A critical step in performance-based seismic design is the prediction and definition of earthquake performance using terms that are meaningful to building owners. Recently, economic impact, defined as the cost of repairing earthquake damage and the building downtime required to complete the repair work, has been adopted as a meaningful measure of building performance. To enable earthquake engineers to predict the economic impact of earthquake loading, models are required linking the engineering measures used traditionally to define building performance with damage, repair methods, economic loss and repair time.

The work presented here develops these models for older reinforced concrete beam-column joints. The results of previous research are used to develop empirical relationships between damage states and traditional engineering response measures, such as inter-story drift, joint deformation and number of loading cycles. The proposed damage states are characterized by parameters such as concrete crack width, extent of concrete spalling and yielding and buckling of reinforcement. The results of previous research and practical experience by engineers and contractors are used to define a series of repair methods that can be used to restore a damaged joint to its original condition. Each damage state is associated with a specific repair technique, and probabilistic models are developed to enable prediction required repair.

Keywords: beam-column joint, damage, repair, fragility curve.

1. INTRODUCTION

Research at the Pacific Earthquake Engineering Research Center (PEER) and elsewhere to advance performance-based earthquake engineering has resulted in an awareness by the earthquake engineering community of the needs to 1) define performance using terms that are understood by and of valuable to building owners and 2) employ a probabilistic framework that supports the propagation of uncertainty through the process. The PEER framework equation (http://peer.berkeley.edu):
Multiple approaches are appropriate for incorporating information about building response into Eq. 1. For example, a building-specific EDP, such as maximum roof drift, could be used to predict the damage state of the building, or component-specific EDPs could be used to predict the damage state of individual component, with the damage state of the building defined by the cumulative damage states of all of the components. It is generally accepted that the latter approach provides an opportunity to introduce more information and thereby reduce uncertainty. The models developed here linking EDPs with DVs for older joints support the latter approach. Specifically, the results of previous research and practical experience are used to develop probabilistic relationships linking EDPs with DVs for one type of structural component, older beam-column building joints.

2. EXPERIMENTAL DATA

A critical phase of this research effort was the identification of experimental data characterizing the progression of earthquake damage in older beam-column joints. The criteria used to choose laboratory test specimens, the characteristics of the experimental test specimens, and variation in test specimens that could be expected to affect damage progression are discussed in the following sections.

2.1.1. Criteria Used to Identify Appropriate Laboratory Test Specimens

Three criteria were used to identify experimental data sets for use in this study. First, only laboratory specimens representative of pre-1979 design were considered. Prior to 1971 the ACI design recommendations did not explicitly address joint design; “modern” design requirements were first introduced in 1983 and revised to the current requirements in 1995. A review of construction drawings for buildings designed prior to 1979 conducted by Mosier (2000) were used as a basis for defining design details for older joints. Table 1 lists Mosier’s statistics for critical joint design parameters. The joints test specimens used in the current study fell within the ranges observed by Mosier for joints designed prior to 1979.

<table>
<thead>
<tr>
<th>Design year</th>
<th>Volumetric Transverse Steel Ratio</th>
<th>Shear Stress Demand / f′c</th>
<th>Beam Bar Anchorag Length / d_b</th>
</tr>
</thead>
<tbody>
<tr>
<td>pre-1967</td>
<td>0.002, 0.000, 0.002</td>
<td>0.15, 0.06, 0.29</td>
<td>12, 14, 43</td>
</tr>
<tr>
<td>1967-1979</td>
<td>0.009, 0.000, 0.021</td>
<td>0.21, 0.09, 0.03</td>
<td>21, 12, 38</td>
</tr>
</tbody>
</table>

Second, only laboratory test specimens with the same basic configuration and load history were used. All of the specimens represented sub-assemblages from two-dimensional building frames and comprised the joint, beams framing into the joint and extending to mid-span, columns framing into the joint and extending to mid-height. Lateral loading was applied as a shear load the top of the column and reacted by shear loads at the base of the column and beam ends. If it is assumed that under earthquake loading beams and columns develop a point of contra-flexure at mid-span, then this load distribution is representative of earthquake loading. Simulated earthquake load was applied pseudo-statically by forcing the top of the column through a prescribed cyclic displacement history (relative to the beam ends and column base) consisting of one or more cycles to increasing maximum displacement demands. In some cases, a constant axial load was applied at the top of the column to represent gravity load.

