NCEER-ATC Joint Study on Fragility of Buildings

by

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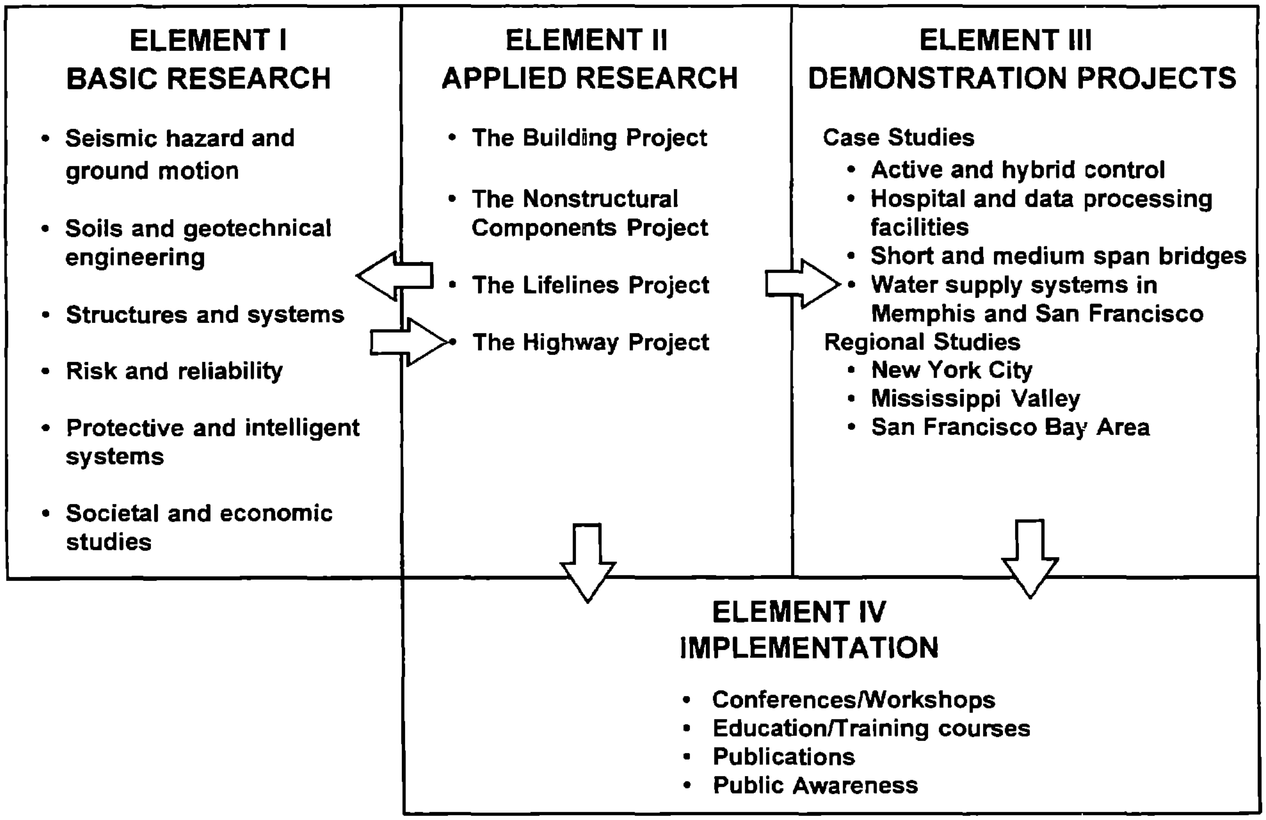
PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established to expand and disseminate knowledge about earthquakes, improve earthquake-resistant design, and implement seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures in the eastern and central United States and lifelines throughout the country that are found in zones of low, moderate, and high seismicity.

NCEER's research and implementation plan in years six through ten (1991-1996) comprises four interlocked elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten. Element III, Demonstration Projects, have been planned to support Applied Research projects, and will be either case studies or regional studies. Element IV, Implementation, will result from activity in the four Applied Research projects, and from Demonstration Projects.



ELEMENT I
BASIC RESEARCH

- Seismic hazard and ground motion
- Soils and geotechnical engineering
- Structures and systems
- Risk and reliability
- Protective and intelligent systems
- Societal and economic studies

ELEMENT II
APPLIED RESEARCH

- The Building Project
- The Nonstructural Components Project
- The Lifelines Project
- The Highway Project

ELEMENT III
DEMONSTRATION PROJECTS

Case Studies
  - Active and hybrid control
  - Hospital and data processing facilities
  - Short and medium span bridges
  - Water supply systems in Memphis and San Francisco

Regional Studies
  - New York City
  - Mississippi Valley
  - San Francisco Bay Area

ELEMENT IV
IMPLEMENTATION

- Conferences/Workshops
- Education/Training courses
- Publications
- Public Awareness

Research in the **Building Project** focuses on the evaluation and retrofit of buildings in regions of moderate seismicity. Emphasis is on lightly reinforced concrete buildings, steel semi-rigid frames, and masonry walls or infills. The research involves small- and medium-scale shake table tests and full-scale component tests at several institutions. In a parallel effort, analytical models and computer programs are being developed to aid in the prediction of the response of these buildings to various types of ground motion.

Two of the short-term products of the **Building Project** will be a monograph on the evaluation of lightly reinforced concrete buildings and a state-of-the-art report on unreinforced masonry.

The **risk and reliability program** constitutes one of the important areas of research in the **Building Project**. The program is concerned with reducing the uncertainty in current models which characterize and predict seismically induced ground motion, and resulting structural damage and system unserviceability. The goal of the program is to provide analytical and empirical procedures to bridge the gap between traditional earthquake engineering and socioeconomic considerations for the most cost-effective seismic hazard mitigation. Among others, the following tasks are being carried out:

1. Study seismic damage and develop fragility curves for existing structures.
2. Develop retrofit and strengthening strategies.
3. Develop intelligent structures using high-tech and traditional sensors for on-line and real-time diagnoses of structural integrity under seismic excitation.
4. Improve and promote damage-control design for new structures.
5. Study critical code issues and assist code groups to upgrade seismic design code.
6. Investigate the integrity of nonstructural systems under seismic conditions.

This report provides improved damage-motion relationships that can be used in regional earthquake damage and loss studies. Three main areas for modification of the existing ATC-13 damage probability matrices were investigated. The first was to develop detailed descriptions of the original 40 building classes defined in ATC-13. These descriptions clarified assumptions made regarding the load carrying system and the standard design practices. The second approach for modifying the motion damage relationships was through collecting existing data. It was found that these data are not particularly useful because they were collected under different formats and with different interpretations by the individuals gathering the data. In addition, ground motions were not available for the majority of the data.

The third modification considered the development of fragility formulations based on the information from the ATC-13 damage probability matrices. For that purpose, the original mean and 90% expert opinion values of damage at each intensity level were used to develop fragility curves for all 40 building classes. Then a lognormal function was fitted through the fragility curve to enable easy implementation of these fragility curves. These curves can be easily implemented in large regional damage and loss estimation studies.

ABSTRACT

The objective of this report is to provide improved damage-motion relationships that can be used in regional earthquake damage and loss studies. Three main areas for modification of the existing ATC-13 damage probability matrices were identified and are discussed in the report. The first improvement is the development of detailed descriptions of the original 40 building classes defined in ATC-13. These descriptions clarify the assumptions that were made regarding the load carrying system and the standard design practices.

The second approach for modifying the motion damage relationships was through existing data. Thus an attempt was made to collect data on building damage from recent significant earthquakes. After a considerable effort in search of such data, it was found that these data are not particularly useful because they were collected under different formats and with different interpretations by the individuals gathering the data. In addition, ground motions are not available for majority of the data. Thus, these data could not be used to improve the existing motion-damage relations given in ATC-13. A summary of the sources of data reviewed and gathered is included in this report.

The third modification of the motion-damage relationships considered the development of fragility formulations based on the information from the ATC-13 damage probability matrices. For that purpose, the original mean and 90% expert opinion values of damage at each intensity level were used to develop fragility curves for all 40 building classes. Then a lognormal function was fitted through the fragility curve to enable easy implementation of these fragility curves. The curves and parameters for each building class are included in the report. Fragility curves are provided for six damage states corresponding to damage factor of 0.1%, 1%, 10%, 30%, 60%, and 99%. These curves can be easily implemented in large regional damage and loss estimation studies.

ACKNOWLEDGMENTS

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SECTION 1
INTRODUCTION

Despite the enormous monetary and life loss that has occurred in this century due to the occurrence of damaging earthquakes, little useful quantitative data exist relating level of damage to earthquake characteristics. Relationships between earthquake size and damage are essential tools in estimating regional damage for the purposes of developing earthquake preparedness plans and making decisions about allocations of resources for structural rehabilitation, hazard mitigation, post-earthquake recovery and earthquake insurance. In 1985, with the publication of ATC-13, "Earthquake Damage Evaluation Data for California", motion-damage relationships were developed for 78 classes of structures (building and non-building) found in California. To overcome the limitations of available data, questionnaires were submitted to panels of experts to obtain probabilities of occurrence of damage to each structure type at various levels of ground motion intensity. A Delphi procedure was used to gather the expert opinion data, and damage probability matrices were developed by aggregating the responses of experts.

While this document was an enormous step forward in the effort to quantify earthquake damage, several improvements can be made. First, more detailed descriptions are needed to define the building classes in the ATC-13 document. This is needed in order to facilitate classification of existing buildings and to distinguish between standard, nonstandard and special construction. In ATC-13, buildings and other structures such as bridges, pipelines and tanks were described by short descriptors, for example engineering facility class - is defined as "medium rise braced steel frame". As the document was used, it became apparent that more information was needed about the load paths of these buildings and the design standards and criteria that were implied in the building descriptors, in order to distinguish between an average or "Standard" building and a below average or "Non-Standard" building. Since ATC-13 applied only to California buildings, attempts have been made to modify the damage probabilities developed in ATC-13 to perform loss studies in other parts of the United States such as Charleston, South Carolina and Boston, Massachusetts. (Harlan and Lindbergh, 1988; Massachusetts Civil Defense Agency, 1989). It is difficult to modify these damage motion relationships for local design and construction practices unless it is understood what the experts were thinking of when the original relationships were developed.

One of the objectives of this project is to develop detailed descriptions of the original 40 ATC-13 building types in order to clarify the assumptions that were made regarding load carrying systems and standard design practices. These detailed descriptions, based on original notes of and discussions with those involved in the development of the relationships, are presented in Section 2.

A second area where damage motion relationships could be improved would be to compare and combine the expert generated ATC-13 damage probabilities with damage data that have been collected in recent California earthquakes. There have been several damaging earthquakes in California since 1985 in which damage data have been collected. Ideally these data should be formally included in the motion-damage relationships. However, much of the data that have been collected are not in a format that would make them useful for developing damage probabilities. Building type, location, ground motion magnitude or amount of damage in this report are usually missing in these data. In this report, data from recent earthquakes were reviewed and evaluated for their usefulness in developing or modifying damage probabilities. Data that were reviewed are summarized in Section 3. Comparisons were made between ATC-13 cumulative probability distributions based on expert opinion and data, for the few classes of buildings for which data were available. These comparisons are presented in Section 3.

A third difficulty in using damage probabilities in ATC-13 has been that it is not always easy to distinguish between building types when performing loss studies. In most cases building inventories used in loss estimation studies are compiled from various sources such as assessors files, insurance data, census data, and specialized local government databases. Most often these are incomplete or inexact in their descriptions of the structural system. Even if one were to do a building by building sidewalk survey, it is difficult to determine building type unless one has access to drawings. An example might be that it is difficult to look at an existing structure and identify it as a moment resisting perimeter frame or a moment resisting distributed frame. One option is to assign probabilities to the building type based on land use, year of construction, typical local building practices and other characteristics of the built environment. Using inference rules in an expert system an assignment can be made, such as 0.6 probability the frame is distributed and 0.4 probability it is perimeter. Then damage probability matrices can be combined, weighted by the associated probabilities (in this case 0.6 and 0.4). Another approach is to compare the damage probability functions for all of the building types and determine if any of them are similar enough to be combined into a single structural class. Thus, if the probability damage matrices or the equivalent fragility curves for several classes of buildings are similar, then a precise classification of structures may not be necessary for these structural classes. The fragility curves presented in Section 3 are reviewed in Section 4 and recommendations are made as to whether any classes could be combined.

Damage motion relationships used in loss estimation are most frequently expressed in the form of damage probability matrices (DPMs) or fragility curves. Both formulations provide information on the probability of experiencing some level of damage given a certain level of ground motion. In this project the DPMs developed in ATC-13 were converted into fragility curves in order to provide an alternate representation of the damage - motion relationships for structures in California. The fragility curves have a logarithmic representation that enables a simple analytical combination of probabilities. The procedure was used for converting the DPMs to fragility curves is presented in Section 4. Some observations and several conclusions are summarized in Section 5 of this report.

SECTION 2
BUILDING DESCRIPTIONS

ATC-13 classifies buildings and other structures in terms of earthquake engineering characteristics and in terms of social function. Table 3.1 in ATC-13, entitled Earthquake Engineering Facility Classification, contains 40 building classes (78 total facility classes) that are based on height (number of stories), structural framing type, and structural material. For each of the 78 facility classes, a damage probability matrix (DPM) relating damage to ground motion intensity (MMI) was developed from expert opinion surveys. The DPMs were developed based on "Standard" construction, with simplified rules (or modifiers) for adjusting the DPMs to account for design and construction quality. After the completion of ATC-13, it became apparent that the omission of detailed descriptions of the facility classes limited the applicability of the DPMs. For example, since Standard, Nonstandard and Special construction were not specifically defined for each facility class, it was difficult to determine when modifiers should be used. In addition, ATC-13 DPMs, modified for local building and design practices, have been used in loss estimation studies for regions outside of California. It has been difficult to modify the DPMs to account for non-California practices without a detailed description of the design and construction assumptions associated with each building class.

The 40 building classes in ATC-13 can be reduced to 17 types if only framing type and structural material are considered. Thus, descriptions were developed for 17 construction categories (Rojahn, 1993) as shown in Table 2-I. The detailed descriptions are based upon the notes and review comments of key developers of the ATC-13 facility classifications and DPMs, and review of descriptions of building types found in subsequent ATC studies.

Each description contains information on the structural framing system, presented in terms of construction materials, gravity load carrying system, and lateral load resisting system. Also included are features, if any, that designate structures as *Nonstandard*, *Standard*, and *Special* construction. *Standard* construction includes all structures except those designated as special or nonstandard. *Special* construction includes structures that have special earthquake damage control features, and *Nonstandard* construction includes those structures that are more susceptible to earthquake damage than Standard construction. These descriptions are found in Tables 2-III through 2-XIX.

TABLE 2-I Construction Categories for Which Detailed Descriptions were Developed

Construction Category	ATC-13 Facility Number(s)
Wood Frame	1
Light Metal	2
Unreinforced Masonry Bearing Wall	75, 76
Unreinforced Masonry with Load Bearing Frame	78, 79, 80
Reinforced Concrete Shear Wall with Moment-Resisting Frame	3, 4, 5
Reinforced Concrete Shear Wall without Moment-Resisting Frame	6, 7, 8
Reinforced Masonry Shear Wall with Moment-Resisting Frame	84, 85, 86
Reinforced Masonry Shear Wall without Moment-Resisting Frame	9, 10, 11
Braced Steel Frame	12, 13, 14
Moment Resisting Steel Perimeter Frame	15, 16, 17
Moment Resisting Steel Distributed Frame	72, 73, 74
Moment Resisting Ductile Concrete Frame	18, 19, 20
Moment Resisting Non-Ductile Concrete Frame	87, 88, 89
Precast Concrete Frame	81, 82, 83
Long Span	91
Tilt-up	21
Mobile Homes	23

Descriptions such as those found in Tables 2-III through 2-XVIII serve to clarify the building classification system and use of construction quality modifiers found in ATC-13. A general description of the interpretation of each of the damage states (slight, light, moderate, heavy, major, and destroyed) for each of the 17 construction categories would provide additional improvement to the information in these tables. Simplified definitions of these states can be found in ATC-13 (Chapter 2) and are repeated in Table 2-XVIII of this report.

It can be seen that the generality and simplicity of these definitions allow for a great deal of interpretation. This makes comparison of ATC-13 damage probabilities with available damage data difficult. In addition to the definitions, each of the above damage states is associated with a damage factor (dollar loss/replacement cost) range. However, it is not clear that damage state definitions and damage factors are consistent across all construction classes. Significant damage to components in steel structures may be considerably different than significant damage to a concrete structure, each resulting in different dollar losses. Thus, it is recommended that the damage state definitions be defined for each structural engineering class. If damage to specific structural components is defined for each damage state and building type, then the estimation of repair costs can be more rationally based. The development of definitions for damage state specific to each structural class, however, requires considerable effort and is beyond the scope of this study.

TABLE 2-II Simplified Definitions of Damage States from ATC-13

State	Definition
None	No damage
Slight	Limited localized minor damage not requiring repair
Light	Significant localized damage of some components generally not requiring repair
Moderate	Significant localized damage of many components warranting repair
Heavy	Extensive damage requiring major repairs
Major	Major widespread damage that may result in the facility being condemned, demolished or repaired
Destroyed	Total destruction of the majority of the facility

It should be noted that while detailed descriptions of damage states for each structural category will improve understanding, problems still arise in collecting and comparing data. For example, often it is difficult to identify damage from the street without a detailed investigation of the interior of the structure. Some forms of damage may not be readily identified as they may be hidden by architectural finishes. Therefore, collection of reliable damage data for the purposes of improving damage motion relationships requires a well organized and detailed study of damaged structures.

TABLE 2-III Detailed Description of Wood Frame Construction

General Description. Wood-frame buildings can be of two types: (1) low-rise single-family and multi-family dwelling units with structural systems of repetitively used wood studs and joists; and (2) commercial and industrial buildings with structural systems of beams and columns composed of wood and/or steel.
Gravity Loads. Gravity loads are transferred from floor and roof sheathing to joists that span between stud walls or larger beams. The interior wood posts that support these elements are typically founded on individual concrete footings.
Lateral Loads. Wood dwelling units normally are non-engineered but usually have the components of a lateral-force resisting system. Lateral loads are transferred by floors and roofs, acting as diaphragms, to walls, acting as shear walls. Shear walls can be exterior walls sheathed with plank siding, stucco, or plywood, and interior partitions sheathed with plaster or gypsum board. These buildings usually have high chimneys. Wood commercial and industrial buildings are usually engineered structures with lateral force resisting systems that can be similar to those for wood dwelling units, or there may be rod bracing between columns. Large openings for stores and garages often require post-and-beam framing. Wall openings may have steel rigid frames or diagonal bracing.
Standard Construction. The designation of *Standard* wood construction pertains to wood structures built in or after 1940 and before 1976, i.e., prior to the enforcement of modern seismic provisions.
Nonstandard Construction. *Nonstandard* wood buildings are pre-1940 structures, many of which used unsheathed cripple studs with perimeter wall foundations and lacked anchorage of the wood sill plates to the foundation. Estimates of damage would be equivalent to that for two intensities higher than for *Standard* construction.
Special Construction. The designation of *Special* construction pertains to buildings built in 1976 or thereafter, when modern seismic provisions were assumed to be in widespread enforcement throughout California. Estimates of damage would be equivalent to that for two intensities lower than for *Standard* construction.

TABLE 2-IV Detailed Description of Light-Metal Construction

General Description. Light-metal buildings are pre-engineered, prefabricated, single-story, usually utilitarian structures with transverse rigid frames and longitudinal rod bracing. The roof and walls consist of lightweight panels. The frames have tapered beam and column sections built up of light plates.

Gravity Loads. Gravity loads are transferred from the roof elements to steel purlins or open web joists that span between main framing lines. The main transverse beams or trusses then transfer loads to the steel columns on the building perimeter and/or interior.

Lateral Loads. Lateral loads in the transverse direction are resisted by the rigid frames, with loads distributed to them by shear elements. Loads in the longitudinal direction are resisted entirely by shear elements. The shear elements can be either the roof and wall sheathing panels or an independent system of tension-only rod bracing, or a combination of panels and bracing.

Standard Construction. The designation of *Standard* light-metal construction pertains to the vast majority of structures in this category.

Nonstandard Construction. There is no designation of *Nonstandard* construction for this structure type.

Special Construction. The designation of *Special* construction pertains to buildings that have been engineered on a site-specific basis. Estimates of damage would be equivalent to that for one intensity lower than for *Standard* construction.

TABLE 2-V Detailed Description of Unreinforced Masonry Bearing Wall Construction

General Description. Buildings of this type have perimeter walls, and possibly some interior walls, of unreinforced masonry (URM). Prior to 1900, the majority of floor and roof construction consisted of wood sheathing supported by wood subframing. Cast-in-place concrete floors, supported by the unreinforced masonry bearing walls and/or steel or concrete interior framing, were commonly used in large multi-story structures. Post-1950 unreinforced masonry buildings with wood floors usually have plywood rather than board sheathing.
Gravity Loads. Gravity loads are transferred from floor and roof wood-sheathed diaphragms to joists that span between exterior masonry walls and interior partition walls. Concrete diaphragm buildings are supported by the exterior masonry walls and interior frames of either concrete or steel.
Lateral Loads. Lateral loads are transferred from the diaphragm elements to the exterior walls through wall anchors. Interior partitions may contribute to the lateral force resisting system by limiting both inter-story drift and diaphragm displacement. Wall anchors secure the wall to the diaphragm for perpendicular loads.
Standard Construction. The designation of *Standard* unreinforced masonry bearing wall construction pertains to URM buildings with good brick and mortar, i.e., buildings normally built in or after 1950, although some older buildings also have good quality construction materials.
Nonstandard Construction. *Nonstandard* URM buildings are those with substandard (lime-sand) mortar, often a characteristic of pre-1950 URM buildings. Estimates of damage would be equivalent to that for one intensity higher than for *Standard* construction.
Special Construction. The designation of *Special* construction pertains to buildings that have been completely seismically retrofitted according to formal criteria (e.g., Los Angeles Division 88). Estimates of damage would be equivalent to that for one intensity lower than for *Standard* construction.

TABLE 2-VI Detailed Description of Unreinforced Masonry with Load Bearing Frame

General Description. Buildings of this type are older structures with load bearing frames of concrete or steel and unreinforced masonry infill walls. The infill walls may be located between columns or offset from exterior frame members, and wrapped around them, presenting a smooth masonry exterior with no indication of the frame. Floor and roof diaphragms may be composed of straight or diagonally sheathed wood supported by wood subframing. Cast-in-place concrete slabs may also be used. The infill walls may consist of solid clay brick, concrete block, or hollow clay tile.

Gravity Loads. Gravity loads are transferred from floor and roof diaphragms to subframing, which is supported by the steel or concrete frame. The frames may also support the weight of the infill masonry walls and/or partitions.

Lateral Loads. Although it is often assumed that lateral loads are resisted by the frame elements only, stiffness of the infill walls may significantly affect lateral response. In the elastic range (i.e., for low levels of excitation), the stiffness of the infill may cause buildings of this type to respond as stiff, shear-wall structures. Once cracks form along the boundary between the infill and the frame, the infill in compression can act as a diagonal strut (i.e. like a braced frame). If the cyclic response continues, the masonry cracks can become more severe, and spalling may commence. As the stiffness of the masonry infill degrades, lateral loads are increasingly resisted by frame action.

Standard Construction. The designation of *Standard* construction for this building type pertains to buildings with good grade brick and mortar, i.e., buildings normally built in or after 1950, although some older buildings also have good quality construction materials.

Nonstandard Construction. *Nonstandard* URM-infill frame buildings are those with soft brick and substandard (lime-sand) mortar, often a characteristic of pre-1950 URM buildings. Estimates of damage would be equivalent to that for one intensity higher than for *Standard* construction.

Special Construction. The designation of *Special* construction pertains to buildings that have been seismically retrofitted. Estimates of damage would be equivalent to that for one intensity lower than for *Standard* construction.

TABLE 2-VII Detailed Description of Reinforced Concrete Shear Wall with Moment-Resisting Frame

General Description. Buildings of this type have shear walls of reinforced concrete and moment resisting frames of concrete or steel. Floor and roof diaphragms are typically composed of cast-in-place concrete slabs, but they can be of almost any material. In older buildings, the concrete walls are often quite extensive, and the entire exterior may be a concrete shear wall system.
Gravity Loads. Gravity loads are transferred from floor and roof slabs to the framing elements, such as one-way joists or waffle joists, or through flat slab action to larger beams, walls or columns. The frame columns and concrete walls support the major floor framing elements and transfer the gravity loads to the foundation.
Lateral Loads. Typically, the reinforced concrete shear walls are designed to carry at least 75% of the lateral loads, whereas the frames are designed to carry 25% of the lateral loads.
Standard Construction. All buildings of this type are assumed to be of *Standard* construction. Existing buildings of this type are expected to perform in a similar fashion, as there are no easily discernible differences in design and construction practices, particularly with respect to design date.
Nonstandard Construction. There is no designation of *Nonstandard* construction for this structure type.
Special Construction. There is no designation of *Special* construction for this structure type.

TABLE 2-VIII Detailed Description of Reinforced Concrete Shear Wall without Moment-Resisting Frame

General Description. Buildings of this type have shear walls of reinforced concrete and may have vertical-load bearing frames of concrete or steel. The shear walls may be bearing walls; they may be of any extent (a few or many); and they may be located anywhere in the building (interior or exterior). Floor and roof diaphragms are generally composed of cast-in-place concrete slabs, or metal decking with concrete fill. Exterior walls may be either metal, concrete, or precast concrete panels.
Gravity Loads. Gravity loads are transferred from floor and roof slabs to the framing elements (beams and joists) which normally carry only vertical loads, and/or to load bearing walls.
Lateral Loads. Lateral loads are primarily resisted by the concrete shear walls.
Standard Construction. The designation of *Standard* construction for this building type pertains to structures built before 1976, i.e., prior to the enforcement of modern seismic provisions.
Nonstandard Construction. There is no designation of *Nonstandard* construction for this structure type.
Special Construction. The designation of *Special* construction pertains to buildings built in 1976 or thereafter, when modern seismic provisions were assumed to be in widespread enforcement throughout California. Estimates of damage would be equivalent to that for one intensity lower than for *Standard* construction.

TABLE 2-IX Detailed Description of Reinforced Masonry Shear Wall With Moment Resisting Frame

General Description. Buildings of this type have shear walls of reinforced masonry and moment resisting frames of concrete or steel. Floor and roof diaphragms are typically composed of precast concrete elements, such as planks, T-beams, or slabs; they may or may not include a concrete topping slab. The walls typically consist of either grouted brick or concrete block masonry.
Gravity Loads. Gravity loads are transferred from floor and roof slabs to the exterior masonry walls and/or interior framing elements (beams and columns) and masonry walls.
Lateral Loads. Typically, the reinforced masonry shear walls are designed to carry at least 75% of the lateral loads, whereas the frames are designed to carry 25% of the lateral loads.
Standard Construction. All buildings of this type are assumed to be of *Standard* construction. Existing buildings of this type are expected to perform in a similar fashion, as there are no easily discernible differences in design and construction practices, particularly with respect to design date.
Nonstandard Construction. There is no designation of *Nonstandard* construction for this structure type.
Special Construction. There is no designation of *Special* construction for this structure type.

TABLE 2-X Detailed Description of Reinforced Masonry Shear Wall Without Moment Resisting Frame

General Description. Buildings of this type have shear walls of reinforced masonry and may have vertical-load bearing frames of wood or steel. The shear walls may be reinforced brick or concrete block masonry, may be bearing walls, and may be located anywhere in the building (interior or exterior). Floor and roof diaphragms are typically composed of plywood, or straight or diagonal sheathing. Metal deck with or without concrete fill may also be used for diaphragm elements.
Gravity Loads. Gravity loads are transferred from floor and roof diaphragms to the masonry walls and/or framing elements, which may be wood joists and beams supported by interior wood posts or steel columns, or steel beams supported by steel columns.
Lateral Loads. Lateral loads are resisted by the masonry shear walls.
Standard Construction. The designation of *Standard* construction for this building type pertains to structures built before 1976, i.e., prior to the enforcement of modern seismic provisions.
Nonstandard Construction. There is no designation of *Nonstandard* construction for this structure type.
Special Construction. The designation of *Special* construction pertains to buildings built in 1976 or thereafter, when modern seismic provisions were assumed to be in widespread enforcement throughout California. Estimates of damage would be equivalent to that for one intensity lower than for *Standard* construction.

TABLE 2-XI Detailed Description of Braced Steel Frame

General Description. Structural systems of braced steel frame buildings consist of steel columns, beams and girders, and diagonal braces spanning between floor levels. The roof and floor diaphragms are generally composed of either metal decking with concrete fill or cast-in-place concrete slabs. Exterior walls may be either metal or precast concrete panels. In older buildings, the exterior may be composed of masonry or concrete, with an architectural facing.

Gravity Loads. Gravity loads are transferred from floor and roof slabs to the floor/roof framing elements composed of steel beams or open web joists. Floor/roof girders are supported by steel columns that transfer loads to the foundation.

Lateral Loads. Lateral loads are transferred from the floor diaphragms to collector elements and to the braced frames. Vertical truss action of the beams, columns, and diagonals transfer these forces through axial stresses to the foundation. Simple connections are often used at the braced frame connections. Buildings of this type may or may not have a complete gravity load resisting moment frame as a secondary lateral force resisting system.

