USING CSMIP DATA TO TEST FEMA P58 METHODOLOGY AND THE POTENTIAL FOR AUTOMATED LOSS ESTIMATION

P. Martuscelli, J. P. Moehle

Department of Civil and Environmental Engineering
University of California, Berkeley

Abstract

The research objective is to study whether results of FEMA P58 loss assessment methodology are consistent with damage observations from real earthquakes and to develop automated software routines for projecting losses using the FEMA P58 methodology and instrumental recordings from building responses. To this purpose, a direct loss analysis approach is developed in which instrumental recordings from the California Strong Motion Instrumentation Program are used to define Engineering Demand Parameters (EDPs) for input to the FEMA P58 methodology, which is then used to estimate damage and losses. A subset of instrumented buildings that suffered earthquake damage are selected for the study. Calculated and observed damage are compared to evaluate the reasonableness of the procedures. The study is part of a larger study that will also compare results of the direct loss analysis approach with results obtained by a full implementation of the FEMA P58 methodology.

Introduction

Rapid developments in performance-based seismic design procedures in the United States occurred in the 1980s and 1990s in response to societal reactions to the nearly annual occurrence of damaging earthquakes in the Western United States during this period. These earthquakes did not cause collapse or life safety endangerment in many cases, but they amply demonstrated that the building code provisions permitted extensive damage and economic loss and could readily impair the functionality of important facilities. Interest by owners and tenants to understand performance of new buildings or of seismic upgrades spurred the development of performance-based standards and guidelines and, ultimately, the development of a new engineering methodology implemented in FEMA P58 [6] to calculate expected performance of buildings. The methodology offers the capability to express earthquake losses in probabilistic terms for individual buildings, considering metrics such as capital repair costs, downtime, and casualties.

The FEMA P58 methodology is based on the PEER (Pacific Earthquake Engineering Research Center) PBEE (Performance-Based Earthquake Engineering) framework [14]. The PEER PBEE methodology seeks to treat the seismic risk assessment problem in a probabilistically consistent manner, from expected hazard and building performance, to expected losses, downtime, and casualties. Figure 1 illustrates the various steps in the process. The uncertainties of these steps (hazard analysis, structural analysis, damage analysis, and loss analysis) are explicitly accounted for to create probability distributions for performance measures of interest. In the usual application, the seismic hazard representation is developed through a probabilistic seismic hazard analysis, representative earthquake ground motions are developed, these are input into a numerical model of the building, engineering demand parameters are calculated, and then these are used to quantify Decision Variables including expected capital losses, downtime, and casualties.

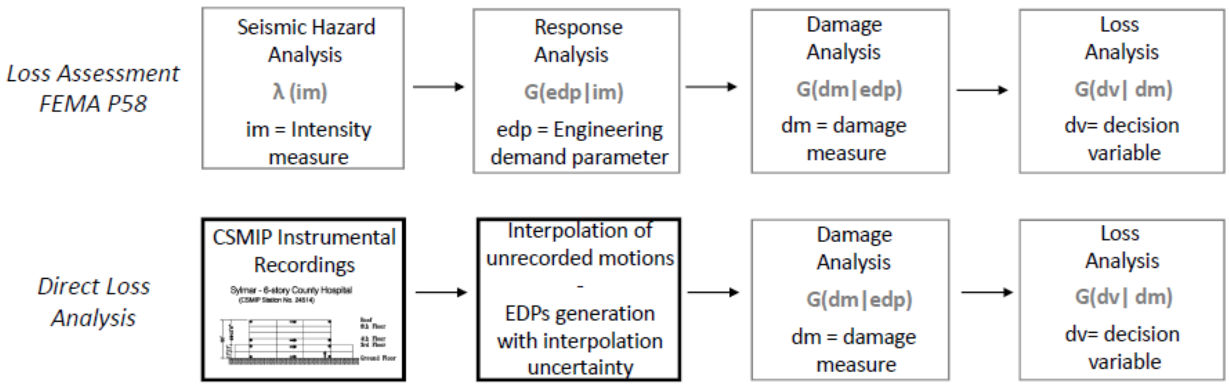


Fig.1: Direct Loss Analysis and FEMA P58 methodology flowcharts

To test the reasonableness of the FEMA P58 calculation of the Decision Variables, we have implemented a direct loss analysis approach that uses CSMIP instrumental recordings for loss estimation of buildings that have been shaken in past earthquakes. Because strong motion instruments are placed sparsely in most instrumented buildings, it is necessary to develop a procedure to reconstruct motions at locations where instruments are absent. For this purpose, an archetype building study is performed to test interpolation techniques and to infer statistical parameters that can be used to characterize approximation uncertainty for the different interpolation methods.

Three CSMIP instrumented buildings that suffered earthquake damage are selected as case-study structures. Results of calculated damage and losses are compared with post-earthquake damage surveys. This initial phase of the study is part of a larger study that will also apply the full FEMA P58 methodology to some of the buildings.

Direct Loss Analysis

Method overview

The direct loss analysis aims to provide an estimation of damage of instrumented buildings by using the instrumental recordings as direct input for loss assessment. As shown in Figure 1, the direct loss analysis process circumvents two steps of the FEMA P58 methodology, namely, the seismic hazard analysis and the structural analysis. Instead, recorded ground and floor motions are used directly to recreate EDPs in terms of peak floor accelerations (PFA) and peak story drift ratios (SDR), which are then used as input for damage and loss analysis using the fragility and consequence functions of FEMA P58.

Common instrumentation layouts for buildings do not include accelerometers at each building floor. The absence of instrumental recording at each level requires a reconstruction technique for unrecorded floor motions. The direct loss analysis aims to reconstruct missing floor responses through simple interpolation techniques. Then, depending on the chosen interpolation method, an interpolation uncertainty is assigned to the predicted quantity, which then is used through a Monte Carlo simulation to define a probability distribution of the unrecorded EDPs.

In FEMA P58 terms, the direct loss analysis intent is to provide a scenario-based loss estimation, where ground motion uncertainty is negligible due to instrumental data and the only sources of uncertainties are coming from prediction error of the interpolation techniques used to reconstruct unrecorded floor motions and from uncertainties in building contents and their fragilities.

Reconstruction of EDPs

Direct loss analysis aims to reconstruct unrecorded motions and EDPs through simple interpolation techniques, such as: 1) linear interpolation; 2) cubic spline with not-a-knot end conditions; and 3) shape-preserving piecewise cubic interpolation. These interpolation methods represent usual schemes adopted in seismic response reconstruction [4, 10, 12] and their formulations are described in detail by [3, 7]. These techniques can be used to reconstruct missing data either at each time step or exclusively with respect to maximum recorded response. Through the chosen interpolation technique, missing data are reconstructed and EDPs are defined in terms of peak floor acceleration and peak story drift:

$$PFA_j = max|a_j|$$

$$SDR_j = \frac{max|\Delta_j - \Delta_{j-1}|}{h_j}$$

where a_j and Δ_j are the acceleration and displacement at floor j and h_j is the height of story j. A general MATLAB routine, called *CSMIPDataInterpreter*, that elaborates CSMIP instrumental data to define EDPs was developed for the study and will be adopted to explore the feasibility of an automatic loss assessment procedure.

