness (mm) |
|---|---|---|---|---|---|
| Roof | W27 × 84 | — | — | — | 35.7 |
| 12th floor | W33 × 118 | W14 × 167 | 371.5 | 25.4 | 54.7 |
| 11th floor | W33 × 118 | W14 × 167 | 371.5 | 25.4 | 60.3 |
| 10th floor | W33 × 130 | W14 × 246 | 384.2 | 31.8 | 66.7 |
| 9th floor | W33 × 130 | W14 × 246 | 384.2 | 31.8 | 65.0 |
| 8th floor | W33 × 141 | W14 × 287 | 390.5 | 34.9 | 71.4 |
| 7th floor | W33 × 141 | W14 × 287 | 390.5 | 34.9 | 70.9 |
| 6th floor | W33 × 152 | W14 × 314 | 403.2 | 41.3 | 70.9 |
| 5th floor | W33 × 152 | W14 × 314 | 403.2 | 41.3 | 70.4 |
| 4th floor | W33 × 152 | W14 × 398 | 409.6 | 44.5 | 70.4 |
| 3rd floor | W33 × 152 | W14 × 398 | 409.6 | 44.5 | 73.0 |
| 2nd floor | W33 × 152 | W14 × 426 | 422.3 | 50.8 | 73.0 |
| 1st floor | W36 × 230 | W14 × 426 | 422.3 | 50.8 | 106.4 |
| Plaza | W36 × 194 | W14 × 500 | 447.7 | 63.5 | 87.3 |
| Ground | — | W14 × 500 | 447.7 | 63.5 | — |



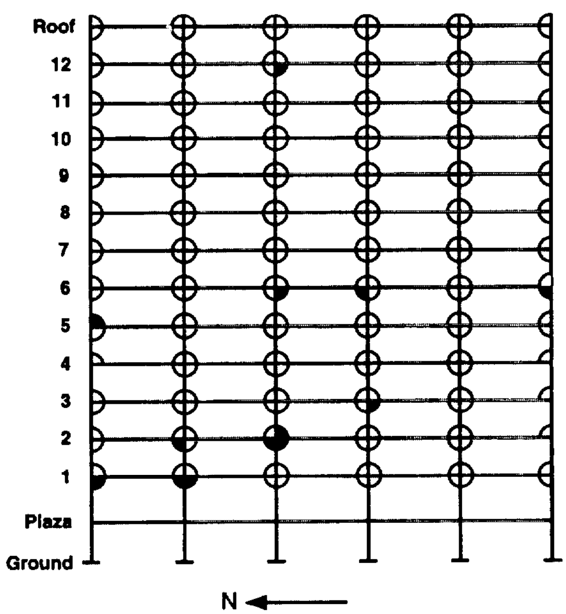




*Figure 9* Damage to north–south frames

## 4. Analysis results

The bare steel lateral force resisting system described above was modeled using the IDASS computer program. Calculations indicated that the beams would yield before the panel zones so panel zone elements were not included in the analyses. Panel zone flexibility was approximated using centerline dimensions and no rigid offsets.

Several two-dimensional time–history analyses were conducted using the east–west and north–south acceleration records. The roof displacement is used to compare the analytical results with observed performance. Additionally, predicted damage is compared with observed connection damage obtained from surveys as noted above.

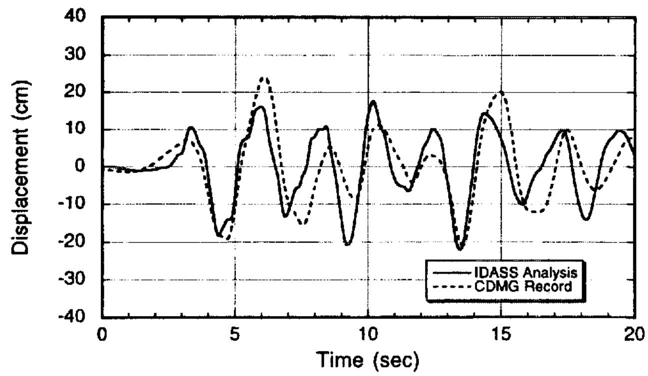



*Figure 10* Roof displacement for east–west acceleration record

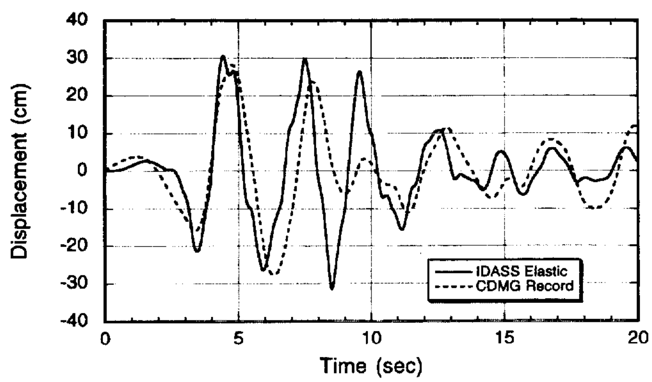



*Figure 11* Results of elastic analysis for north–south acceleration record

### *4.1. E–W acceleration*

Results from a time–history analysis using the east–west acceleration record indicate that the structure remains elastic. A comparison between the computed and measured (CDMG) lateral displacement of the roof is shown in *Figure 10*. Recall that very little damage was reported for the east–west frames.