Third, only test specimens for which sufficient data characterizing the progression of damage in the beam-column joint were used. While all experimental researchers provide data characterizing the load-displacement response of laboratory test specimens, the detail and consistency with which damage data are reported in the literature varied substantially. In many cases, the lack of sufficient damage data eliminated joint specimens for use in this study.

2.2 Experimental Data Used in the Study

A review of the literature resulted in five test programs and twenty-five test specimens that met the above criteria:
• Meinheit and Jirsa (1977) investigated the impact of joint transverse reinforcement on response. Data from one (MII) of the eleven specimens tested by Meinheit and Jirsa are used; sufficient data are provided for this specimen.
• Durrani and Wight (1982) also investigated joint transverse steel volume. The researchers concluded that joint damage is a function of joint shear stress demand, the number of layers of joint hoops, and the presence of transverse beams and slabs. Data from three specimens (X1, X2, and X3) are used.
• Pessiki et al. (1990) investigated the earthquake response of older building components, including joints. Pessiki et al. conclude that the joint failure mechanism depends on the amount of reinforcing steel within the joint and beam-bar anchorage lengths. Data from eight (P2 through P9) of the Pessiki test specimens are used.
• Joh et al. (1991a, 1991b) investigated the impact on earthquake response of 1) joint transverse reinforcement, 2) beam transverse reinforcement and 3) torsion due to beam eccentricity. They conclude that increasing the volume of transverse reinforcement results decreased bar slip, increased energy dissipation, and increased post-cracking joint stiffness. Three specimens from these studies (JXO-B8-LH, JXO-B1, and JXO-B2) are used.
• Walker, Lehman, and Stanton (2001) and Alire, Lehman, and Stanton (2002): evaluated the impact of joint shear stress and load history. These studies conclude that joints maintain strength and adequate stiffness when drift demand less than 1.5% and shear stress demand is less than $10\sqrt{f'c}$.

Data from all of the specimens are used.

Design details and loading data for these specimens are listed in Table 2.