Multiple SDR input vector for torsional response

To account for building torsional response and better represent building damage, multiple story drift ratio (SDR) vectors at different building locations can be calculated from recorded and reconstructed motions and used as input for damage and loss analysis. Calculation of each set of story drift EDPs is performed by identification of displacements at any point of the floors, under the assumption of rigid diaphragm, as described by *Naeim et al.* [12] and shown in Figure 2:

$$\begin{bmatrix} A_x \\ A_y \\ \theta \end{bmatrix} = \begin{bmatrix} \frac{y_c - y_3}{x_2 - x_1} & \frac{y_3 - y_c}{x_2 - x_1} & 1 \\ \frac{x_2 - x_c}{x_2 - x_1} & \frac{x_c - x_1}{x_2 - x_1} & 0 \\ -\frac{1}{x_2 - x_1} & \frac{1}{x_2 - x_1} & 0 \end{bmatrix} \begin{bmatrix} A_1 \\ A_2 \\ A_3 \end{bmatrix}$$

where A1, A2 and A3 are the recorded motions; (x_1, y_1), (x_2, y_2), and (x_3, y_3) are the coordinates of the sensors; and (x_c, y_c) is the coordinate of the point of interest.

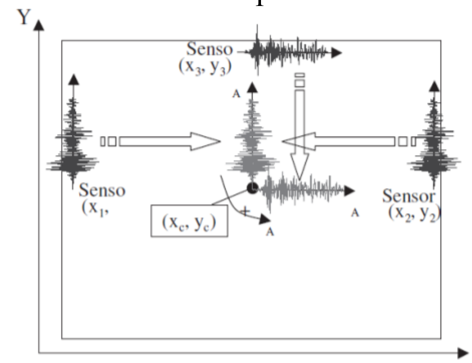


Fig.2: Displacement time-series transformation, after *Naeim et al.* [12]

By being able to calculate story drift ratios at any location of the floor plan and in any direction, it is feasible to assign location-specific story drift EDPs for each structural and non-structural assembly. Inputting more than one SDR vector into the damage and loss analysis framework is allowed by a second Matlab routine developed for this study, called IBLA (Instrumented Building Loss Analyzer).

Uncertainty quantification

The direct loss analysis differs from FEMA P58 in that the analysis does not need to consider the following types of uncertainty: modeling epistemic uncertainty β_m; ground motion uncertainty β_{gm}; and drift and acceleration record-to-record variability ($\beta_{a\Delta}$ and β_{aa}). Other uncertainties, however, need to be considered for direct loss analysis, specifically: instrumentation recording uncertainty β_r; and the uncertainty of unrecorded floor motion prediction through interpolation methods β_i. It is assumed that instrumentation recording uncertainty is negligible and that only the uncertainty introduced by interpolation methods needs to be evaluated here.

For each investigated interpolation method, uncertainty in prediction of EDPs is going to be tested through an archetypes database that is described in a later section. Results from this study will concur to define dispersion for reconstructed peak floor acceleration PFA (β_{PFA}) and peak story drift ratio SDR (β_{SD}), that are:

$$\beta_{SDR} = \sqrt{\left(\beta_{i,SDR}\right)^2 + {\beta_{m,D}}^2}$$

$$\beta_{PFA} = \sqrt{\left(\beta_{i,PFA}\right)^2 + {\beta_{m,D}}^2}$$

where $\beta_{i,SD}$ and $\beta_{i,PFA}$ are the uncertainties in approximation of SDR and PFA through different interpolation techniques and $\beta_{m,D}$ is the epistemic uncertainty related to modeling of the archetype buildings.

Use of FEMA P58 fragility and consequence functions

For the loss analysis, we need to define the structural and non-structural components. The structural components are determined through specific knowledge of the gravity and lateral resisting system of the building as obtained from the structural drawings. In the absence of more detailed knowledge, building content population is defined through the FEMA "Normative Quantitative Estimation Tool" for generic non-structural components. Then, given EDPs and their distribution, likelihood of damage and losses is calculated through uniform random number generation using fragility and consequence functions for each structural and non-structural component. The database of more than 700 fragility and consequence functions, developed for the FEMA P58 project, is adopted for loss calculation here.

Archetypes Study to Test Interpolation Techniques

An archetype buildings database is currently being developed with the intent to test different interpolation techniques for approximating building response at locations within instrumented buildings where there are no instrumental recordings. Of interest are methods for interpolating the peak floor accelerations (PFA) and peak story drift ratios (SDR), including information on bias and uncertainty for each interpolation method. The archetype models are representative of non-ductile reinforced concrete frames designed and constructed in California during 1960s. The archetypes are designed according to the 1961 Uniform Building Code.

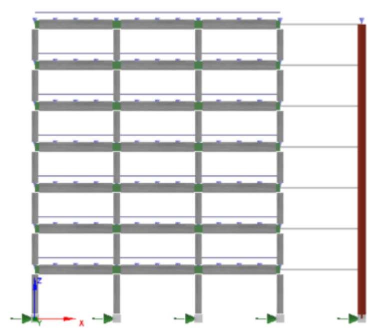


Fig.3: Archetype structural configuration – SeismoStruct by Seismosoft

Preliminary results of this investigation are presented for a set of archetype models that are representative of the first case-study building, a 7-story hotel. The range of design parameters for the archetype models is described in Table 1. The current set of analyses, for five archetype configurations, comprises a total of 200 nonlinear response history analyses that are consistent with the seismic intensity experienced by the 7-story Hotel during 1994 Northridge event.

	Columns					Beams			Structure					
Model Name	Axial ($N/A_g f'_c$)	Size [in]		f'c [ksi] Story		Size [in]		f'c [ksi] Story	Geometry		Design Base Shear Coefficient	First Mode Period [s]	Strong Column Weak Beam	Section Aggregator for Shear
Mod.#	Gf	b	h	1,2	3,7	b	h	1,7	N_{story}	N_{bay}		T_1		
A4.3	0.31	26	24	5	4	20	30	4	7	3	0.080	0.81	1.0	No
A4.4	0.31	26	24	5	4	24	32	4	7	3	0.070	0.73	0.9	No
A3.4	0.36	24	22	5	4	24	32	4	7	3	0.060	0.78	0.8	No
A2.4	0.38	24	20	5	4	24	32	4	7	3	0.058	0.84	0.7	No
A1.1	0.43	20	18	5	4	18	24	4	7	3	0.050	1.27	0.7	No

Table.1: Archetypes modeling and design parameters

Structural models for each archetype configuration consist of a two-dimensional representation of the building lateral resisting system; for the first set of architypes presented herein, the two-dimensional representation is characterized by three bays and seven stories (Figure 3). Nonlinear material response is modeled through distributed plasticity elements with force-based formulation; geometric non-linearities are explicitly considered. For several archetype performance groups, shear strength during the analysis is continuously calculated according to ASCE 41-17 equation (10-3) and ACI 318-11 equations (11-5; 11-27; and 11-28) and if demand exceeds capacity, the latter is reduced to a residual value equal to 20%. The two-dimensional structural models include a leaning column that is modeled to represent vertical loads carried by the internal gravity framing system and their effect on system stiffness and stability.