Standard Construction. The designation of *Standard* construction for this building type pertains to buildings built between 1960 and 1988, i.e., prior to the enforcement of modern seismic provisions.

Nonstandard Construction. *Nonstandard* braced steel frame buildings are those built prior to 1960, the initial benchmark year in which earthquake design standards were substantially improved. Estimates of damage would be equivalent to that for one intensity higher than for *Standard* construction.

Special Construction. The designation of *Special* construction pertains to buildings built in 1988 or thereafter, when modern seismic provisions were assumed to be in widespread enforcement throughout California. Estimates of damage would be equivalent to that for one intensity lower than for *Standard* construction.

TABLE 2-XII Detailed Description of Moment Resisting Steel Perimeter Frame

General Description. Structural systems of moment resisting steel perimeter frame buildings are comprised of steel columns, beams and girders. Lateral loads are resisted by the moment action of the perimeter frames, whereas the interior girder-column connections are simple connections designed to support only vertical loads. The roof and floor diaphragms are generally composed of either metal decking with concrete fill or cast-in-place concrete slabs. Exterior walls may be either metal, precast concrete panels, or brick masonry.

Gravity Loads. Gravity loads are transferred from floor and roof slabs to the floor/roof framing elements composed of steel beams or open web joists. Floor/roof girders are supported by steel columns that transfer loads to the foundation.

Lateral Loads. Lateral loads are transferred from the floor/roof diaphragms to the moment resisting perimeter frames. Moment frame action between the steel girders and columns is produced by full or partial moment connections.

Standard Construction. The designation of *Standard* construction for this building type pertains to buildings built between 1960 and 1976, prior to the enforcement of modern seismic provisions.

Nonstandard Construction. *Nonstandard* moment resisting steel perimeter frame buildings are those built prior to 1960, the initial benchmark year in which earthquake design standards were substantially improved. Estimates of damage would be equivalent to that for one intensity higher than for *Standard* construction.

Special Construction. The designation of *Special* construction pertains to buildings built in 1976 or thereafter, a second benchmark year when substantial improvements in seismic design provisions were assumed to be in widespread enforcement throughout California. Estimates of damage would be equivalent to that for one intensity lower than for *Standard* construction.

TABLE 2-XIII Detailed Description of Moment Resisting Steel Distributed Frame

General Description. Similar to moment resisting steel perimeter frames, the structural systems of moment resisting steel distributed frame buildings are comprised of steel columns, beams and girders. In this building type, however, lateral loads are resisted by the moment action of the entire frame, i.e., by both the interior and exterior girder-column connections. The roof and floor diaphragms are generally composed of either metal decking with concrete fill or cast-in-place concrete slabs. Exterior walls may be either metal, precast concrete panels, or brick masonry.
Gravity Loads. Gravity loads are transferred from floor and roof slabs to the floor/roof framing elements composed of steel beams or open web joists. Floor/roof girders are supported by steel columns that transfer loads to the foundation.
Lateral Loads. Lateral loads are transferred from the floor/roof diaphragms to the moment resisting frames located throughout the structure. Moment frame action between the steel girders and columns is produced by full or partial moment connections.
Standard Construction. The designation of *Standard* construction for this building type pertains to buildings built in 1960 or thereafter.
Nonstandard Construction. *Nonstandard* moment resisting steel distributed frame buildings are those built prior to 1960, the initial benchmark year in which earthquake design standards were substantially improved. Estimates of damage would be equivalent to that for one intensity higher than for *Standard* construction.
Special Construction. The designation of *Special* construction pertains to buildings of special design built after 1976, a second benchmark year when substantial improvements in seismic design provisions were assumed to be in widespread enforcement throughout California. Estimates of damage would be equivalent to that for one intensity lower than for *Standard* construction.

TABLE 2-XIV Detailed Description of Moment Resisting Ductile Concrete Frame

General Description. The structural systems of moment resisting ductile concrete frame buildings are comprised of concrete columns, joists, beams and girders. The term "ductile" indicates that the frame meets certain concrete confinement and reinforcing anchorage details that were specified for buildings over 160 feet in height built in California after 1967 and for all concrete frames built in California after 1976. The roof and floor diaphragms are typically composed of cast-in-place concrete slabs. Exterior walls may be veneer or cladding of various materials.

Gravity Loads. Gravity loads are transferred from floor and roof slabs to the floor/roof framing elements such as one-way joists or waffle joists, or through flat slab action to large beams or girders. Concrete columns support the major floor framing elements and transfer gravity loads to the foundation.

Lateral Loads. Lateral loads are transferred from the floor/roof slabs to the moment resisting frames.

Standard Construction. The designation of *Standard* construction for this building type pertains to all concrete frame buildings over 160 feet in height built in California after 1967 and to all concrete frames built after 1976.

Nonstandard Construction. Due to the expected similar performance of *Standard* ductile concrete frame buildings, there is no designation of *Nonstandard* construction for this building type.

Special Construction. Due to the expected similar performance of *Standard* ductile concrete frame buildings, there is no designation of *Special* construction for this building type.

TABLE 2-XV Detailed Description Moment Resisting Non-Ductile Concrete Frame

General Description. Generally similar to moment resisting ductile concrete frames, the structural systems of moment resisting non-ductile concrete frame buildings are comprised of concrete columns, joists, beams and girders. The term "non-ductile" indicates that the frame does not meet certain concrete confinement and reinforcing anchorage details that were specified for buildings over 160 feet in height built in California after 1967 and for all concrete frames built in California after 1976. The roof and floor diaphragms are typically composed of cast-in-place concrete slabs. Exterior walls may be veneer or cladding of various materials.
Gravity Loads. Gravity loads are transferred from floor and roof slabs to the floor/roof framing elements such as one-way joists or waffle joists, or through flat slab action to large beams or girders. Concrete columns support the major floor framing elements and transfer gravity loads to the foundation.
Lateral Loads. Lateral loads are transferred from the floor/roof slabs to the moment resisting frames.
Standard Construction. The designation of *Standard* construction for this building type pertains to all concrete frame buildings built in or before 1967 and to all concrete frame buildings less than 160 feet in height built in or before 1976.
Nonstandard Construction. Due to the expected similar performance of *Standard* non-ductile concrete frame buildings, there is no designation of *Nonstandard* construction for this building type.
Special Construction. Due to the expected similar performance of *Standard* non-ductile concrete frame buildings, there is no designation of *Special* construction for this building type.

TABLE 2-XVI Detailed Description of Precast Concrete Construction

General Description. Buildings of this type have structural systems comprised of precast concrete frames and/or shear walls, which may be cast-in-place or precast panels. Roof and floor diaphragms are typically composed of precast concrete elements with or without cast-in-place concrete topping slabs. Closure strips between precast floor elements and beam-column joints are usually cast-in-place concrete. Welded steel inserts are often used to interconnect precast elements.
Gravity Loads. Gravity loads are transferred from precast floor/roof elements to precast concrete girders. Floor/roof girders are supported by precast concrete columns and/or concrete shear walls that transfer the loads to the foundation.
Lateral Loads. Lateral loads are transferred from the floor/roof diaphragms to the concrete shear walls or the moment resisting precast frames.
Standard Construction. All buildings of this type are assumed to be of *Standard* construction. Existing buildings of this type are expected to perform in a similar fashion, as there are no easily discernible differences in design and construction practices, particularly with respect to design date.
Nonstandard Construction. There is no designation of *Nonstandard* construction for this structure type.
Special Construction. There is no designation of *Special* construction for this structure type.

TABLE 2-XVII Detailed Description of Long Span Construction

General Description. Long span buildings typically house facilities, such as gymnasiums or auditoriums, that require large open areas. Typically these building types are low rise, with roof systems supported by long-span steel or wood trusses. Exterior bearing walls are normally shear walls of reinforced masonry or concrete, but may have frames of steel.
Gravity Loads. Gravity loads are transferred from the roof diaphragm to the wood or steel trusses. The trusses span to the perimeter bearing walls, which transfer the loads to the foundations.
Lateral Loads. Lateral loads are transferred from the roof diaphragm by the steel or wood trusses to the exterior bearing walls, which are typically designed to carry 100% of the lateral forces.
Standard Construction. All buildings of this type are assumed to be of *Standard* construction. Existing buildings of this type are expected to perform in a similar fashion, as there are no easily discernible differences in design and construction practices, particularly with respect to design date.
Nonstandard Construction. There is no designation of *Nonstandard* construction for this structure type.
Special Construction. There is no designation of *Special* construction for this structure type.

TABLE XVIII Detailed Description of Tilt-up Construction

General Description. Buildings of this type are low-rise structures with precast concrete wall panels that are often poured on the ground and "tilted" into place. The wall panels may or may not be interconnected with poured-in-place concrete corbels. Roof diaphragms are generally composed of plywood sheathing, but may consist of metal deck with or without concrete fill, or precast concrete elements. Floor diaphragms are typically metal deck with concrete fill, plywood, or precast concrete elements.

Gravity Loads. Gravity loads are transferred from floor and roof diaphragms to the wood or steel joists and beams, or open web joists. The major floor framing elements span to the exterior bearing walls or interior columns, which transfer the loads to the foundations.

Lateral Loads. Lateral loads are transferred from the diaphragms to the exterior bearing walls. The precast walls may act as single elements, or as a succession of individual panels, depending on the shear capacity of the connection between panels.

Standard Construction. The designation of *Standard* construction for this building type pertains to structures built before 1973, i.e., prior to the enforcement of modern seismic provisions.

Nonstandard Construction. There is no designation for *Nonstandard* construction.

Special Construction. The designation of *Special* construction pertains to buildings built in 1973 or thereafter, when modern seismic provisions were assumed to be in widespread enforcement throughout California. Estimates of damage would be equivalent to that for one intensity lower than for *Standard* construction.

TABLE 2-XIX Detailed Description of Mobile Homes

General Description. Mobile homes are prefabricated dwelling units that are transported to the housing site on wheels or truck-pulled platforms. At the site the units are placed on isolated piers and leveled, and, in some cases, masonry block foundations may be constructed. Floor and roof diaphragms and walls are typically constructed of plywood; outside surfaces are often covered with sheet metal.
Gravity Loads. Gravity loads are transferred from floor and roof diaphragms to the walls, which are supported on isolated piers or masonry block foundations.
Lateral Loads. Lateral loads are transferred from the floor/roof diaphragms to the foundation piers or walls. Anchorage between the dwelling unit and the foundation piers or walls may or may not be provided.
Standard Construction. The designation of *Standard* construction pertains to mobile homes that are not anchored to their foundations.
Nonstandard Construction. There is no designation of *Nonstandard* construction for this structure type.
Special Construction. *Special* construction pertains to mobile homes that are anchored to their foundations. Estimates of damage would be equivalent to that for one intensity lower than for *Standard* construction.

SECTION 3
REVIEW OF EARTHQUAKE DAMAGE DATA

Several earthquakes have occurred in California since the completion of ATC-13, including Whittier Narrows and Loma Prieta. As a first step in converting the existing ATC-13 damage probability matrices to fragility curves, the investigators of this project reviewed sources of damage data for several California earthquakes. These included: Whittier Narrows 1987, Loma Prieta 1989, Coalinga 1983, San Fernando 1971, Long Beach 1933, and San Francisco 1906. Available data were reviewed in order to determine if they could be used to update existing DPMs. Ultimately, the goal is to use Bayesian analytical techniques to combine the existing expert opinion with earthquake data as they become available. Potential sources of data included existing technical and reconnaissance reports, FEMA Damage Survey Reports (DSR), city and county permit records, city and county databases developed specifically to track damaged properties, and red tag building reports. After reviewing much of the available data, both published and unpublished, it is clear that a more systematic form of data collection must become standard if damage vulnerability relationships are to be improved.

The following documents and sources were reviewed:

1) ATC - 31, Evaluation of the Performance of Seismically Retrofitted Buildings, 1992. This document contains the results of a study to evaluate the effectiveness of retrofitting techniques relative to the performance of retrofitted buildings in earthquakes. Data were collected through a questionnaire sent to Structural Engineers Association of California (SEAOC) members, city and county building departments and local engineering firms. The data include 113 retrofitted unreinforced masonry buildings, 43 retrofitted concrete tilt-up buildings and a few other buildings of unspecified structural type that were subjected to either the Loma Prieta or the Whittier Narrows earthquake. The results of the analyses are damage probability matrices (DPMs) for each of the structural types and retrofitting schemes. The DPMs were developed for both MMI and PGA as ground motion parameters. The damage scale is the same one used in ATC-13. The DPMs are compared with DPMs from the ATC-13 study and conclusions are drawn. These data are very useful and are in a format that could be used to update DPMs based on expert opinion.

2) USGS Bulletin 1939-A, Earthquake Losses to Single-Family Dwellings: California Experience, 1990. This study includes loss data primarily from the Long Beach, San Fernando, Coalinga and Whittier Narrows earthquakes. Data were collected from insurance companies in the form of paid insurance claims and from field inspections. The data are tabulated in terms of percent maximum probable loss and loss over deductible. This study includes a significant amount

of data, however, the data are not very useful for the development of fragility curves or DPMs since they do not include information on intensity or ground motion.

3) Thiel and Zsutty, Earthquake Parameters and Damage Statistics, 1987. The appendix of this document contains the raw data that were used in developing and testing the authors' model. Earthquakes included in the appendix are San Francisco - 1906, Santa Barbara - 1925, Long Beach - 1933, Kern County - 1952, Puget Sound - 1965, San Fernando -1971, Coalinga - 1983, Tangshan - 1976, Santa Barbara - 1978 (mobile homes only), and Imperial Valley - 1979 (mobile homes only). In most of the earthquakes, damage data are limited to commercial unreinforced masonry structures. The damage is summarized using the Wailes and Horner scale. This scale contains 5 damage states. Each damage state is associated with a damage range. For example, State A corresponds to 0% - 4% damage. The difficulty in using this set of data is that the summaries do not contain information on ground motion intensity or MMI. The Coalinga data include street addresses so that a correlation with MMI could be obtained.

4) FEMA - Damage Survey Reports (DSRs) for publicly-owned buildings in the Loma Prieta earthquake. The database contains 10,000 entries, of which approximately 3,600 are buildings. DSRs are submitted to FEMA by cities, counties, special districts, the State of California, and miscellaneous non-profit organizations. The DSRs are classified as follows: A) debris removal, B) emergency work, C) roads, D) utilities, E) buildings, F) civil work, and G) miscellaneous. In the buildings category, the DSR represents the cost of reconstruction rather than the cost of damage which might also include debris removal and emergency work. About 40% of the DSRs are based on the costs of reconstruction that has been completed or is in progress, while 60% of the DSRs are based on construction estimates. The difficulties in using this data set are 1) it does not contain information about ground motion intensity, 2) while it does contain zipcode information, exact addresses are not provided, 3) it does not contain information on structural type, 4) it does not contain information on total building value, thus making it difficult to estimate cost of damage relative to the value of the structure.

5) Hart et al., Masonry Building Performance Survey from the Whittier Narrows Earthquake, 1988. This database contains information on good and bad performance of all reinforced and unreinforced masonry buildings for selected cities in the epicentral region of the Whittier Narrows earthquake. For each building the following data were collected: address, year built, building size and shape, number of stories, ground motion, soil type, damage information, ATC damage state. This would be an excellent source of data for updating DPMs, however, the investigators were not able to obtain this database. The database in its electronic form appears to have been lost. Apparently hard copies of the original survey data do exist and may be useful in reconstructing the electronic database.

6) Rutherford and Chekene (1991), Damage to Unreinforced Masonry Buildings in the October 17, 1989 Loma Prieta Earthquake. Information was obtained for 6,878 unreinforced masonry buildings (strengthened and unstrengthened) in San Francisco, Santa Cruz, Watsonville, Hollister, Los Gatos, Salinas, San Jose, Campbell, Emeryville, Oakland, Gilroy and many other small communities in the 10 county area affected by the Loma Prieta earthquake. Several statistical studies were performed on the data. One of the most useful statistical analyses for incorporation into the current study contains summaries of MMI and ATC-13 rating for URMs in the cities of Campbell, Gilroy, Hollister, Los Gatos, Santa Cruz and Watsonville (169 buildings total). In another analysis, average damage ratios are compared for URMs built on different soil types. This may be useful in developing "fragility curve modifiers" to incorporate soil type. According to this document three PARADOX3 databases, containing information on performance of URMs in San Francisco and the surrounding areas during the Loma Prieta earthquake, are available.

7) City of Palo Alto Records. The City of Palo Alto has a list of all buildings that generated building permits as a result of the Loma Prieta earthquake. This list contains addresses, permit numbers, permit fees and plan check fees. From these fees one can estimate the cost of reconstruction. The majority of the buildings that were damaged were single- and multi-family wood framed residences. The list contains 85 records. Further information on these buildings could be collected by investigating the city files on a file-by-file basis. At the present time this list contains no information about structural type or amount and type of damage. A flaw in the use of this type of data is that reconstruction work that was done without a permit is omitted from the database. Similarly, upgrades and modifications to the structure cannot be easily separated from the costs of repairing only earthquake damage. Because of the enormous effort that would be required to evaluate the buildings on a file-by-file basis, perhaps the best use of a data source such as this is to eliminate all wood frame residences and only collect data on the other types of structures, which are few in number.

8) Santa Cruz County Planning Department Records. The County of Santa Cruz developed a computerized database to track structures that were damaged in the Loma Prieta earthquake. The database contains addresses and assessor's parcel numbers, two or three line verbal descriptions of the damage, and some information about what was done. In addition to these data, the County has maintained a list of properties with assessed values (land and improvements) before and after the earthquake. These two databases can be correlated through assessor's parcel numbers. Determining structural type from this database could be accomplished through use of the assessor's files and inference rules. However, this would be costly and thus was beyond the scope of this study. The County has also kept original copies of damage assessment forms which can be reviewed for additional information.

9) Red Tag Buildings Interim Status Report, Draft #10. (August 15, 1990). This document contains information on all the city and county of San Francisco buildings that were red tagged after the Loma Prieta earthquake. There were 376 buildings that fell into this category. The report for each building contains an entry for each time the structure was inspected and for each time the status of the building was changed (e.g. red to yellow, or red to demolish). In some cases the permit numbers and dates of issue are included. In addition to a history of the building's status, the report indicates address, owner, year built, UBC construction type (I through V), number of stories, number of dwelling units, whether or not the structure is unreinforced masonry, and a one line description of the damage to the building. There is little information that could be easily converted to percent damage. This type of document could be a first step in obtaining more information about structures that were damaged. For each structure, one would need to review city and county building department files to obtain information on type of building and repair work done. It should be mentioned that this document does not contain any information about buildings that were originally yellow tagged and thus does not include many structures. At the present time there is no yellow tag building report available. Furthermore, the data must be carefully studied and classified as some red tagged buildings are included in the database because an adjacent property was creating a life safety hazard. Some buildings that have significant damage do not appear in the data because they did not pose a life safety hazard.

10) ATC 25, Seismic Vulnerability and Impact of Disruption of Lifelines in the Conterminous United States, 1991. In this report vulnerability functions and restoration curves are developed for lifelines. The curves are based on a regression analysis of the expert opinion data in ATC-13. Descriptions of lifeline facilities and typical seismic damage are provided. No new damage data was available from this report.

11) French et al., Damage to Urban Infrastructure and Public Property from the Loma Prieta Earthquake, 1992. The FEMA Damage Survey Reports (DSRs) were used in their study to investigate damage patterns in roads, bridges, water and sewer systems and public buildings. This is the same FEMA database described earlier. The FEMA database does not include damage to federally supported highways or privately owned infrastructure such as telephone and electric power facilities. The authors compare the level and type of damage in the Loma Prieta earthquake to that in the Whittier Narrows earthquake. Detailed analyses were made for water and sewer systems. Damage probability matrices from inventories obtained from several communities were compared to the DPMs from ATC-13. The same difficulties were encountered using these data as were discussed earlier in relation to the FEMA - DSR database. However, the study by French et al. does contain a comparison of actual damage claims with those predicted from ATC-13, thus providing a useful means of calibration of the DPMs.

12) Yanev et al., The Performance of Steel Buildings in Past Earthquakes, 1991. This report includes detailed studies of the performance of individual steel buildings in 12 recent earthquakes. The study contains information on two California earthquakes, San Fernando and Loma Prieta. The San Fernando data is from a report published by Steinbrugge. Much of the Loma Prieta data is from an unpublished survey performed by the Building Owners and Managers Association of San Francisco. The information in this report is not useful for updating DPMs because it does not contain information on how many steel buildings were exposed to each earthquake, nor does it contain information about the ground motion for each building. However, since individual buildings are identified, ground motion information could be obtained. With a great deal of effort, the number of steel structures exposed to each earthquake could also be obtained.

In general, earthquake damage data are collected for purposes other than to correlate damage with ground motion. Government agencies and insurance companies are trying to assess the extent (dollar value) of the damage, and thus are not much interested in issues such as structural type or ground motion intensity. Much of the data that are collected after an earthquake are not usable for the purpose of developing or updating DPMs or fragility curves because these data are missing valuable components such as structural type, building height or square footage, ground motion intensity, a consistent indicator of level of damage, or a location indicator to correlate data with intensity maps. Often in reviewing records, it is difficult to determine what percentage of costs can be attributed to other factors such as demolition, debris removal, or seismic upgrading. A very troubling and prevalent problem is the inadequate inventories of existing buildings. Without an estimate of the total number of buildings in a particular class, it is difficult to determine the number of structures in the class "undamaged".

In a few cases, and for a limited number of facility classes, extensive data have been collected that can or have been used in developing DPMs (ATC, 1992; Rutherford and Chekene, 1991 and 1993). In these studies the investigators systematically collected data on damage to URMs after the Loma Prieta earthquake and compiled the data in a usable format. As a first step in developing a technique to combine damage data with relationships derived from expert opinion, the URM data from Rutherford and Chekene (R & C) were compared to the ATC-13 Beta cumulative probability distributions. R & C "Level 2" data, consisting of 2,356 URMs from nine cities, were used to plot empirical cumulative probability distributions of damage factor (dollar loss/replacement value) for URMs subjected to different levels of modified Mercalli intensity (MMI = VI, VII, VIII, IX).

An example of this comparison for MMI=VI is shown in Figure 3-1. The solid line in this figure represents the empirical cumulative probability distribution derived from 592 buildings. The R & C data set was not sorted by building height, thus comparisons are made with both low rise (ATC-13 facility type 75) and medium rise (ATC-13 facility type 76) URM bearing wall buildings. The comparison suggests that the ATC-13 curves overestimate (at least for the Loma Prieta earthquake) the damage for URMs. From Figure 3-1, the ATC-13 curves predict that the damage factor is less than or equal to 5% with a probability of about 0.5. In comparison the data gives an estimate of this probability to be about 0.9. In fact, 409 out of 592 buildings in the MMI=VI zone were in the state "no damage", whereas the ATC-13 damage probability distributions would suggest almost no buildings in the "no damage" state. The deviation of the ATC-13 curves from the URM damage data becomes more pronounced with larger MMI as shown in Figure 3-2. This figure suggests that even with relatively large ground motions a large number of the URM buildings experience light to moderate damage, whereas ATC-13 predicts the majority of structures to be in the heavy to major damage state. It should be noted that only 27 data points were used to derive the empirical cumulative probability distribution in Figure 3-2.

The discrepancies between the data and the ATC-13 damage probabilities confirms the need to develop a methodology to combine new data with the existing curves and update the curves as new earthquake damage data become available. However, it should be noted when making comparisons, as was done in Figures 3-1 and 3-2, that these data are only from the Loma Prieta earthquake and may not be representative of all California earthquakes. Secondly, the plotted ATC-13 curve represents a best estimate of the damage probability distribution, and uncertainty in this estimate is not shown. Thus comparing the ATC-13 distributions with one earthquake may be misleading. For example, it is possible that the database is biased by San Francisco buildings that have complied with the parapet law by bracing parapets. These buildings were included in the database as unstrengthened URMs. Of the 6,716 buildings in the database 1,962 are in San Francisco. Furthermore, while ATC-13 provides some generalized definitions of what is meant by damage in each of the seven damage states, it does not give descriptions of damage states for each facility class. Thus a great deal of interpretation is used when collecting data which may skew the data one way or another.

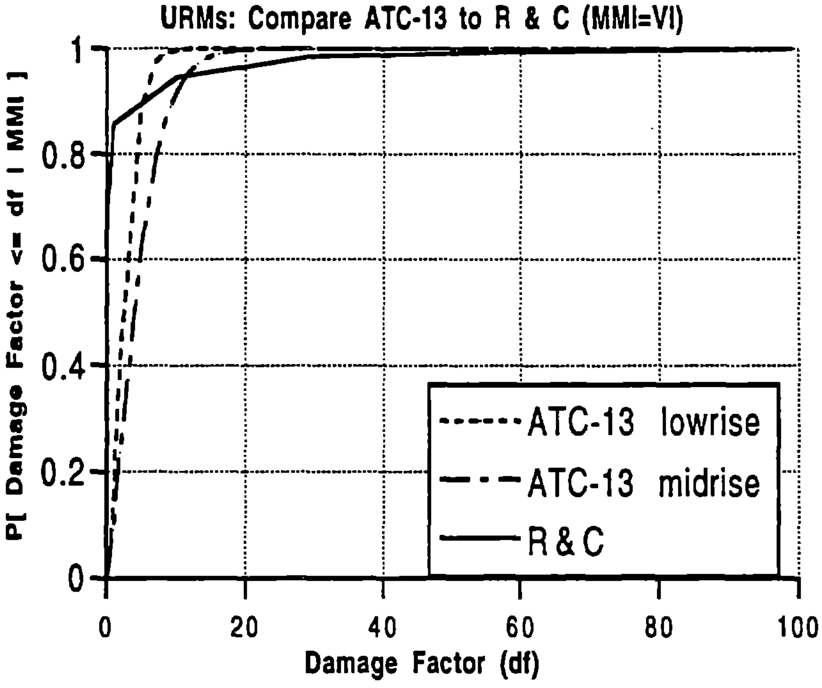


FIGURE 3-1 Comparison of "Level 2" unreinforced masonry building damage data from Rutherford and Chekene (1993) with ATC-13 Beta cumulative probability distributions for URM bearing wall buildings (facility types 75 and 76). The data is for URMs subjected to MMI = VI.

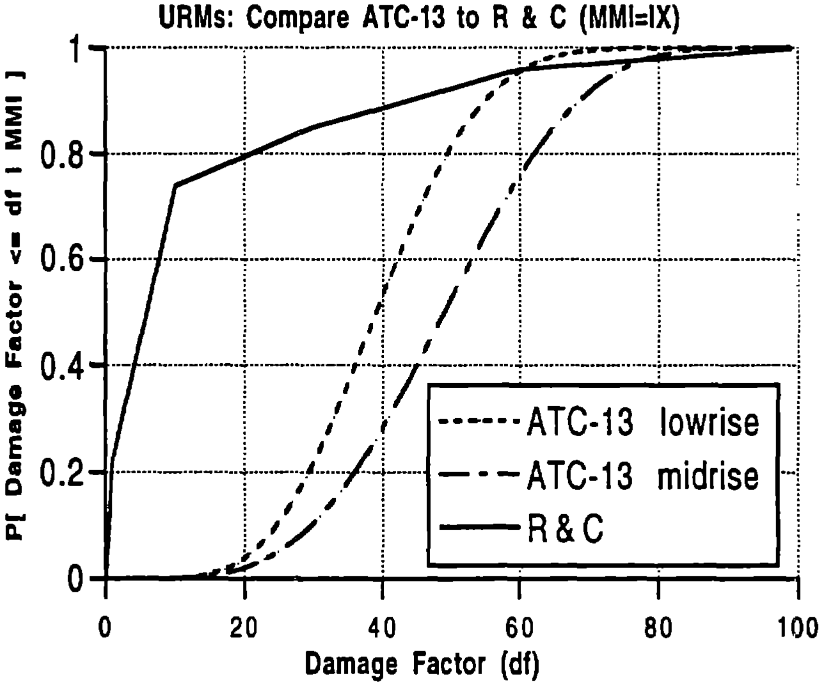


FIGURE 3-2 Comparison of "Level 2" unreinforced masonry building damage data from Rutherford and Chekene (1993) with ATC-13 Beta cumulative probability distributions for URM bearing wall buildings (facility types 75 and 76). The data is for URMs subjected to MMI = IX.

SECTION 4
DEVELOPMENT OF FRAGILITY RELATIONSHIPS

Damage probabilities for structures and components of structures or equipment are often expressed in the form of fragility curves. In this project the ATC-13 DPMs have been transformed into a fragility curve representation. There are several advantages to this representation. One advantage is that graphical representation of damage motion relationships as fragility curves provides a visual means of comparing damage probabilities for different building types. Another advantage is that researchers are developing fragility curves for some building types based on experimental data or analytical techniques. These curves can be compared with fragility curves from expert opinion and perhaps combined. In addition in a very few cases, the authors were able to identify inconsistencies in the expert opinion that caused damage probabilities to decrease with increasing MMI. This occurred only for two buildings classes. These inconsistencies were removed in developing the fragility curves.

Fragility curves were originally developed for use in the nuclear industry, and in that application the curves represented a plot of probability of failure or frequency of failure versus some input parameter such as spectral acceleration or zero period acceleration. In this study the definition of fragility curve is modified to represent the probability of experiencing some damage level or damage state as a function of ground motion. Since multiple damage states are defined, multiple fragility curves are developed for each building type. This type of representation of damage motion relationships has been used in previous loss studies such as the regional loss study of the Mississippi Valley (Allen and Hoshall, 1985).