### Table 2. Design Details and Load Data for Experimental Test Specimens

<table>
<thead>
<tr>
<th>Specimen Tag</th>
<th>Ratio of Beam to Column Width</th>
<th>Joint Cross Sectional Area (in²)</th>
<th>$f_c$ (psi)</th>
<th>Joint Shear Stress / $f_c$</th>
<th>Joint Trans. Steel Ratio (vol.)</th>
<th>Beam Bar Anchorage Length / $d_b$</th>
<th>Column Splice Above Joint</th>
<th>Column Axial Load / $(Ag*f'_c)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PEER 14</td>
<td>1.00</td>
<td>288</td>
<td>4606</td>
<td>0.16</td>
<td>0.0000</td>
<td>29</td>
<td>no</td>
<td>0.11</td>
</tr>
<tr>
<td>PEER 22</td>
<td>1.00</td>
<td>288</td>
<td>5570</td>
<td>0.20</td>
<td>0.0000</td>
<td>21</td>
<td>no</td>
<td>0.09</td>
</tr>
<tr>
<td>CD15 14</td>
<td>1.00</td>
<td>288</td>
<td>4322</td>
<td>0.18</td>
<td>0.0000</td>
<td>29</td>
<td>no</td>
<td>0.12</td>
</tr>
<tr>
<td>CD30 14</td>
<td>1.00</td>
<td>288</td>
<td>6171</td>
<td>0.14</td>
<td>0.0000</td>
<td>29</td>
<td>no</td>
<td>0.08</td>
</tr>
<tr>
<td>CD30 22</td>
<td>1.00</td>
<td>288</td>
<td>5533</td>
<td>0.21</td>
<td>0.0000</td>
<td>21</td>
<td>no</td>
<td>0.09</td>
</tr>
<tr>
<td>PADH 14</td>
<td>1.00</td>
<td>288</td>
<td>6218</td>
<td>0.15</td>
<td>0.0000</td>
<td>29</td>
<td>no</td>
<td>0.08</td>
</tr>
<tr>
<td>PADH 22</td>
<td>1.00</td>
<td>288</td>
<td>5259</td>
<td>0.22</td>
<td>0.0000</td>
<td>21</td>
<td>no</td>
<td>0.10</td>
</tr>
<tr>
<td>PEER 09</td>
<td>1.00</td>
<td>288</td>
<td>9500</td>
<td>0.13</td>
<td>0.0000</td>
<td>21</td>
<td>no</td>
<td>0.10</td>
</tr>
<tr>
<td>PEER 15</td>
<td>1.00</td>
<td>288</td>
<td>9500</td>
<td>0.19</td>
<td>0.0000</td>
<td>24</td>
<td>no</td>
<td>0.10</td>
</tr>
<tr>
<td>PEER 41</td>
<td>1.00</td>
<td>288</td>
<td>9500</td>
<td>0.17</td>
<td>0.0000</td>
<td>16</td>
<td>no</td>
<td>0.10</td>
</tr>
<tr>
<td>P2</td>
<td>0.88</td>
<td>256</td>
<td>5000</td>
<td>0.19</td>
<td>0.0000</td>
<td>14</td>
<td>yes</td>
<td>0.27</td>
</tr>
<tr>
<td>P3</td>
<td>0.88</td>
<td>256</td>
<td>4000</td>
<td>0.21</td>
<td>0.0000</td>
<td>16</td>
<td>yes</td>
<td>0.34</td>
</tr>
<tr>
<td>P4</td>
<td>0.88</td>
<td>256</td>
<td>4000</td>
<td>0.20</td>
<td>0.0000</td>
<td>16</td>
<td>yes</td>
<td>0.34</td>
</tr>
<tr>
<td>P5</td>
<td>0.88</td>
<td>256</td>
<td>3750</td>
<td>0.22</td>
<td>0.0021</td>
<td>16</td>
<td>yes</td>
<td>0.34</td>
</tr>
<tr>
<td>P6</td>
<td>0.88</td>
<td>256</td>
<td>3750</td>
<td>0.19</td>
<td>0.0064</td>
<td>16</td>
<td>yes</td>
<td>0.36</td>
</tr>
<tr>
<td>P7</td>
<td>0.88</td>
<td>256</td>
<td>3000</td>
<td>0.19</td>
<td>0.0000</td>
<td>16</td>
<td>yes</td>
<td>0.46</td>
</tr>
<tr>
<td>P8</td>
<td>0.88</td>
<td>256</td>
<td>3000</td>
<td>0.19</td>
<td>0.0000</td>
<td>16</td>
<td>yes</td>
<td>0.46</td>
</tr>
<tr>
<td>P9</td>
<td>0.88</td>
<td>256</td>
<td>4000</td>
<td>0.15</td>
<td>0.0000</td>
<td>16</td>
<td>yes</td>
<td>0.10</td>
</tr>
<tr>
<td>MII</td>
<td>0.85</td>
<td>234</td>
<td>6060</td>
<td>0.25</td>
<td>0.0046</td>
<td>18</td>
<td>no</td>
<td>0.25</td>
</tr>
<tr>
<td>X1</td>
<td>0.77</td>
<td>203</td>
<td>4647</td>
<td>0.20</td>
<td>0.0115</td>
<td>19</td>
<td>no</td>
<td>0.06</td>
</tr>
<tr>
<td>X2</td>
<td>0.77</td>
<td>203</td>
<td>4863</td>
<td>0.19</td>
<td>0.0172</td>
<td>19</td>
<td>no</td>
<td>0.06</td>
</tr>
<tr>
<td>X3</td>
<td>0.77</td>
<td>203</td>
<td>4400</td>
<td>0.16</td>
<td>0.0115</td>
<td>19</td>
<td>no</td>
<td>0.06</td>
</tr>
<tr>
<td>JXO-B1</td>
<td>0.67</td>
<td>139</td>
<td>3901</td>
<td>0.12</td>
<td>0.0108</td>
<td>24</td>
<td>no</td>
<td>0.17</td>
</tr>
<tr>
<td>JXO-B2</td>
<td>0.50</td>
<td>139</td>
<td>3269</td>
<td>0.24</td>
<td>0.0108</td>
<td>24</td>
<td>no</td>
<td>0.17</td>
</tr>
<tr>
<td>JXO-B8-LH</td>
<td>0.93</td>
<td>139</td>
<td>3429</td>
<td>0.19</td>
<td>0.0108</td>
<td>24</td>
<td>no</td>
<td>0.15</td>
</tr>
</tbody>
</table>

The specimens listed in Table 2 have design details typical of pre-1980s construction and were subjected to similar simulated earthquake load histories in the laboratory. However, as suggested by the data in Table 2 there are variations in both the design details and gravity loading. These variations contribute to variability in the observed damage patterns and progression.
3. IDENTIFYING DAMAGE STATES, ENGINEERING DEMAND PARAMETERS AND METHODS OF REPAIR

3.1 Engineering Demand Parameters

Within the context of this study, an engineering demand parameter (EDP) is a scalar or functional quantity that defines the earthquake demand on a joint at any point in the load history. Since the objective of the current study is to develop models for use in predicting joint damage given an EDP value, we are seeking to find the EDP that most accurately and precisely predicts joint damage. Since data characterizing the response of the laboratory test specimens discussed previously are used to develop models linking EDPs with damage states and methods of repair, the domain of potential EDPs is limited to the data published by the experimental researchers.