Tested Interpolation methods

The interpolation techniques currently being tested are: 1) linear interpolation; 2) cubic spline with not-a-knot end conditions; and 3) shape-preserving piecewise cubic interpolation. Acceleration response histories are interpolated either at each time step (later referred as **@t_i**) or with respect to maximum recorded response (later referred as **@t_{max}**). Regarding the latter, it is important to underline that for the purpose of the study it is enough to characterize absolute maximum response since loss assessment only requires peak floor acceleration as input. Instead, to characterize absolute maximum story drift, interpolation is preferably performed at each time step.

Instrumentation configurations

The ability of the interpolation techniques to correctly predict response at not instrumented floors is evaluated considering different instrumentation layouts. Four instrumentation configurations are considered (referred as **C1, C2, C3** and **C4**) and are shown in the following figure:

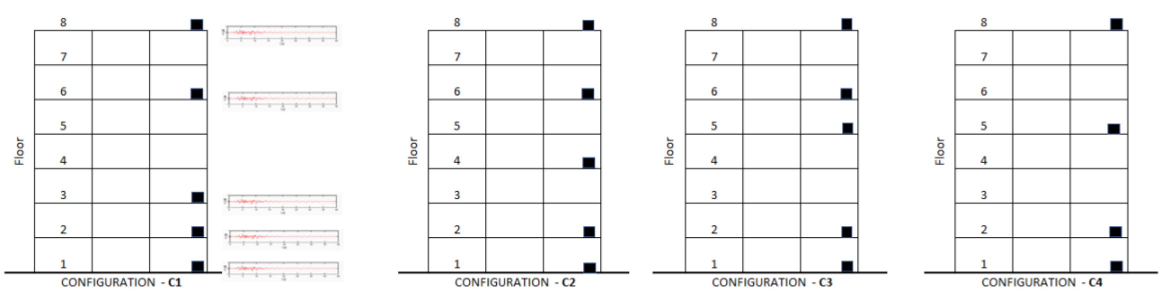


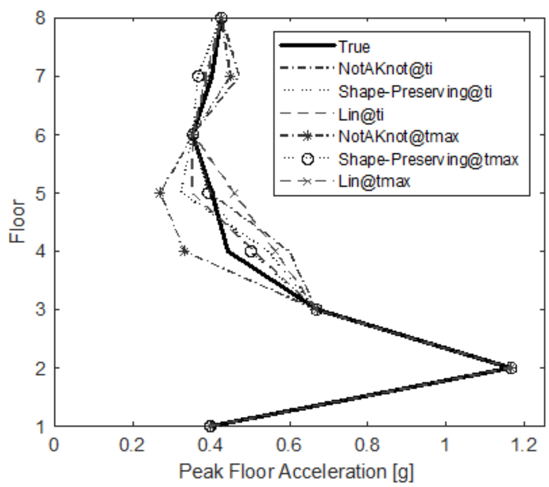
Fig.4: Tested instrumentation configurations

Error measures

The following metric is defined to characterize the interpolation error for the reconstruction of EDPs:

$$E_t = \frac{x_p - \tilde{x}}{\tilde{x}}$$

where x_p represents the interpolation prediction of story drift ratio (SDR) and peak floor acceleration (PFA), and $\tilde{x}$ is the actual value from the nonlinear dynamic structural analysis. It is to be underlined that regardless of how interpolation is done (at each time step or at maximum recorded response), the error is calculated only in terms of absolute maximum SDR and PFA, since these are the only quantities of interest for loss assessment. For each instrumentation configuration, the error is calculated at each "non-instrumented" floor, as shown in the next figure:



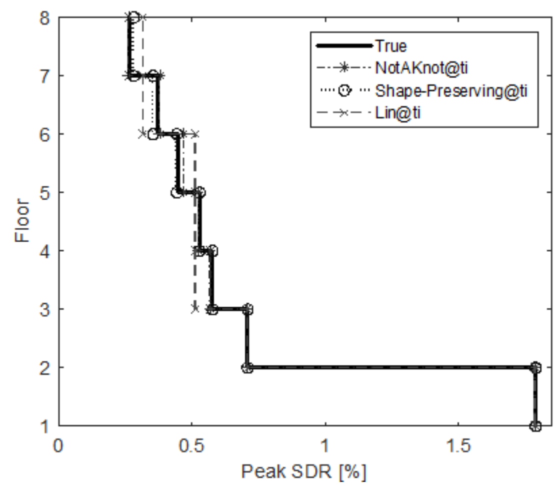


Fig.5: Different interpolation techniques for one NLRHA – Configuration **C1**

An analysis of multiple nonlinear response history analyses shows that there is little variation of interpolation prediction error over the archetype model height, which allows to aggregate the error data for all the floors. The next figures present the interpolation prediction error for different techniques for PFA and peak SDR, considering instrumentation configuration **C1**:

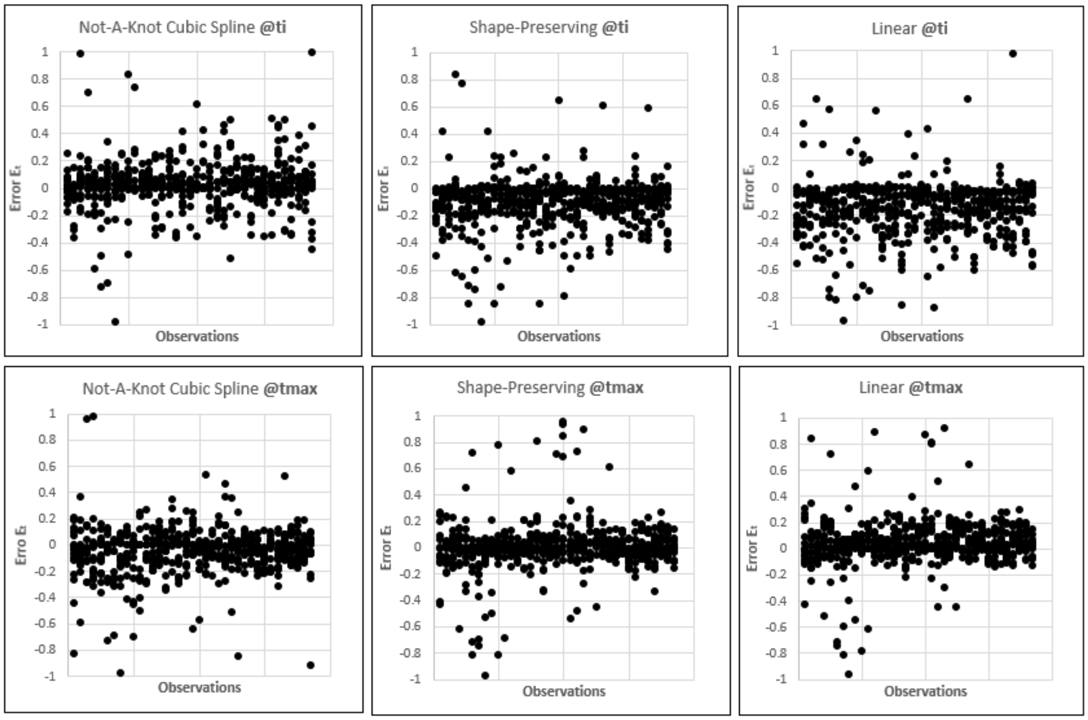


Fig.6: Interpolation error for **PFA** – Configuration **C1**

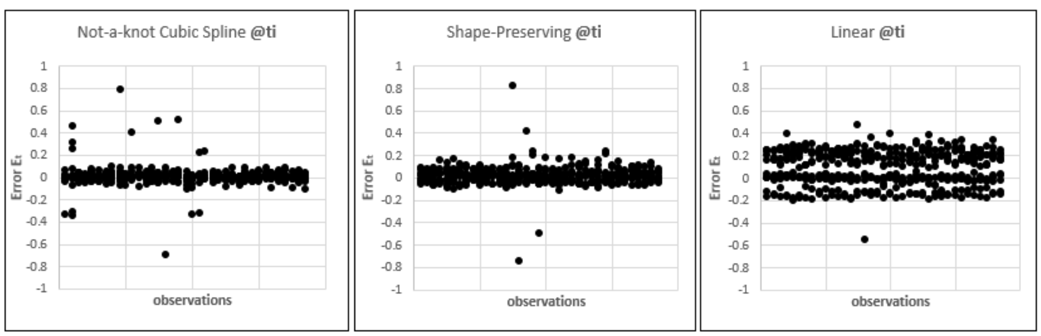


Fig.7: Interpolation error for peak **SDR** – Configuration **C1**

The aggregated data of prediction error for PFA and peak SDR, in their preliminary form, allows to define prediction error statistics for the different interpolation techniques:

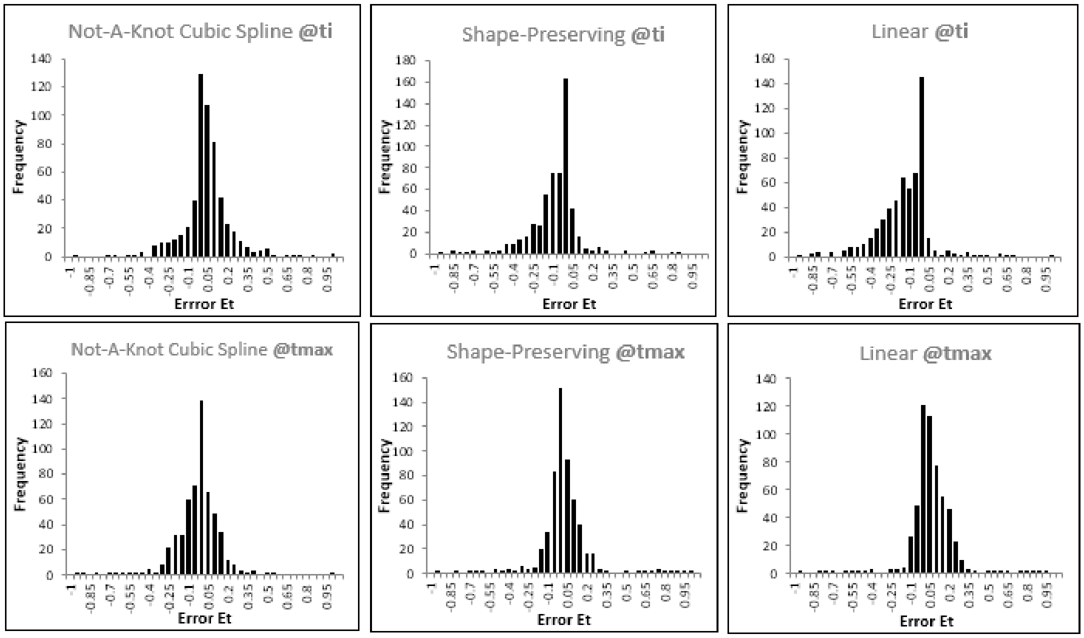


Fig.8: Et frequency distribution for **PFA** – Configuration **C1**

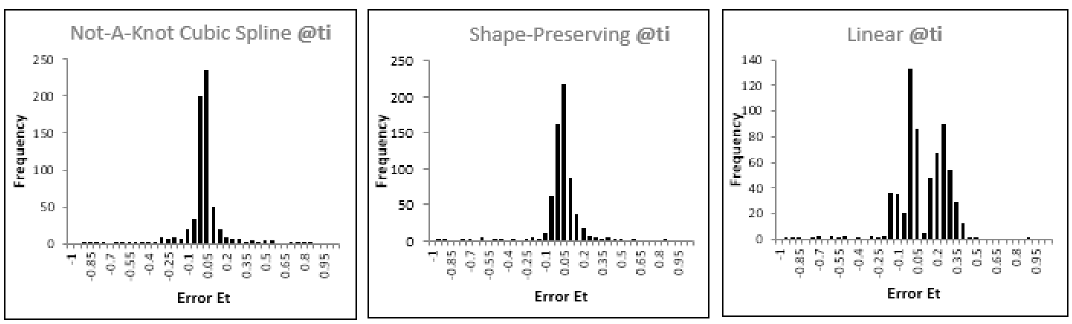


Fig.9: Et frequency distribution for peak **SDR** – Configuration **C1**

The prediction error for the different interpolation methods is calculated also for the other investigated instrumentation configurations, namely **C2**, **C3** and **C4**. The following figure summarizes findings for each interpolation technique and instrumentation layout:

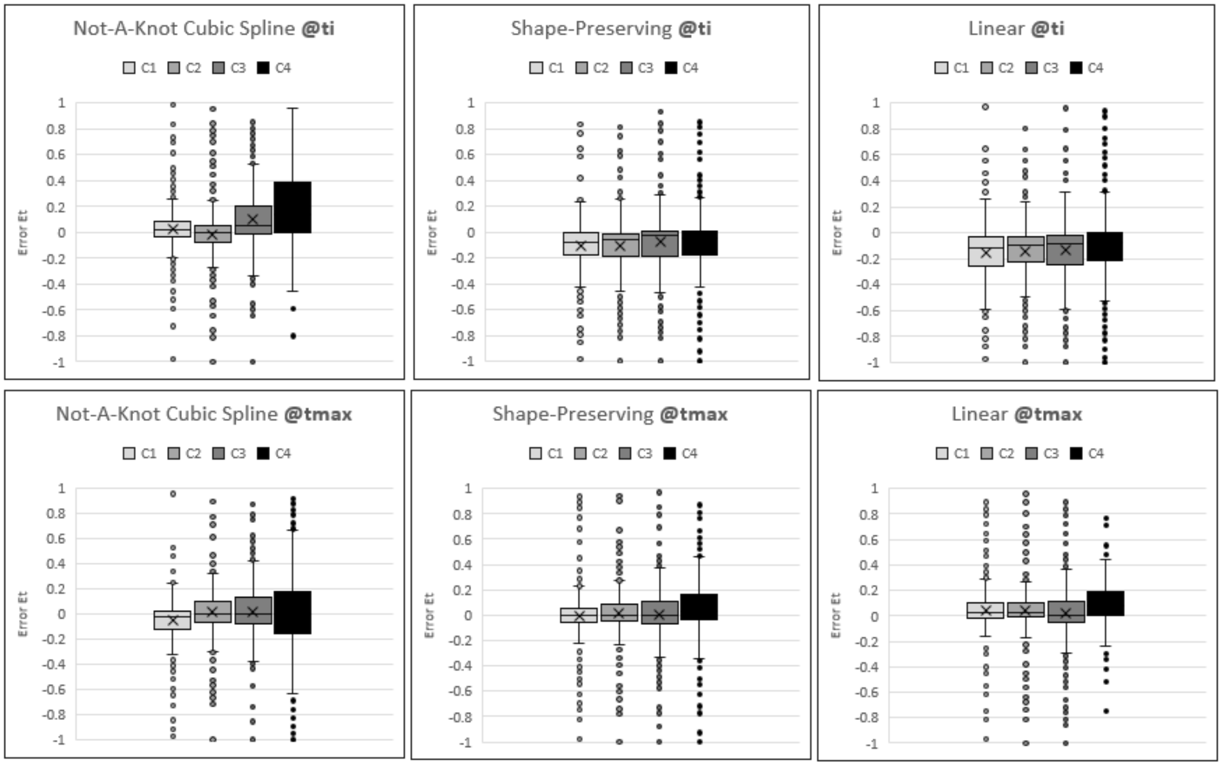


Fig.10: Interpolation error for PFA – Configuration **C1, C2, C3, C4**

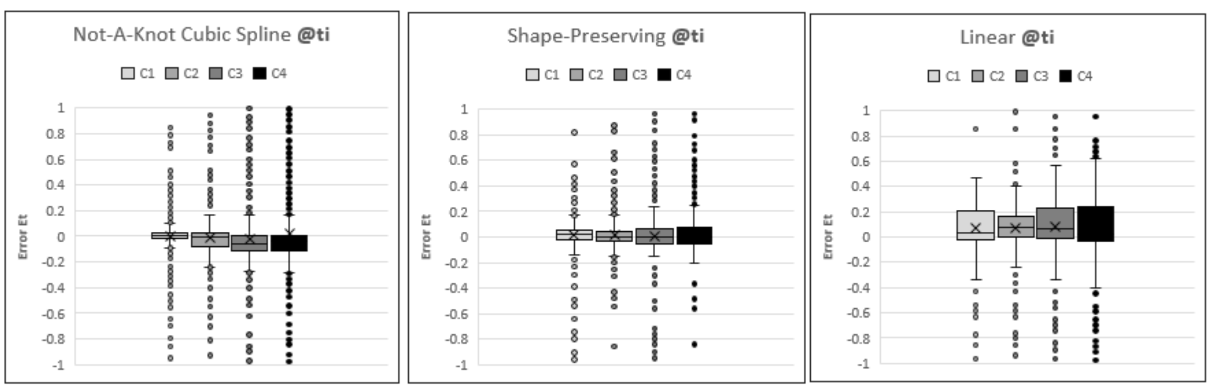


Fig.11: Interpolation error for peak SDR – Configuration **C1, C2, C3, C4**

Once completed in later stages of this study, the archetypes study results will be used to infer dispersion for interpolation prediction error that will then be adopted to determine a probability distribution for unrecorded EDPs.

First Case-Study Building: 7-story Hotel

Building description

The first building selected for the study is a 7-story hotel [CSMIP Station #24386], shown in Figure 12. The building has a seven-story reinforced concrete structure with a floor plan of approximately 150 by 62 feet. The building was designed in 1965 according to the Uniform Building Code (UBC 1961) and constructed in 1966. It is characterized by a lateral-force-resisting system made of non-ductile perimeter moment resisting frames and an internal gravity system comprising a two-way flat slab supported by square columns. The construction cost of the building was $1.3 million in 1966 dollars as reported by John A. Blume & Associates (1973). The 1966 construction cost is equivalent to $6.7 million in 1994 dollars, the year of the 1994 Northridge earthquake, which is assumed as a reference for the following analyses.

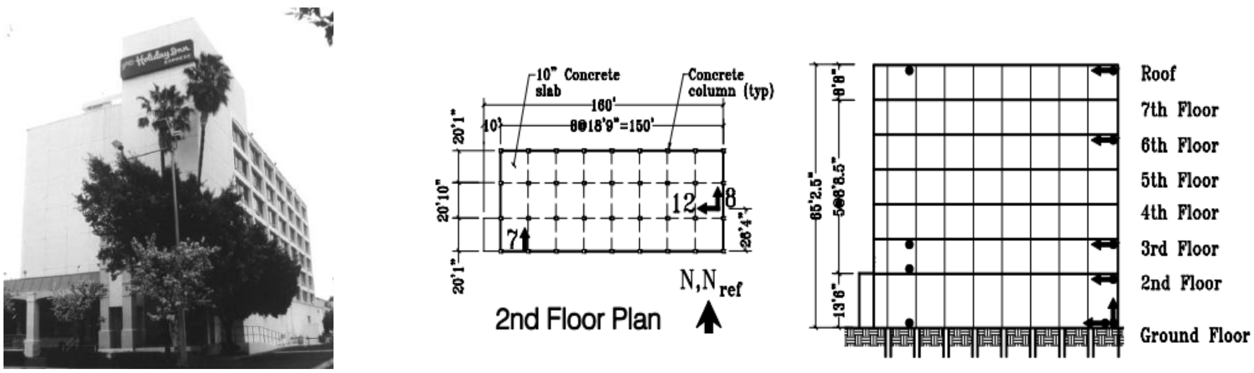


Fig.12: 7-story hotel

The building was instrumented prior to 1971 and its response to major earthquakes was studied extensively [2, 5, 13, 17, 18], including after the 1994 Northridge earthquake that caused severe structural damage, as shown in Figure 13.

Fig.13: Structural damage during Northridge earthquake

During the Northridge earthquake the building suffered also some light non-structural damage, shown in the next figure:

Fig.14: Non-structural damage during Northridge earthquake

Outline of the analyses

The hotel response is studied through direct loss analysis first for the Northridge earthquake and then for the Landers and Big Bear earthquakes. For the Northridge event, three different types of analyses are performed in terms of input story drift: 1) As Is analysis (**As Is**), where story drift ratio (SDR) is calculated at the instrument locations; 2) geometric center analysis (**GC**), where SDR is calculated at the geometric center of the building floors; and 3) single frame analysis (**SF**), where SDR is calculated at the location and in the direction of each perimeter moment frame, to better account for building torsion. All direct loss analyses are performed in a simplified form in that uncertainties for unrecorded EDPs is not considered.

Reconstruction on of EDPs

The hotel is characterized by an instrumentation layout that is consistent with the investigated configuration **C1** (Figure 4). Floor motions and EDPs are reconstructed for all the analyses adopting a not-a-knot cubic spline interpolation technique at each time step. As shown in previous sections, this method is characterized by minimum prediction errors for story drift ratio (SDR) reconstruction, which is the most important input given the severe structural damage that was observed after the Northridge earthquake. Peak story drift is evaluated, as explained in the outline, at different locations (Figure 15), which are: 1) instruments locations; 2) building geometric center; and 3) center of each perimeter moment frame.

