4. LEVEL 1 DETAILED POSTEARTHQUAKE EVALUATIONS

4.1 Introduction

Detailed evaluation is the second step of the postearthquake evaluation process. It should be performed for all buildings that are estimated to have experienced potentially damaging ground motions, using the screening procedures of Section 3.2 of these *Recommended Criteria*. As detailed evaluation can be a time consuming process, it is recommended that a preliminary evaluation, in accordance with the procedures of Chapter 3, be conducted prior to detailed evaluation, to permit rapid identification of those buildings that may have been so severely damaged that they pose an immediate threat to life safety.

Many steel moment-frame buildings damaged in past earthquakes have displayed few outward signs of structural or nonstructural damage. Consequently, except for those structures which have been damaged so severely that they are obviously near collapse, brief evaluation procedures, such as those of Chapter 3, are unlikely to provide a good indication of the extent of damage or its consequences. In order to make such determination, it is necessary to perform detailed inspections of the condition of critical structural components and connections. If structural damage is found in the course of such inspections, it is then necessary to make a determination as to the effect of discovered damage on the structure's ability to resist additional loading. Ultimately, decisions as to the significance of damage, whether occupancy should be permitted in a building and whether specific types of damage should be repaired must be made on the basis of quantitative evaluation and engineering judgment.

This chapter provides simplified procedures for a quantitative evaluation method in which occupancy and repair decisions are assisted based on the calculation of a damage index, related to the distribution and severity of different types of damage in the structure. In order to apply this method, termed a Level 1 evaluation, it is necessary to obtain an understanding of the distribution of damage in the structure. This must be obtained by performing visual inspections of critical framing and connections. It is preferred that damage indices be calculated based on a determination of the condition of all critical connections in the building; however, it is permissible to infer a distribution of damage, and calculate a damage index, based on an appropriately selected sample of connections.

Chapter 5 provides recommended criteria for an alternative method of quantitative evaluation, termed a Level 2 evaluation, based on performing structural analysis of the damaged structure's ability to resist additional strong ground shaking. In order to perform a Level 2 evaluation, it is necessary to conduct a complete inspection of all fracture-susceptible connections in the building.

Commentary: The Level 1 evaluation approach contained in this chapter is based upon a methodology originally presented in FEMA-267, modified to account for experience gained in the application of the FEMA-267 guidelines to real buildings and also calibrated to expert opinion on the severity of various types of damage. The Level 2 evaluation is a more comprehensive approach that is *compatible with the overall approach developed for performance evaluation of structures.*

The Level 1 detailed evaluation procedure consists of gathering available information on construction of the structure and a multi-step inspection, evaluation, decision and reporting process. Although it is preferable to conduct a complete inspection of all fracture-susceptible connections, it is permissible to inspect only a selected portion of the elements and connections and to use statistical methods to estimate the overall condition of the building. A damage index is introduced to quantify the severity of damage in the building. This damage index is calculated based on individual connection damage indices, d_i, assigned to the individual inspected connections. These connection damage indices vary between 0 and 4, with 0 representing no significant earthquake damage and 4 representing severe damage. A story-level damage index, D_{max}, is introduced which varies between 0 and 1.0, depending on the severity of damage. Based on the maximum damage index obtained for any floor level, D_{max}, or if full inspections were not made of all connections, the probability that the damage index exceeds a specified threshold, recommendations are provided to the structural engineer regarding the appropriate damage condition designation as well as decisions regarding occupancy restrictions and repair actions.

4.2 Data Collection

Prior to performing a detailed inspection and evaluation, available information on the building's construction should be collected and reviewed. This review should be conducted in a manner similar to that indicated in Section 3.3.2, but extended to include greater knowledge, for example, of the primary lateral and gravity load-resisting systems, typical detailing, and presence of irregularities. Pertinent available engineering and geotechnical reports, including any previous damage survey reports, such as the preliminary postearthquake evaluation report prepared in accordance with Chapter 3 of these *Recommended Criteria*, and current ground motion estimates, should also be reviewed. Specifications (including the original Welding Procedure Specifications), shop drawings, erection drawings, and construction inspection records should be reviewed when available.

When structural framing information is not available, a comprehensive field study should be undertaken to determine the location and configuration of all lateral-force-resisting frames, and the details of their construction, including members' sizes, material properties, and connection configurations. See Section 5.2 for additional discussion.

4.3 Evaluation Approach

Analyses of buildings with brittle connections, such as those damaged by the 1994 Northridge earthquake, show that although damage occurs slightly more often in locations predicted by analysis to have high stress and deformation demands, damaged connections tend to be widely distributed throughout building frames, often at locations that analyses would not predict. This suggests that there is some randomness in the distribution of the damage. To detect reliably all such damage, it is necessary to subject each fracture-susceptible connection to detailed inspections. Fracture-susceptible connections include:

- Moment-resisting beam-column connections in which the beams are connected to columns using full penetration welds between the beam flanges and column, and in which yield behavior is dominated by the formation of a plastic hinge within the beam at the face of the column, or within the column panel zone.
- Splices in exterior columns of moment-resisting frames when the splices consist of partial penetration groove welds between the upper and lower sections of the column, or of bolted connections that are incapable of developing the full strength of the upper column in tension.

The inspection of all such connections within a building can be a costly and disruptive process. Although complete visual inspections of fracture-susceptible connections are recommended as part of a Level 1 evaluation, this evaluation methodology permits a representative sample of the critical connections to be selected and inspected. When only a sample of connections is inspected use is made of statistical techniques to project damage observed in the inspected sample to that likely experienced by the entire building.

In order to obtain valid projections of a building's condition, when the sampling approach is selected, samples should be broadly representative of the varying conditions (location, member sizes, structural demand) present throughout the building and samples should be sufficiently large to permit confidence in the projection of overall building damage. Two alternative methods for sample selection are provided. When substantial damage is found within the sample of connections, additional connections should be inspected to provide better, more reliable information on the building condition.

Once the extent of building damage is determined, (or estimated if a sampling approach is utilized) the structural engineer should assess the residual structural capacity and safety, and determine appropriate repair and/or modification actions. General recommendations are provided, based on calculated damage indices. As an alternative to this approach, direct application of engineering analysis (Level 2 evaluation) may also be used as provided for in Chapter 5 of these *Recommended Criteria*.

4.4 Detailed Procedure

Postearthquake evaluation should be carried out under the direct supervision of a structural engineer. Two alternative procedures are presented below depending on whether all connections in the building are inspected, or only a sample of the connections in the building are inspected. Section 4.4.1 describes the procedure when all connections are inspected. Section 4.4.2 describes the procedure when a sample of connections are inspected.

As used in these *Recommended Criteria*, the term "connection" means that assembly of elements including the beam, column, plates, bolts, and welds, that connect a single beam to a single column. Interior columns of plane frames will typically have two connections (one for each beam framing to the column) at each floor level. Exterior columns of plane frames will have only one connection at each floor level.