The damage states used in this study are the same as those defined in ATC-13. These damage states are summarized in Table 4-I below.

Damage factor is defined as the cost of repair divided by the replacement cost. Each state is associated with a range of damage factors. The central damage factor is defined as the midpoint value of the damage factor range.

TABLE 4-I ATC-13 Damage States

Damage State	Description	Damage Factor Range (%)	Central Damage Factor (%)
1	None	0	0
2	Slight	0 -1	0.5
3	Light	1 -10	5
4	Moderate	10 -30	20
5	Heavy	30 - 60	45
6	Major	60 -100	80
7	Destroyed	100	100

The statistical summaries of the expert opinion data found in ATC-13, Appendix G, were used as a basis for transforming the DPMs to lognormal fragility relationships for the 40 building classes described in Section 2 of this report. The statistics in Appendix G consist of best estimates of the low, mean and high damage factor, in percent, for a given California building type when subjected to a specified level of ground motion (MMI). An example is shown in Table 4-II for building class 1, low rise wood frame.

TABLE 4-II Summaries of Best Estimates of Damage Factors for Low Rise Wood Frame Structures (From ATC-13)

MMI	Low Damage Factor (%)	Mean Damage Factor (%)	High Damage Factor (%)
6	0.2	0.8	2.6
7	0.7	1.5	4.8
8	1.8	4.7	11.0
9	4.5	9.2	19.7
10	8.8	19.8	39.7
11	14.4	24.4	47.3
12	23.7	37.3	61.3

In ATC-13, beta probability distributions were fitted to the above statistics for each building class and each MMI. The low and high damage factor estimates were defined in the surveys as the 90% probability bounds. The best estimate and 90% probability bounds were used in developing the parameters of the beta distributions. That is, for a given MMI there is a probability of 0.9 that the damage factor will be between the low and high estimates; and the best estimate represents the mean value of damage. Using this information, the parameters of the beta distribution at each intensity level were estimated. These parameters completely define the equation of the distribution. An example of the beta probability density function is shown in Figure 3 below. Beta distributions can be symmetrical, skewed to the left or skewed to the right depending on the values of its parameters. This characteristic was one of the main reasons for selecting the beta distribution to represent probabilities of damage. For low levels of ground motion, it is expected that the probability density function will be skewed to the left, that is toward lower levels of damage. At higher levels of ground motion the probability density function will be skewed to the right reflecting the greater likelihood of higher level of damage. The DPMs found in ATC-13 were developed by discretizing these beta distributions according the ranges of damage factor given in Table 4-I.

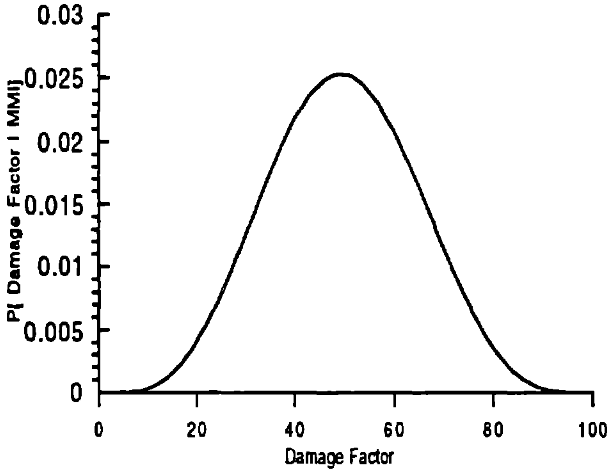


FIGURE 4-1 Typical beta probability density function developed for each building type and MMI level in ATC-13.

A beta probability density function such as the one shown in Figure 4-1 was developed for each building type and each MMI. Thus for each building type, seven Beta probability density functions were developed (MMI = VI, VII, VIII, IX, X, XI, XII). For a given building type and MMI, the density function can be used to determine the probability of experiencing a certain level of damage (expressed by damage factor, df). The probability that the damage is in the range df to df + Δdf is represented by the area under the probability density function between df and df + Δdf. To determine the probability that the damage will be greater than or equal to a certain level, for example greater than or equal to 60%, the area under the curve must be computed as shown in Figure 4-2.

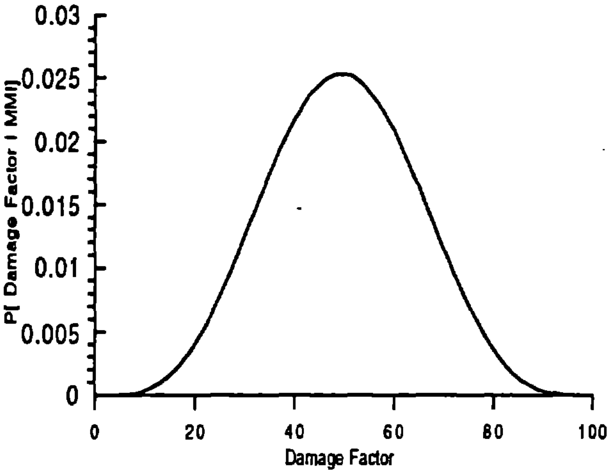


FIGURE 4-2 Area representing the probability of experiencing damage greater than or equal to 60%

As discussed earlier, the fragility curves represent the probability of experiencing damage greater than or equal to some damage state as a function of ground motion. For example, to be in state moderate or worse, the level of damage must be greater than or equal to 10% (see Table 4-I). Thus, for each facility class and MMI, fragility curves were developed by integrating the beta density functions to develop probabilities of experiencing a damage factor (df) greater than or equal to a specified level. Using least squared error techniques, lognormal curves were fitted through the resulting points. A lognormal probability density function is of the form:

$$f_Y(y) = \frac{1}{y \;\sigma_X \;\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{\ln y \; - \; m_X}{\sigma_X} \right)^2 \right] \qquad (4.1)$$

where X=ln(Y), and σ_X and m_X are parameters of the function. Specifically, σ_X is the standard deviation of ln(Y) and m_X is the mean (or median) of ln(Y). The lognormal cumulative probability function is of the form:

$$P[Y \le y] = F_Y(y) = \int_0^y \frac{1}{y \;\sigma_X \;\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{\ln y \; - \; m_X}{\sigma_X} \right)^2 \right] dy \qquad (4.2)$$

This integral does not have a closed form solution but is tabulated in a standardized.

The lognormal relationships that were used to represent the fragility curves are similar to that shown in equation 4-2. For the fragility curves y represents MMI and x represents ln(MMI). The functions are of the form:

$$P[\text{damage factor} \geq \text{df} \mid \text{MMI}] = \int_0^{\text{MMI}} \frac{1}{\text{MMI}\ \sigma_x \sqrt{2\pi}} \exp\left[-\frac{1}{2} \left(\frac{\ln \text{MMI} - m_x}{\sigma_x} \right)^2 \right] d\text{MMI} \quad (4\text{-}3)$$

It should be understood that equation 4-3 is not a cumulative probability function of MMI nor of damage factor. Instead it is a function with the same form as the lognormal cumulative probability function that describes the probability of experiencing a given damage factor or larger as a function of MMI. Lognormal relationships were chosen because of computational efficiencies that can occur when combining multiple curves. In most cases the lognormal functions fit the discrete points developed from ATC-13 expert opinion very well. Figure 4-3 shows a comparison of the best fit lognormal fragility curves with the data points for low rise wood frame structures.

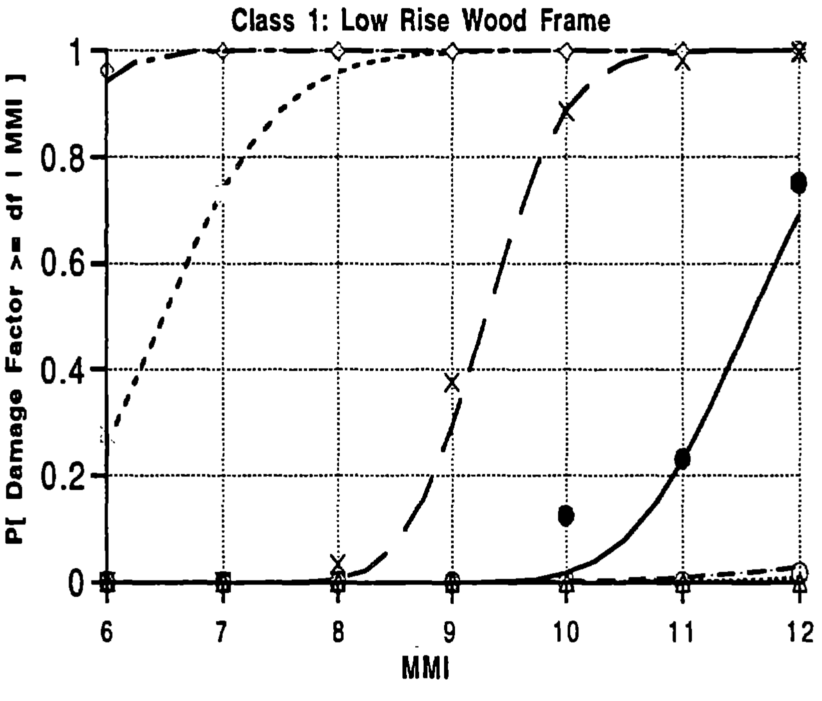


FIGURE 4-3 Comparison of best fit lognormal fragility curves with points used to generate the curves.

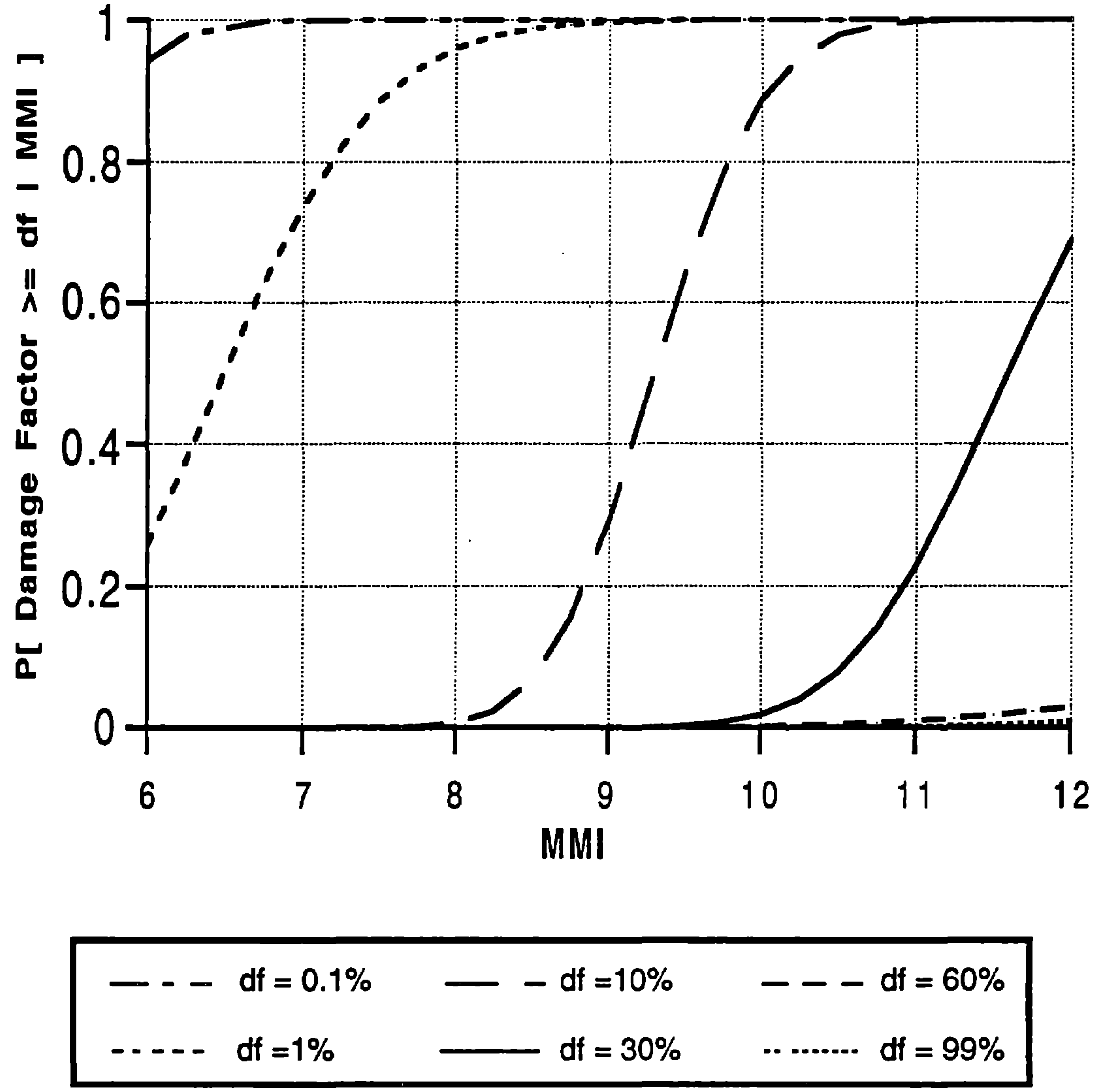
The fragility curves for the 40 ATC-13 building classes are shown in Figures 4-4 through 4-48. Each figure contains six curves. Each curve represents the probability of experiencing damage equal to or more severe than a particular damage state. For example, the dashed curve representing P[damage factor ≥ 10% | MMI] is the probability of being in the moderate damage state (damage state 4 =10% to 30%) or in one of the more severe damage states (heavy, major or destroyed). The solid line, P[damage factor ≥ 30% | MMI] represents the probability of being in damage states heavy, major or destroyed. The parameters for each of these curves σ_X and m_X, denoted respectively as the "standard deviations of ln(MMI)" and the "median of ln(MMI)", are listed in the tables below the fragility curves for each structural class. It is again emphasized that these values are only used to define the fragility curves and are not probability distributions of MMI, as may be implied by the use of the terminology for the parameters σ_X and m_X.

In comparing the fragility curves for the 40 building classes some similarities in fragility curves were noted. These similarities are summarized in Table 4-III.

TABLE 4-III Building Classes with Similar Fragility Curves

Building Class	Similar to Building Class
3 - Low rise RC Shear Wall w/ MRF	84 - Low rise RM Shear Wall w/ MRF
4 - Medium rise RC Shear Wall w/ MRF	85 - Medium rise RM Shear Wall w/ MRF
5 - High rise RC Shear Wall w/ MRF	86 - High rise RM Shear Wall w/ MRF
6 - Low rise RC Shear Wall w/o MRF	9 - Low rise RM Shear Wall w/o MRF
7 - Medium rise RC Shear Wall w/o MRF	10 - Medium rise RM Shear Wall w/o MRF
8 - High rise RC Shear Wall w/o MRF	11 - High rise RM Shear Wall w/o MRF
16 -Medium Rise Moment Resisting Steel Perimeter Frame	19 -Medium Rise Moment Resisting Ductile Concrete Distributed Frame
17 -Medium Rise Moment Resisting Steel Perimeter Frame	20 -High Rise Moment Resisting Ductile Concrete Distributed Frame

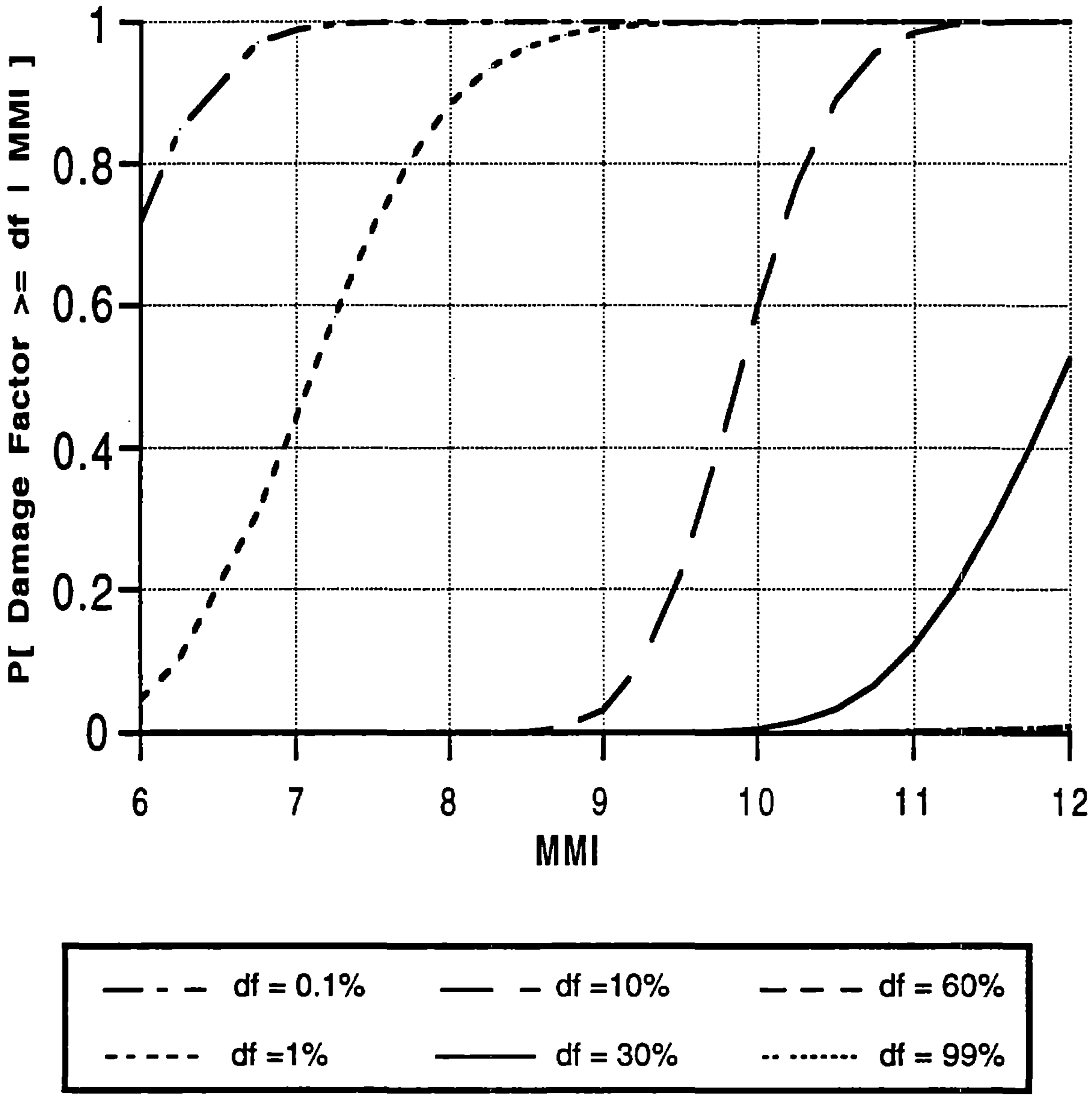
This information may be useful in reducing the total number of curves (facility classes). The justification for reducing the number of facility classes is that, it may be difficult to distinguish building construction types from available inventories when performing regional loss estimation studies. For example, a moment resisting steel perimeter frame may be impossible to distinguish from a moment resisting distributed frame without a detailed investigation of a structure. Individual site visits are not common in developing regional inventories, except for unique or important structures. Therefore if the fragility curves of two facility classes are comparable, it is logical to combine these into one class with one DPM or one set of fragility curves.



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 1

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.65	0.0898
1	1.87	0.12
10	2.23	0.0599
30	2.45	0.0699
60	2.84	0.188
99	2.84	0.15

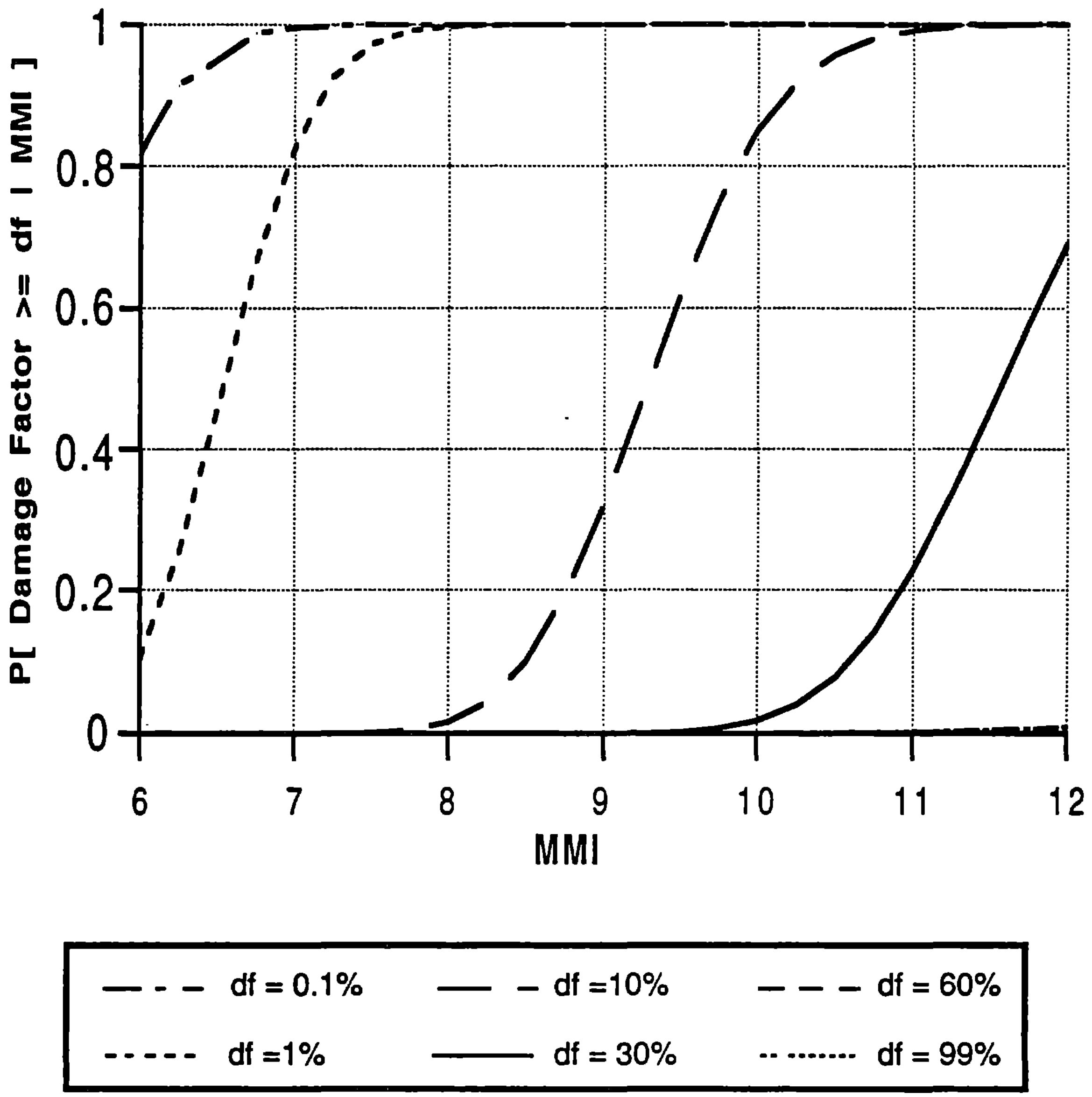
FIGURE 4-4 Fragility curves and parameters for Structure Class 1, Low Rise Wood Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 2

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.74	0.0898
1	1.96	0.0998
10	2.29	0.05
30	2.48	0.0699
60	2.84	0.15
99	2.84	0.15

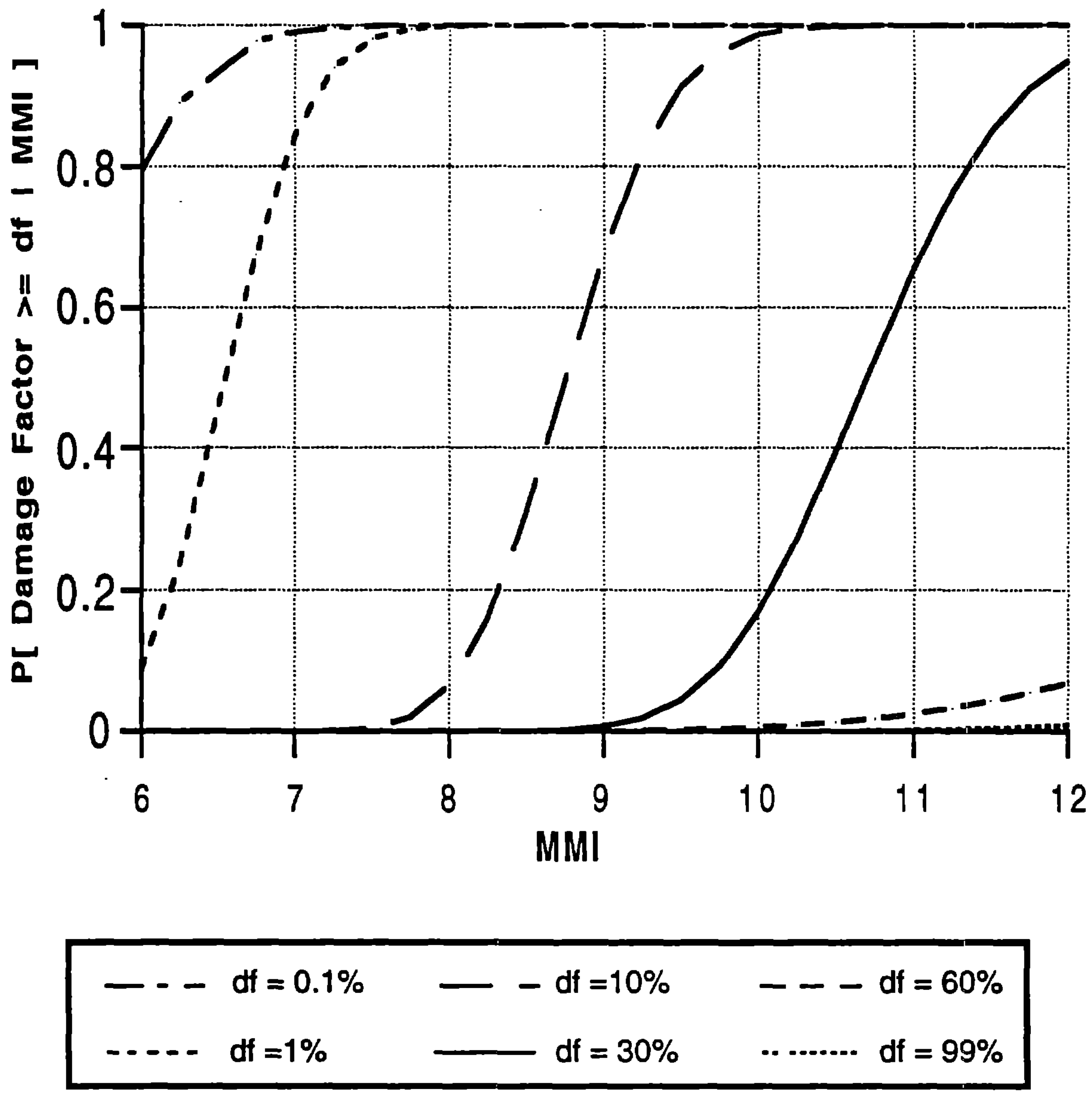
FIGURE 4-5 Fragility curves and parameters for Struture Class 2, Light Metal



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 3

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.71	0.0898
1	1.88	0.07
10	2.23	0.07
30	2.45	0.0699
60	2.84	0.15
99	2.84	0.15