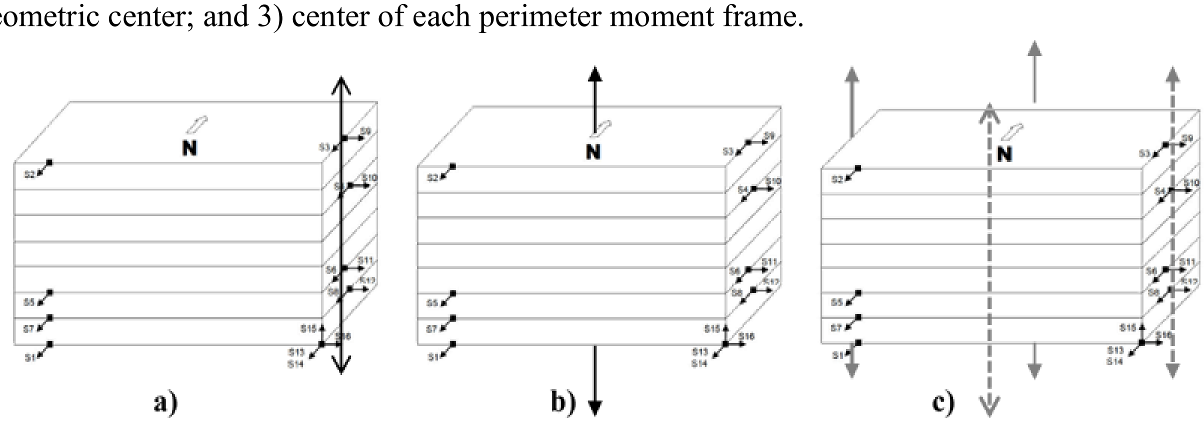


Fig.15: peak SDR location for the different direct loss analysis, a) As Is; b) GC; and c) SF

Figure 16 presents EDPs calculated for the three analyses. It can be appreciated that transposing the displacement to the building geometric center has no effect on East-West direction response. This is because instruments used to infer EDPs are located at mid-span of the North-South building framing, as shown in Figure 12. Also, the EW direction is slightly affected by torsional building response, as it can be seen from north and south frame story drift.

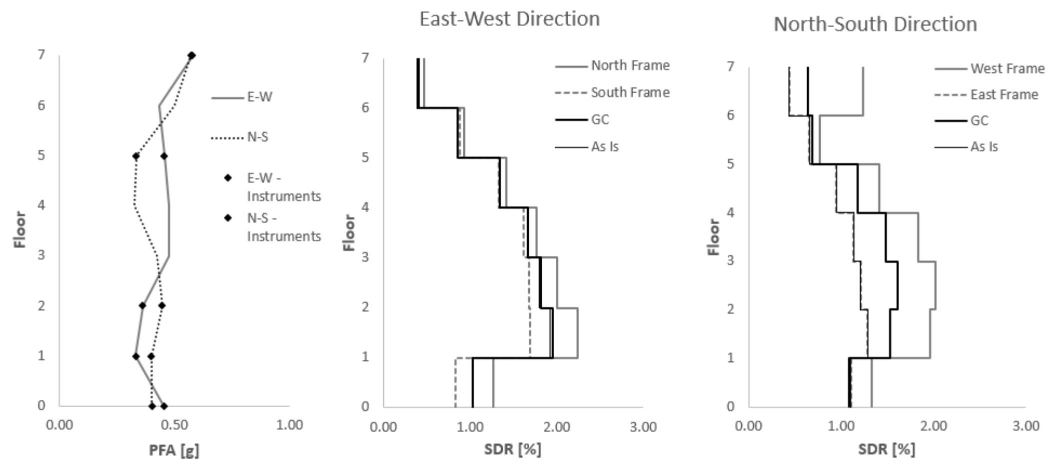
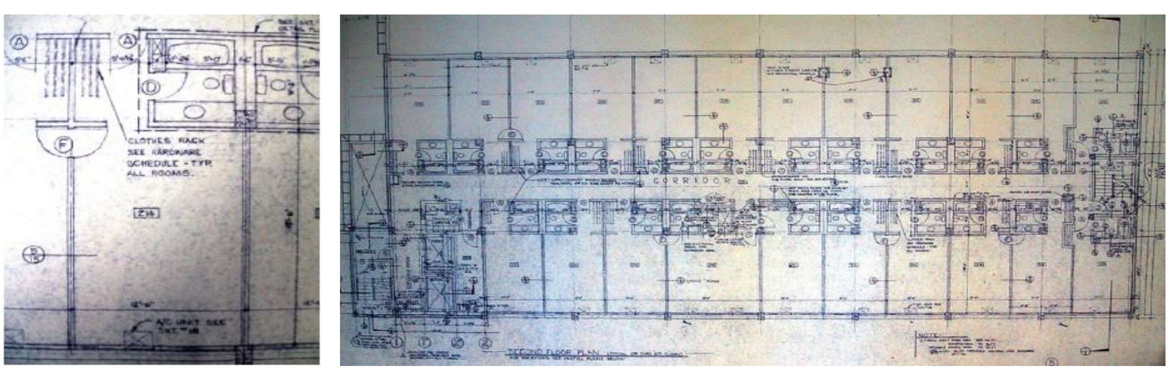


Fig.16: Calculated Engineering Demand Parameters for the building

Direct loss analyses

The loss analyses require the identification of building contents to evaluate likelihood of damage and capital repair costs. Building content population is defined using FEMA P58 "Normative Quantitative Estimation Tool" for generic non-structural components. Structural components and additional non-structural components, such as furniture, were added to the building components population, on the basis of specific knowledge of the building. Detailed information was available for the building, including architectural floors plan and suite floor plan, as shown in Figure 17.



a) **b)**

Fig.17: a) typical suite floor plan; b) second floor architectural plan. After *Porter et al.* [2]

The Instrumented Building Loss Analyzer (IBLA) is used to evaluate damage and losses for the structural components, for all types of analyses. The algorithm allows using multiple sets of peak SDR as input for the analysis and it defines the unit repair costs based on the overall number of damaged components in the building. Therefore, in the case of single frame analysis, it aggregates structural damage of each frame before performing the loss analysis. Non-structural components damage and losses are defined through the commercial software SP3 by Haselton Baker Risk Group.