### *4.2. N–S acceleration*

An elastic analysis using the north–south acceleration record was conducted for comparison purposes and results for the computed and measured roof displacement are shown in *Figure 11*. The peak displacements are fairly closely predicted as would be expected for a structure with a fundamental period of approximately 3 s[20]. When an elastic–plastic hysteresis model is used, the results for the roof displacement are shown in *Figure 12*. Analytically pre-

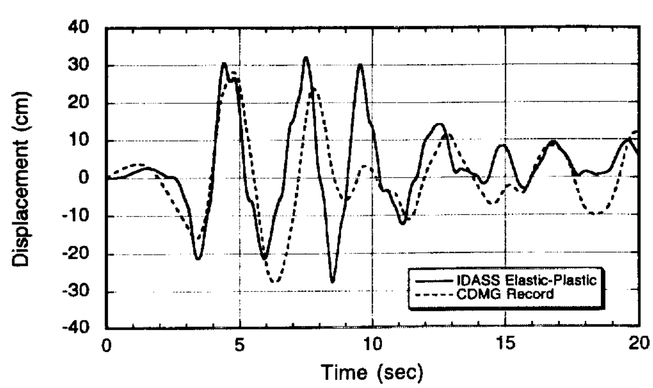



*Figure 12* Roof displacement for N–S acceleration, elastic–plastic model

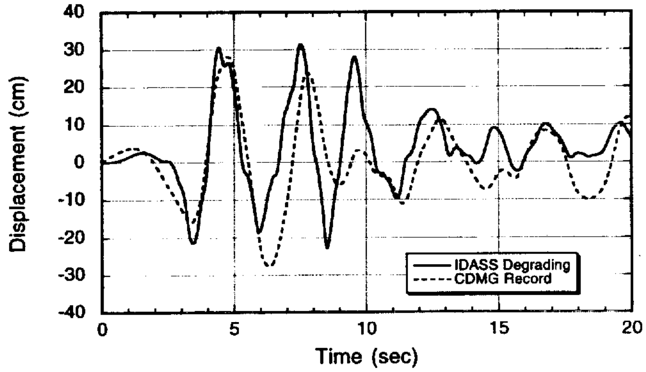



*Figure 13* Roof displacement for N–S acceleration, degrading stiffness hysteresis

dicted displacements, particularly in the range of 10–20 seconds, appear to be closer to measured displacements. Similarly, when the degrading hysteresis model is used, as shown in *Figure 13*, the third negative peak is further reduced, which again is closer to the measured response.

In contrast to the results obtained with the east–west record, the north–south acceleration record produced many connection failures in levels 3–7. The predicted connection failures for the analysis using degrading stiffness are shown in *Figure 14*. The pattern of predicted failures is similar to that of observed failures (*Figure 9*). The greater damage to frames orientated in the north–south direction is consistent with observations of the general population of buildings in the San Fernando Valley[9].

## 5. Discussion

It should be kept in mind that several approximations inherent in the analyses conducted here may influence the results. For instance, only the intended lateral force resisting system was modeled; no account was taken of the resistance provided by the interior members, nor were composite floor effects considered. Likewise, no account was taken of the resistance of non-structural components including cladding, interior walls, etc. Actual material properties were not known and fracture initiation was estimated to

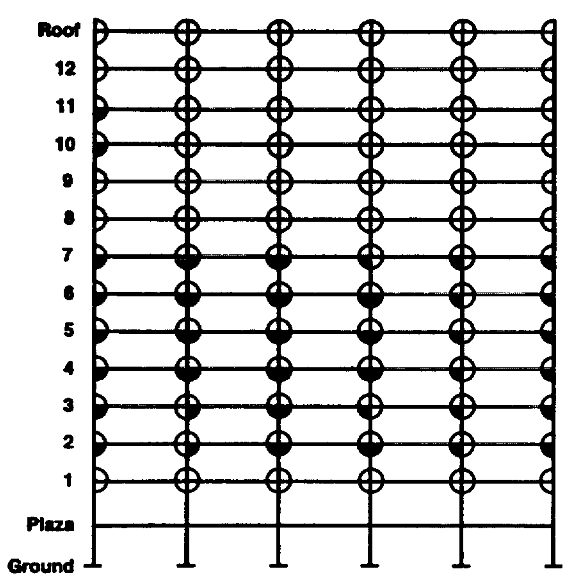



*Figure 14* Predicted connection failure using degrading hysteresis model

be at the onset of beam flange yielding. Finally, there is uncertainty in the parameters used to characterize the degrading hysteresis model.

Nevertheless, analytically predicted roof displacements presented here showed reasonable agreement with displacements obtained from accelerations recorded at the roof level. It should be noted that, for the relatively long period structure analyzed here, one would not expect the displacements resulting from an inelastic analysis to be significantly different from those resulting from an elastic analysis. The location and extent of connection fracture was, in general, also predicted within reason. Again, many modeling assumptions were made and uncertainties remain in characterizing the onset of fracture and subsequent loss of stiffness and strength.

The research described herein should be considered preliminary as much additional work needs to be done to validate the degrading strength and stiffness hysteresis model. Specifically, the merits of the degrading hysteresis model may be more apparent in smaller buildings, say four to five stories, where failure of just a few connections may lead to a significant change in stiffness. Also, low-rise buildings will have a much shorter fundamental period and differences between elastic and inelastic response will be more pronounced. The approach described herein shows promise and deserves further attention.

## Acknowledgements

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