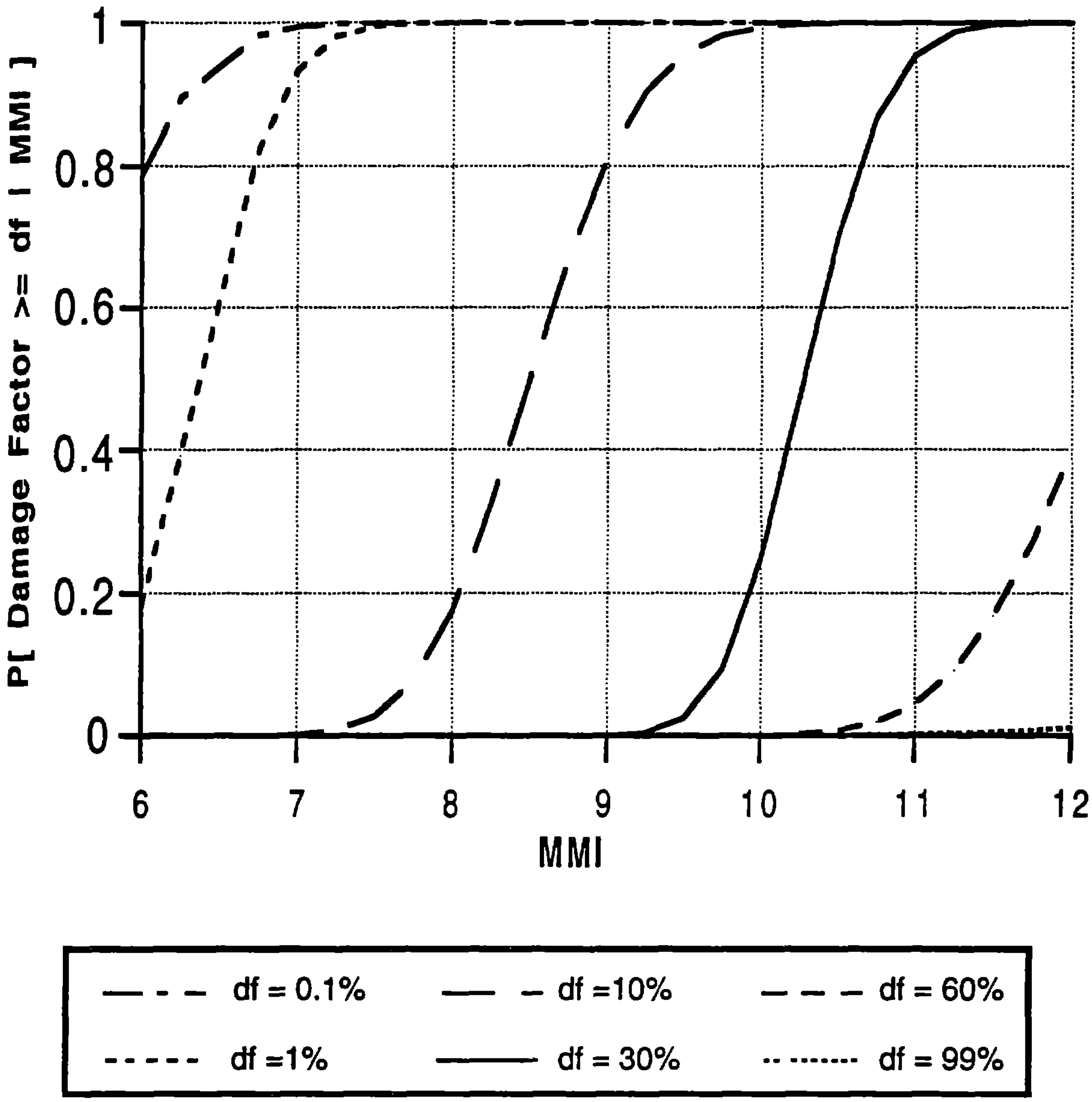
FIGURE 4-6 Fragility curves and parameters for Structure Class 3, Low Rise RC Shear Wall (w/ MRF)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 4

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.71	0.0998
1	1.88	0.065
10	2.17	0.0599
30	2.37	0.07
60	2.75	0.179
99	2.84	0.15

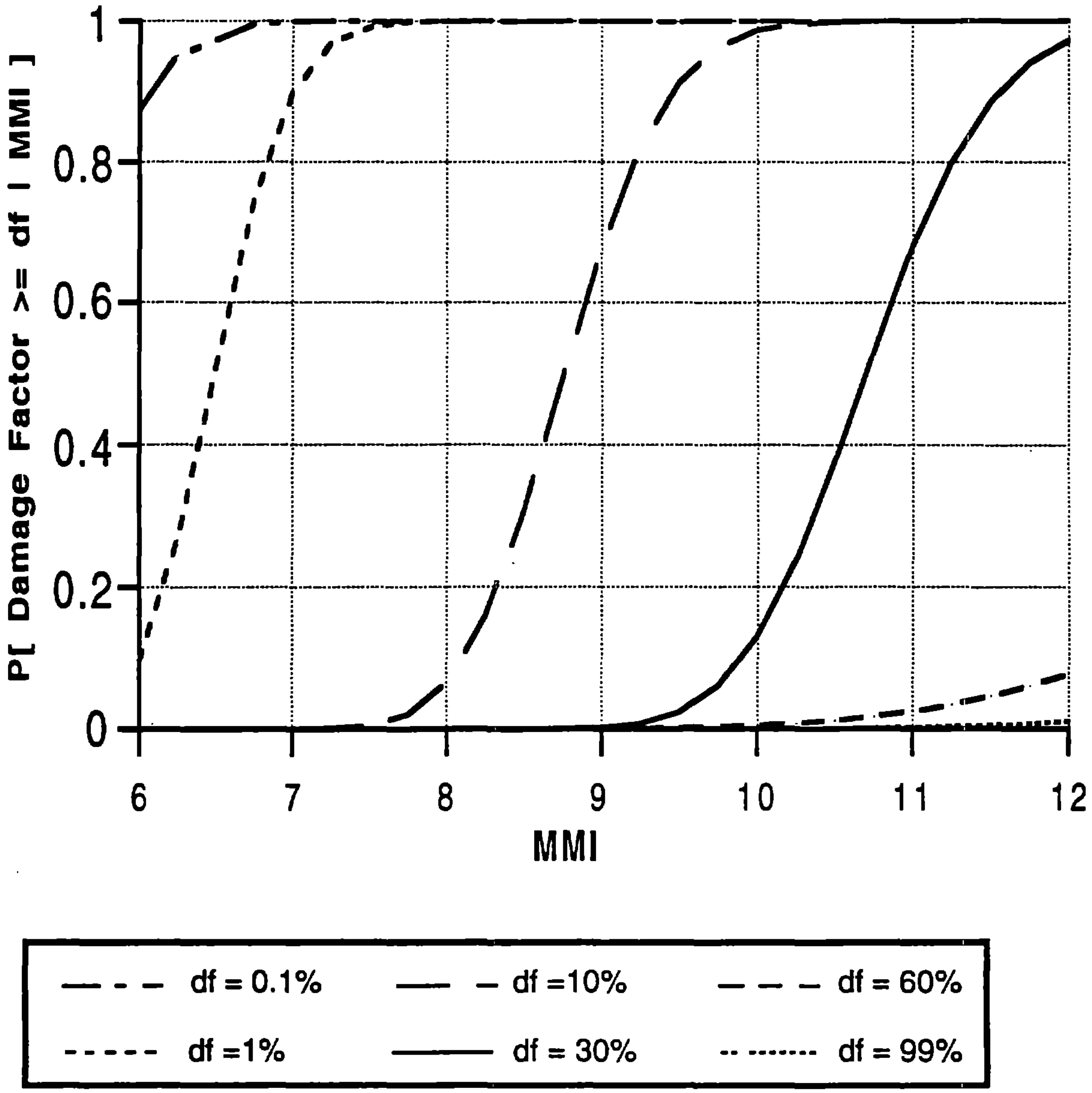
FIGURE 4-7 Fragility curves and parameters for Structure Class 4, Medium Rise RC Shear Wall (w/ MRF)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 5

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.72	0.0898
1	1.85	0.064
10	2.14	0.065
30	2.33	0.04
60	2.50	0.0599
99	2.83	0.15

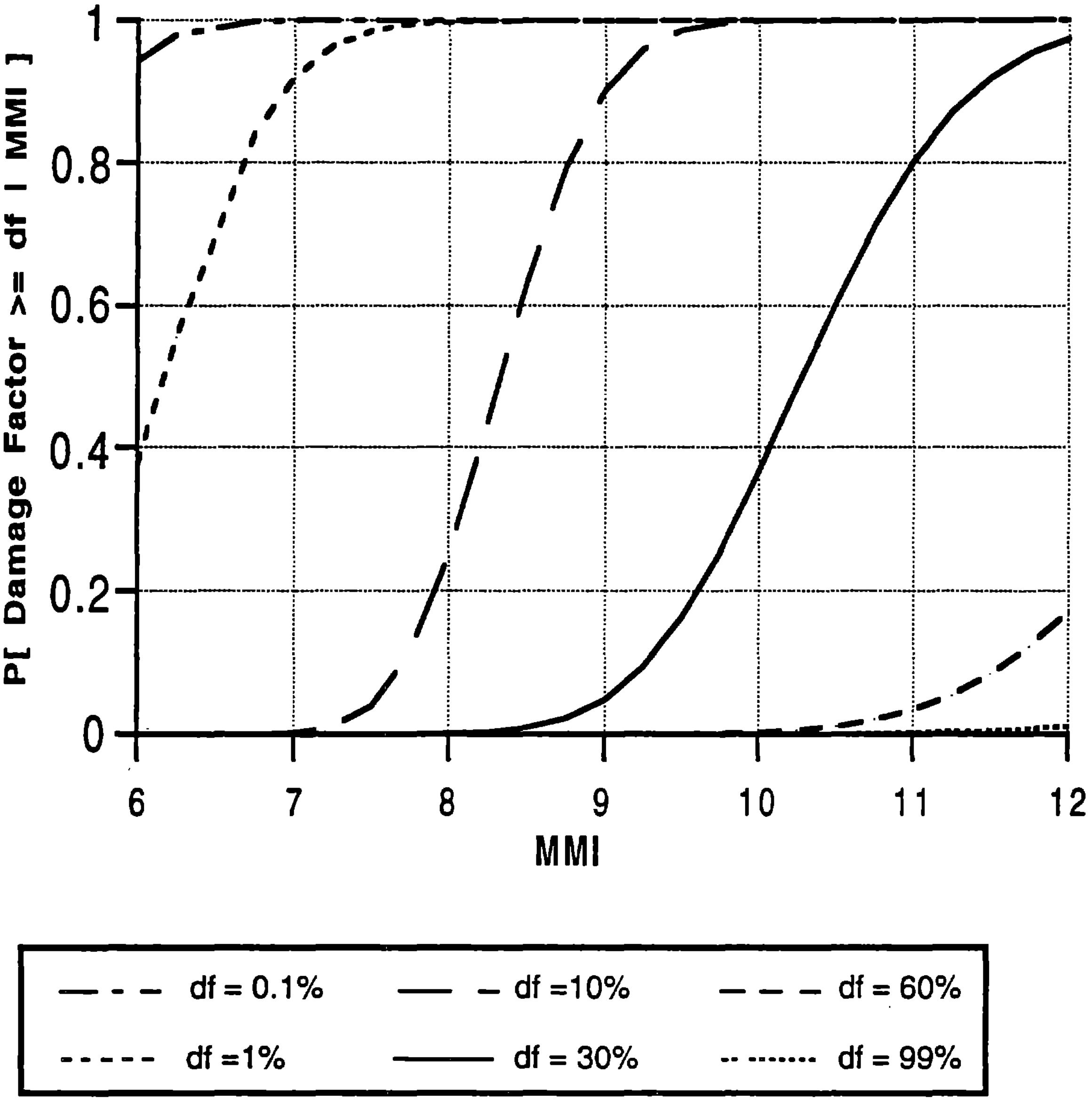
FIGURE 4-8 Fragility curves and parameters for Structure Class 5, High Rise RC Shear Wall (w/ MRF)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 6

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.70	0.08
1	1.87	0.0599
10	2.17	0.0599
30	2.37	0.0599
60	2.71	0.159
99	2.83	0.15

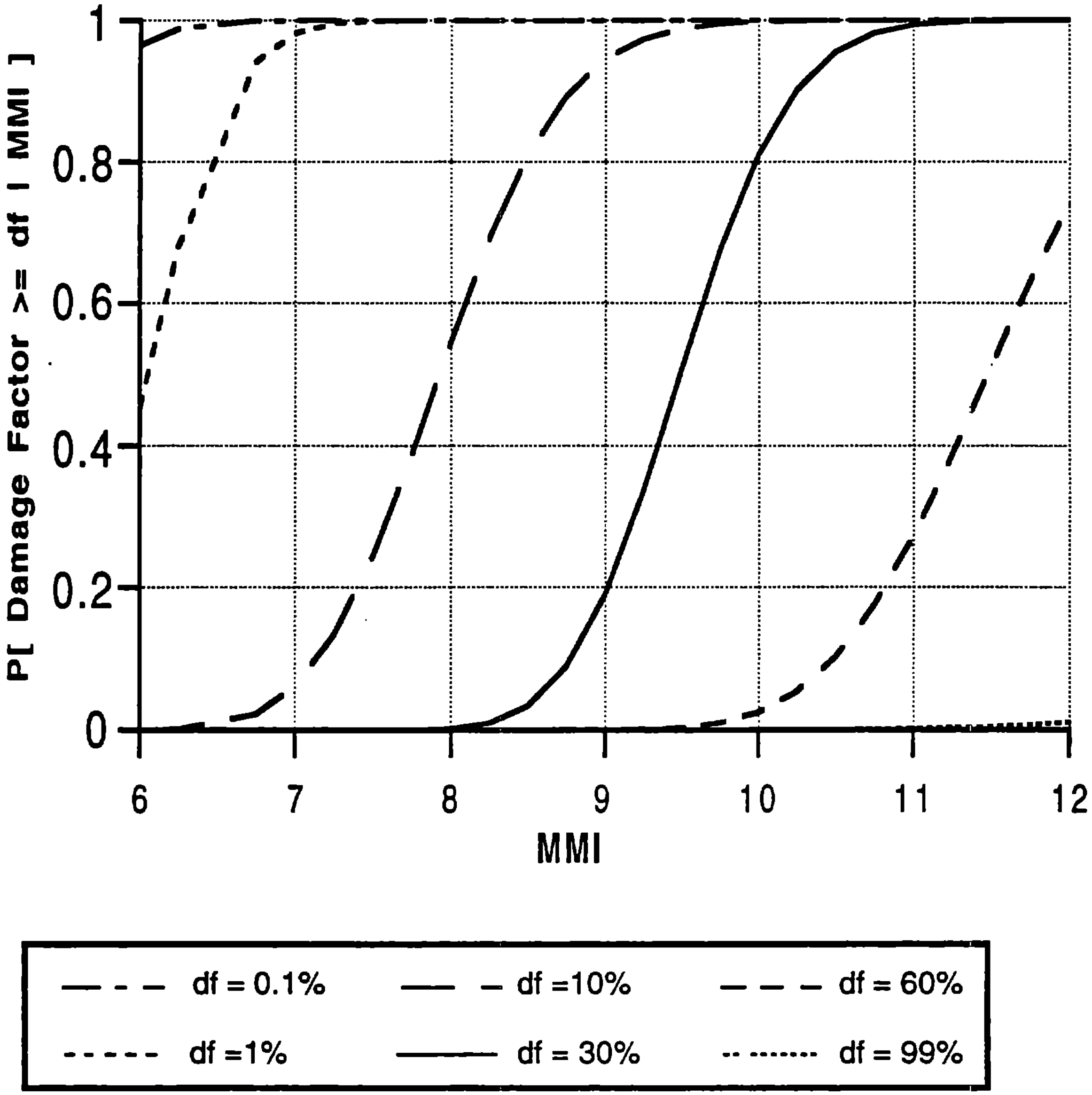
FIGURE 4-9 Fragility curves and parameters for Structure Class 6, Low Rise RC Shear Wall (w/o MRF)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 7

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.65	0.0898
1	1.82	0.09
10	2.12	0.0599
30	2.33	0.0799
60	2.58	0.0998
99	2.83	0.15

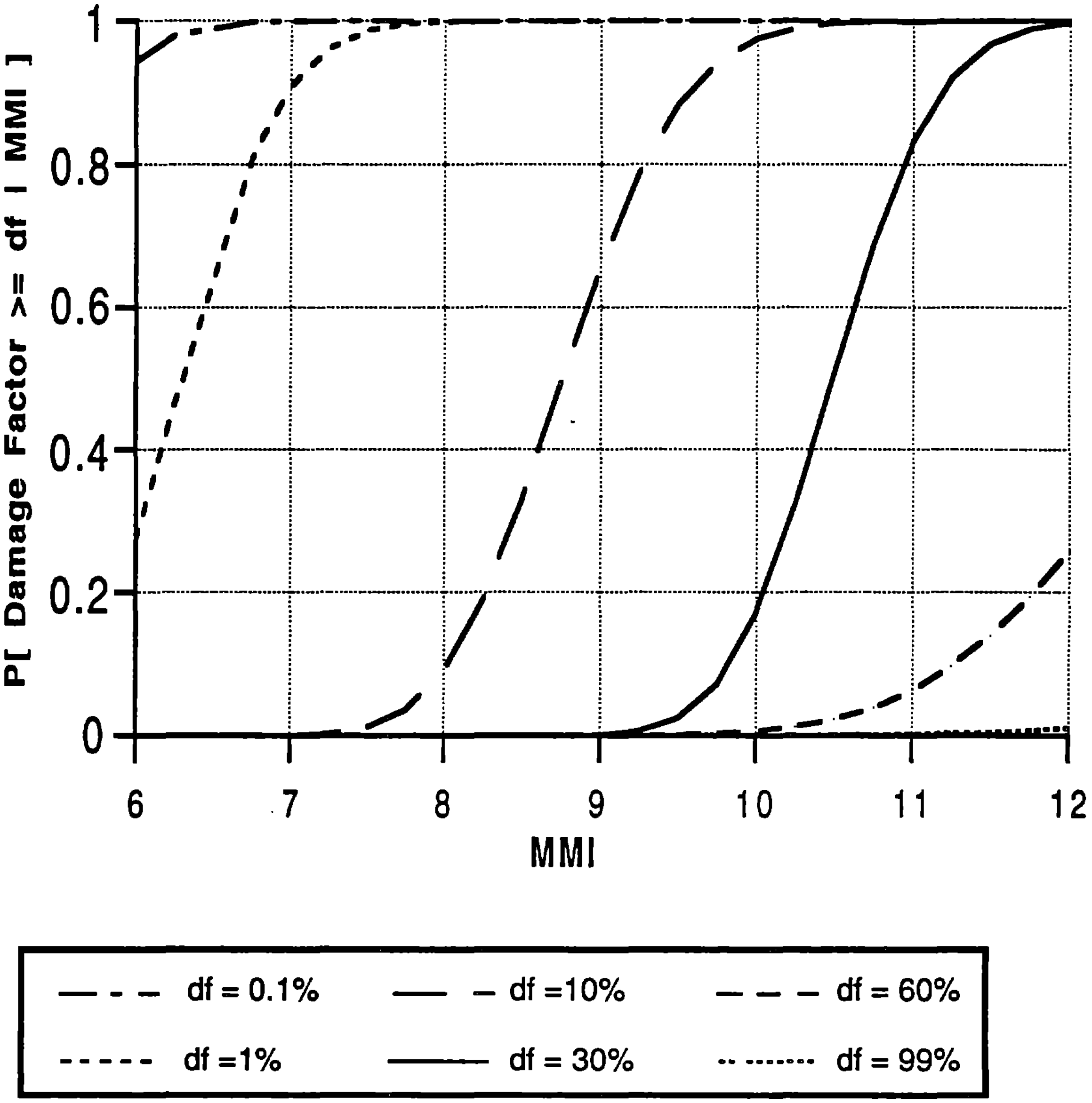
FIGURE 4-10 Fragility curves and parameters for Structure Class 7, Medium Rise RC Shear Wall (w/o MRF)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 8

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.63	0.0898
1	1.80	0.07
10	2.07	0.0799
30	2.25	0.0599
60	2.44	0.0699
99	2.83	0.15

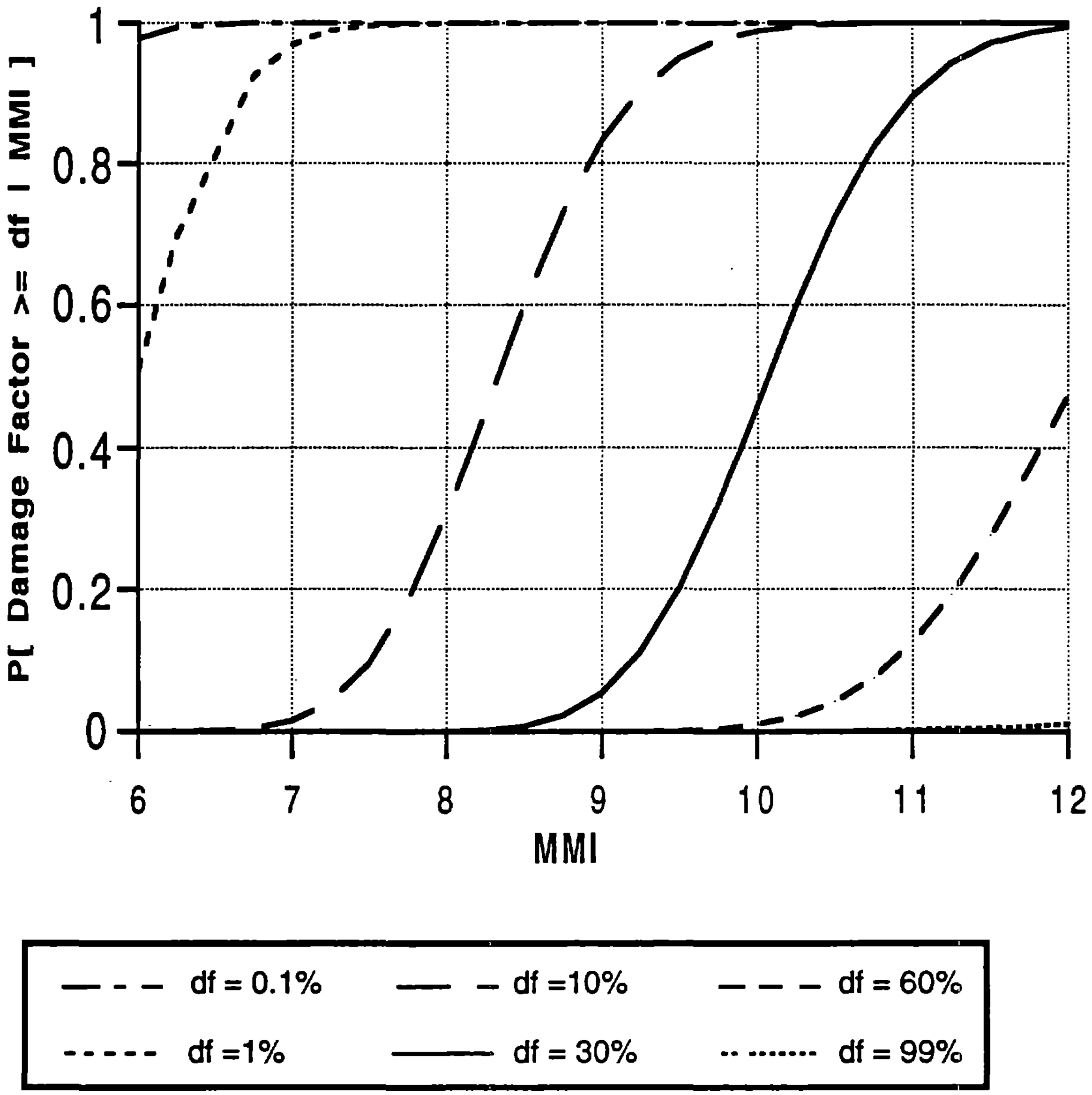
FIGURE 4-11 Fragility curves and parameters for Structure Class 8, High Rise RC Shear Wall (w/o MRF)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 9

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.65	0.0898
1	1.84	0.0799
10	2.17	0.068
30	2.35	0.05
60	2.55	0.0998
99	2.83	0.15

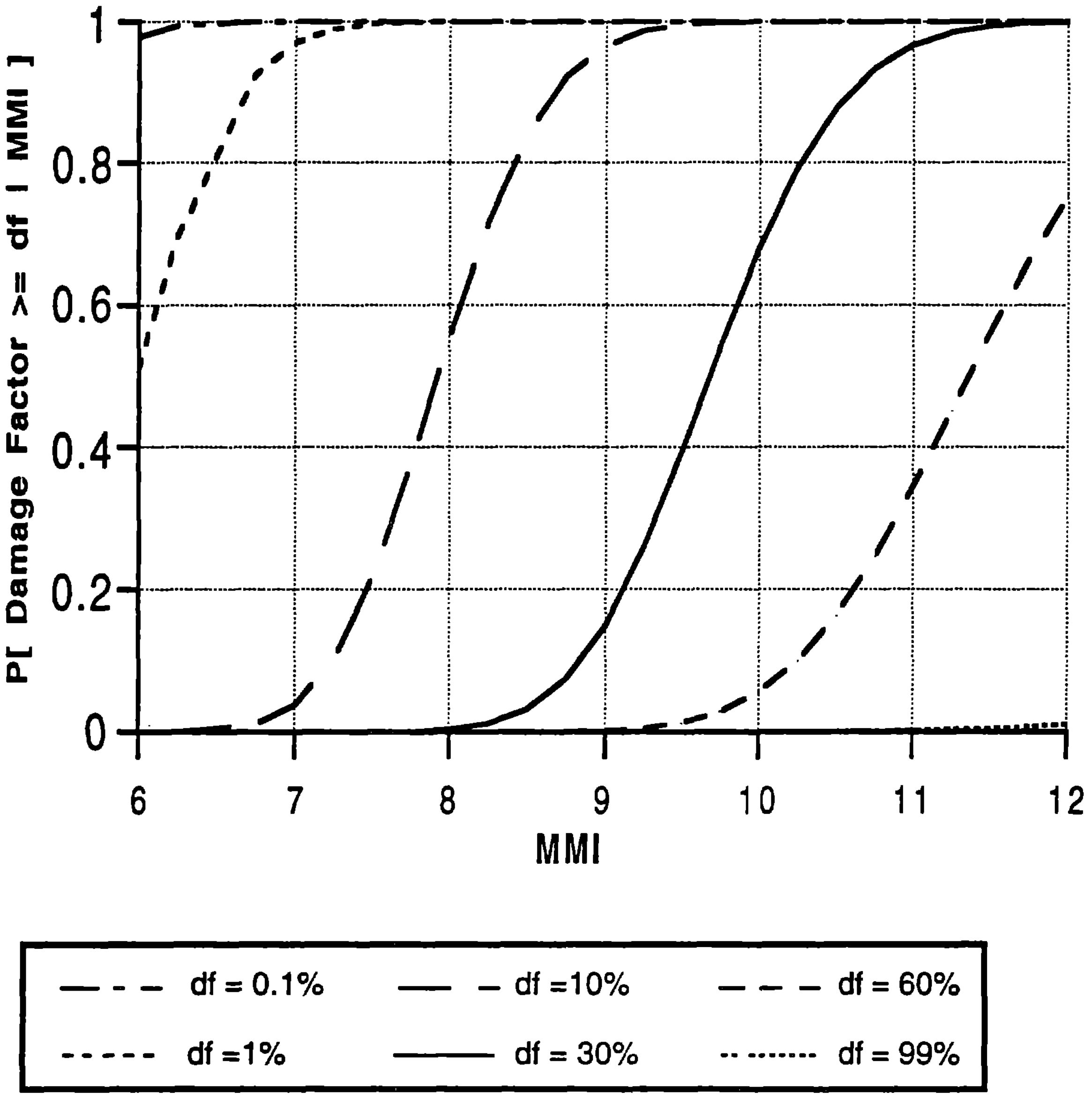
FIGURE 4-12 Fragility curves and parameters for Structure Class 9, Low Rise RM Shear Wall (w/o MRF)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 10

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.63	0.0799
1	1.79	0.083
10	2.12	0.0799
30	2.31	0.0699
60	2.49	0.0799
99	2.83	0.15

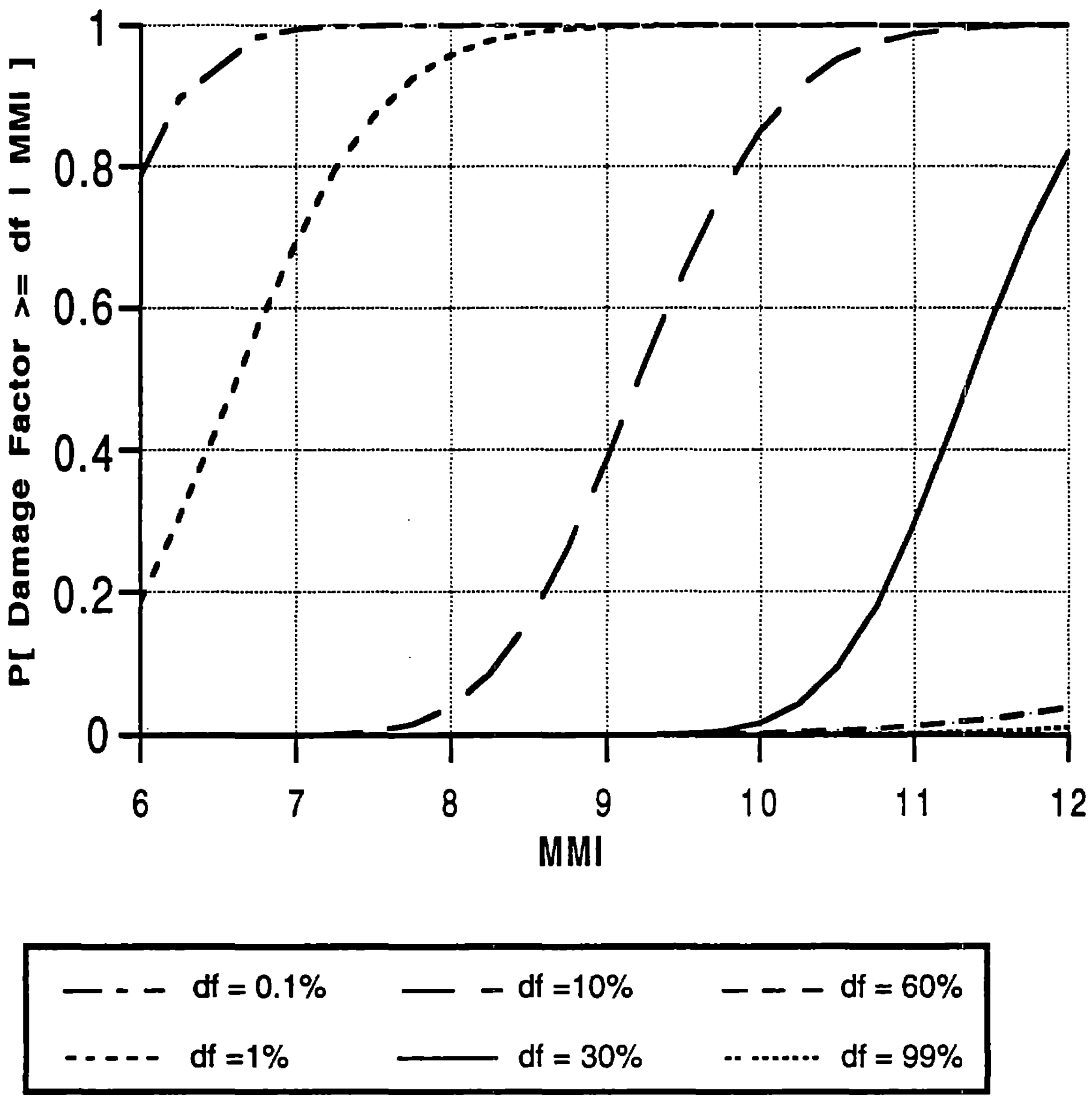
FIGURE 4-13 Fragility curves and parameters for Structure Class 10, Medium Rise RM Shear Wall (w/o MRF)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 11