A review of the literature and the experimental data provides a basis for identifying a series of five potential EDPs:

- **Inter-story drift**: Drift is a simply demand measure provided by all researchers, and there is consensus within the earthquake engineering community that drift is a measure of earthquake demand. However, inter-story drift comprises flexural deformation of beams and columns as well as joint deformation. Thus, it is an imperfect measure of joint deformation demand.
- **Number of load cycles**: Like drift, the number of load cycles is a simply demand measure provided by all researchers and there is consensus within the earthquake engineering community that the number of load cycles has an impact on the observed response of components.
- **Drift in combination with the number of load cycles**: The results of previous research suggest that earthquake demand on a component is best characterized by a function that includes a measure of displacement demand and a measure of the number of load cycles. Given the availability of drift as a measure of joint deformation demand, a functional EDP that includes inter-story drift and number of load cycles is proposed:

\[
F = aD^b + cN^d
\]  

where \(D\) is the maximum drift and \(N\) is the number of load cycles. Empirical parameters in Eq. 2 are defined as follows: \(a=7.7\), \(b=0.85\), \(c=0.87\), and \(d=0.8\).
- **Joint shear strain**: Joint shear strain represents a substantial improvement over inter-story drift, since it is a measure only of joint deformation demand. However, joint shear strain data are provided by few researchers; the sparsity of these data may increase model uncertainty substantially.
- **Joint shear strain in combination with the number of load cycles**: A functional EDP that includes both maximum joint shear strain and number of load cycles is considered the most desirable EDP. Here the function defined by Eq. 2 is employed with \(D\) defining the maximum joint shear strain and empirical parameters are defined as follows: \(a=20\), \(b=0.25\), \(c=7.12\), and \(d=0.36\).

3.2 Damage States

Damage measures (DMs) describe the damage sustained by a component during an earthquake. In this study, damage is quantized into discrete damage states. Data characterizing the development of damage in the previously discussed laboratory test specimens as well as research results, documentation providing guidelines for post-earthquake repair and interviews with professional were used to identify a series of damage states that 1) best characterize the progression of damage in reinforced concrete beam-column joints and 2) best determine the appropriate method of repair for the component. These damage states are:

0. Initial cracking at the beam-column interface
1. Initial cracking within the joint area
2. Maximum crack width is less than 0.02 in. (5 mm)
3. Maximum crack width is greater than 0.02 in. (5 mm)
4. Beam longitudinal reinforcement yields
5. Maximum crack width is greater than 0.05 in. (1.3 mm)
6. Spalling of at least 10% joint surface concrete
7. Joint shear strength begins to deteriorate
8. Spalling of more than 30% joint surface concrete
9. Cracks extend into the beam and/or column
10. Spalling of more than 80% joint surface concrete
11. Crushing of concrete extends into joint core
12. Failure due to a) buckling of longitudinal steel reinforcement, b) loss of beam longitudinal steel anchorage within the joint core, or c) pull-out of discontinuous beam longitudinal steel reinforcement

3.3 Methods of Repair

The method of repair required to restore a component to its original, pre-earthquake condition, provides a basis for estimating the economic impact of earthquake loading. Information was collected from multiple sources to identify appropriate techniques for repairing earthquake damage to RC components and to link these repair methods with the range of previously identified damage states. The primary references consulted were *FEMA 308 Repair of Earthquake Damaged Concrete and Masonry Wall Buildings* (1998) and *ACI 546R-96 Concrete Repair Guide* (1996). In addition, the results of previous research by others were used to verify the adequacy of repair methods and to identify which repair methods would be employed for which damage states (Filiatrault 1996, Tasai 1992, Karayannis 1998, Jara et al. 1989). Additionally, practicing engineers and contractors were consulted to verify linkages between repair methods and damage states (Coffman and Kapur 2003, Runacres 2003, Savage 2003).