4.4.1 Method 1 - Inspection of All Connections

The following five-step procedure may be used to determine the condition of the structure and to develop occupancy, repair and modification strategies when all critical connections in a building are inspected and the extent of damage to all connections is known:

Step 1: Conduct a complete visual inspection of all fracture-susceptible connections in the building in accordance with Section 4.3. Moment-resisting connections should be inspected in accordance with Section 4.4.1.1, with supplemental nondestructive examination, as suggested in that section.

Step 2: Assign a connection damage index, d_i, to each inspected connection in accordance with Section 4.4.1.2.

Step 3: Calculate the floor damage index at each floor, D_j, pertinent to lateral force resistance of the building in each of two orthogonal directions, in accordance with Section 4.4.1.3. Determine the maximum of the floor damage indices, D_{max}.

Step 4: Based on the calculated floor damage indices, determine appropriate occupancy, and structural repair strategies, in accordance with Section 4.4.1.5. If deemed appropriate, the structural engineer may conduct detailed structural analyses of the building in the as-damaged state, to obtain improved understanding of its residual condition and to confirm that the recommended strategies are appropriate or to suggest alternative strategies. Recommendations for such detailed evaluations are contained in Chapter 5.

Step 5: Report the results of the inspection and evaluation process to the building official and building owner.

4.4.1.1 Detailed Connection Inspections

In order to perform a detailed inspection of beam-column joints, it is necessary to remove any fireproofing or other obscuring finishes to allow direct visual observation of the connection area. Detailed inspections may be conducted in stages. An initial stage inspection may be performed by removing only the limited amount of fireproofing indicated in Figure 4-1 and following the inspection checklist of Section 4.4.1.1.1. If such initial inspection indicates the presence or potential presence of damage, than a complete inspection, in accordance with the checklist of Section 4.4.1.1.2 should be performed at each connection where such damage is detected. To accommodate a complete inspection, removal of fireproofing as indicated in Figure 4-2 is necessary. At the discretion of the engineer, a complete inspection in accordance with Section 4.4.1.1.2 may be performed without first performing the initial inspection of Section 4.4.1.1.1. Refer to Chapter 3 for cautions with regard to removal of fireproofing materials.

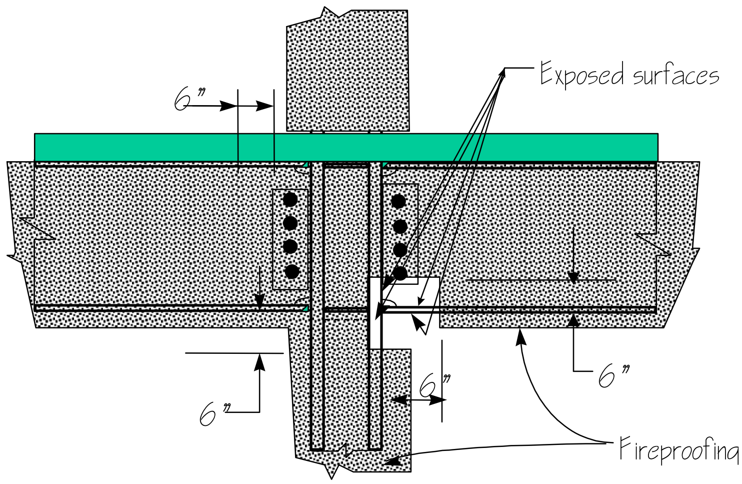

Figure 4-1 Fireproofing Removal for Initial Connection Inspection

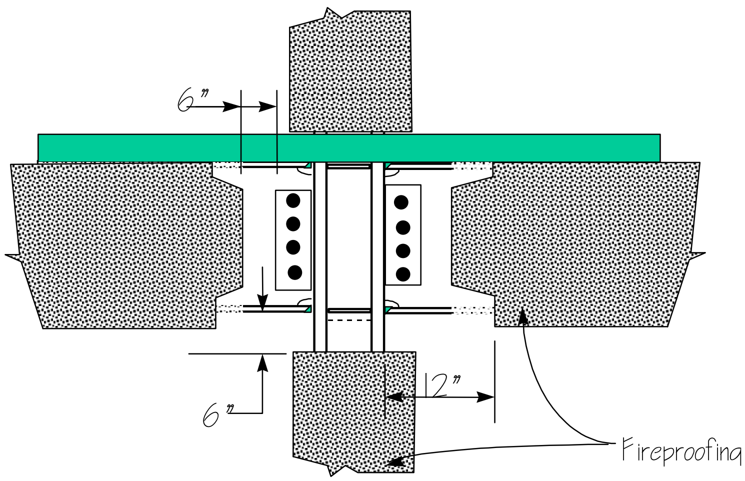

Figure 4-2 Fireproofing Removal for Complete Connection Inspection

The findings of detailed inspections of moment-resisting connections should be recorded on appropriate forms, documenting the location of the connection, the person performing the inspection, the date of the inspection, the extent of the inspection, the means of inspection (visual or nondestructive testing), the location and type of any observed damage, and, if no damage was observed, an indication of this. Appendix C includes forms suggested for this purpose. Detected damage should be classified in accordance with the system of Chapter 2.

Commentary: The largest concentration of reported damage following the 1994 Northridge earthquake occurred at the welded joint between the bottom girder flange and column, or in the immediate vicinity of this joint. To a much lesser extent, damage was also observed in some connections at the joint between the top girder flange and column. If damage at either of these locations is substantial, then damage is also possible in the panel zone or shear tab areas. For this reason, and to minimize inspection costs, these Recommended Criteria *suggest that it is appropriate to initially inspect only the welded joint of the bottom beam flange to the column, and only if damage is found at this location to extend the inspection to the remaining connection components.*

4.4.1.1.1 Initial Inspections

The checklist below may be used as a guide for initial inspections. Prior to performing the inspection, remove fireproofing (see Section 3.3.3), as indicated in Figure 4-1. If there are indications of damage, then perform a complete inspection in accordance with the procedures of Section 4.4.1.1.2.

- ☑ Observe the beam framing into the connection for trueness to line, and potential indications of lateral flexural-torsion buckling (damage type G8, Section 2.2.1).
- ☑ Observe condition of fireproofing along the beam within one beam depth of the column for cracking or spalling of the fireproofing material along the beam surface, indicating potential yielding or buckling of the beam flanges (damage types .G1, G2, Section 2.2.1).
- ☑ Observe the top and bottom surface of the exposed beam bottom flange for fractures (damage types G3, G4, Section 2.2.1).
- ☑ Observe the exposed surfaces of the complete joint penetration weld between the beam bottom flange and column for fractures (damage types W2, W3, W4 Section 2.2.3).
- ☑ Observe the exposed surfaces of the column flange for fractures (damage types C1, C2, C3, Section 2.2.2).
- ☑ Observe the condition at the bottom of weld backing on the bottom flange. If gaps are present, insert feeler gauge to detect potential damage (damage types C1, C4, C5, Section 2.2.2).
- ☑ Observe the bottom surface of the top flange fireproofing at the locations where the beam flanges join the column flanges (or continuity plates for minor axis connections) for cracks or losses of fireproofing material that could indicate cracking at the complete joint penetration weld (damage types G3, Section 5.3.1; C1, C3 and C4 Section 2.2.2; W2, W3, W4, Section 2.2.3).
- ☑ Observe the condition of the fireproofing at the beam web, in the vicinity of the connection from the beam web to the column for loosened, cracked or spalled material indicative of potential damage to shear tabs (damage types S1 through S5, Section 2.2.4).
- ☑ Observe the condition of the fireproofing at the column panel zone for cracks, loosened or spalled material, indicative of damage to the panel zone or continuity plates (damage types P1 through P8, Section 2.2.5).