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.63	0.0799
1	1.79	0.083
10	2.07	0.0699
30	2.27	0.0699
60	2.43	0.0799
99	2.83	0.15

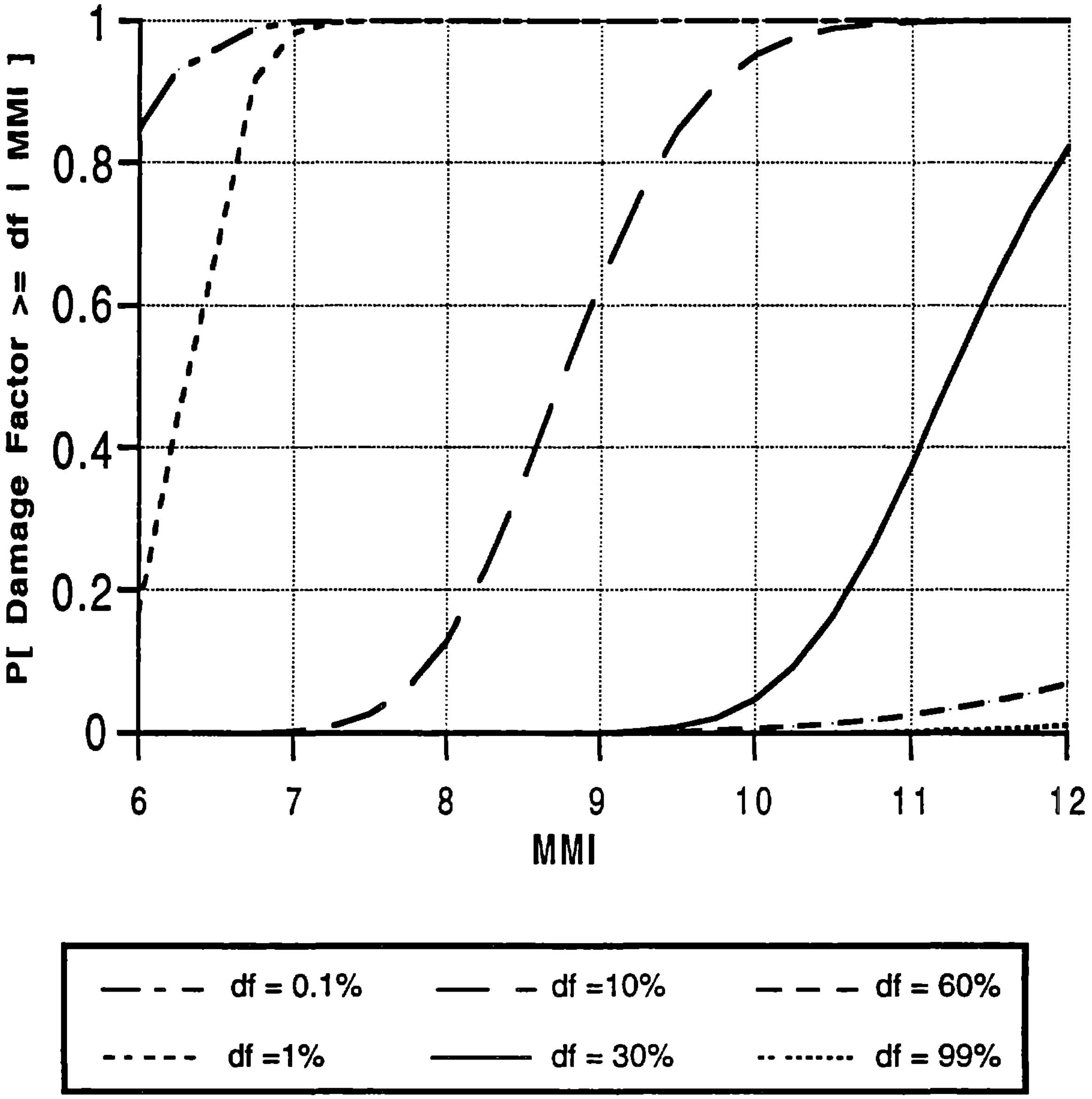
FIGURE 4-14 Fragility curves and parameters for Structure Class 11, High Rise RM Shear Wall (w/o MRF)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 12

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.72	0.0898
1	1.89	0.11
10	2.22	0.0799
30	2.43	0.0599
60	2.80	0.179
99	2.83	0.15

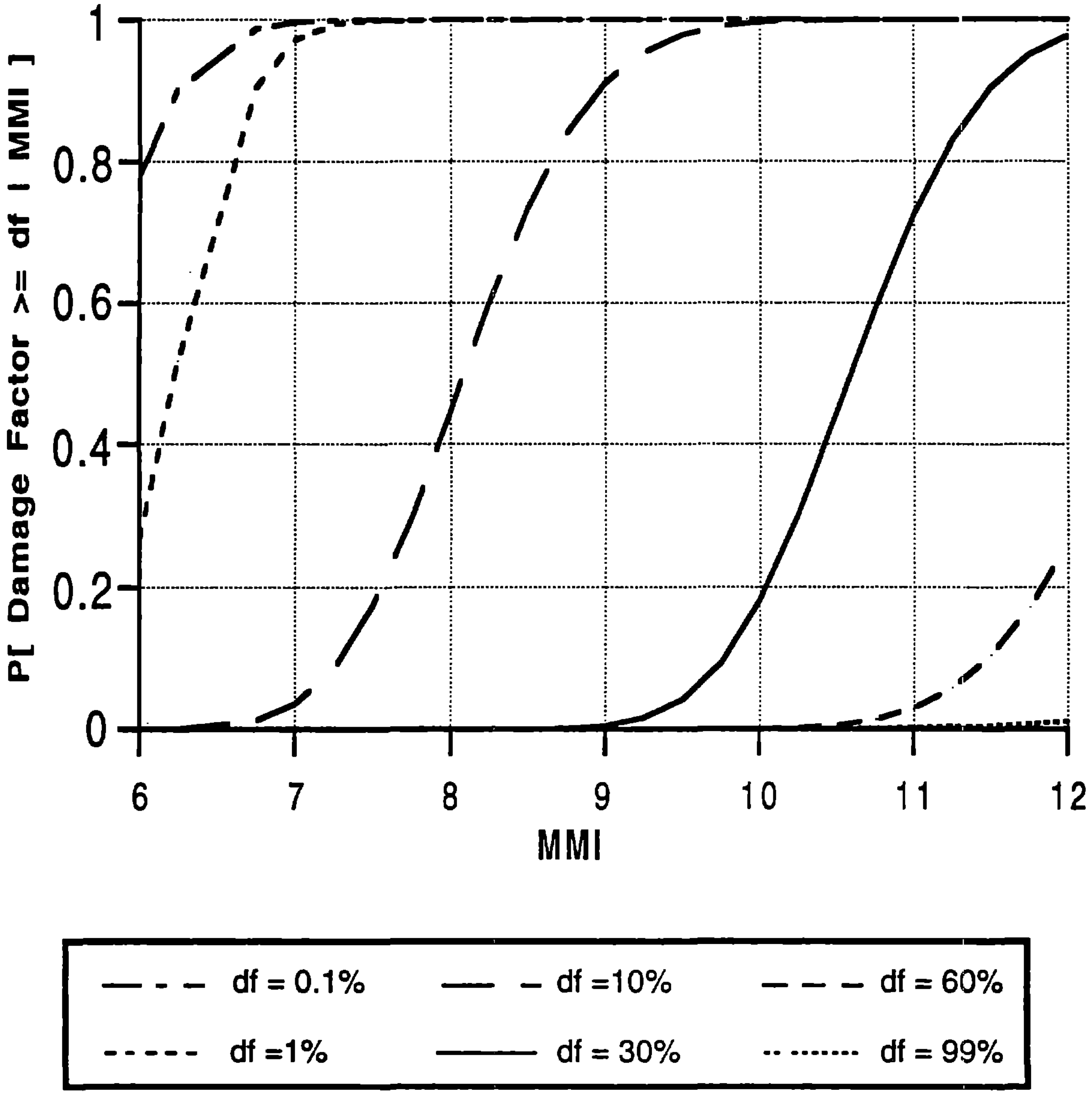
FIGURE 4-15 Fragility curves and parameters for Structure Class 12, Low Rise Braced Steel Frame



Parameters of Lognormal Distributions for Fragility Curves for Building Class 13

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.70	0.0898
1	1.84	0.05
10	2.17	0.0799
30	2.42	0.0699
60	2.75	0.179
99	2.83	0.15

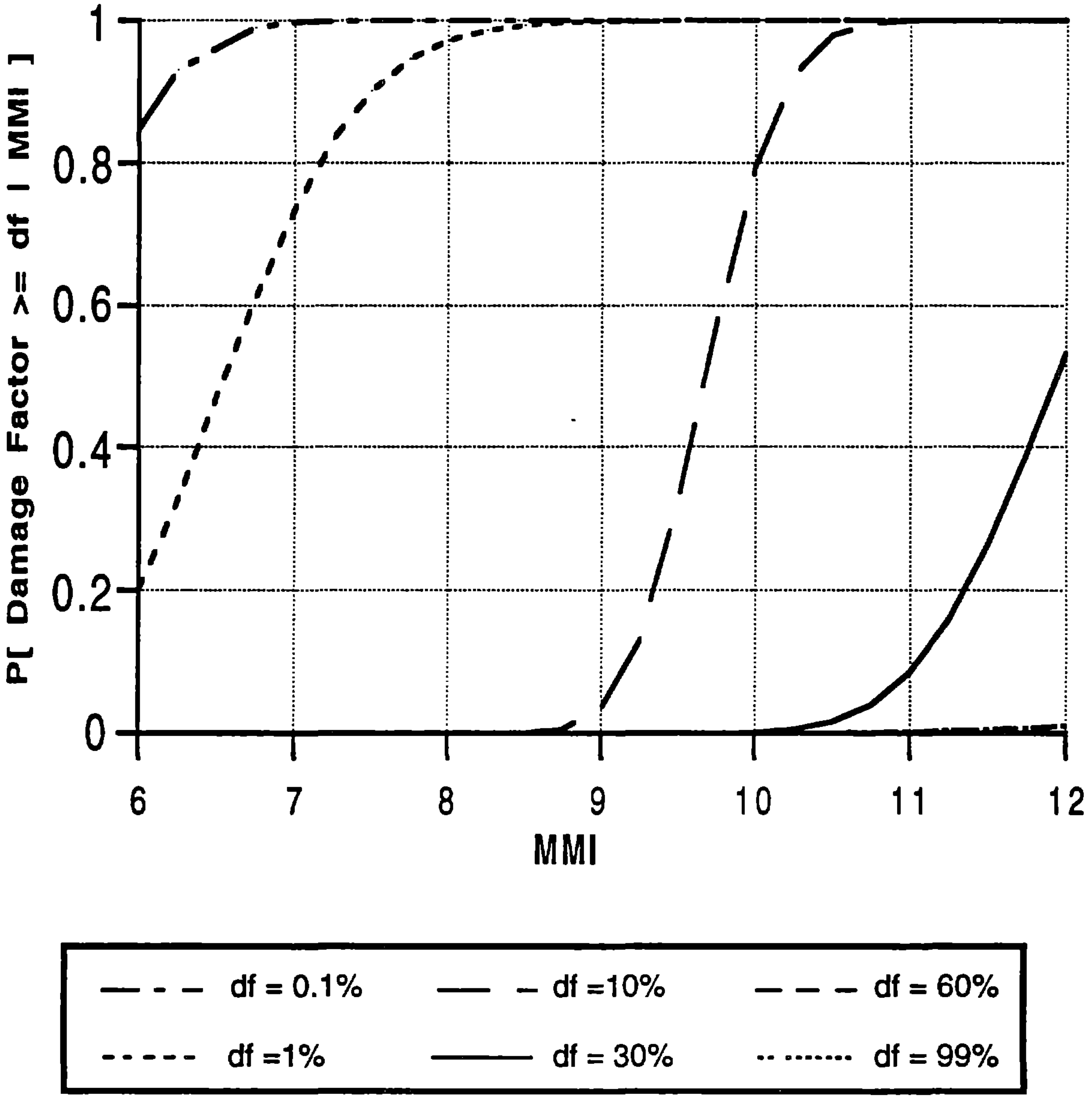
FIGURE 4-16 Fragility curves and parameters for Structure Class 13, Medium Rise Braced Steel Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 14

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.73	0.0799
1	1.83	0.061
10	2.09	0.0799
30	2.36	0.063
60	2.53	0.0699
99	2.83	0.15

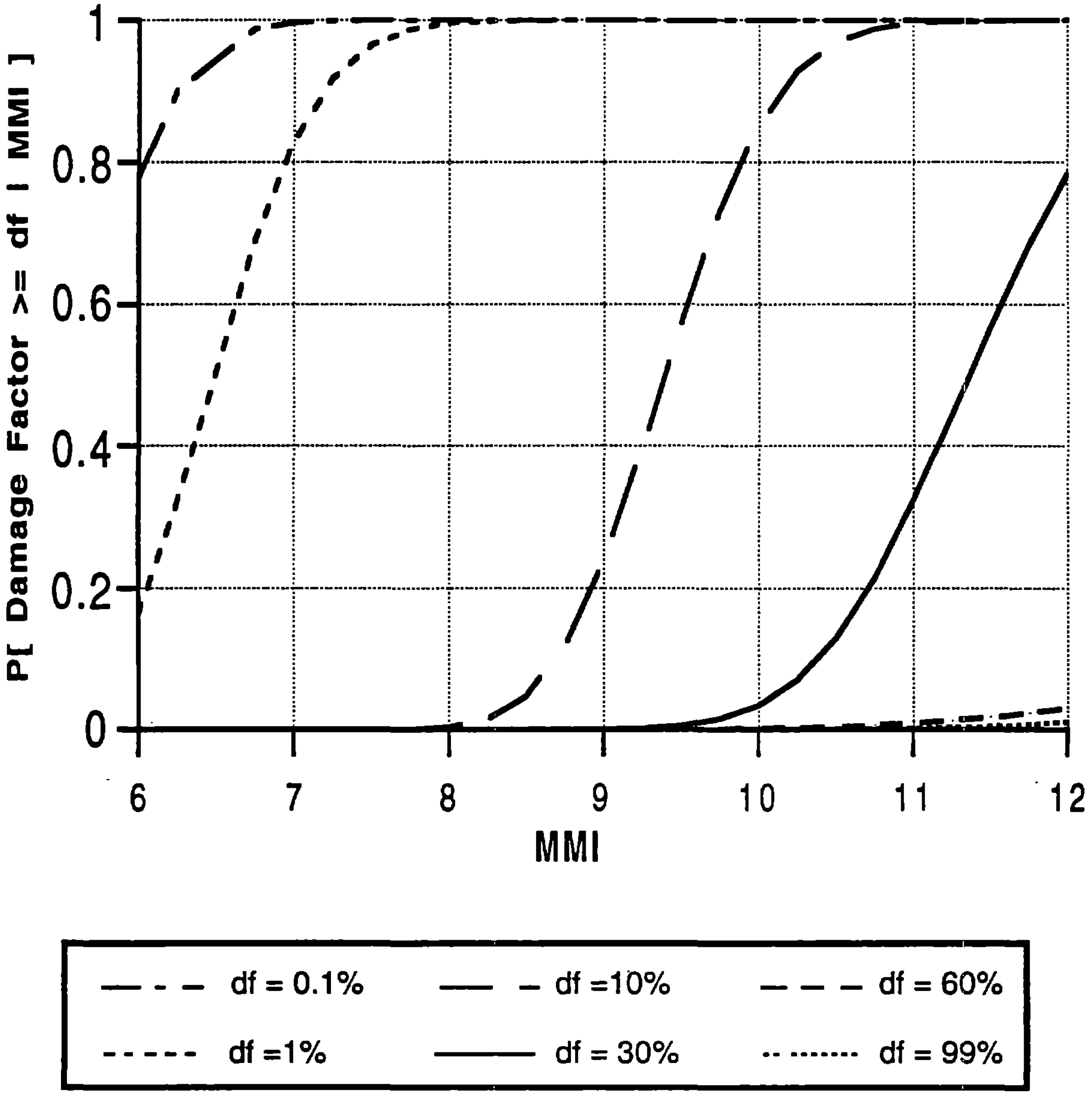
FIGURE 4-17 Fragility curves and parameters for Structure Class 14, High Rise Braced Steel Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 15

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.70	0.0898
1	1.88	0.105
10	2.27	0.04
30	2.48	0.0599
60	2.83	0.15
99	2.83	0.15

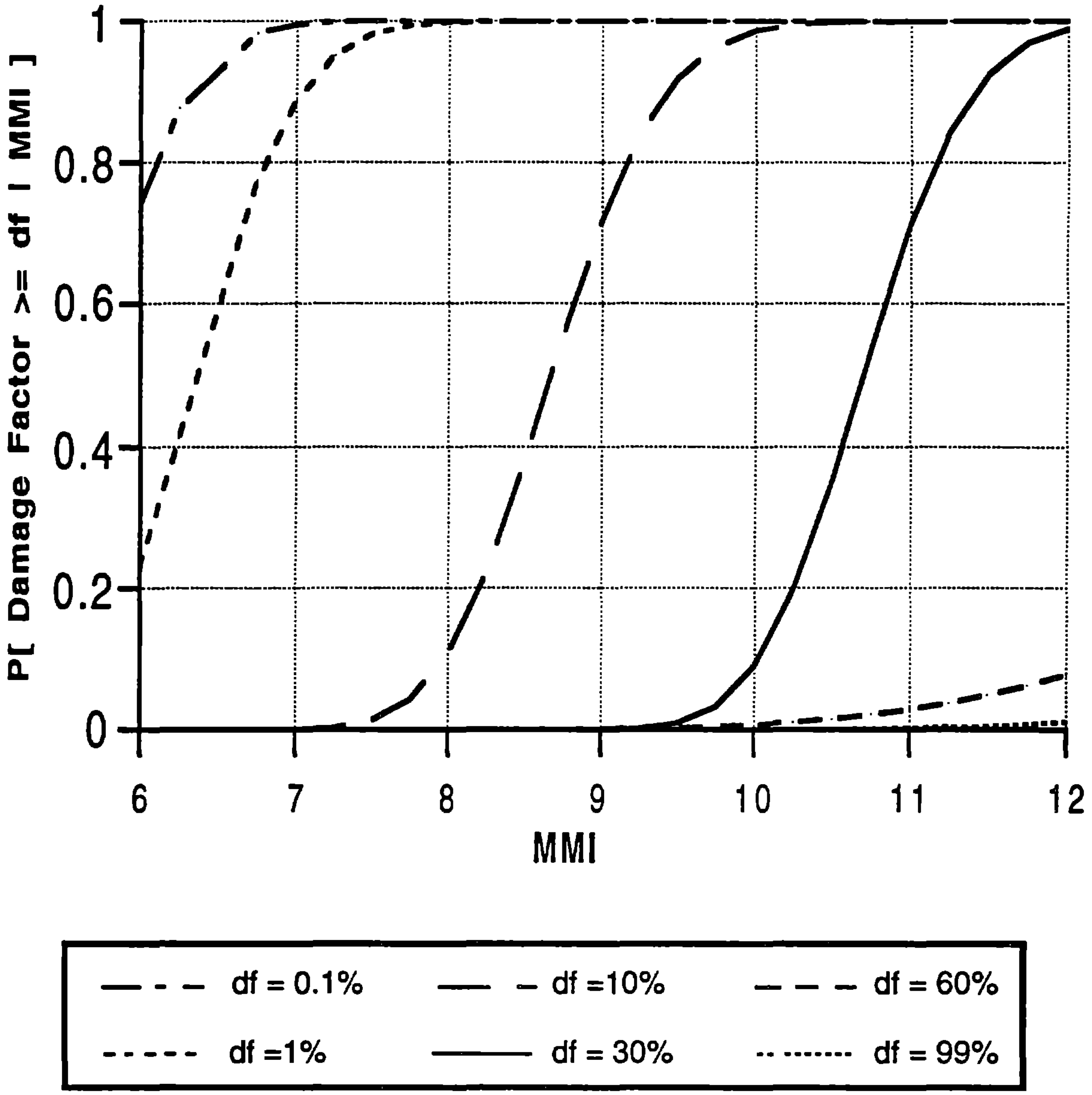
FIGURE 4-18 Fragility curves and parameters for Structure Class 15, Low Rise Moment-Resisting Steel Perimeter Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 16

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.73	0.0799
1	1.87	0.0799
10	2.24	0.0599
30	2.43	0.0699
60	2.82	0.179
99	2.83	0.15

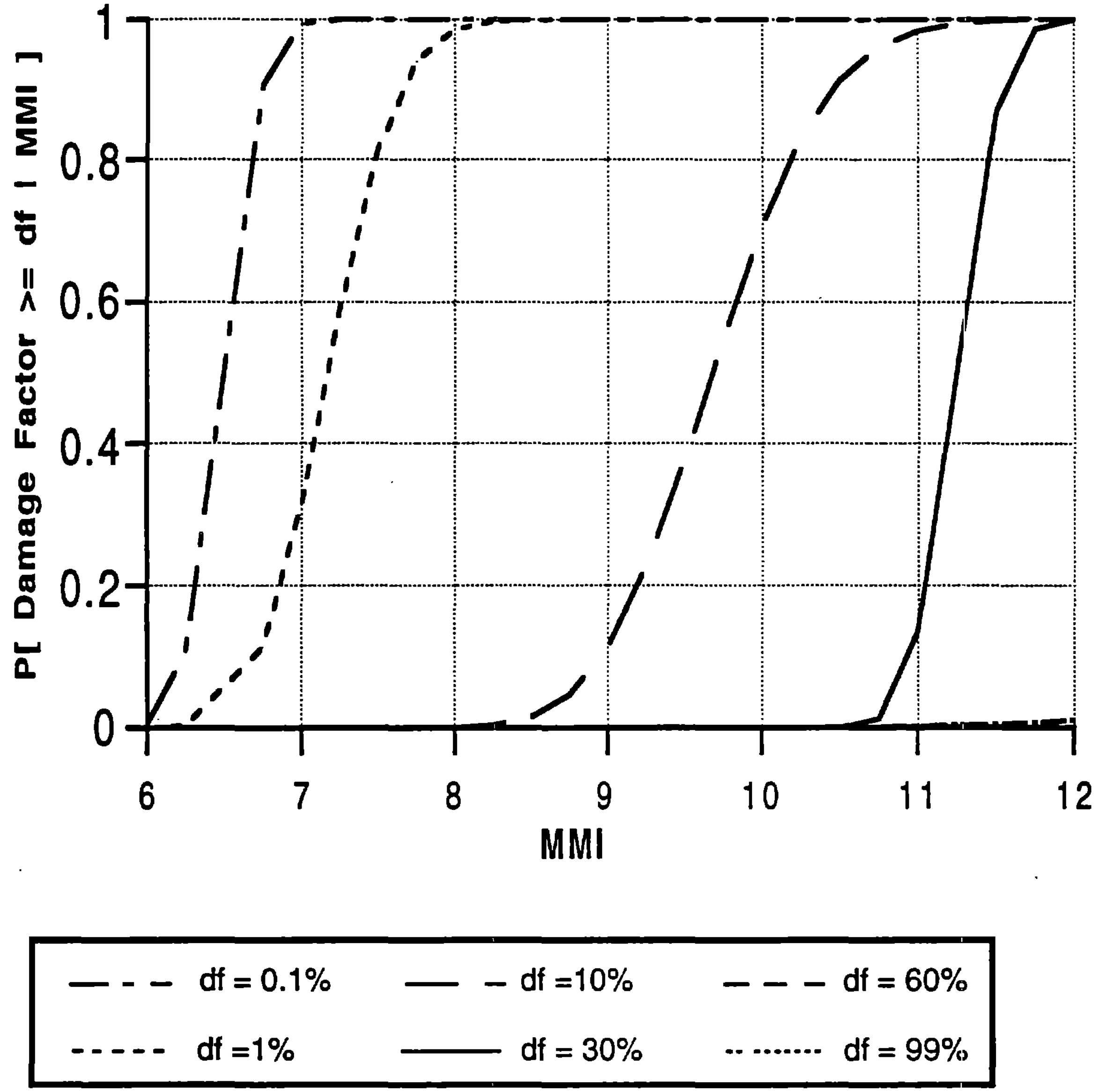
FIGURE 4-19 Fragility curves and parameters for Structure Class 11, Medium Rise Moment-Resisting Steel Perimeter Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 17

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.74	0.0799
1	1.85	0.079
10	2.16	0.065
30	2.37	0.05
60	2.74	0.179
99	2.83	0.15

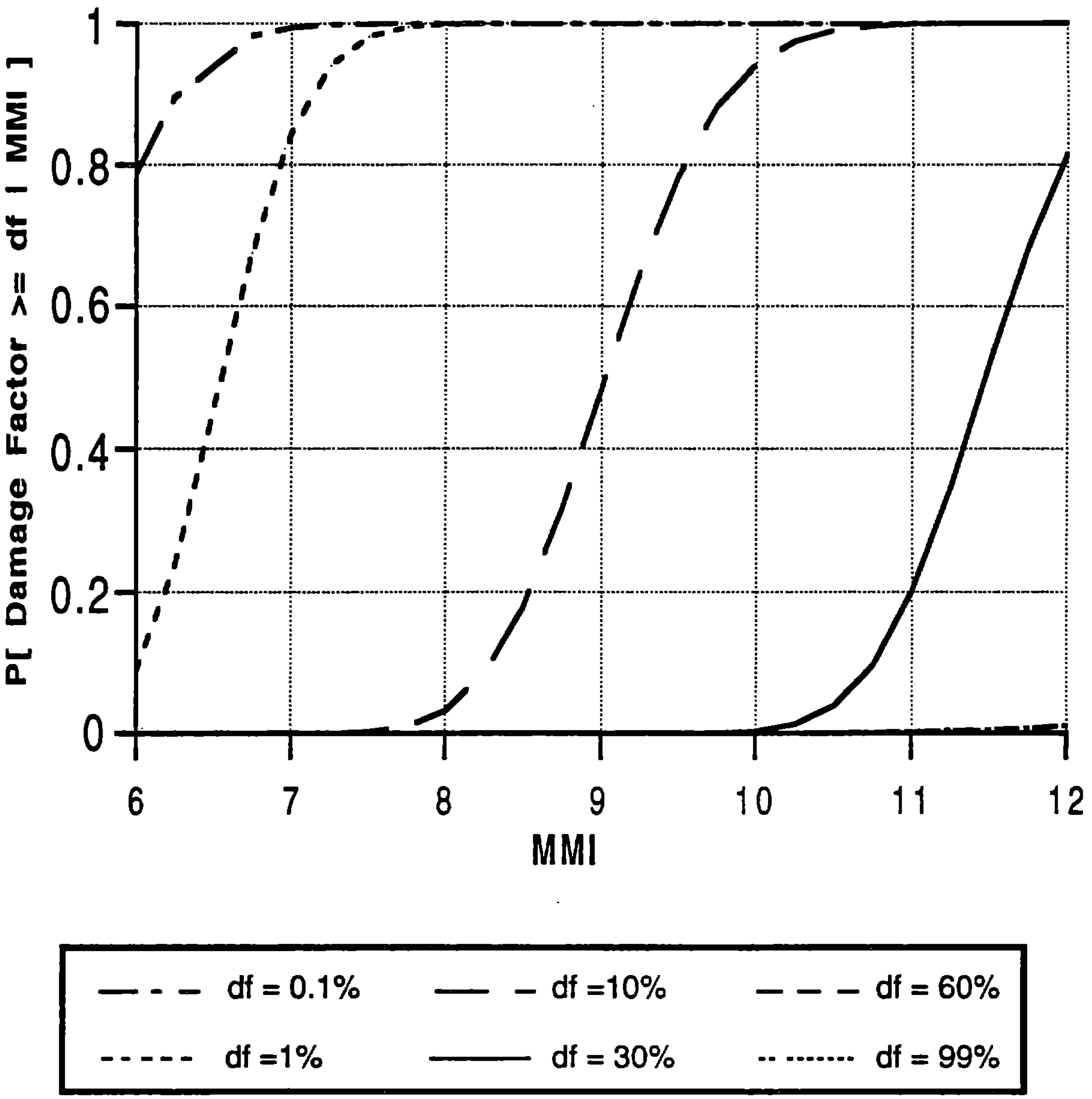
FIGURE 4-20 Fragility curves and parameters for Structure Class 17, High Rise Moment-Resisting Steel Perimeter Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 18