Review of the relevant sources results in identification of five methods of repair that would be appropriate for restoring joints to original condition (Table 3). These methods of repair include five basic repair techniques: repair cosmetic finishes, epoxy inject concrete cracks, patch spalled concrete, remove and replace crushed concrete, replace reinforcing steel. Review of the relevant sources also provided a basis for linking these methods of repair with specific damage states (Table 3). While the probabilistic framework employed for prediction of economic impact (Eq. 1) would suggest that there should be a probabilistic relationship linking each repair method with a set of damage states, there are no data available to calibrate such models.

**Table 3. Methods of Repair for Joints**

<table>
<thead>
<tr>
<th>Method of Repair</th>
<th>Activities</th>
<th>Damage States</th>
</tr>
</thead>
<tbody>
<tr>
<td>0. Cosmetic Repair</td>
<td>Replace and repair finishes.</td>
<td>0,1</td>
</tr>
<tr>
<td>1. Epoxy Injection</td>
<td>Inject cracks with epoxy and replace finishes.</td>
<td>2-5</td>
</tr>
<tr>
<td>2. Patching</td>
<td>Patch spalled concrete, epoxy inject cracks and replace finishes.</td>
<td>6-8</td>
</tr>
<tr>
<td>3. Replace Concrete</td>
<td>Remove and replace damaged concrete, replaces finishes</td>
<td>9-11</td>
</tr>
<tr>
<td>4. Replace Joint</td>
<td>Replace damaged reinforcing steel, remove and replace concrete, and replace finishes.</td>
<td>12</td>
</tr>
</tbody>
</table>

4. PREDICTION OF REQUIRED REPAIR METHOD

Using the experimental data collected and the damage states and methods of repair identified from the literature, a series of fragility curves were developed for use in predicting damage as well as repair method given an EDP.

4.1 Damage Versus EDP

Experimental data characterizing the progression of damage for the test specimens were used to generate data sets linking the thirteen damage states with the three primary EDPs: drift, number of load cycles and joint shear strain. The functional EDPs, defined by Eq. 2, were calibrated to minimize the dispersion of the data for all damage states. Figure 1 shows damage-EDP data for the three primary EDPs.
The scatter of the data in Figure 1 reinforces the need for probabilistic models linking EDPs with damage and repair. The variability in these data is due in part to variability in test specimen design and loading, as discussed previously. However, the variability is due also to the tremendous uncertainty that is necessarily introduced in compiling these data. The typically procedure for conducting an pseudo-static experimental investigation of component response to earthquake loading is:

1. A half-cycle of loading to a new maximum displacement demand, at which point loading is paused to allow for identification of new cracks and regions of spalling, measurement of new and existing cracks and picture taking.
1. Loading in the reversed direction to a new minimum displacement demand, at which point loading is paused to allow for data collecting as above.
2. Multiple additional full load cycles, typically two additional cycles, to the new maximum and minimum displacement demand levels.

Thus, in monitoring the progression of damage, it is not possible to know exactly the displacement demand level at which damage occurred, only that it occurred prior to reaching a particular maximum displacement demand level. Further it is not possible to differentiate between damage that occurs during the second cycle to a maximum displacement demand level from that which occurs during the third cycle or from that which occurs during the first cycle to an increased maximum displacement demand.

4.2 Predicting the Required Method of Repair

4.2.1. Grouping Damage Data for Using is Prediction Method of Repair

The data presented in Figure 1 were used to develop models defining the probability of earthquake damage requiring, at least, the use of a specific method of repair. These data could have been used to generate fragility curves defining the probability that joint damage would meet or exceed a specific damage state. However, since the ultimate objective of this effort was the prediction of economic impact, the development of damage-state prediction models was not considered to be necessary.

To generate repair-method prediction models, the data in Figure 1 were combined so that individual data points define a specific EDP value and the required method of repair associated with that EDP value. This combination was accomplished using the relationships in Table 3. Because several damage states are linked with each methods of repair, there are several plausible approaches to combining the data:

- Method One: For each individual specimen, only the EDP-damage state pair for the lowest damage state associated with each method of repair is used. This method results in no more than 25 data points for each method of repair.
- Method Two: Similar to Method One with the exception that if a particular specimen has no data points for a particular method of repair, then the EDP for a more extensive method of repair is included. This method results in exactly 25 data points for each method of repair. However, this method also increases the dispersion of the data and skews the data towards higher EDP levels.
- Method Three: Only data for the lowest damage state are used for each method of repair. This method results in the fewest data for each method of repair.