☑ Observe the flanges of the column at and beneath the joint with the beam flange for loosened, spalled or cracked material, indicative of buckled or yielded sections (damage type C6, Section 2.2.2).

4.4.1.1.2 Detailed Inspections

When an initial inspection conducted in accordance with Section 4.4.1.1.1 indicates the presence or likely presence of damage in a connection, the more detailed inspections and observations indicated in the checklist below should be performed for that connection. Prior to performing the inspection, remove fireproofing (see Section 3.3.3), as indicated in Figure 4-2. Note that inspection of the top surface of the top flange of the beam and the adjacent column flange will typically be obscured by the diaphragm. If inspections from the exposed bottom surface of the top beam flange indicate a potential for damage to be present, then the diaphragm should be locally removed to allow a more thorough inspection.

☑ Observe the beam framing into the connection for trueness to line, and potential indications of lateral flexural-torsion buckling (damage type G8, Section 2.2.1).

☑ Observe condition of fireproofing along the beam within one beam depth of the column for cracking or spalling of the fireproofing material along the beam surface, indicating potential yielding or buckling of the beam flanges (damage types G1, G2, Section 2.2.1).

☑ Observe the top and bottom surface of the exposed beam bottom flange and the bottom surface of the top flange for fractures (damage types G3, G4, Section 2.2.1).

☑ Observe the exposed surfaces of the complete joint penetration welds between the beam top and bottom flanges and column for fractures (damage types W2, W3, W4 Section 2.2.3).

☑ Observe the exposed surfaces of the column flanges for fractures (damage types C1, C2, C3, Section 2.2.2).

☑ Observe the condition at the bottom of weld backing on the top and bottom flanges. If gaps are present, insert feeler gauge to detect potential damage (damage types C1, C4, C5, Section 2.2.2). See Chapter 2 for additional information.

☑ Observe the condition of the shear tab for deformation of the tab, fractures or tearing of the welds and loosening or breaking of the bolts (damage types S1 through S5, Section 2.2.4).

☑ Observe the column panel zone for cracks, or distortion (damage types P1 through P8, Section 2.2.5).

☑ Observe the exposed flanges of the column for distortion (damage type C6, Section 2.2.2).

4.4.1.2 Damage Characterization

Characterize the observed damage at each of the inspected connections by assigning a connection damage index, d_j, obtained either from Table 4-1a or Table 4-1b. Table 4-1a presents damage indices for individual classes of damage. Table 4-1b provides indices for the more common combinations of damage and also provides a method for developing indices for other combinations.

Commentary: The connection damage indices provided in Table 4-1 (ranging from 0 to 4) represent judgmental estimates of the relative severity of the various types of damage. Damage severity is judged in two basic respects, the impact of the damage on the connection's ability to participate in the frame's global stability and lateral resistance, and the impact of the damage on the local gravity load carrying capacity of the individual connection. An index of 0 indicates no impact on either global or local stability while an index of 4 indicates very severe impact.

When initially developed, in support of the publication of FEMA-267, these connection damage indices ranged from 0 to 10 and were conceptualized as estimates of the connection's lost capacity to reliably participate in the building's lateral-force-resisting system in future earthquakes (with 0 indicating no loss of capacity and 10 indicating a complete loss of capacity). However, due to the limited data available, no direct correlation between these damage indices and the actual residual strength and stiffness of a damaged connection was possible. In these Recommended Criteria, the damage indices have been simplified, to remove the apparent accuracy implied by a scale ranging from 0 to 10. It should be noted that although the damage indices do not correlate directly with the loss of strength or stiffness experienced by a connection, they do provide a convenient qualitative measure of the extent of damage that various connections in a building have experienced.

Analyses conducted to explore the effect of connection fractures on the global behavior of frames have revealed that the loss of a single flange connection (top or bottom) at each joint, consistently throughout a moment-resisting frame results in only a modest increase in the vulnerability of the structure to developing P-delta instability and collapse. However, if a number of connections develop fractures at both flanges of the beam-column connection, significant increase in vulnerability occurs. As a result of this, damage that results in the loss of effectiveness of a single flange joint to transfer flexural tension stress is assigned a relatively modest damage index of 2, if not combined with other types of damage at the connection. Damage types that result in an inability of both flanges to transfer flexural demands are assigned a high damage index, of 4, as are types of damage that could potentially result in impairment of a column or beam's ability to continue to carry gravity loads. Other types of damage are assigned proportionately lower damage indices, depending on the apparent effect of this damage on structural stability and load carrying capacity.

Table 4-1a Connection Damage Indices

Type	Location	Description[1]	Index d_j
G1	Girder	Buckled Flange	2
G2	Girder	Yielded Flange	0
G3	Girder	Top or Bottom Flange fracture in Heat Affected Zone (HAZ)	2
G4	Girder	Top or Bottom Flange fracture outside HAZ	2
G5	Girder	Not used	-
G6	Girder	Yielding or Buckling of Web	2
G7	Girder	Fracture of Web	4
G8	Girder	Lateral-torsional Buckling	2
C1	Column	Minor column flange surface crack	1
C2	Column	Flange tear-out or divot[4]	2
C3	Column	Full or partial flange crack outside HAZ	3
C4	Column	Full or partial flange crack in HAZ	3
C5	Column	Lamellar flange tearing	2
C6	Column	Buckled Flange	3
C7	Column	Fractured column splice	4
W2	CJP weld	Crack through weld metal exceeding t/4	2
W3	CJP weld	Fracture at girder interface	2
W4	CJP weld	Fracture at column interface	2
S1	Shear tab	Partial crack at weld to column	2
S2	Shear tab	Crack in Supplemental Weld (beam flanges sound)	1
S3	Shear tab	Fracture through tab at bolt holes	4
S4	Shear tab	Yielding or buckling of tab	3
S5	Shear tab	Damaged, or missing bolts[3]	2
S6	Shear tab	Full length fracture of weld to column	4
P1	Panel Zone	Fracture, buckle, or yield of continuity plate[2]	1
P2	Panel Zone	Fracture of continuity plate welds[2]	1
P3	Panel Zone	Yielding or ductile deformation of web[2]	0
P4	Panel Zone	Fracture of doubler plate welds[2]	1
P5	Panel Zone	Partial depth fracture in doubler plate[2]	1
P6	Panel Zone	Partial depth fracture in web[2]	3
P7	Panel Zone	Full (or near full) depth fracture in web or doubler plate[2]	4
P8	Panel Zone	Web buckling[2]	2
P9	Panel Zone	Fully severed column	4

Notes To Table 4-1a:

1. See Figures 2-2 through 2-6 for illustrations of these types of damage.
2. Panel zone damage should be reflected in the damage index for all moment connections that are attached to the damaged panel zone within the assembly.
3. Missing or loose bolts may be a result of construction error rather than damage. The condition of the metal around the bolt holes, and the presence of fireproofing or other material in the holes can provide clues to this. Where it is determined that construction error is the cause, the condition should be corrected and a damage index of “0” assigned.
4. Damage type C2 is very similar to type W3, the primary differentiation being the depth of the concave fracture surface into the column flange. If the fracture surface is relatively shallow within the column flange and does not result in the removal of substantial column flange material, type C2 fractures may be classified as type W3 and the corresponding damage index utilized.