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.87	0.03
1	1.97	0.05
10	2.27	0.0599
30	2.42	0.02
60	2.83	0.15
99	2.83	0.15

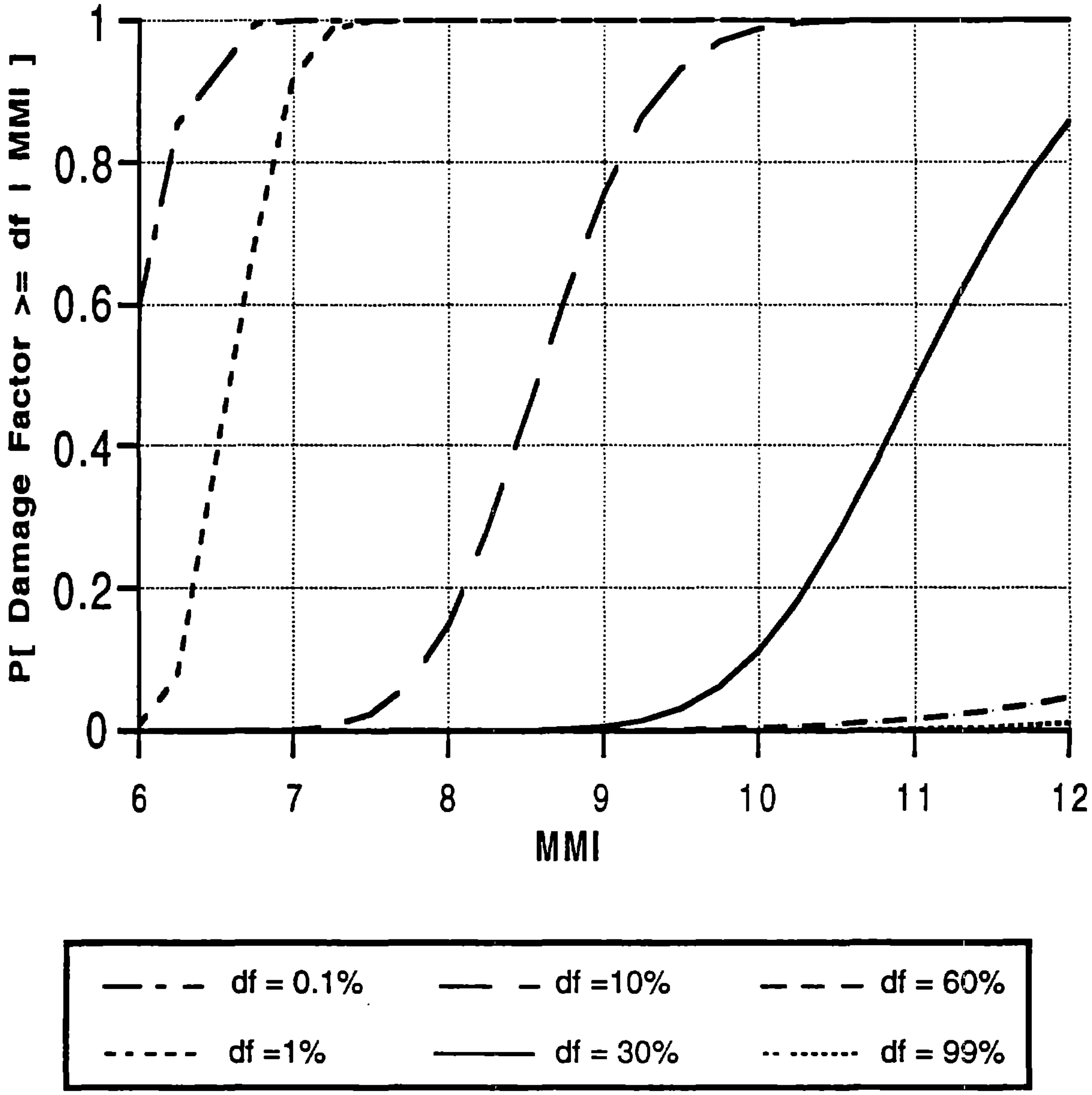
FIGURE 4-21 Fragility curves and parameters for Structure Class 18, Low Rise Moment-Resisting Ductile Concrete Dist. Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 19

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.72	0.0898
1	1.88	0.065
10	2.20	0.065
30	2.44	0.05
60	2.83	0.15
99	2.83	0.15

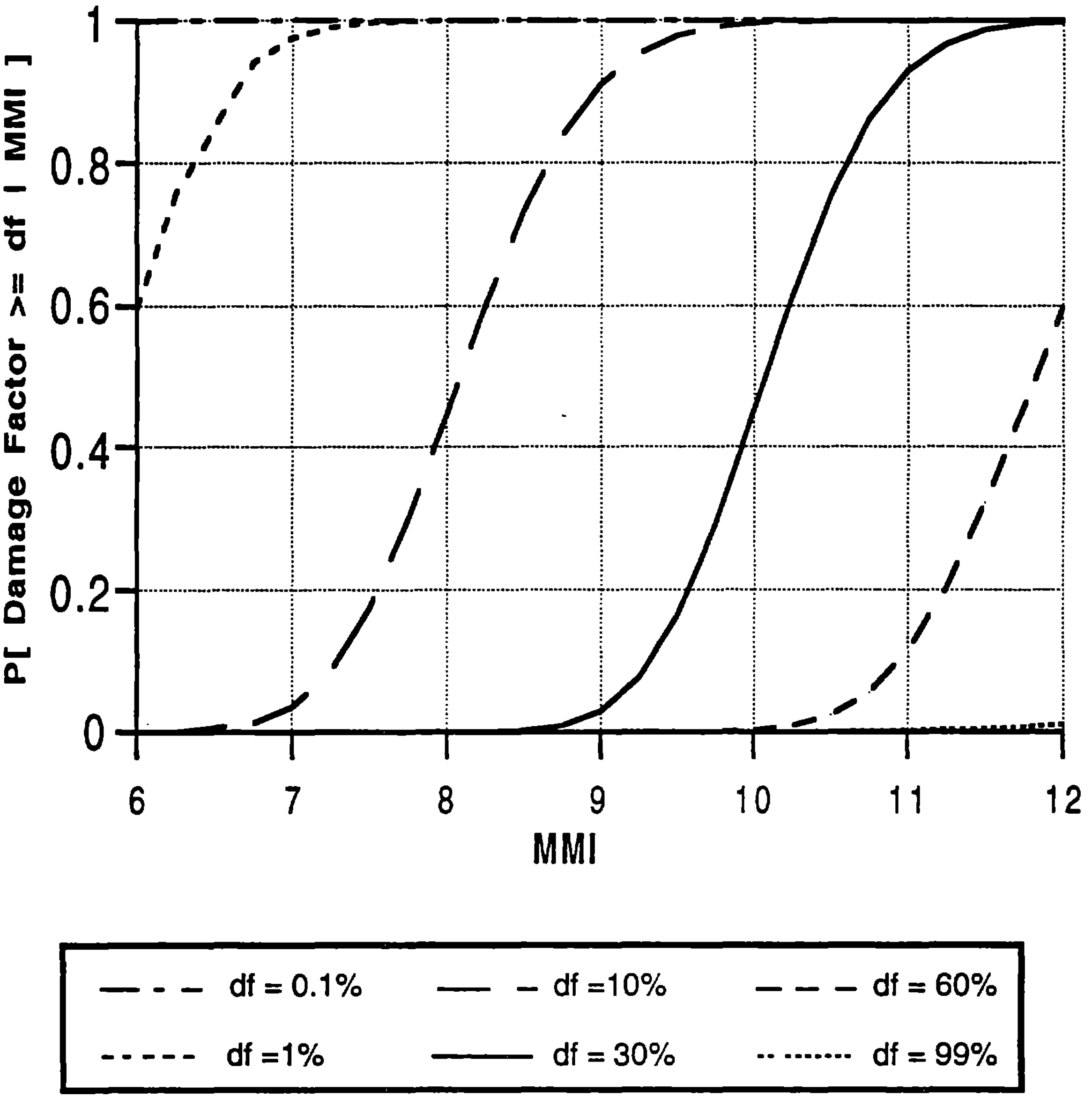
FIGURE 4-22 Fragility curves and parameters for Structure Class 19, Medium Moment-Resisting Ductile Concrete Dist. Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 20

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.78	0.05
1	1.89	0.04
10	2.15	0.068
30	2.40	0.0799
60	2.80	0.188
99	2.83	0.15

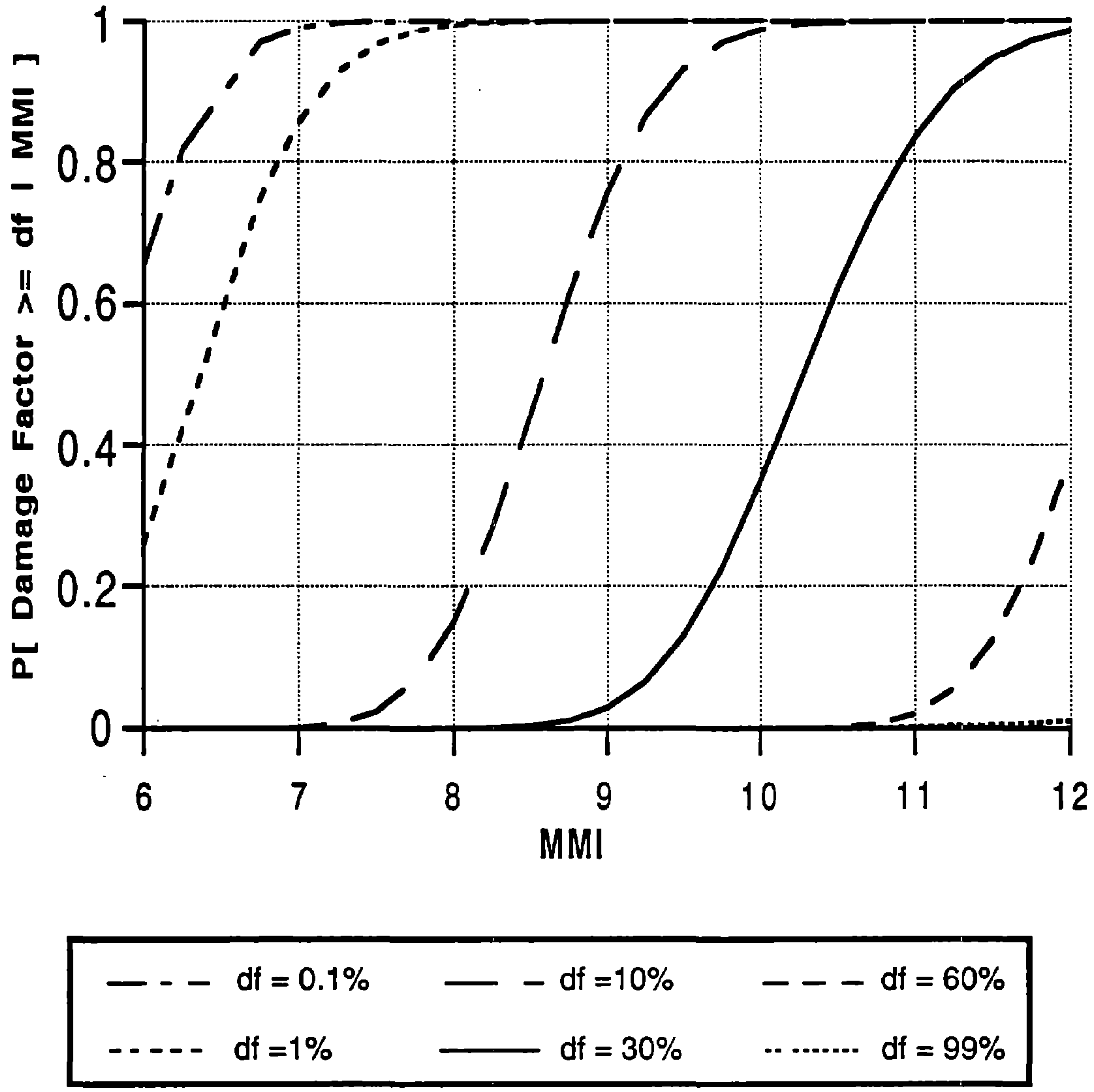
FIGURE 4-23 Fragility curves and parameters for Structure Class 20, High Rise Moment-Resisting Ductile Concrete Dist. Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 21

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.53	0.0898
1	1.77	0.0898
10	2.09	0.0799
30	2.31	0.0599
60	2.47	0.0599
99	2.83	0.15

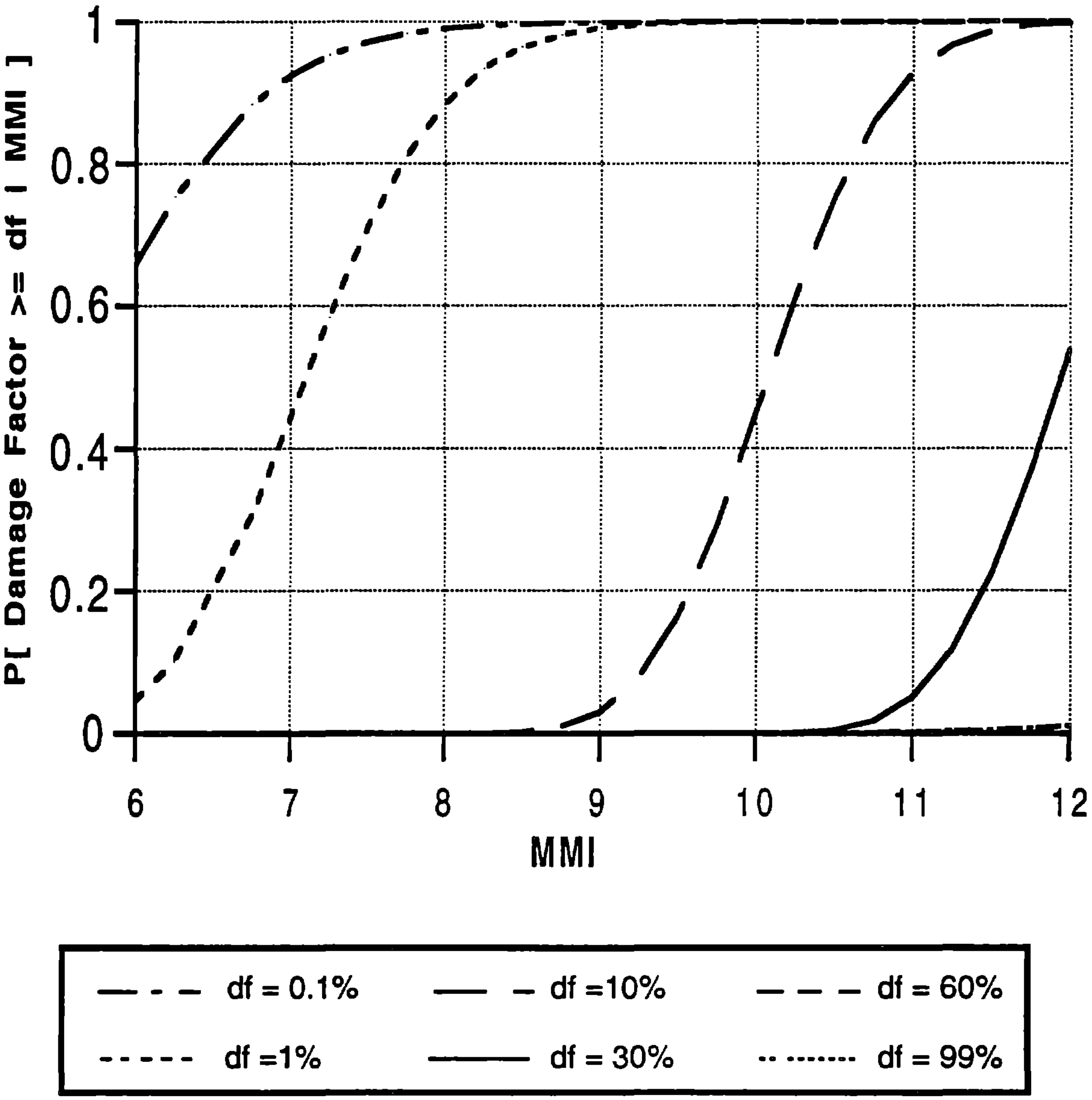
FIGURE 4-24 Fragility curves and parameters for Structure Class 21, Low Rise Tilt-up



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 23

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.76	0.0799
1	1.85	0.09
10	2.15	0.068
30	2.33	0.0699
60	2.50	0.05
99	2.83	0.15

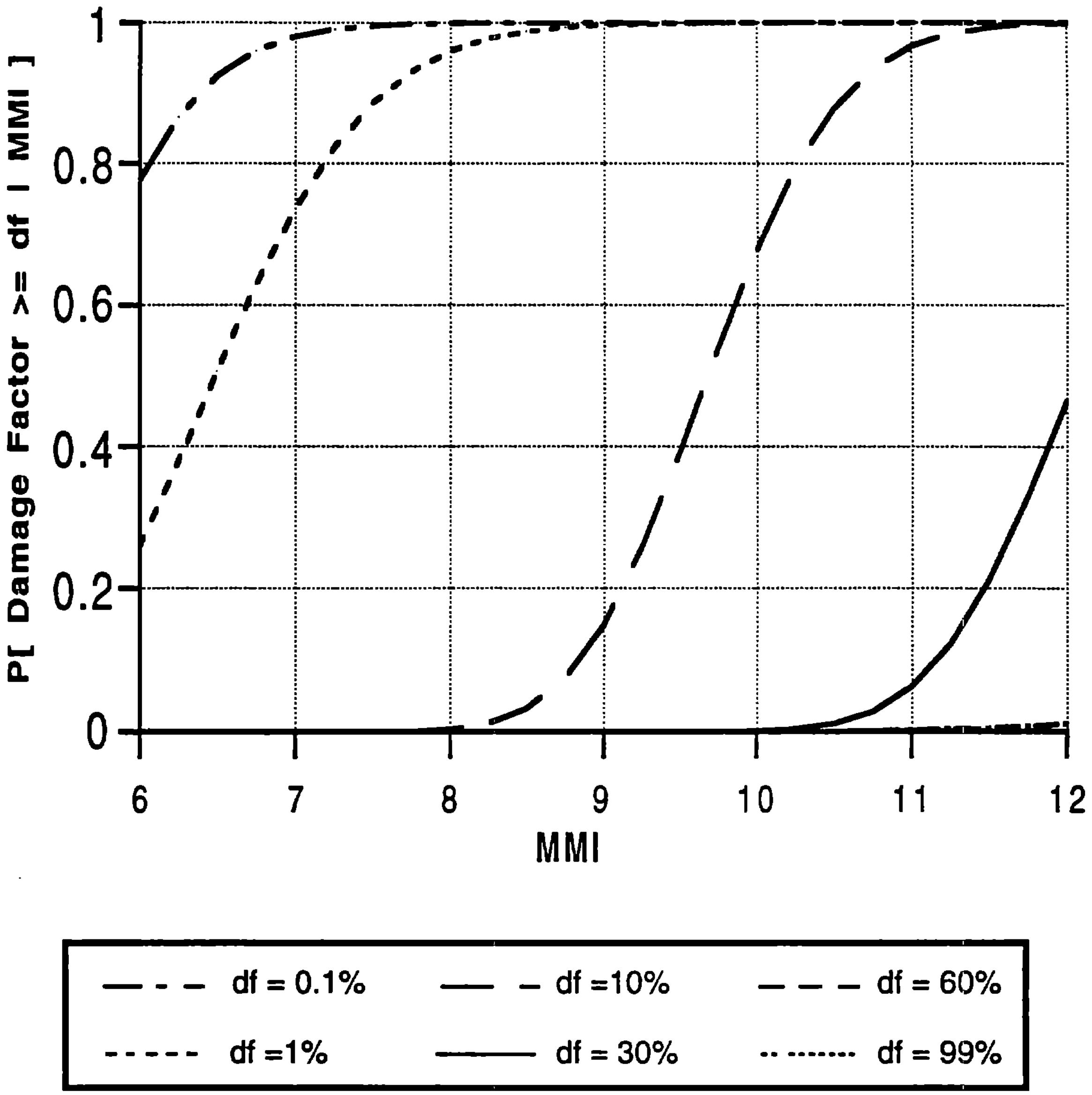
FIGURE 4-25 Fragility curves and parameters for Structure Class 23, Mobile Homes



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 72

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.73	0.15
1	1.96	0.0998
10	2.31	0.0599
30	2.48	0.05
60	2.83	0.15
99	2.83	0.15

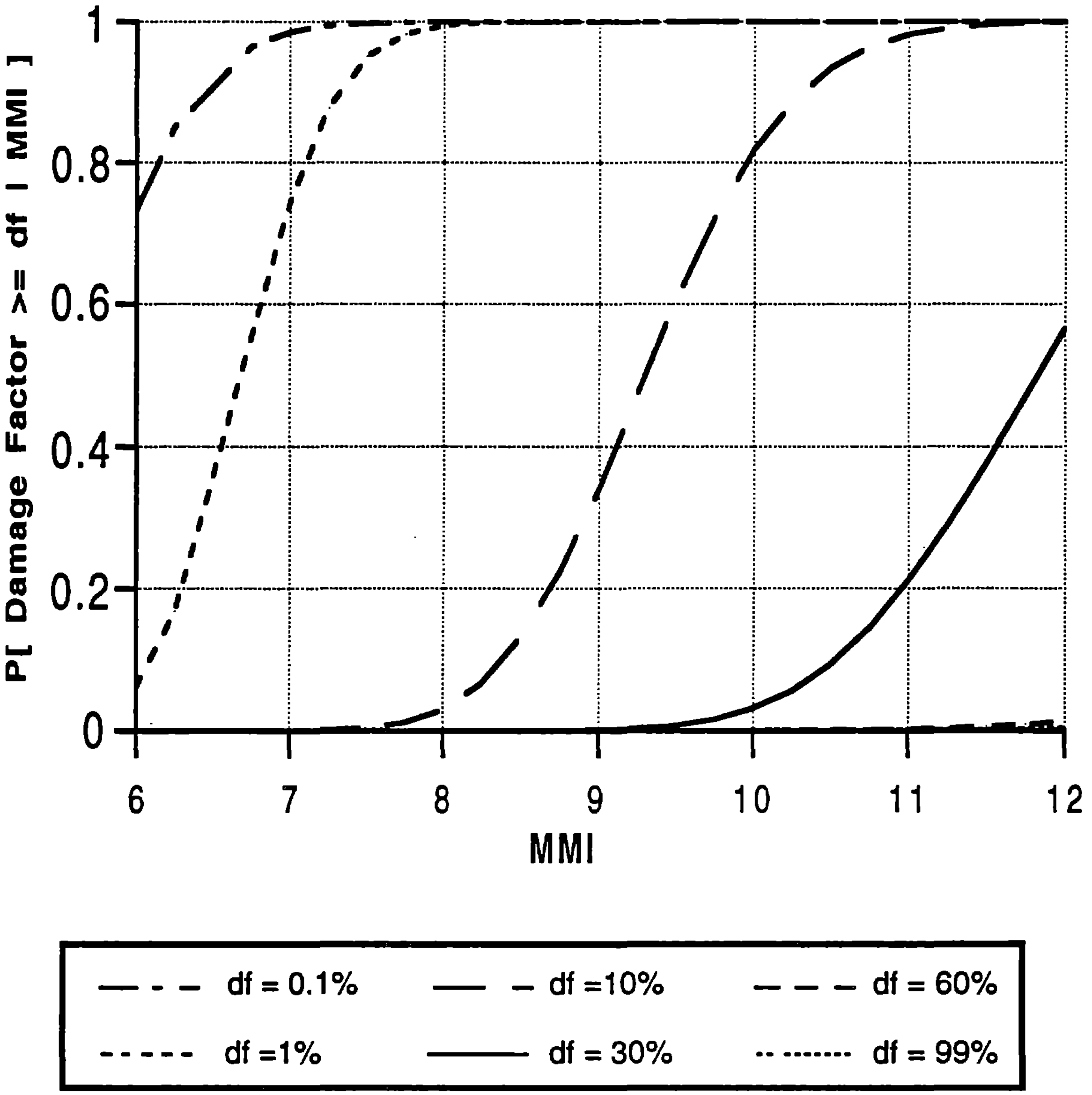
FIGURE 4-26 Fragility curves and parameters for Structure Class 72, Low Rise Moment-Resisting Distributed Steel Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 73

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.70	0.12
1	1.87	0.12
10	2.27	0.0699
30	2.49	0.0599
60	2.83	0.15
99	2.83	0.15

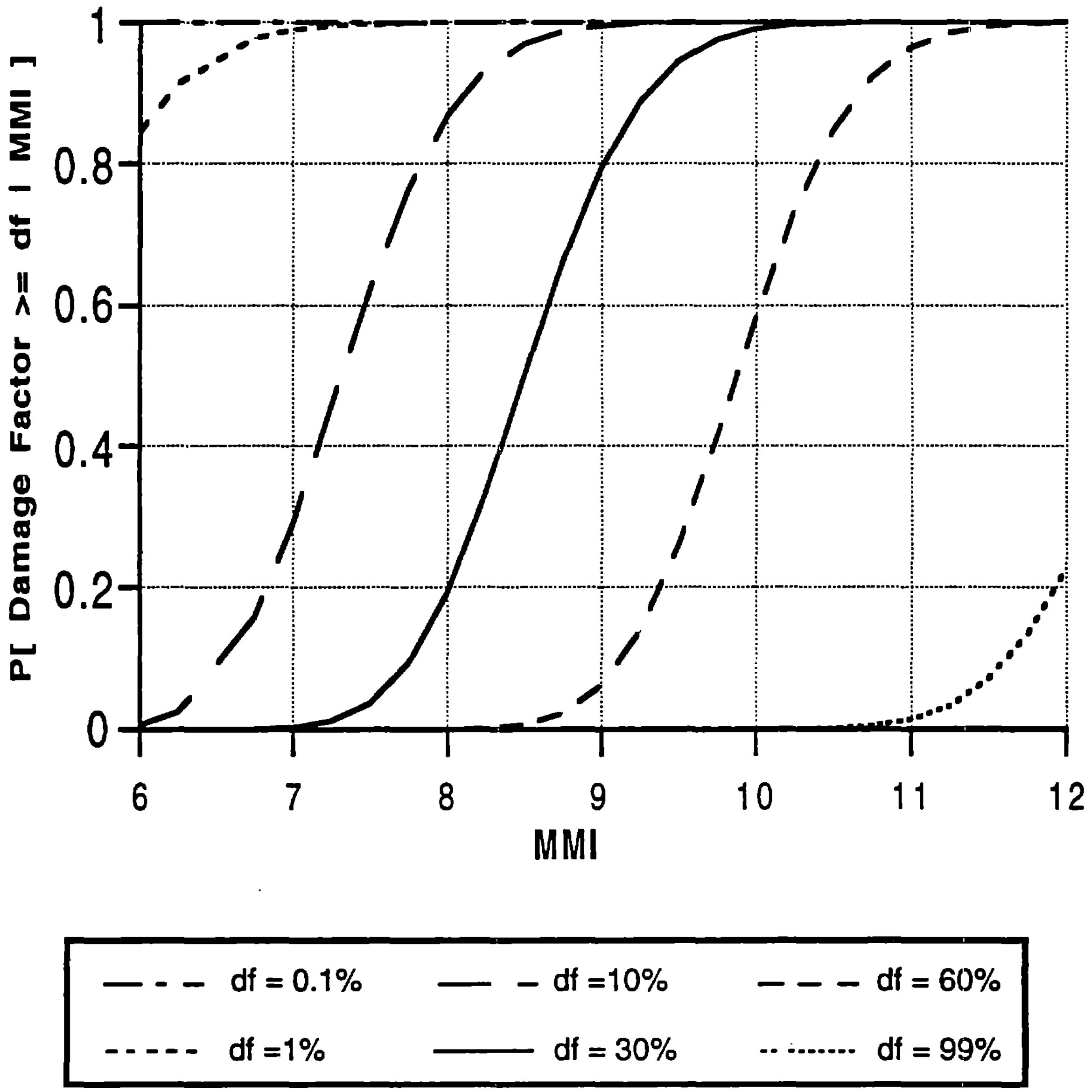
FIGURE 4-27 Fragility curves and parameters for Structure Class 73, Medium Rise Moment-Resisting Distributed Steel Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 74

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.73	0.099
1	1.90	0.0699
10	2.23	0.0799
30	2.47	0.0898
60	2.82	0.15
99	2.83	0.13