All three approaches were employed for demand defined by inter-story drift, and the resulting probabilistic repair-prediction models were used to evaluate the methods.
4.2.2. Standard Probabilistic Models Considered

The data in Figure 1, combined into method of repair groups as discussed in Section 5.2.1, may be used to define an empirical cumulative probability density function (CDF) that defines the probability of meeting or exceeding a particular method of repair for a given EDP. This “empirical” CDF is the Stepwise CDF, and is generated simply by ranking the data points for each method of repair from lowest to highest EDP. The rank of a particular EDP value relative to the total number of data points for the method of repair is the CDF value for that particular EDP.

However, to facilitate the use of the probabilistic repair-prediction models, standard CDFs were calibrated using the data and tested to determine which represented a best-fit to the Stepwise CDF. The standard distributions considered included the following:

- Normal distribution: Commonly employed distribution. This distribution includes negatively valued data, so it is not ideal for the current application.
- Lognormal distribution: Commonly employed distribution; includes only positively valued data.
- Weibull distribution: Less commonly used distribution. The distribution allows for a stronger influence of extreme-valued data. This is desirable for the current application where small-demand values are important.
- Beta distribution: Less commonly used distribution. Allows for an upper and lower bound to be defined for the distribution, which may be desirable for the current application.

Two different methods may be used to calibrate the four standard CDFs using the empirical data: The Method of Moments and the Method of Maximum Likelihood. The Method of Moments relies on calculation of the mean and variance of the population using the sample data. Thus, this method introduces some uncertainty into the calculation process because the mean and variance of the sample are only estimates of the population statistics. To avoid this error, the Method of Maximum Likelihood may be used to calibrate the CDFs. Using this method, a likelihood function is defined as the product of the derivative of the CDF with respect to the random variable evaluated at each of the data points. The CDF parameters are computed to maximize this likelihood function.

4.2.3. Evaluation of the Models and Model Calibration Methods

The three methods for combining damage data, the four CDFs and the two calibration methods were evaluated using two standard tests: the Kolmogorov-Smirnov (K-S) and the Chi-Square. The K-S test numerically confirms a visual inspection of the distribution function in comparison with the Stepwise CDF and is defined by the maximum difference between the predicted and observed value. The Chi-Square test provides an averaged evaluation of the model over the range of observed EDPs.

The results of this evaluation indicated that the preferred approach for predicting the required method of repair employed 1) Method One for combination of damage data to develop method of repair – EDP data, 2) a lognormal distribution, and 3) the Method of Maximum Likelihood for calibration of the probability function. Several approaches, including the preferred approach, resulted in similar average K-S and Chi-Square test parameters. However, the preferred approach resulted in the minimum coefficient of variation across the different methods or repair.

4.2.4. Evaluation of EDPs

Figure 2 shows the computed probabilistic models linking method of repair with each of the five EDPs. Given a specific EDP value and a specific method of repair, these models define the probability that joint damage will be such that, at a minimum, the specific method of repair will be required to restore the joint to its original condition. Ideally the curves that represent different methods of repair have well spaced means and low coefficients of variation, such that the progression from a relatively low probability to a relatively high probability occurs over a very small EDP range. Visually it is clear that the models defined by functional EDPs (Figure 2d and e), with the exception of method of repair 4, come closest to this ideal while the models in Figure 2c are perhaps farthest from this ideal. Evaluation of the EDPs using the K-S test confirms that the functional EDPs provide the best fit to the data.

5. CONCLUSIONS

The results of previous research as well as the practical experience of structural engineers and contractors were used as a basis for developing probabilistic defining the method of repair required to restore a pre-
1980s joint to original condition as a function of traditional engineering demand parameters. These models provide a basis for evaluating the economic impact of earthquake loading of a building. Multiple approaches were considered in developing the probabilistic model; it was found that defining engineering demand using a nonlinear function of inter-story drift and number of load cycles resulted in the best model, as evaluated using the K-S test. It was found also that the defining demand using a nonlinear function of joint shear strain and number of load cycles offers the potential for reduced model uncertainty. However, insufficient data were available to define this model for all methods of repair. Additional data are required defining the progression of damage in older joints as a function of the three primary EDPs: drift, number of load cycles and joint shear strain.

Figure 2. Probability of meeting or exceeding a method of repair for all EDPs
6. REFERENCES


ACI (1968). “Epoxy with Concrete.” *ACI SP-21*.