Table 4-1b Connection Damage Indices for Common Damage Combinations

Girder, Column or Weld Damage	Shear Tab Damage	Damage Index	Girder, Column or Weld Damage	Shear Tab Damage	Damage Index
G3 or G4 Fracture of Girder Top or Bottom Flange	S1	4	C5 Column Flange Tearing parallel to rolling direction	S1	4
	S2	3		S2	3
	S3	4		S3	4
	S4	3		S4	3
	S5	4		S5	4
	S6	4		S6	4
C2 Column Flange Tear-out or Divot	S1	4	W2, W3, or W4 CJP Weld Fracture	S1	4
	S2	3		S2	3
	S3	4		S3	4
	S4	3		S4	3
	S5	4		S5	4
	S6	4		S6	4
C3 or C4 Column Flange Crack	S1	4			
	S2	4			
	S3	4			
	S4	4			
	S5	4			
	S6	4			

Note: For other combinations of damage, indices are obtained as follows:

a. Two types of damage with individual $d_i \le 1$, Combination $d_i = 2$
b. Two types of damage with both individual $d_i > 1$, Combination $d_i = 4$.
c. Two types of damage with only one individual $d_i \ge 2$, Combination d_i = largest individual $d_i + 1 \le 4$.
d. Three types of damage with all $d_i \le 1$, Combination $d_i = 3$.
e. Three types of damage with any $d_i \ge 2$, Combination $d_i = 4$.
f. More than three types of damage, Combination $d_i = 4$.

4.4.1.3 Determine Damage Index at Each Floor for Each Direction of Response

Divide the connections in the building into two individual groups. Each group of connections should consist of those connections, which are part of frames that provide primary lateral-force resistance for the structure in one of two orthogonal building directions. For example, one group of connections will typically consist of all those connections located in frames that provide north-south lateral resistance, while the second group will be all those connections located in frames that provide east-west lateral resistance.

For each group of connections, determine the value of the damage index for the group at each floor, from the equation:

$$D_i = \frac{1}{n}\sum_{j=1}^{n}\frac{d_j}{4} \tag{4-1}$$

where D_i is the floor damage index at floor "i" for the group,
n is the number of connections in the group at floor level "i," and.
d_j is the damage index, from Tables 4-1a and 4-1b, for the j^{th} connection in the group at that floor.

4.4.1.4 Determine Maximum Floor Damage Index

Determine the maximum floor damage index for the building, D_{max}, consisting of the largest of the D_i values calculated in accordance with the Section 4.4.1.3.

4.4.1.5 Determine Recommended Recovery Strategies for the Building

Recommended postearthquake recovery strategies are as indicated in Table 4-2, based on the maximum damage index, D_{max}, determined in accordance with Section 4.4.1.4.

Table 4-2 Recommended Repair and Modification Strategies

Values of D_{max}[2]	Recommended Strategy	Note
$0 < D_{max} \le 0.5$	Repair all connections discovered to have $d_j \ge 1$	
$D_{max} > 0.5$	A potentially unsafe condition should be deemed to exist unless a Level 2 evaluation is performed and indicates that acceptable confidence is provided with regard to the lateral stability of the structure. Notify the building owner of the potentially unsafe condition. Inspect all connections in the building. Repair all connections with $d_j \ge 1$.	1

Notes to Table 4-2:
1. The determination that an unsafe condition may exist should be maintained until either:
   a. Level 2 analyses indicate that a dangerous condition does not exist, or
   b. recommended repairs are completed for all connections having $d_j \ge 2$.
2. See Section 4.4.1.4

Commentary: Recommendations to close a damaged building to occupancy should not be made lightly, as such decisions will have substantial economic impact, both on the building owner and tenants. A building should be closed to occupancy whenever, in the judgment of the structural engineer, damage is such that the building no longer has adequate lateral-force-resisting capacity to withstand additional strong ground shaking, or if gravity-load-carrying elements of the structure appear to be unstable.

When a building has been damaged, it is recommended that in addition to repair, consideration also be given to upgrade. This is particularly the case when damage is severe (computed D_{max} exceeding 0.5) and the estimated ground shaking that caused the damage is substantially less than that which would be used to design the building under currently applicable building codes. In such conditions, it can reasonably be expected that the building would not be able to reliably resist the levels of ground shaking that could credibly occur at the building site. In addition to these basic safety considerations, there are also economic reasons to consider upgrading a building concurrently with damage *repair. A significant portion of structural upgrade costs are a result of the need to move occupants out of construction areas as well as the need to selectively demolish and replace building finishes and utilities in areas affected by the work. Often the magnitude of such costs required to implement repairs are comparable to those that would be incurred in performing an upgrade, permitting improved future performance to be attained with relatively little increment in construction cost. Structural repair, by itself, will not typically result in substantial reduction in the vulnerability of the structure to damage from future earthquakes, while selected connection upgrade has the potential to greatly reduce future damage and losses.*

A companion document to this publication, FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings, provides guidelines for assessing the probable performance of steel moment-frame buildings and for designing upgrades to improve this performance.

4.4.2 Method 2 – Inspection of a Sample of Connections

The following eight-step procedure may be used to determine the condition of the structure and to develop occupancy, repair and modification strategies when only a sample of the building's critical connections are inspected:

Step 1: Categorize the moment-resisting connections in the building into two or more groups comprising connections expected to have similar probabilities of being damaged.

Complete steps 2 through 7 below, for each group of connections.

Step 2: Determine the minimum number of connections in each group that should be inspected and select the specific sample of connections to be inspected.

Step 3: Inspect the selected sample of connections using the procedures of Section 4.4.1 and determine connection damage indices, d_j, for each inspected connection.

Step 4: If inspected connections are found to be seriously damaged, perform additional inspections of connections adjacent to the damaged connections.

Step 5: Determine the average damage index d_{avg} for connections in each group, and then the average damage index at a typical floor for each group.

Step 6: Given the average damage index for connections in each group, determine the probability P that, had all connections been inspected, the connection damage index for any group, at a floor level, would exceed 0.50, and determine the probable maximum floor damage index, D_{max}.

Step 7: Based on the calculated damage indices and statistics, determine appropriate occupancy and structural repair strategies. If deemed appropriate, the structural engineer may conduct detailed structural analyses of the building in the as-damaged state, to obtain improved understanding of its residual condition and to confirm that the recommended strategies are appropriate or to suggest alternative strategies. However, for such analyses to be meaningful, full inspections of all connections are required. Procedures for such detailed evaluations are contained in Chapter 5.

Step 8: Report the results of the inspection and evaluation process to the building official and building owner.