Figure 18 and 19 present the results of the three type of analysis in terms of capital repair costs, normalized with respect to building construction cost, and also an overview of the direct loss analyses results for three earthquakes (Landers, Big Bear, and Northridge) of the 1990s. Figure 18 shows the four largest contributors to loss (structural components; walls, partitions, and external shell; elevators and stairs; and furniture). The next largest loss items were: 1) Ceiling; 2) Piping; 3) Heating, HVAV and VAV; 4) Electrical & fire protection; and 5) Concrete tile roof. Loss for these non-structural components were too small to be identified in Figure 18

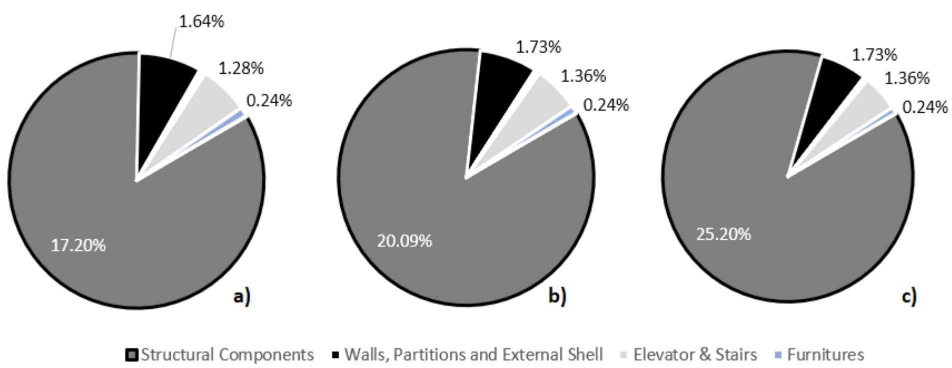


Fig.18: Direct loss analyses results, a) As Is analysis; b) GC analysis; and c) SF analysis

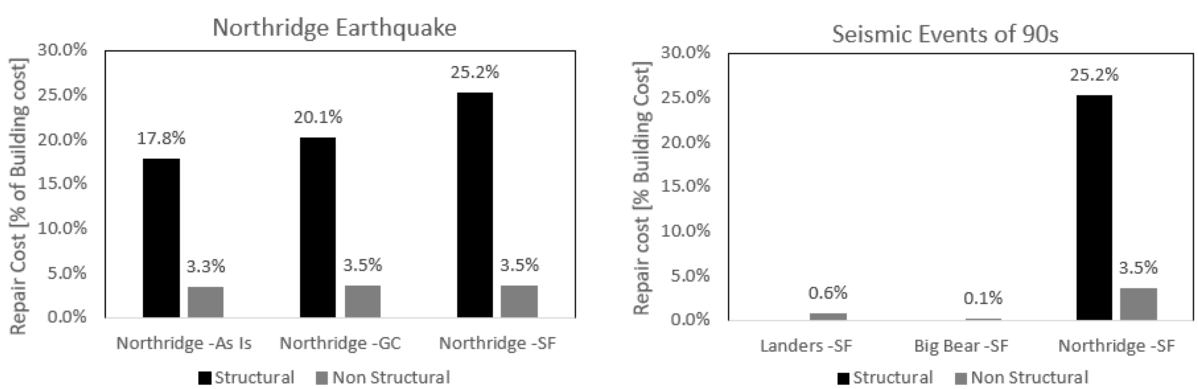


Fig.19: Summary of direct loss analyses results and loss evaluation for the Landers, Big Bear, and Northridge earthquakes

Results comparison with observed damage

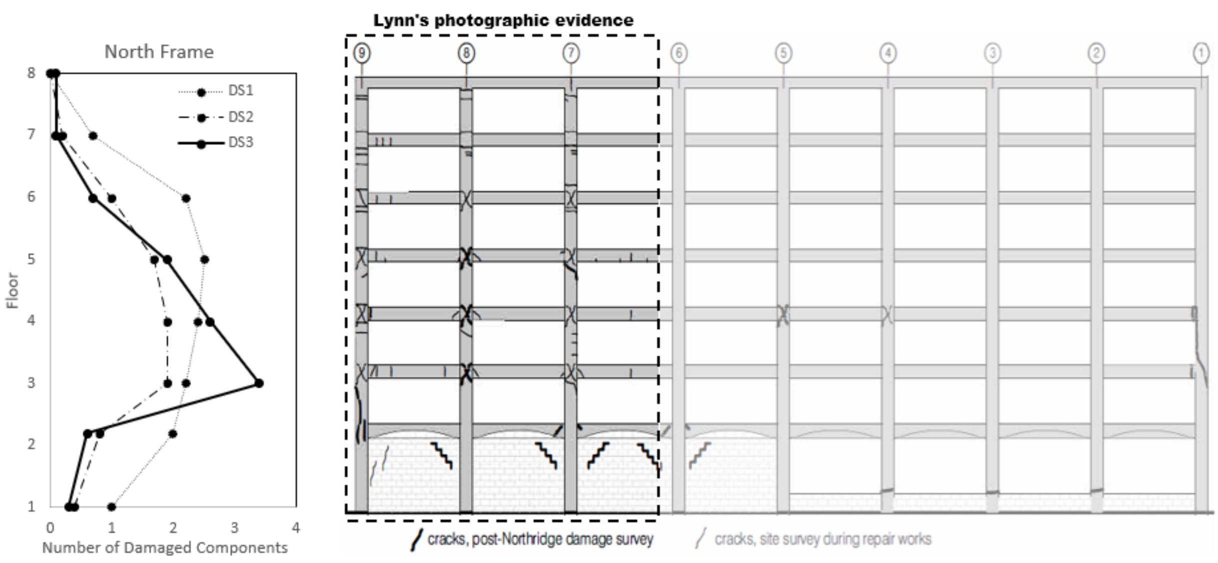
Figure 20 shows a site survey of the building during post-Northridge repair works, where significant use of epoxy injection is apparent [11]. Evidence from this photograph is used to update an existing sketch of observed damage for the north frame [17], which does not sufficiently identify the spread of damage in the reinforced concrete beams, columns and joints.

Estimated structural damage for the north frame with the single frame (**SF)** analysis is compared with the above-mentioned damage sketch. The direct loss assessment for structural components seems to be reasonably representing the extent of damage for the north frame, as shown in Figure 21, where the different calculated damage states are identified. For instance, the number of components in damage state DS 3, between third to fifth floor, is consistent with observed damage to beam-column joints and to the frame external columns. Similarly, the observed spread of elements cracking seems to be reasonably identified by the number of elements in damage state DS 1.

Regarding non-structural components, several sources including the *Earthquake Engineering Field Investigation Team* [5] have reported limited damage in the aftermath of the Northridge earthquake. Damage was limited to: 1) minor cracking to external and internal partition walls; 2) tilted furniture; 3) minor damage to elevators (loss of hydraulic fluid); and 4) damage to doors. The single frame loss analysis successfully identified these components as the major non-structural contributors to losses (Figure 18c), except for doors, which were not modeled. For all three of the 1990s earthquakes, namely Landers, Big Bear, and Northridge, non-structural damage calculations seem to be slightly overestimated relative to actual damage. For Big Bear and Landers, damage to non-structural components was not reported, while for Northridge the minor observed damage to elevators and partitions does not seem enough to justify the calculated losses.