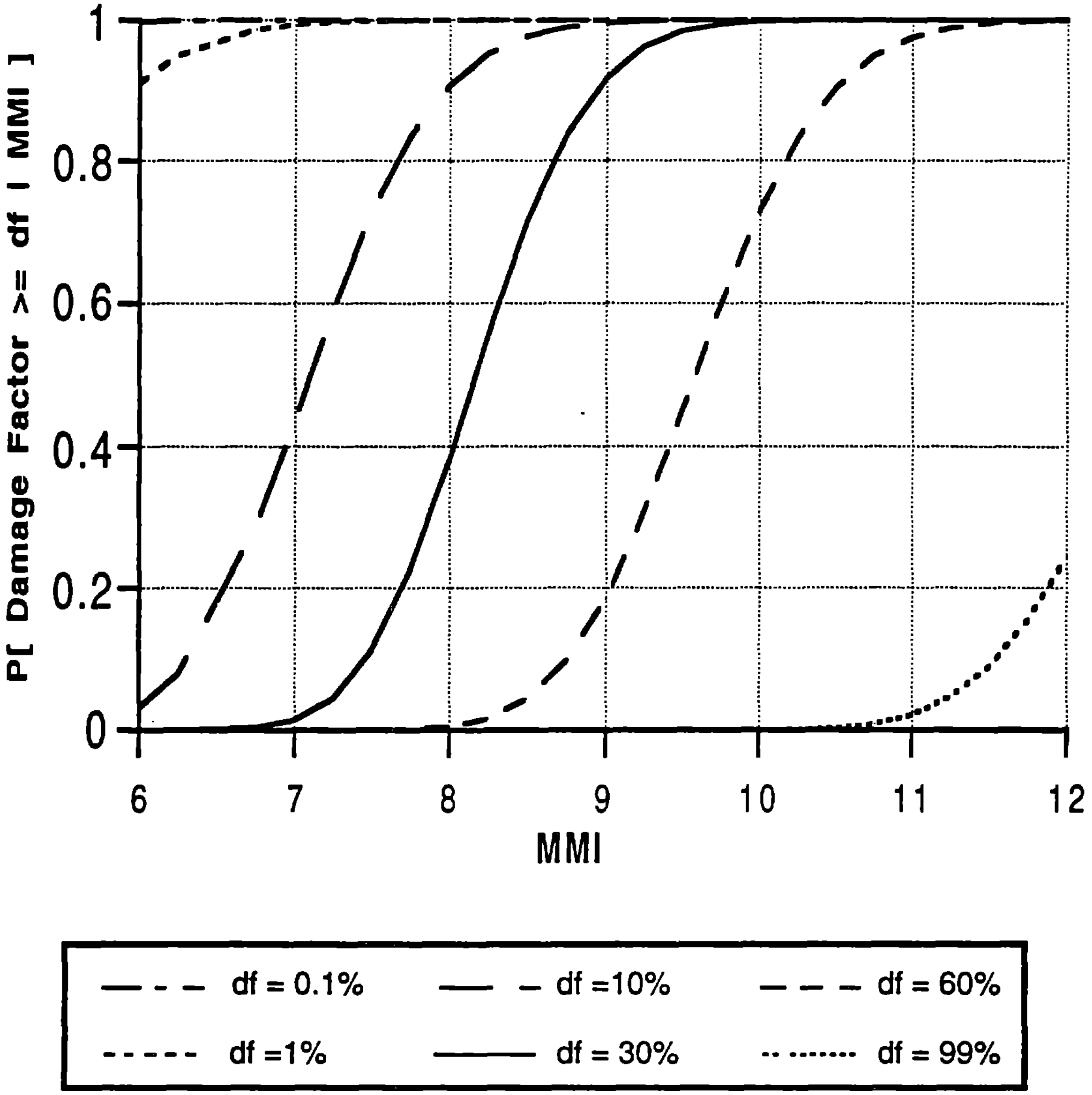
FIGURE 4-28 Fragility curves and parameters for Structure Class 74, High Rise Moment-Resisting Distributed Steel Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 75

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.41	0.0998
1	1.67	0.12
10	1.99	0.0799
30	2.14	0.0699
60	2.29	0.06
99	2.53	0.0599

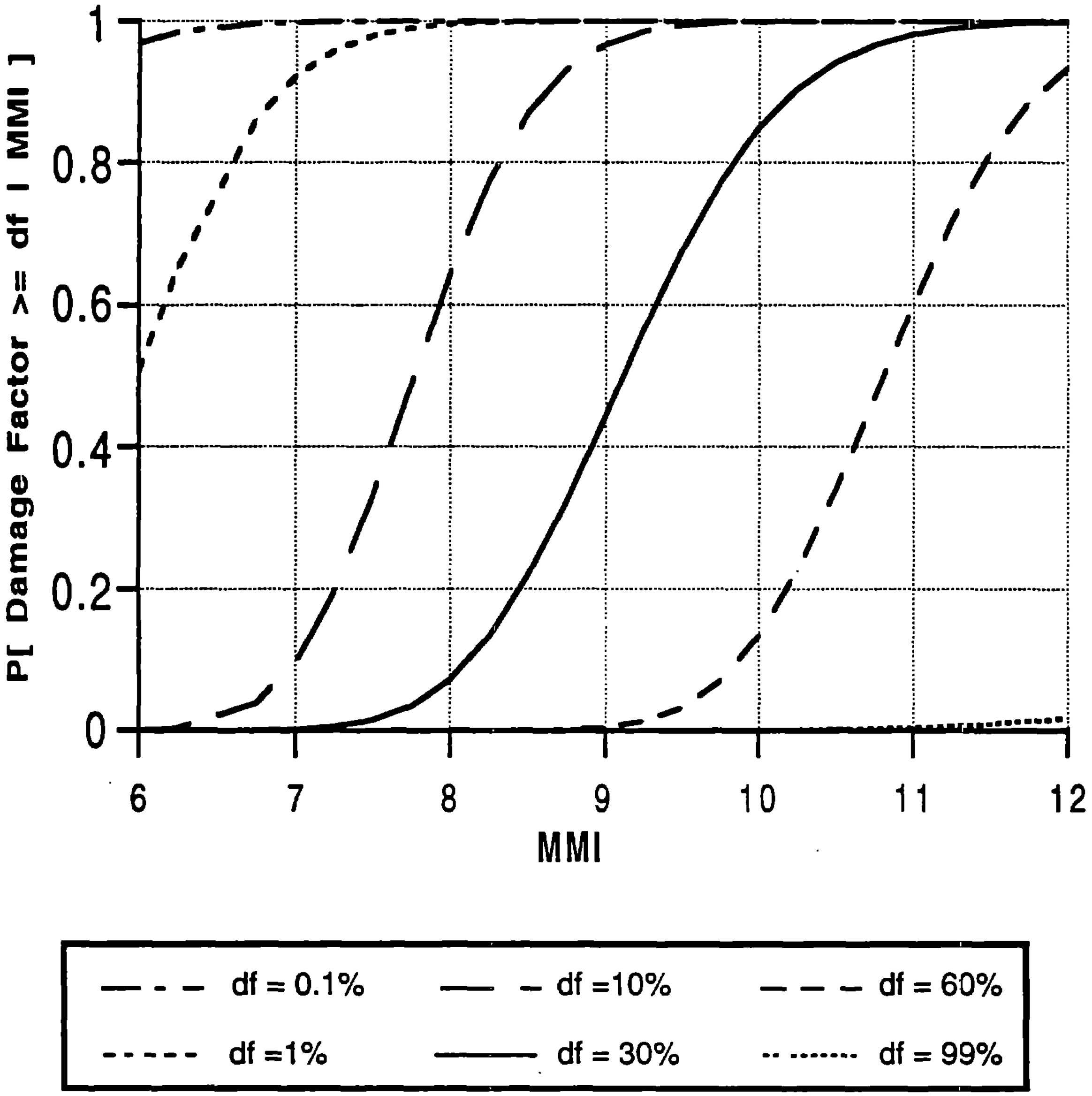
FIGURE 4-29 Fragility curves and parameters for Structure Class 75, Low Rise URM Bearing Wall



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 76

Damage Factor(%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.51	0.099
1	1.61	0.135
10	1.96	0.0898
30	2.10	0.0699
60	2.26	0.0699
99	2.53	0.065

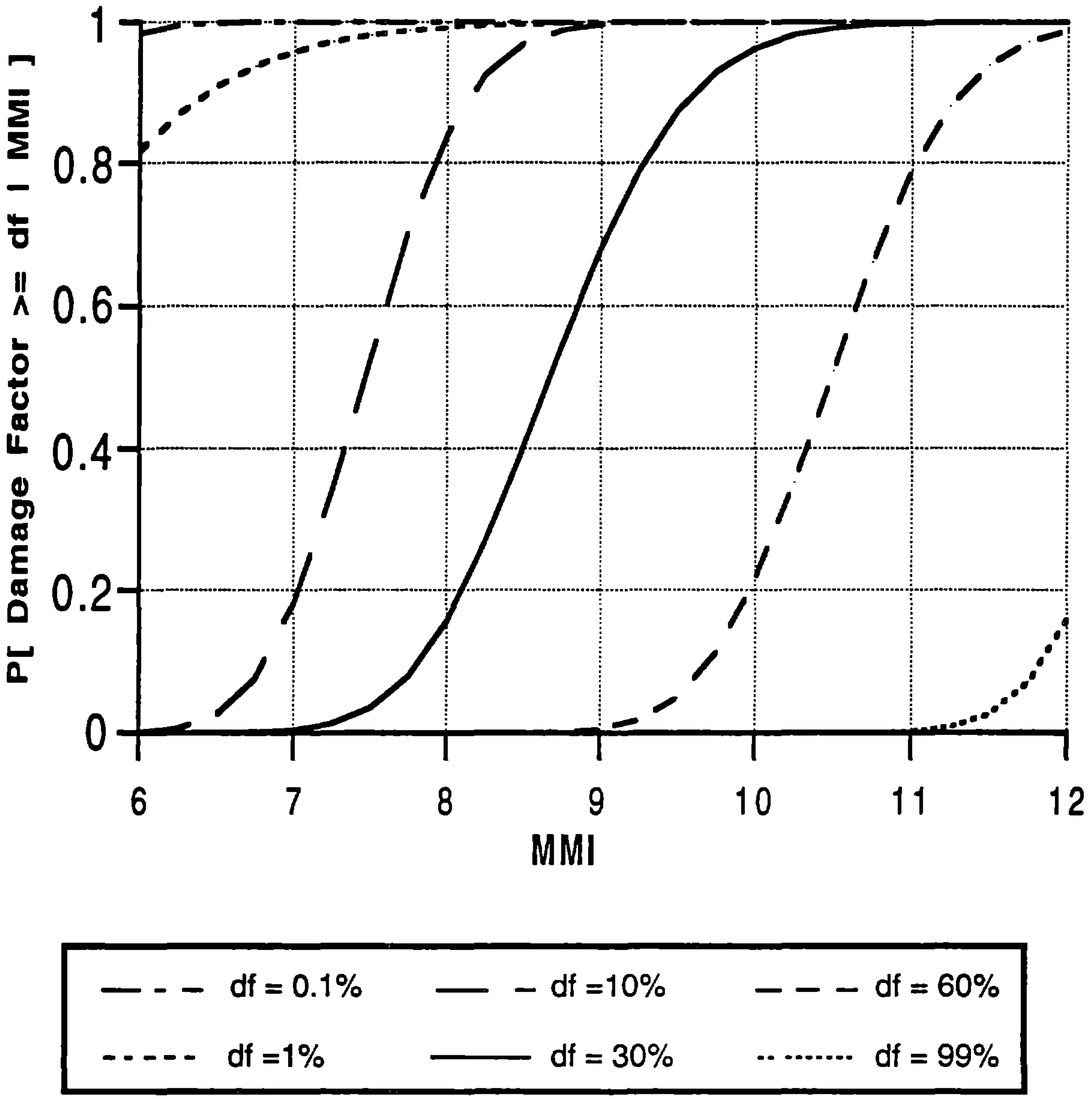
FIGURE 4-30 Fragility curves and parameters for Structure Class 76, Medium Rise URM Bearing Wall



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 78

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.53	0.14
1	1.79	0.11
10	2.05	0.0799
30	2.21	0.0898
60	2.38	0.0699
99	2.82	0.16

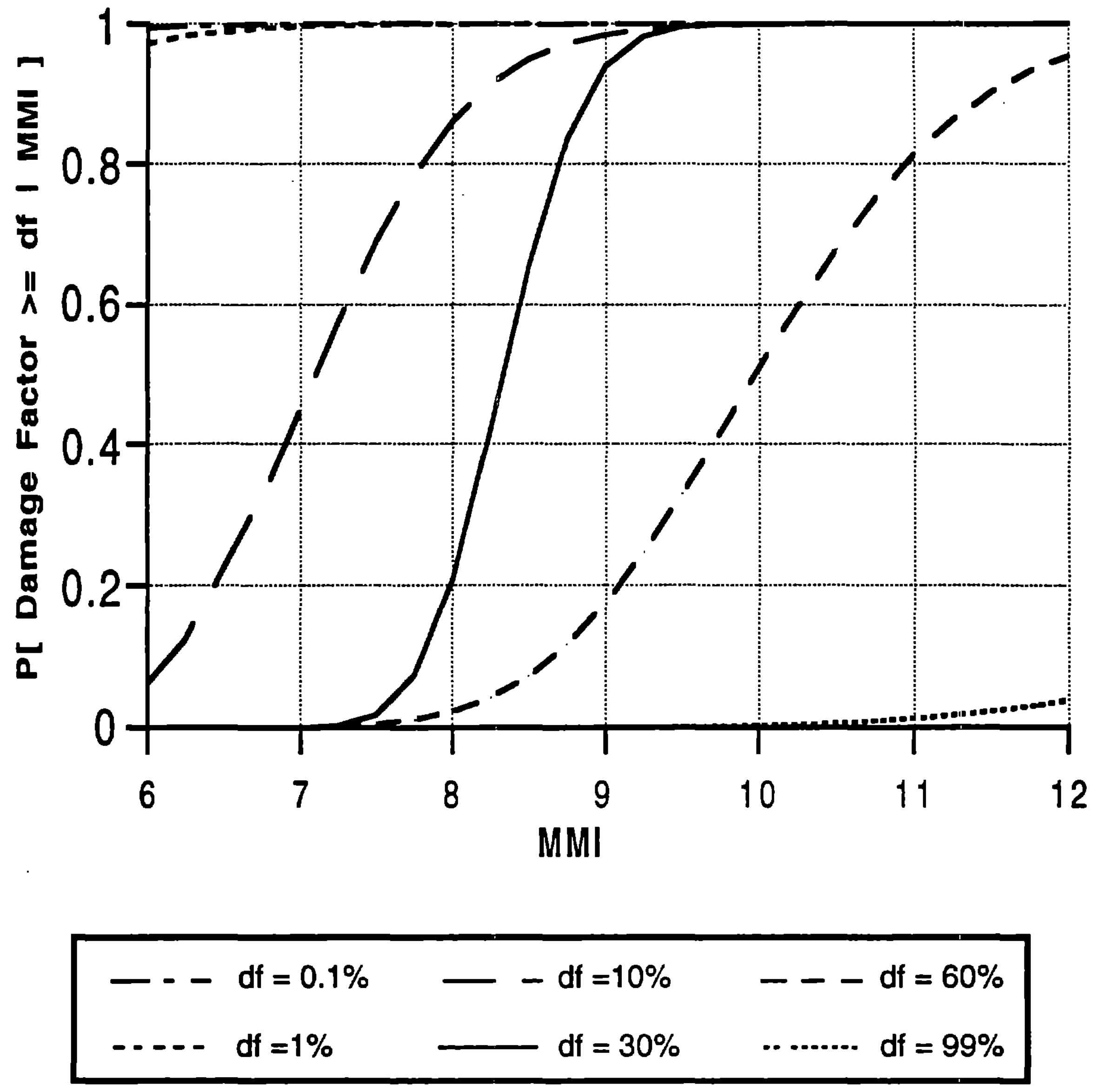
FIGURE 4-31 Fragility curves and parameters for Structure Class 78, Low Rise URM with Load Bearing Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 79

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.60	0.09
1	1.62	0.19
10	2.01	0.0699
30	2.16	0.0799
60	2.35	0.0599
99	2.53	0.045

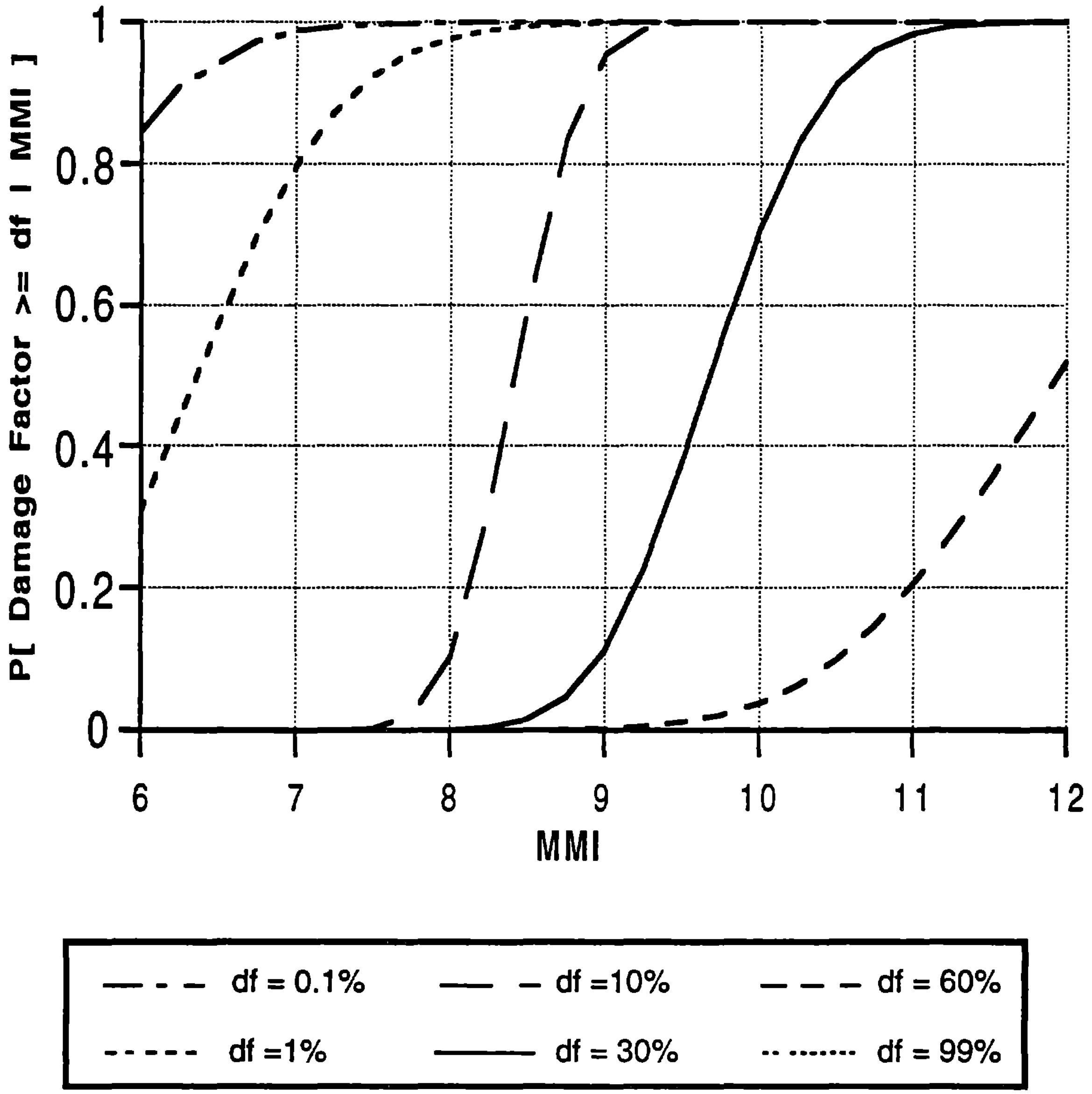
FIGURE 4-32 Fragility curves and parameters for Structure Class 79, Medium Rise URM with Load Bearing Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 80

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.32	0.19
1	1.41	0.2
10	1.96	0.11
30	2.12	0.05
60	2.3	0.11
99	2.8	0.179

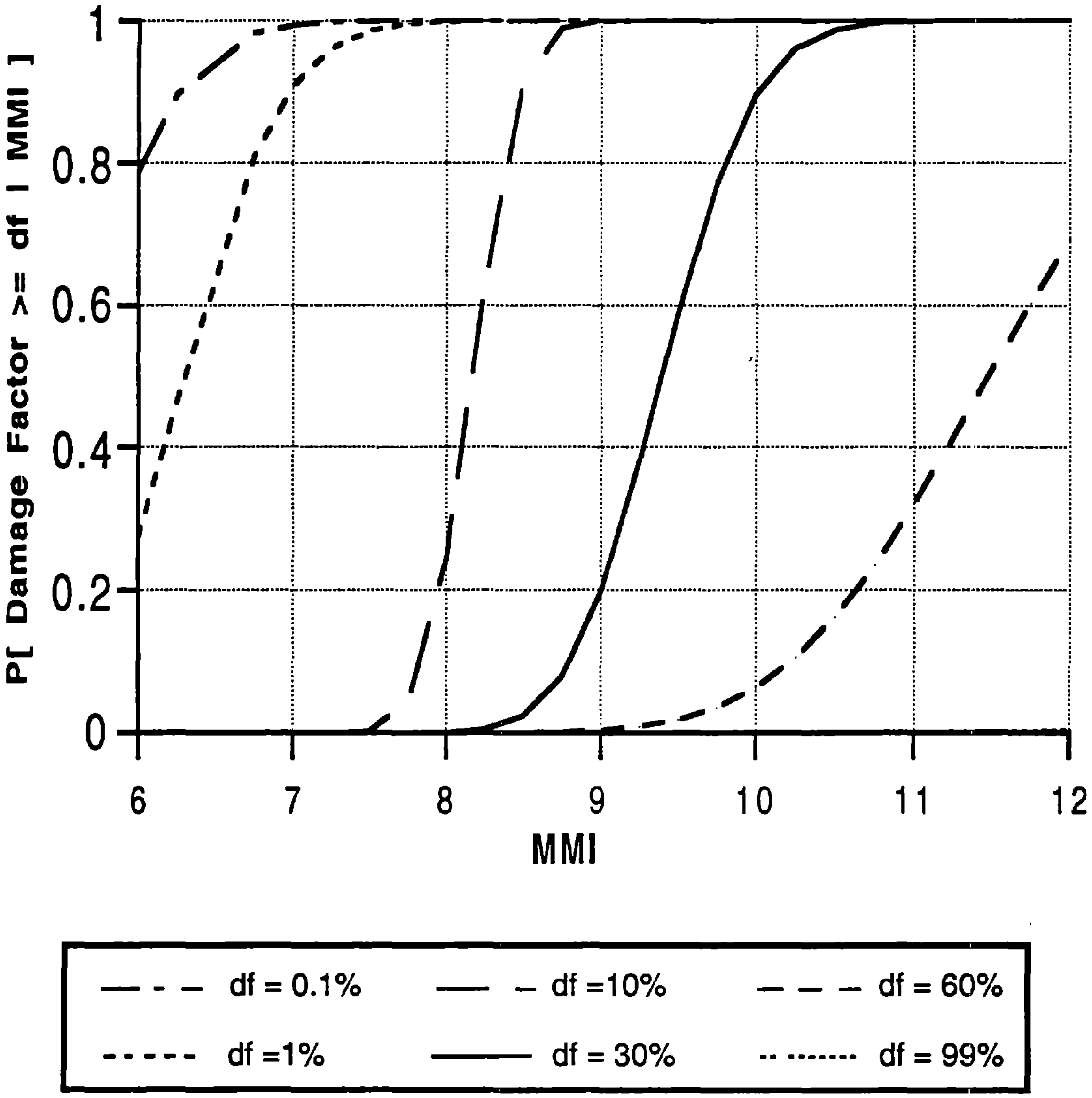
FIGURE 4-33 Fragility curves and parameters for Structure Class 80, High Rise URM with Load Bearing Frame



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 81

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.66	0.13
1	1.85	0.115
10	2.13	0.04
30	2.27	0.0599
60	2.48	0.0998
99	2.75	0.09

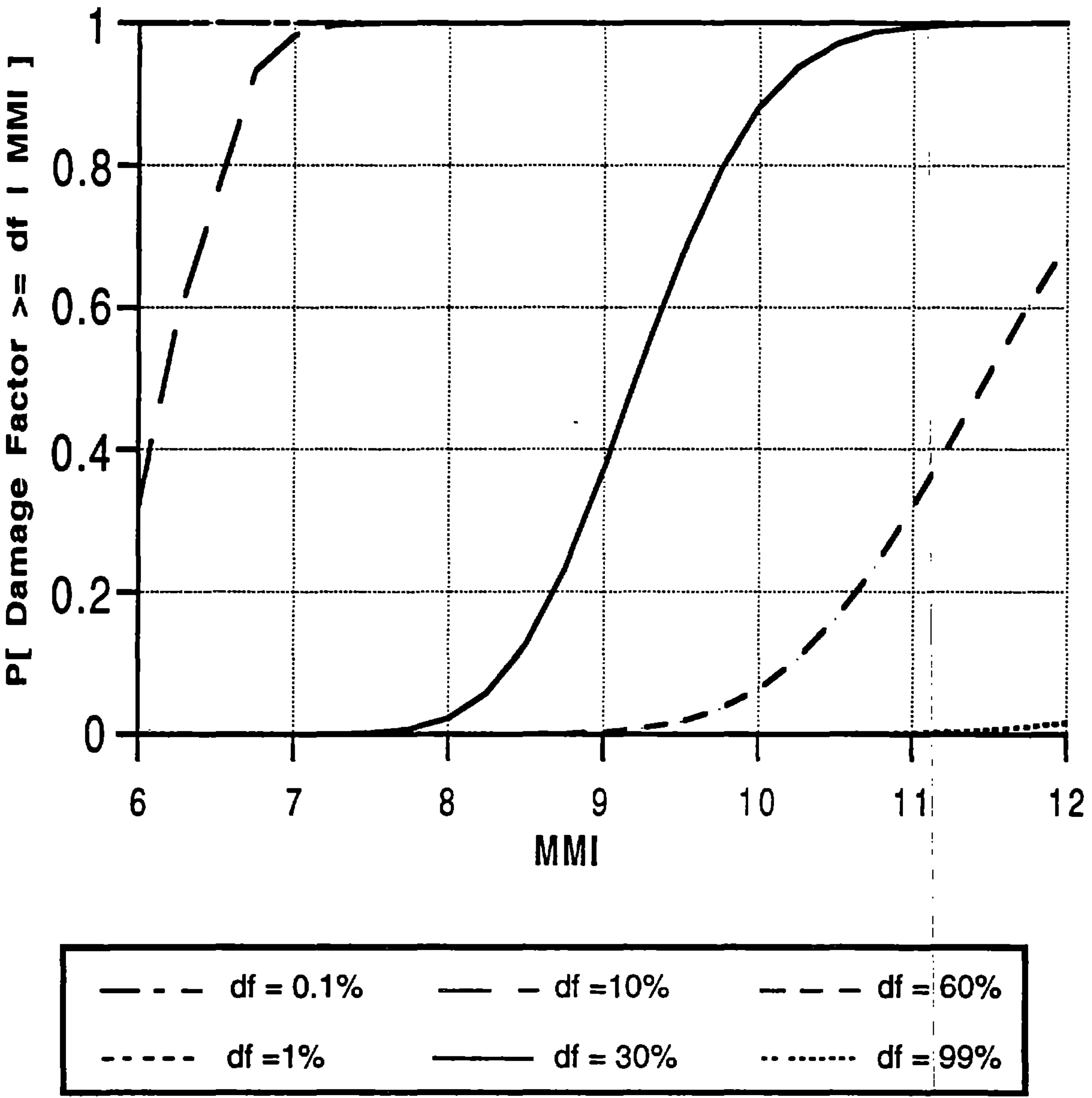
FIGURE 4-34 Fragility curves and parameters for Structure Class 81, Low Rise Precast Concrete (not Tilt-up)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 82

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.72	0.0898
1	1.84	0.0799
10	2.10	0.03
30	2.24	0.05
60	2.44	0.0898
99	2.73	0.09