Sections 4.4.2.1 through 4.4.2.7 indicate, in detail, how these steps should be performed.

Commentary: Following an earthquake, structural engineers and technicians qualified to perform these evaluations may be in high demand. Prudent owners may want to consider having an investigation plan already developed (Steps 1 and 2) before an earthquake occurs, and to have an agreement with appropriate structural engineering and inspection professionals and organizations to give priority to inspecting their buildings rapidly following the occurrence of an earthquake.

4.4.2.1 Evaluation Step 1 — Categorize Connections by Groups

The welded moment-resisting connections participating in the lateral-force-resisting system for the building are to be categorized into a series of connection groups. Each group consists of connections expected to behave in a similar manner (as an example, a group may consist of all those connections that are highly stressed by lateral forces applied in a given direction). As a minimum, two groups of connections should be defined - each group consisting of connections that primarily resist lateral movement in one of two orthogonal directions. It may be appropriate to define additional groups to account for unique conditions, including building configuration, construction quality, member size, grade of steel, or other factors that are likely to result in connection behavior substantially different from other connections in the building. Each connection in the building, including connections at the roof level, should be uniquely assigned to one of the groups, and the total number of connections in each group determined.

In buildings that have significant torsional irregularity, it may be advisable to define at least four groups—one group in each orthogonal direction on each side of an assumed center of resistance.

4.4.2.2 Step 2 — Select Samples of Connections for Inspection

Assign a unique identifier to each connection within each group. Consecutive integer identifiers are convenient to some of the methods employed in this Section.

For each group of connections, select a representative sample for inspection in accordance with either of Methods A or B, below. If the evaluation is being performed to satisfy a requirement imposed by the building official, a letter indicating the composition of the groups, and the specific connections to be inspected should be submitted to the building official prior to the initiation of inspection. The owner or structural engineer may at any time in the investigation process elect to investigate more connections than required by the selected method. However, the additional connections inspected may not be included in the calculation of damage statistics under Step 4 (Section 4.4.2.4) unless they are selected in adherence to the rules laid out for the original sample selection, given below.

Commentary: The purpose of inspection plan submittal prior to the performance of inspections is to prevent a structural engineer, or owner, from performing (1) a greater number of inspections and (2) reporting data only on those which provide a favorable economic result with regard to building disposition. The building official need not perform any action with regard to this submittal other than to file it for later reference at the time the structural engineer's evaluation report is filed. During the inspection process, it may be decided to inspect additional connections to those originally selected as part of the sample. While additional inspections can be made at any time, the results of these additional inspections should not be included in the calculation of the damage statistics, in Step 5, as their distribution may upset the random nature of the original sample selection. If the additional connections are selected in a manner that preserves the distribution character of the original sample, they may be included in the calculation of the damage statistics in Step 5.

4.4.2.2.1 Method A — Random Selection

In this method, connections should be selected for inspection such that a statistically adequate, random sample is obtained. The minimum number of connections to be inspected for each group should be determined in accordance with Table 4-3. For groups containing a population of 100 connections or more, the sample size need not exceed 18, unless damage with $d_j \geq 1.0$ in accordance with Table 4-1a is found in the inspection of these 18 connections. In the event that such damage is found in this initial sample, the sample size shall be expanded to the full amount shown in Table 4-3, while retaining the random character of the selection.

The following limitations apply to the selection of specific connections:

1. Up to a maximum of 20% of the total connections in any sample may be pre-selected as those expected by rational assessment to be the most prone to damage. Acceptable criteria to select these connections could include:
   - Connections shown by a rational analysis to have the highest demand/capacity ratios or at locations experiencing the largest drift ratios.
   - Connections that adjoin significant structural irregularities and which therefore might be subjected to high localized demands. These include the following irregularities:
     - re-entrant corners
     - set-backs
     - soft or weak stories
     - torsional irregularities (connections at perimeter columns)

- diaphragm discontinuities

- Connections incorporating the largest size framing elements.

2. The balance of the sample should be selected randomly from the remaining connections in the group, except that up to 10% of the connections in the sample may be replaced by other connections in the group to which access may more conveniently be made.

For buildings designed and constructed following the 1994 Northridge earthquake, and conforming to the recommendations contained in Chapter 7 of *FEMA-267*, or conforming to the design recommendations for Special Moment Frames contained in the 1997 or later edition of *AISC Seismic Provisions*, the scope of inspection may be reduced to 1/2 the number of connections indicated in Table 4-3. If in the course of this reduced scope of inspection, significant structural damage is found (damage to any connection with a damage index $d_j \geq 1.0$ from Table 4-1 (a or b)), then full inspections should be performed, as for buildings with other types of connections.

Table 4-3 Minimum Sample Size for Connection Groups

Number of connections in Group[1]	Minimum number of connections to be inspected	Number of Connections in Group[1]	Minimum number of connections to be inspected
6	3	200	30
10	4	300	40
15	5	400	50
20	6	500	60
30	8	750	75
40	10	1000	100
50	12	1250	110
75	16	1500	125
100	20	2000	150

Note: 1. For other connection numbers use linear interpolation between values given, rounding up to the next highest integer.

Commentary: The number of connections needed to provide a statistically adequate sample depends on the total number of connections in the group, the amount of damage present in the building, and the amount of damage it is acceptable not to find. Assuming that damage is randomly distributed within a connection group, if no damage is found in a randomly selected inspection sample of 18 connections, this indicates at least a 95% level of confidence that less than 15% of the connections in the group have been damaged for a group of any size. For smaller groups of connections, smaller samples will provide similar levels of confidence. However, if damage is present within the sample of connections selected for a group, then a larger sample size will be required to assure with confidence that the percentage of connections within the group that have been damaged is within a tolerable level. When implemented in the inspection procedures contained in these recommended criteria, the inspection sample sizes *specified in Table 4-3 will produce greater than a 95% level of confidence of finding damage in groups of connections with 20% or more of the connections damaged. Military standard, MIL-STD-105D can be used to determine appropriate sample sizes to obtain other levels of confidence or to obtain similar levels of confidence for reduced levels of damage, if desired.*

If relatively few connections within a group are inspected, the standard deviation for the computed damage index will be large. This may result in prediction of excessive damage when such damage does not actually exist. The structural engineer may elect to investigate more connections than the minimum indicated in order to reduce the standard deviation of the sample and more accurately estimate the total damage to the structure. These additional inspections may be performed at any time in the investigative process. However, care should be taken to preserve the random characteristics of the sample, so that results are not biased either by selection of connections in unusually heavy (or lightly) damaged areas of the structure.

It is recognized that in many cases the structural engineer may wish to pre-select those connections believed to be particularly vulnerable. However, unless these pre-selected connections are fairly well geometrically distributed, a number that is more than about 10% of the total sample size will begin to erode the validity of the assumption of random selection of the sample. If the structural engineer has a compelling reason for believing that certain connections are most likely to be damaged, and that more than 10% should be pre-selected on this basis, either the alternative approach of Method B should be used, or the connections that are believed to have particular vulnerability should be classified as an independent group, and treated accordingly.

It is also recognized that there is often a practical incentive to select connections that are in specific unoccupied or more accessible areas. It is suggested that no more than 10% of the total sample be composed of connections pre-selected for this reason. These connections, rather than having a higher disposition for damage, might well have a lower than average tendency to be damaged. An excessive number of this type of pre-selected connection would quickly invalidate the basic assumption of random selection. It is also recognized that during the inspection process conditions will be discovered that make it impractical to inspect a particular connection, e.g., the architectural finishes are more expensive to remove and replace than in other areas, or a particular tenant is unwilling to have their space disturbed. However, as discussed above, not more than 10% of the total connections inspected should be selected based on convenience.

There are a number of methods available for determining the randomly selected portion of the sample. To do this, each connection in the group (excluding pre-selected connections) should be assigned a consecutive integer *identifier. The sample may then be selected with the use of computer spread sheet programs (many of which have a routine for generation of random integers between specified limits), published lists of random numbers, or by drawing of lots.*

4.4.2.2.2 Method B - Analytical Selection

In this method, connections should be selected for inspection in accordance with the following criteria:

1. The minimum number of connections within the group to be inspected shall be indicated in Table 4-3. As with Method A, if a randomly selected sample of 18 connections from a group is inspected, and found to contain no damage, no further inspections of connections from that group are required.
2. Up to 50% of the connections may be selected based on the results of rational analysis indicating those connections most likely to be damaged.
3. The remaining connections in the group to be inspected are selected such that the sample contains connections distributed throughout the building, including upper, middle and lower stories. The rules of Section 4.4.2.2.1 should be followed in a general way.

Prior to initiation of the inspections, the rational analysis and list of connections to be inspected should be subjected to a qualified independent third party review. The peer review should consider the basis for the analysis, consistency of the assumptions employed, and assure that overall, the resulting list of connections to be inspected provides an appropriate sampling of the building's connections.

During the inspection process, up to 10% of the connections in the sample may be replaced by other connections to which access may more conveniently be made. Substitution for more than 10% of the connection sample may be made provided that the independent third party reviewer concurs with the adequacy of the resulting revised sample.

Commentary: In analyses conducted of damaged buildings, there has been a generally poor correlation of the locations of damage and the locations of highest demand predicted by analysis. This is primarily attributed to the fact that the propensity for a fracture to initiate in a connection is closely related to the workmanship present in the welded joints, which tends to be a randomly distributed quantity. Moreover, typical analysis methods do not capture the complex nonlinear stress state that occurs in actual buildings. However, there has been some correlation. Analysis is a powerful tool to assist the structural engineer in understanding the expected behavior of a structure, damaged or undamaged. The specific analysis procedure used should be tailored to the individual characteristics of the building. It should include consideration of all building elements that are expected to participate in the building's seismic response, including, if appropriate, elements not generally considered to be part of the lateral-force-resisting system. The ground motion characteristics used for *the analysis should not be less than that required by the building code for new construction, and to the extent practical, should contain the spectral characteristics of the actual ground motion experienced at the site. Qualified independent review is recommended to assure that there is careful consideration of the basis for the selection of the connections to be inspected and that a representative sample is obtained.*

4.4.2.3 Step 3 — Inspect the Selected Samples of Connections

4.4.2.3.1 Inspection

All moment-resisting connections within each sample are to be visually inspected as indicated in Section 4.4.1.1.2. Where visual inspection indicates the potential for damage that is not clearly visible, further investigation using nondestructive testing should be performed. Characterize all damage discovered by visual inspection and nondestructive testing for each inspected connection as described in Section 4.4.1.1 An individual data sheet (Appendix C) should be filled out for each connection inspection, recording its location and conditions observed. In addition, plan and elevation sketches for the building's structural system should be developed and conditions of observed damage recorded on these sketches.

Commentary: The largest concentration of reported damage following the 1994 Northridge earthquake occurred at the welded joint between the bottom girder flange and column, or in the immediate vicinity of this joint. To a much lesser extent, damage was also observed in some buildings at the joint between the top girder flange and column. If damage at either of these locations is substantial, then damage is also commonly found in the panel zone or shear tab areas.

For a Level 1 evaluation, these Recommended Criteria permit inspection, by visual means, of all of the potential damage areas for a representative sample of the connections in the building. Most of the damage reported in buildings following the 1994 Northridge earthquake consisted of fractures that initiated at the roots of complete joint penetration welds joining beam flanges to column flanges, and which then propagated through the weld or base metal, leaving a trace that was generally detectable by careful visual examination. Careful visual examination requires removal of all obscuring finishes and fireproofing, and examination from a range of a few inches. Most fractures are visually evident. However, some fractures are rather obscure since deformation of the building following the onset of fracture can tend to close up the cracks. In some cases, it may be appropriate to use magnifying glasses or other means to verify the presence of fractures. If doubt exists as to whether a surface indication is really a fracture, magnetic particle testing and other forms of nondestructive examination can be used to confirm the presence of a fracture. The surface must be carefully cleaned prior to testing.

Some types of fractures extend from the root of the beam flange weld into the column flange and may not be detectable by visual examination. Such fractures, *typified by types C3 and C5 (see Section 2.2.2) can only be detected by removal of the backing, or by nondestructive testing. Often, when such fractures are present, a readily visible gap can be detected between the base of the backing and the column flange. Where such indications are present, a feeler gauge should be inserted into the gap to determine its depth. If the feeler gauge can be inserted to a depth that exceeds the backing thickness, a fracture should be assumed to be present. Removal of the backing, or nondestructive testing, or both, will be required to confirm the extent of the crack.*

The practice of inspecting a small sample of the total connections present in a building, in order to infer the probable overall condition of the structure is consistent with that followed by most engineers in the Los Angeles area, following the 1994 Northridge earthquake. However, the typical practice following that event included the extensive use of ultrasonic testing (UT) in addition to visual inspection. This UT revealed a number of apparent conditions of damage at the roots of the full penetration welds between beam and column flanges. These conditions, which were widespread, were typically reported by testing agencies and engineers as damage. This practice was encouraged by the FEMA-267 guidelines, which classified weld root indications as type W1 "damage".

As a result of limitations in the accuracy of ultrasonic testing techniques it was often found upon removal of weld backing material to allow repair of these root conditions, that the actual condition of the weld root was significantly different from that indicated by UT. Sometimes, no flaws at all were found at the roots of welds reported to have W1 conditions while in other cases, the size and location of actual flaws were found to be significantly different from that indicated by the UT.

In the time since, substantial evidence has been gathered that suggests that many of the W1 conditions reported following the 1994 Northridge earthquake were not damage, but rather latent construction defects, including slag inclusions and lack of fusion that had never been detected during the original construction quality control and quality assurance processes. For these reasons, these Recommended Criteria have de-emphasized, relative to the recommendations of FEMA-267, the importance of employing NDT in the postearthquake inspection process.

4.4.2.3.2 Damage Characterization

The observed damage at each of the inspected connections is characterized by assigning a connection damage index, d_j obtained either from Table 4-1a or Table 4-1b, of Section 4.4.1.2. Table 4-1a presents damage indices for individual classes of damage. Table 4-1b provides combined indices for the more common combinations of damage and a rule for combining indices where a connection has more than one type of damage. Refer to Chapter 2 for descriptions of the various damage types and to Section 4.4.1.2 for commentary relative to these damage indices.