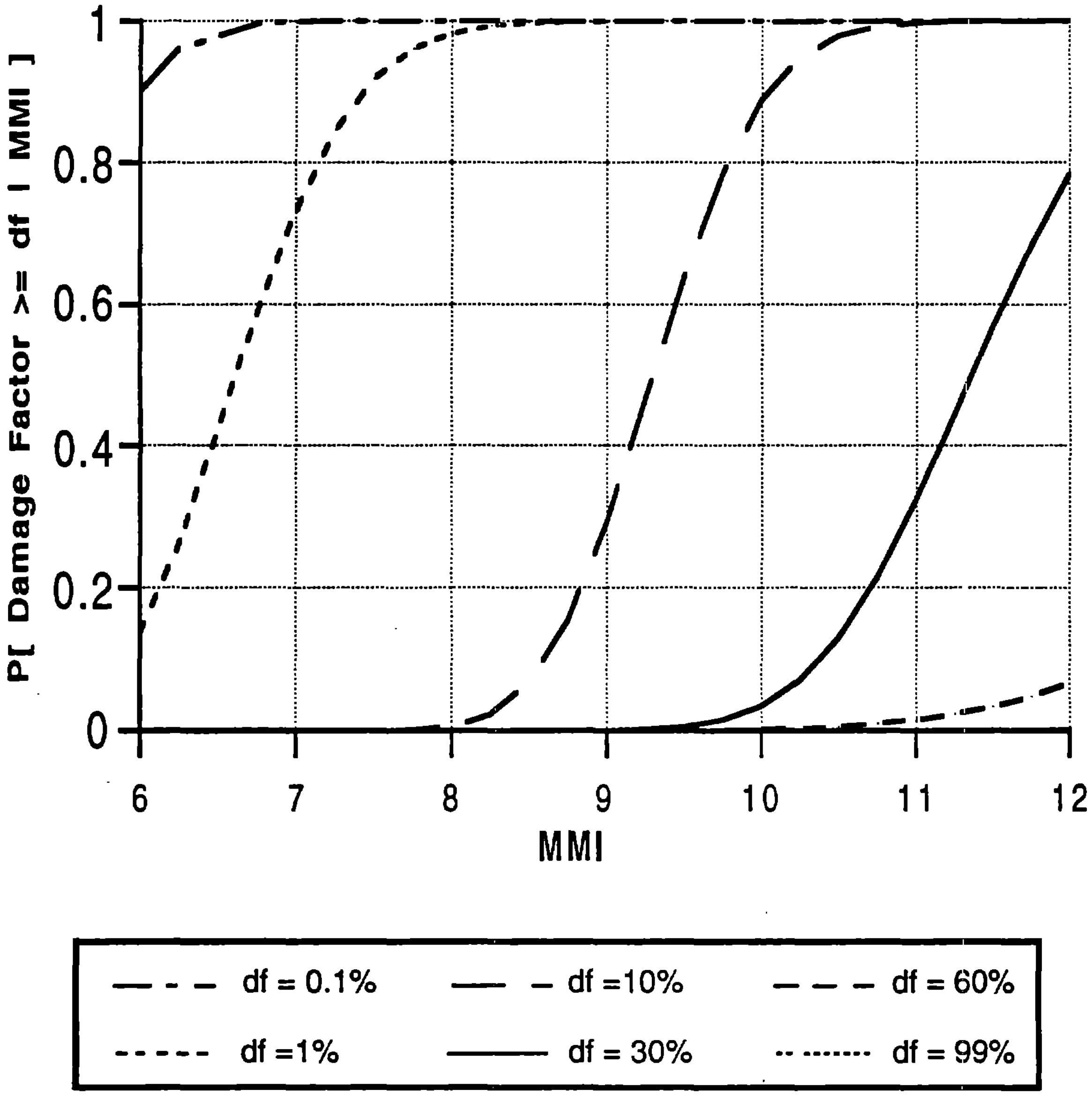
FIGURE 4-35 Fragility curves and parameters for Structure Class 82, Medium Rise Precast Concrete (not Tilt-up)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 83

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.13	0.198
1	1.41	0.0998
10	1.82	0.0599
30	2.22	0.0699
60	2.44	0.0898
99	2.74	0.12

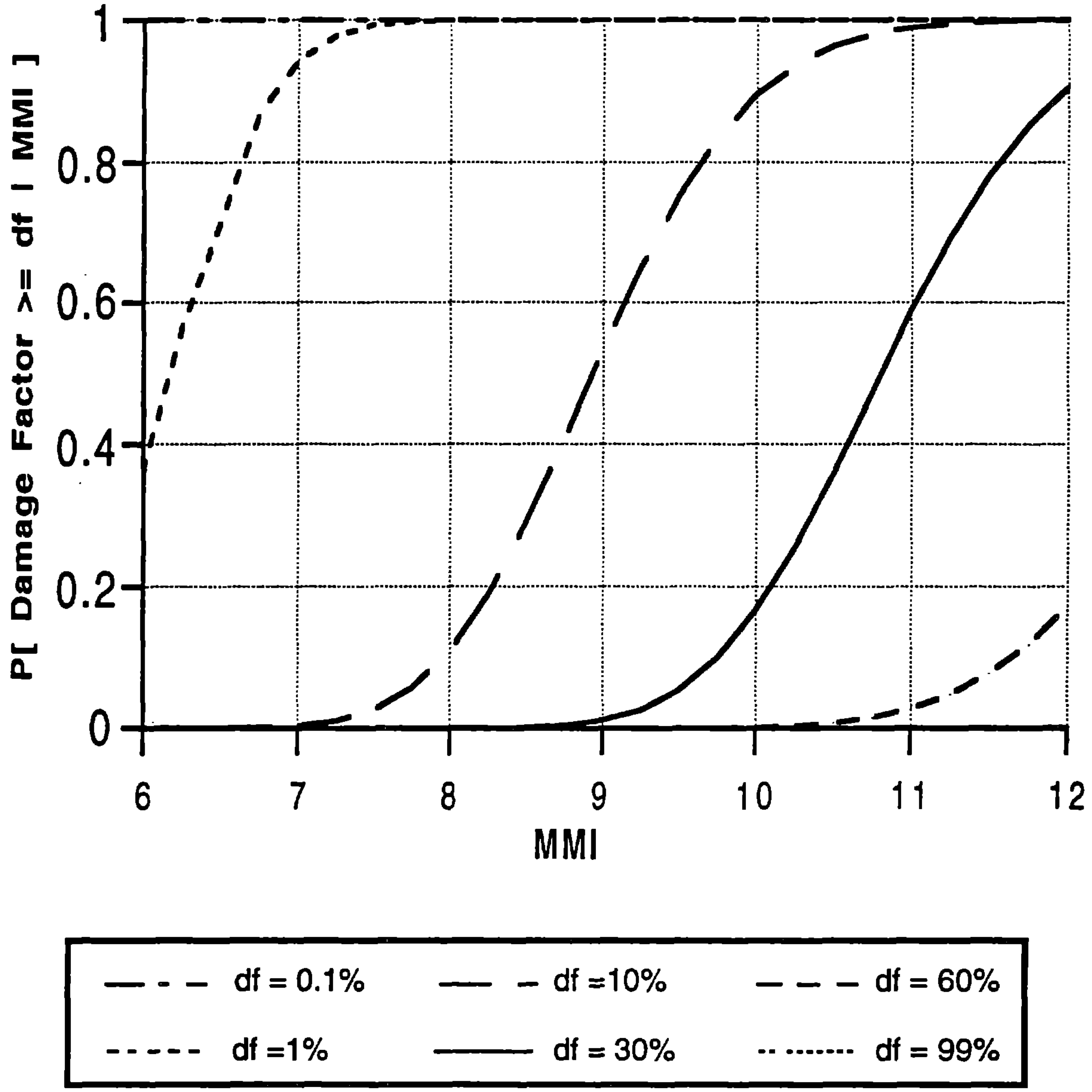
FIGURE 4-36 Fragility curves and parameters for Structure Class 83, High Rise Precast Concrete (not Tilt-up)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 84

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.68	0.087
1	1.89	0.0898
10	2.23	0.0599
30	2.43	0.0699
60	2.68	0.13
99	2.75	0.09

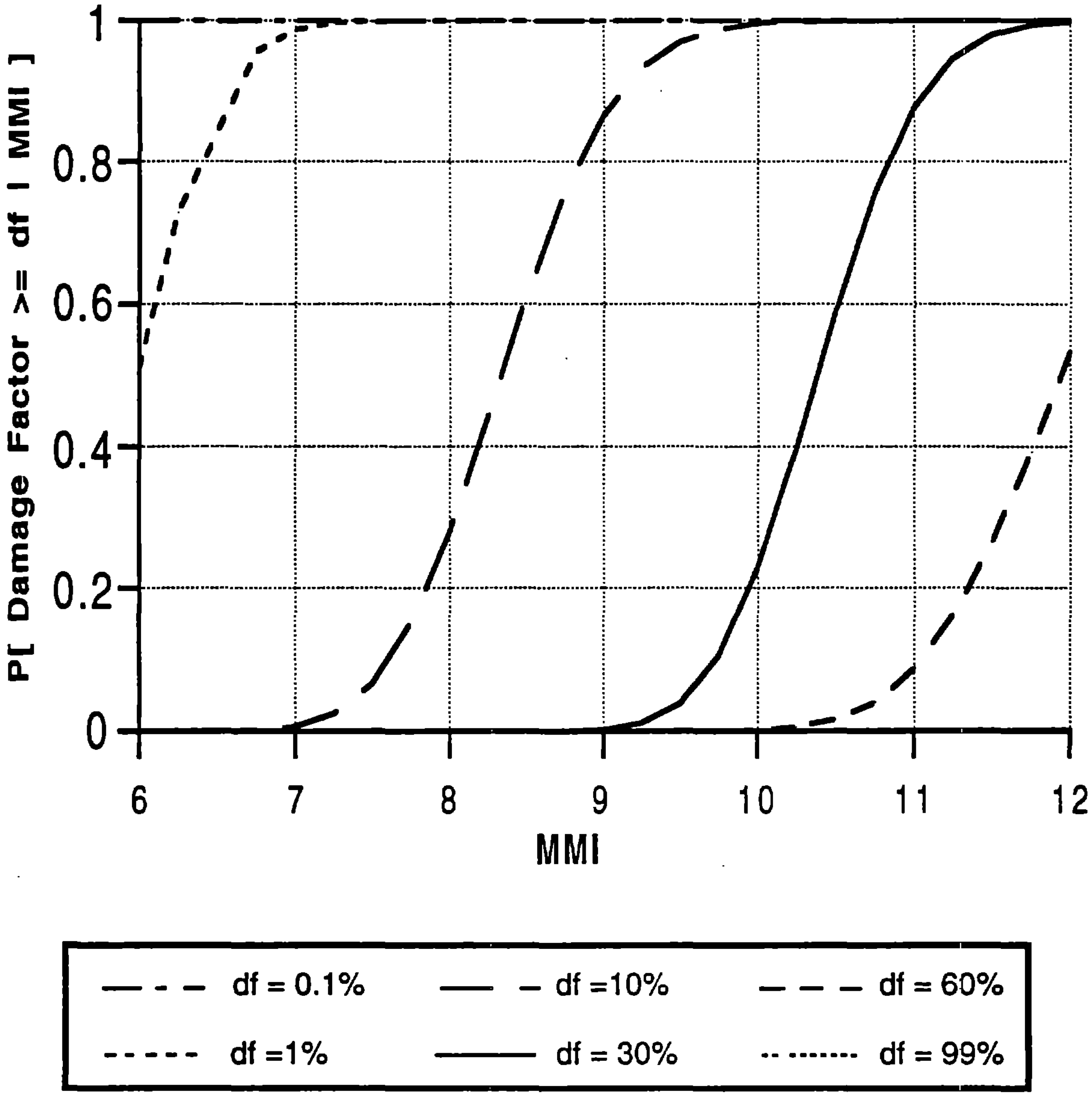
FIGURE 4-37 Fragility curves and parameters for Structure Class 84, Low Rise Reinforced Masonry Shear Wall (w/ MRF)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 85

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.13	0.198
1	1.82	0.0799
10	2.19	0.0898
30	2.38	0.0799
60	2.57	0.0898
99	2.75	0.09

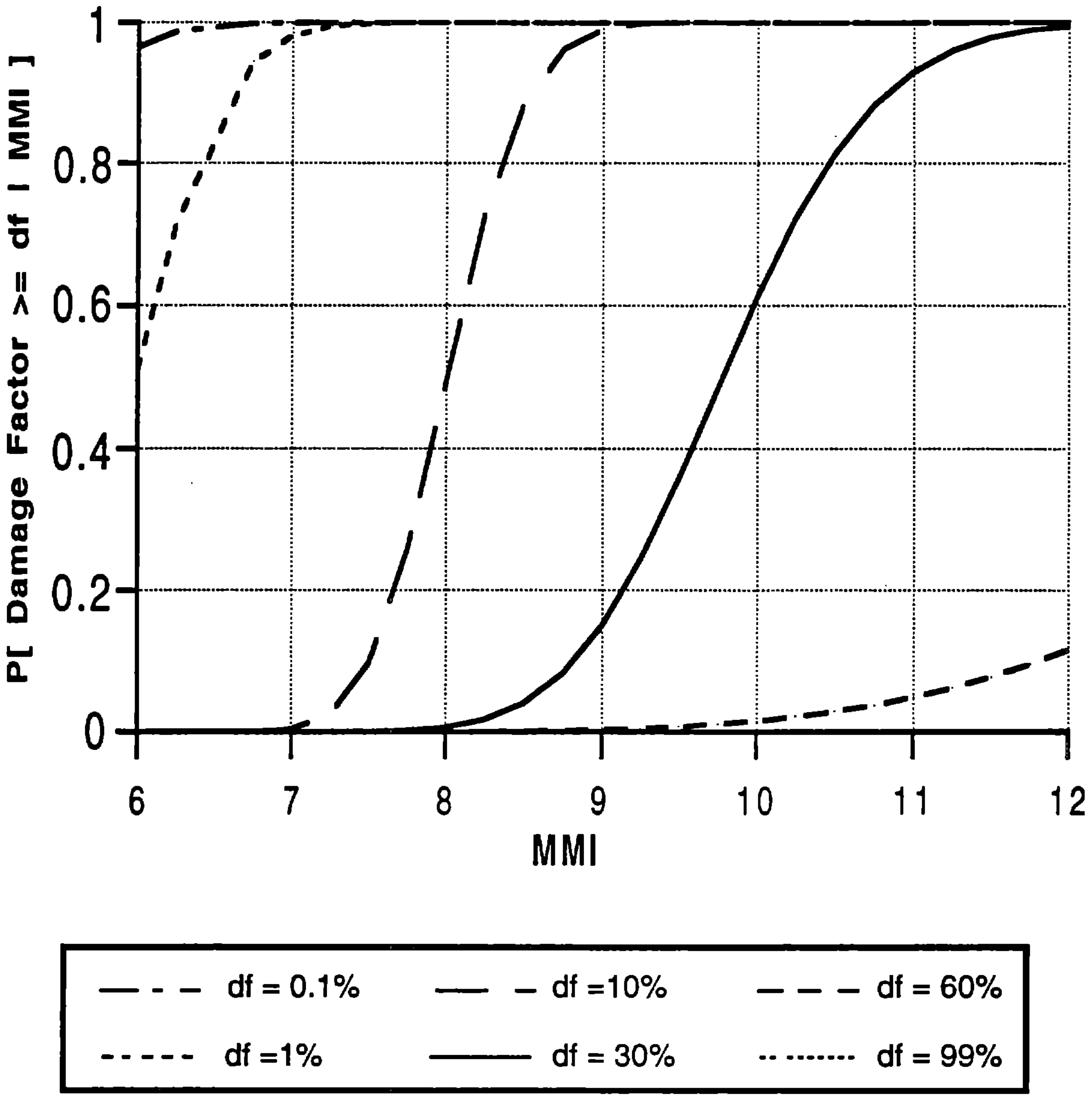
FIGURE 4-38 Fragility curves and parameters for Structure Class 85, Medium Rise Reinforced Masonry Shear Wall (w/ MRF)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 86

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.41	0.0998
1	1.79	0.07
10	2.12	0.0699
30	2.34	0.05
60	2.48	0.0599
99	2.75	0.09

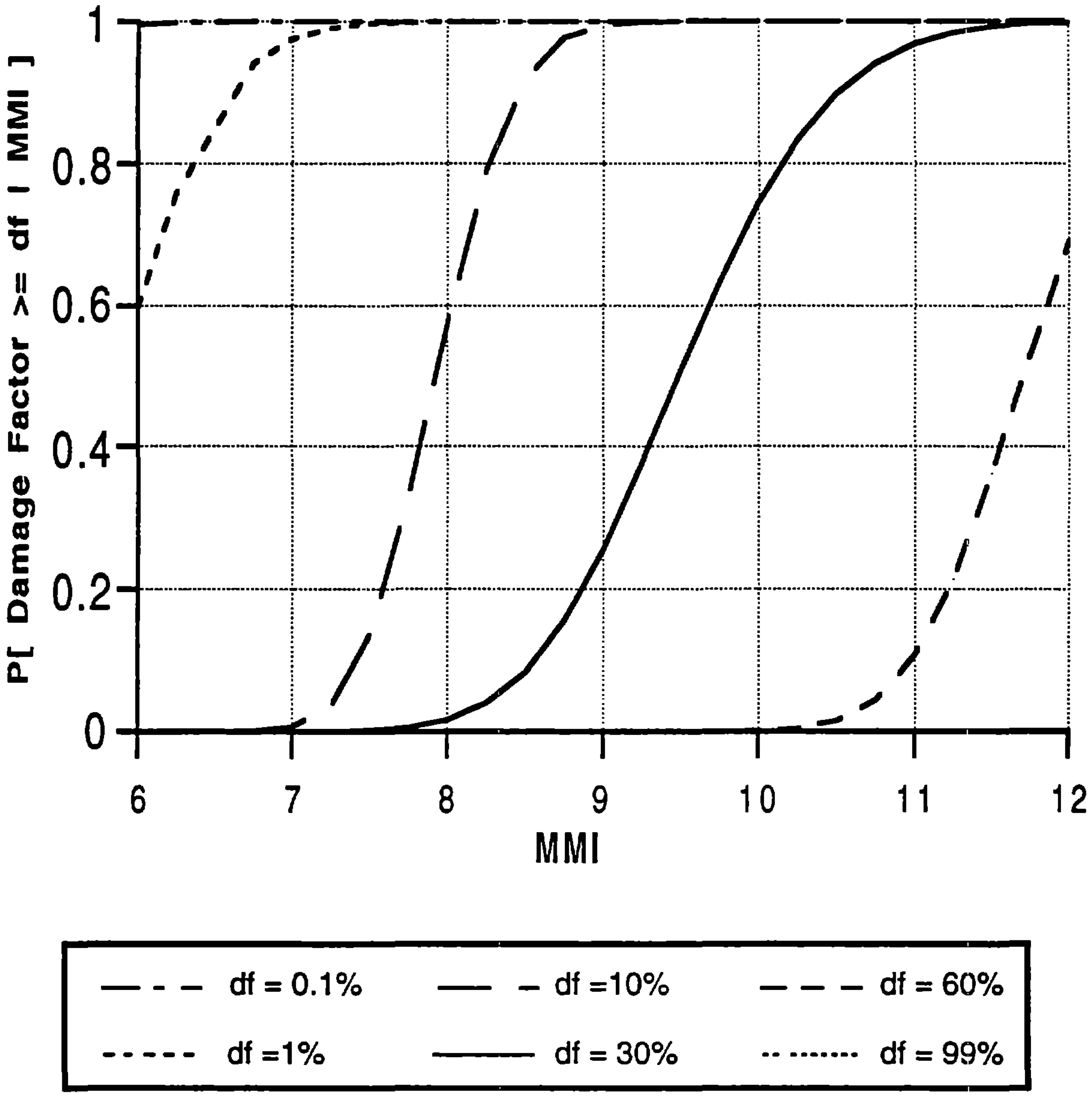
FIGURE 4-39 Fragility curves and parameters for Structure Class 86, High Rise Reinforced Masonry Shear Wall (w/ MRF)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 87

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.61	0.10
1	1.79	0.075
10	2.08	0.05
30	2.28	0.0799
60	2.71	0.188
99	2.75	0.09

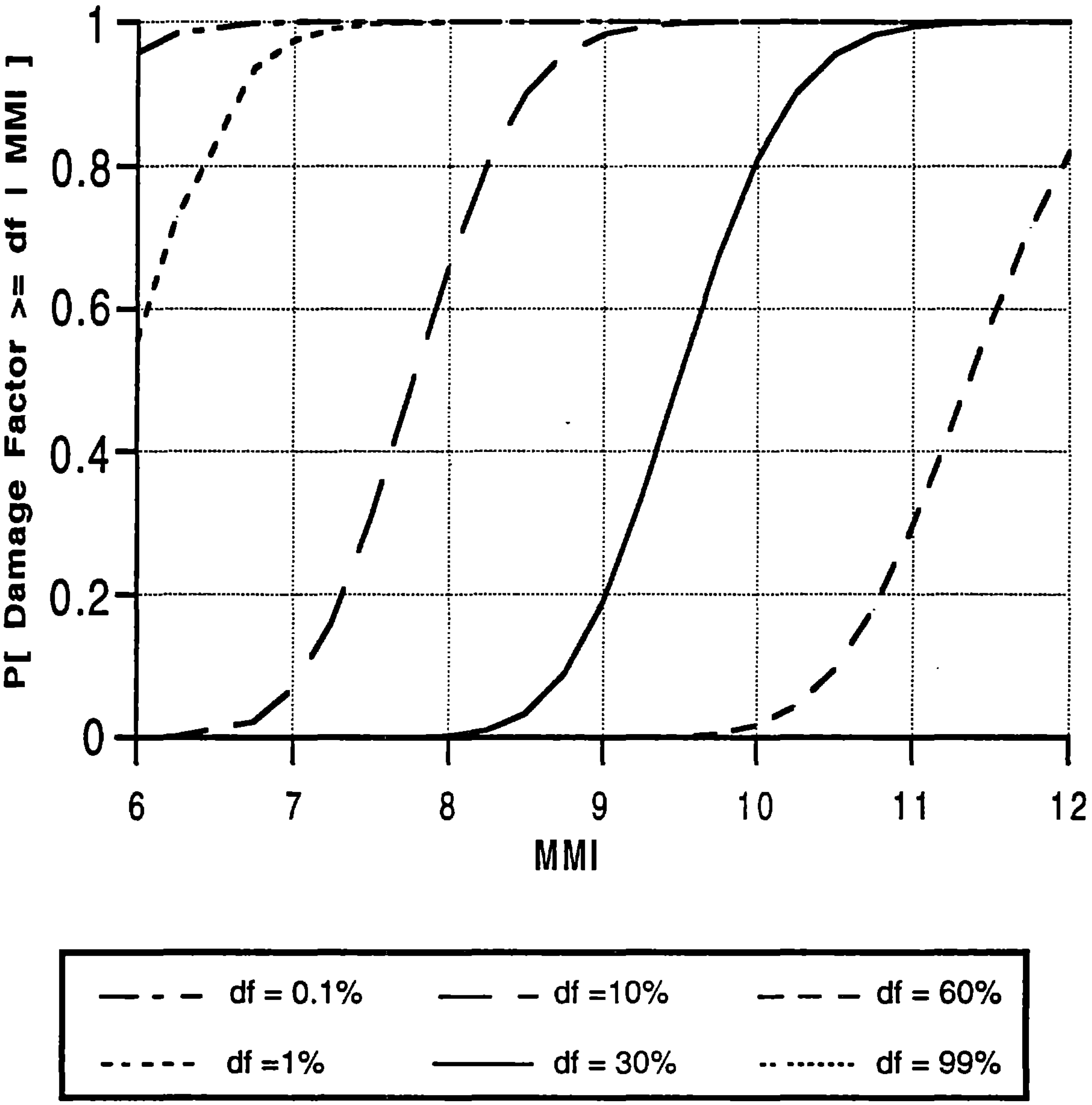
FIGURE 4-40 Fragility curves and parameters for Structure Class 87, Low Rise Non-Ductile Concrete MRF (Distributed)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 88

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.53	0.10
1	1.77	0.0898
10	2.07	0.05
30	2.25	0.0799
60	2.46	0.05
99	2.75	0.09

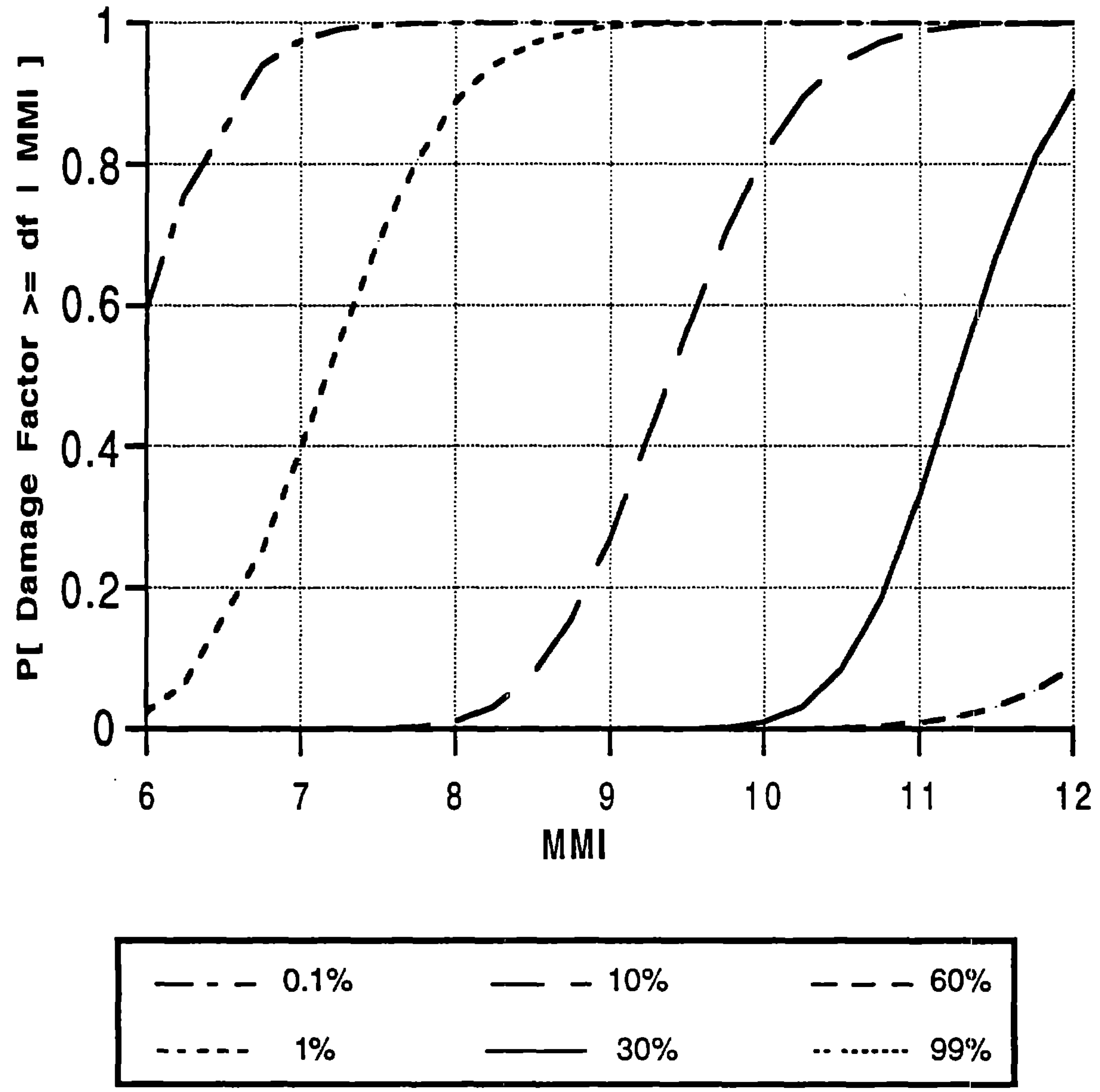
FIGURE 4-41 Fragility curves and parameters for Structure Class 88, Medium Rise Non-Ductile Concrete MRF (Distributed)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 89

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.62	0.10
1	1.78	0.0855
10	2.05	0.0699
30	2.25	0.0599
60	2.43	0.0599
99	2.80	0.09

FIGURE 4-42 Fragility curves and parameters for Structure Class 89, High Rise Non-Ductile Concrete MRF (Distributed)



Parameters of Lognormal Distributions for Fragility Curves
for Building Class 91

Damage Factor (%)	Median of Ln(MMI)	Standard Deviation of Ln(MMI)
0.1	1.77	0.0898
1	1.97	0.0898
10	2.24	0.0699
30	2.42	0.05
60	2.60	0.085
99	2.83	0.09

FIGURE 4-43 Fragility curves and parameters for Structure Class 91, Low Rise Long Span Buildings

SECTION 5
CONCLUSIONS

The detailed descriptions of 17 construction categories found in Section 2 of this report serve to clarify the building classification system and use of construction quality modifiers found in ATC-13. In cases where ATC-13 Damage Probability Matrices are modified to use in loss estimation studies outside of California, the descriptions will help distinguish California design and construction practices from local practices.

An additional improvement to ATC-13 would be to include detailed descriptions of the damage states for each of the 17 construction classes. It is likely that damage states may not be consistent across all building classes. For example, there may be some threshold level above which it is not economical to repair a structure. If a structure experiences damage above this threshold, it is likely that it will be torn down and a new structure built. When this scenario occurs, the damage factor is 100%. Thus, it is possible that above this threshold, all structures should be in the class "destroyed". The threshold may be different for different building classes. For unreinforced masonry buildings the threshold damage factor may be as low as 40%. For steel or concrete structures the threshold may be higher.

Descriptions of damage states would be very useful in collecting damage data after earthquakes. Damage data that are being collected after earthquakes in most cases do not include the types of information that is needed to update or develop damage motion relationships. Most data that were reviewed were missing key descriptors such as building location, construction type, ground motion intensity or a consistent description of damage. In particular, because different data sets use different descriptions of damage, combining data from multiple data sets is difficult. Typically, the data include the number of damaged buildings with no information about the number of undamaged buildings, thus limiting their usefulness in developing damage probabilities.

Difficulties arose when trying to combine available data with existing expert opinion. Specifically, no rational method was identified for weighting the expert opinion. One option is to assign weight to the expert opinion according to the number of experts used to develop the damage relationships. This, however, is not a good alternative because it would take very little data to essentially eliminate the contribution of the experts. Inasmuch as experts have developed their opinions based on investigating many buildings, each expert should be weighted more than one data point. However, it is very difficult to assign an exact number to this expertise.

Another problem that arose in applying Bayesian techniques was how to analytically combine the earthquake data with existing expert based damage probabilities. Many Bayesian techniques rely on the existence of a conjugate prior. The conjugate prior - posterior formulation did not prove to be feasible for this analysis.

Lognormal fragility relationships were developed for 40 building classes for "Standard" construction in California. These curves are based on the expert opinion from ATC-13. Comparisons of these curves may suggest the consolidation of some facility classes if the curves are sufficiently close. Criteria for the combination of classes need to be developed.

SECTION 6
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