4.4.2.4 Step 4 — Inspect Connections Adjacent to Damaged Connections

Regardless of the method used to select the connection sample, perform additional inspections of moment-resisting connections near connections with significant damage as follows:

- When a connection is determined to have a damage index within the range $1 \leq d_j \leq 2$, inspect all of the moment-resisting connections in that line of framing on both sides of the affected column and of the column(s) adjacent to the affected column at that floor level and on the affected column at the floor level immediately above and below the damaged connection (See Figure 4-3). Also inspect any connections for beams framing into the column in the transverse direction at that floor level, at the damaged connection.
- When a connection is determined to have a damage index $d_j \geq 3$, inspect all of the moment-resisting connections in that line of framing on both sides of the affected column and of the column(s) adjacent to the affected column at that floor level and on the affected column at the two floor levels immediately above and below the damaged connection (See Figure 4-4). Also inspect any connections for beams framing into the column in the transverse direction at that floor level at the damaged connection.

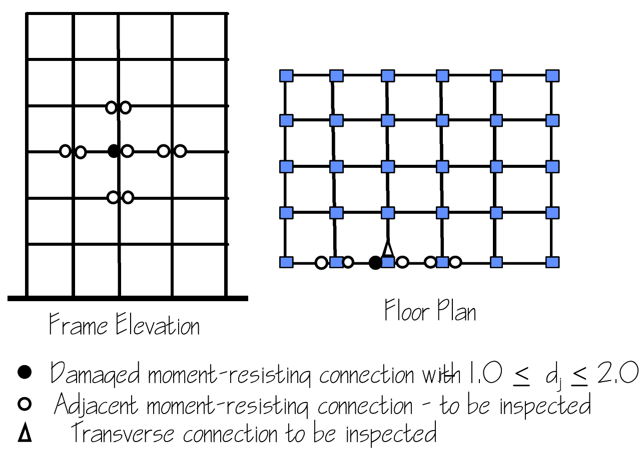

Figure 4-3 Inspection of Connections Adjacent to Damaged Connection ($1 \leq d_j \leq 2$)

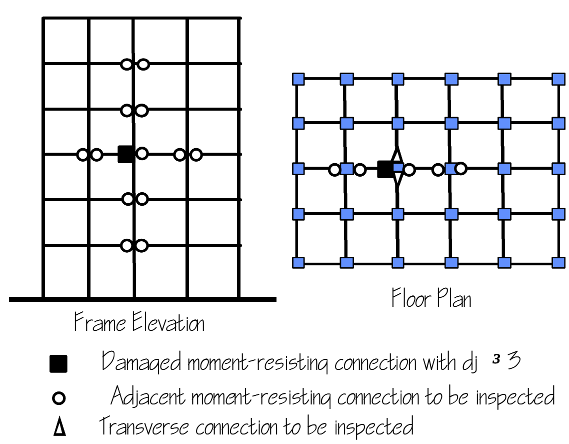

Figure 4-4 Inspection of Connections Adjacent to Damaged Connection ($d_j \geq 3$)

Assign damage indices d_j per Tables 4-1a and 4-1b to each additional connection inspected. If significant damage is found in these additional connections ($d_j \geq 1$), then inspect the connections near these additional connections, as indicated above. Continue this process, until one of the following conditions occurs:

- The additional connection inspections do not themselves trigger more inspections, or
- All connections in the group have been inspected. In this case, proceed with the evaluation of damage indices for this group in accordance with the guidelines of Section 4.4.1.3.

The results of these added connection inspections, performed in this step, are not included in the calculation of average damage index d_{avg} in Section 4.4.2.6, but are included in the calculation of the maximum likely floor damage index D_{max} and the probability of excessive damage P in Section 4.4.2.7.

4.4.2.5 Step 5 — Determine Damage Statistics for Each Group

For each group of connections, determine the estimated average value of the damage index for the group d_{avg} and its standard deviation σ from the equations:

$$d_{avg} = \frac{1}{n}\sum_{j=1}^{n} d_j \qquad (4\text{-}2)$$

$$s = \sqrt{\frac{1}{n-1}\sum_{j=1}^{n}\left(d_j - d_{avg}\right)^2} \qquad (4\text{-}3)$$

where n is the number of connections in the original sample selected for inspection under Step 2 (Section 4.4.2.2), and

d_j is the damage index, from Tables 4-1a and 4-1b, for the j^{th} inspected connection in the original sample

The additional connections selected using the procedure of Section 4.4.2.4 (Step 4) are not included in the above calculation.

4.4.2.6 Step 6 — Determine the Probability that the Connections in a Group at a Floor Level Sustained Excessive Damage

In this procedure, the probable maximum floor damage index at a floor D_{max} is estimated from the damage indices determined for all of the connections actually inspected, including those additional connections inspected in accordance with the requirements of Section 4.4.2.4. In addition, the probability P that had all connections in the building been inspected, D_{max} would exceed a value of 0.50, is determined.

First determine the average floor damage index D and its standard deviation S from the equations:

$$D = \frac{d_{avg}}{4} \tag{4-4}$$

$$S = \frac{s}{4\sqrt{k}} \tag{4-5}$$

where d_{avg} is the average connection damage index, computed from Equation 4-2,

σ is the standard deviation of the connection damage index, computed from Equation 4-3,

k is the total number of connections (both inspected and not inspected) in the group at a typical floor.

Second, determine the probability P that the set of connections within the group at any floor has a floor damage index that is greater than or equal to 0.50. This may be done by using the parameters D and S to calculate a factor b, which represents the number of multiples of the standard deviation of a normal distribution above the mean that would be required to exceed 1/2. The factor b is calculated from the equation:

$$b = \left(\tfrac{1}{2} - D \right) / S \tag{4-6}$$

Using the value of b calculated from equation 4-6, determine P_f, from Table 4-4. P_f is the probability that if all connections had been inspected, the cumulative damage index at any floor would have been found to exceed 0.50. If the probability P_f is high, this strongly suggests the possibility that there has been a significant reduction in seismic resisting capacity.

Next, determine the probability P that if all connections within the group had been inspected, the connections within the group on at least one floor (out of q total floors in the group) would have been found to have a cumulative damage index of 0.50 or more from the equation:

$$P = 1 - (1 - P_f)^q \tag{4-7}$$

Finally, for each floor i in the group for which an inspection has been performed, determine the floor damage index D_i from the equation:

$$D_i = \frac{(k_i - m_i)\, d_{avg}}{4k_i} + \left(\frac{1}{k_i} \right) \sum_{j=1}^{m_i} \frac{d_j}{4} \tag{4-8}$$

where: k_i is the total number of connections in the group at floor i

m_i is the number of inspected connections in the group at floor i including the additional connections inspected under Step 4

Take D_{max} as the largest of the D_i values calculated for each floor of the group.

Table 4-4 P_f as a Function of Parameter *b*

b	P_f	*b*	P_f
-1.2816	0.90	1.2265	0.11
-0.8416	0.80	1.2816	0.10
-0.5244	0.70	1.3408	0.09
-0.2533	0.60	1.4051	0.08
0.0000	0.50	1.4395	0.075
0.2533	0.40	1.4758	0.07
0.5244	0.30	1.5548	0.06
0.8416	0.20	1.6449	0.05
0.8779	0.19	1.7507	0.04
0.9154	0.18	1.8808	0.03
0.9542	0.17	1.9600	0.025
0.9945	0.16	2.0537	0.02
1.0364	0.15	2.1701	0.015
1.0803	0.14	2.3263	0.01
1.1264	0.13	3.0962	0.001
1.1750	0.12	3.7190	0.0001

Note: Intermediate values of P_f may be determined by linear interpolation

Commentary: The criterion for damage evaluation used in these Recommended Criteria is to assume that a cumulative damage index of 0.50 marks the threshold at which a structure may become dangerous. Such a damage index could correspond to cases where 1/2 of the connections at a floor level have been severely damaged, or cases where all of the connections at a floor level have experienced moderate damage, or some combination of these, and therefore represents a reasonable point at which to begin serious consideration of a building's residual ability to withstand additional loads.

Although the actual form of the distribution of the probability of damage for an individual connection is not known, as the number of connections increases, the distribution of damage for a structure tends to a normal distribution, regardless of the form of the distribution for individual connections, by the Central Limit Theorem. Therefore, the probability that a damage index of 0.50 has been exceeded at a floor, in a group with k connections, may be approximated by determining how many multiples b times the standard deviation S, when added to the mean damage index D, equals 1/2. Or, in equation form :

$$D + bS = 0.50 \tag{4-9}$$

Solution of this equation for the multiplier b results in the required relationship of equation 4-6.

In spite of the somewhat arbitrary nature of the 0.50 damage index criterion and the judgmental nature of the suggested way of testing whether that criteria has been exceeded, it is believed that the results of these procedures will lead to reasonable conclusions in most cases. However, it is always the prerogative of *the responsible structural engineer to apply other rational techniques, such as direct analyses of the remaining structural strength, stiffness, and deformation capacity as a verification of the conclusions provided by these procedures. Particularly in anomalous or marginal cases, such additional checks based on engineering judgment are strongly encouraged.*

4.4.2.7 Step 7--Determine Recommended Recovery Strategies for the Building

Recommended postearthquake recovery strategies are as indicated in Table 4-5, based on the calculated damage indices and statistics determined in the previous steps.

Table 4-5 Recommended Condition Designation and Repair Strategies

Values of D_{max} and P	Condition Designation	Recommended Strategy (Cumulative)	Note
$P \leq 10\%$ and $D_{max} \leq 0.2$	Green - 3	Repair all connections discovered to have $d_j \geq 1$	1,2
$10\% < P \leq 25\%$ or $0.2 < D_{max} \leq 0.5$	Green - 3	Inspect all connections in the group. Repair all connections with $d_j \geq 1$	1,2
$P > 25\%$ or $D_{max} > 0.5$	Red – 2	A potentially unsafe condition should be deemed to exist unless a level 2 evaluation is performed and indicates that acceptable confidence is provided with regard to the lateral stability of the structure. Notify the building owner of the potentially unsafe condition. Inspect all connections in the building. Repair all connections with $d_j \geq 1$. Consider structural upgrade.	3

Notes to Table 4-5:

1. Includes damage discovered either as part of Step 2 or Step 3.
2. If all of the discovered damage is relatively minor (d_j <1), at the discretion of the engineer, this need not be repaired. However, if some of the discovered damage is significant ($d_j \geq 1$), all of the damage should be repaired.
3. The determination that an unsafe condition may exist should continue until either:
   a. full inspection reveals that the gravity system is not compromised, and that the damage index at any floor does not exceed 0.50, or
   b. level 2 analyses indicate that a dangerous condition does not exist, or
   c. recommended repairs are completed for all connections having $d_j \geq 1$.

Commentary: Recommendations to close a damaged building to occupancy should not be made lightly, as such decisions will have substantial economic impact, both on the building owner and tenants. A building should be closed to occupancy whenever, in the judgment of the structural engineer, damage is such that the building no longer has adequate lateral-force-resisting capacity to withstand additional strong ground shaking, or if gravity-load-carrying elements of the structure appear to be unstable.

When a building has been damaged, it is recommended that, in addition to repair, consideration also be given to upgrade. Refer to the additional commentary in Section 4.4.1.5.

4.4.3 Additional Considerations

Regardless of the value calculated for the damage indices, in accordance with the previous sections, and the recommended actions of Section 4.4.2.7, the engineer should be alert for any damage condition that results in a substantial lessening of the ability of the structure as a whole, or of any part of the structure to resist gravity loads. Should such a condition be encountered, the engineer should inform those with legal standing to take appropriate steps either to limit entry to the affected portion(s) of the structure, or to ensure that adequate shoring is provide to prevent the onset of partial or total building collapse.

4.5 Evaluation Report

Upon completion of a detailed evaluation, the responsible structural engineer should prepare a written evaluation report and submit it to the person requesting the evaluation, as well as any other parties required by law to receive such a report. In particular, the building official should be notified whenever a hazardous condition is determined to exist. The report should directly, or by attached references, document the inspection program that was performed, and provide an interpretation of the results of the inspection program and a general recommendation as to appropriate repair and occupancy strategies. The report should include but not be limited to the following items:

- Building address
- A narrative description of the building, indicating plan dimensions, number of stories, total square feet, occupancy, the type and location of lateral-force-resisting elements. Include a description of the grade of steel specified for beams and columns and, if known, the type of welding (for example, shielded metal are welding or flux-cored arc welding) present. Indicate if moment connections are provided with continuity plates. The narrative description should be supplemented with sketches (plans and evaluations) as necessary to provide a clear understanding of pertinent details of the building's construction. The description should include an indication of any structural irregularities as defined in the Building Code.
- A description of nonstructural damage observed in the building, especially as relates to evidence of the drift or shaking severity experienced by the structure.
- If a letter was submitted to the building official before the inspection process was initiated that indicated how the connections were to be divided into groups and indicating the specific connections to be inspected, a copy of this letter should be included.
- A description of the inspection and evaluation procedures used, including the signed inspection forms for each individual inspected connection.

- A description, including engineering sketches, of the observed damage to the structure as a whole (e.g., permanent drift) as well as at each connection, keyed to the damage types in Table 4-1a, photographs should be included for all connections with damage index $d_i \geq 1$.
- Calculations of d_{avg}, D_i, and D_{max} for each group, and if all connections in a group were not inspected, P_f and P.
- A summary of the recommended corrective actions (repair and modification measures) and any recommendations on occupancy restrictions.

The report should include identification of any potentially hazardous conditions which were observed, including corrosion, deterioration, earthquake damage, pre-existing rejectable conditions, and evidence of poor workmanship or deviations or alterations from the approved drawings. In addition, the report should include an assessment of the potential impacts of observed conditions on future structural performance and recommendations for remediation of any adverse conditions. The report should include the Field Inspection Reports of damaged connections, as an attachment, and should bear the seal of the structural engineer in charge of the evaluation.