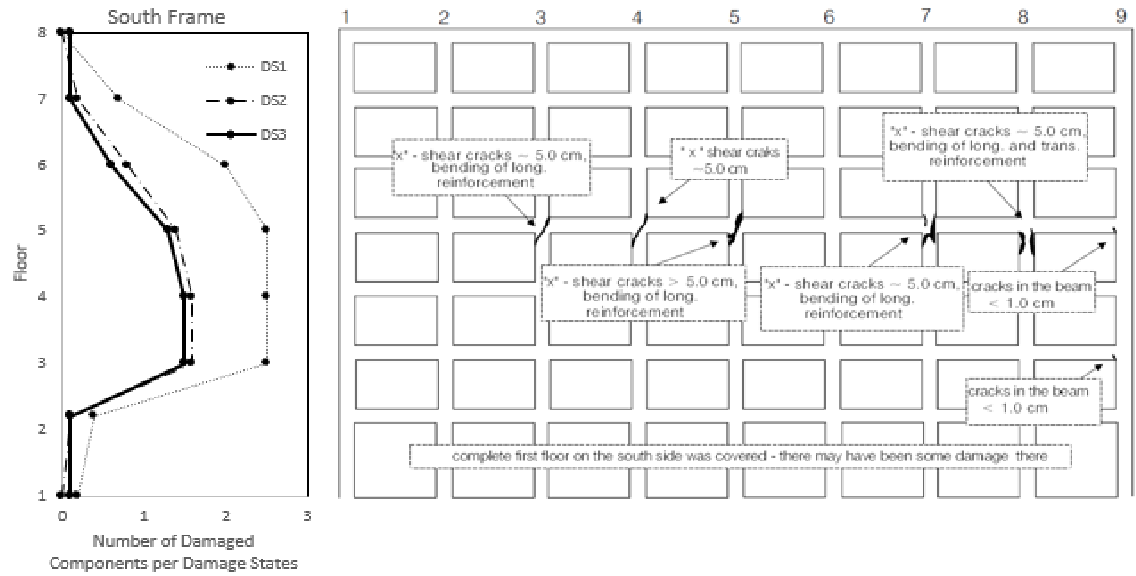
Regarding downtime, the single frame (**SF**) analysis estimates a maximum repair time of 316 days, under the hypothesis that one floor is repaired at a time (FEMA P58). This result seems to underestimate the actual downtime considering that repair works was still undergoing during July 1995, as reported by *Lynn et al.* [11] and shown in Figure 20. The disagreement might be due to impeding factors, such as contracting and permitting, that are currently not considered in FEMA P58 methodology.

Fig.20: Building site survey during repair works - July 1995, after *Lynn et al.* [11]



- **DS1**: Elements exhibit residual crack widths > 0.06 in. No significant spalling. No fracture or buckling of reinforcing.
- **DS2**: Spalling of cover concrete exposes column transverse reinforcement but not longitudinal reinforcement.
- **DS3:** Spalling of column cover concrete exposes a significant length of column longitudinal reinforcement. Crushing of column core concrete may occur. Fracture or buckling of reinforcing requiring replacement may occur.

Fig.21: Comparison of calculated damage with updated post-Northridge damage survey (north frame)



- **DS1**: Elements exhibit residual crack widths > 0.06 in. No significant spalling. No fracture or buckling of reinforcing.
- **DS2**: Spalling of cover concrete exposes column transverse reinforcement but not longitudinal reinforcement.
- **DS3**: Spalling of column cover concrete exposes a significant length of column longitudinal reinforcement. Crushing of column core concrete may occur. Fracture or buckling of reinforcing requiring replacement may occur.

Fig.22: Comparison of calculated damage with existing post-Northridge damage survey (south frame)

Second Case-Study Building: The Imperial County Services Building

Building description

The second building selected for study is an administration building [CSMIP Station #01260], shown in Figure 23. The building has a six-story reinforced concrete structure with a floor plan of approximately 136 by 85 feet of office space. The building was designed in 1967, according to the Uniform Building Code (UBC 1967) and constructed in 1969. It was characterized by a lateral-force-resisting system made of four moment-resisting frames in the longitudinal direction and structural walls in the transverse direction, with the west wall being a coupled wall. Gravity loads were sustained by a slab-joist system spanning in the transverse direction and transferred to the ground through the longitudinal frames. The building was instrumented and its response to the Imperial Valley earthquake was extensively studied [8, 9, 15, 16].

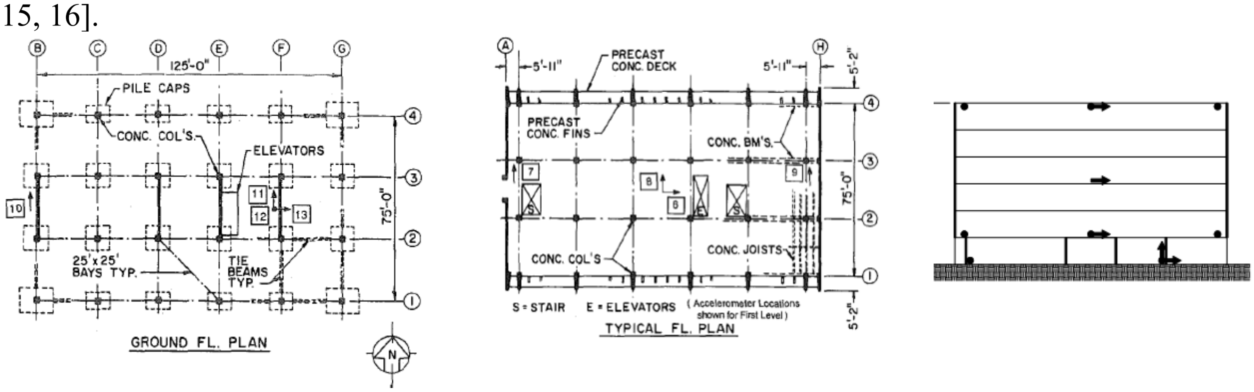


Fig.23: Floor plan and accelerometer locations, after *Sozen et al.* [8]

The building was characterized by significant irregularities in plan and over the building height. At the ground floor, the shear walls were unevenly distributed, likely fostering a building torsional response. Along the building height the shear walls were also discontinuous, as shown in Figure 24. Another irregularity consisted of a column recess provided at the base of all first-story columns, which is also shown in Figure 24. The recess produced a reduced section and required offset column longitudinal reinforcement within a lightly confined length of the column.

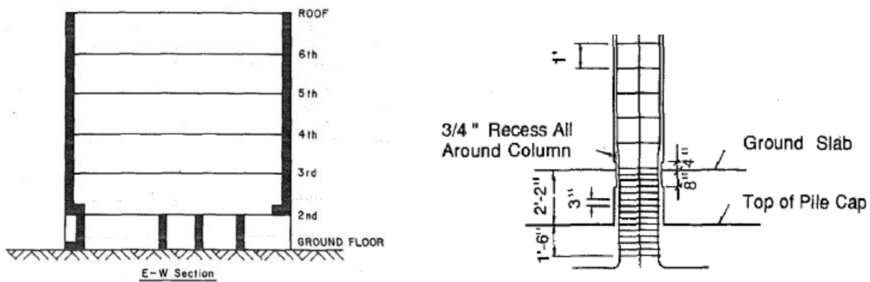


Fig.24: Building irregularities, after *Sozen et al.* [8]

The building suffered extensive structural damage during the 1979 Imperial Valley earthquake, with failure and significant shortening of the east end columns that caused a partial collapse of the same end of the building, as shown in Figure 25. Due to severity of damage the building was later demolished. As reported by *Whitaker et al.* [1], an estimate was made regarding the construction cost of a new building of the same size and design to replace the damaged building, which amounted to $ 6.8 million. This value, in 1980 dollars, is assumed as a reference for the following loss analyses.

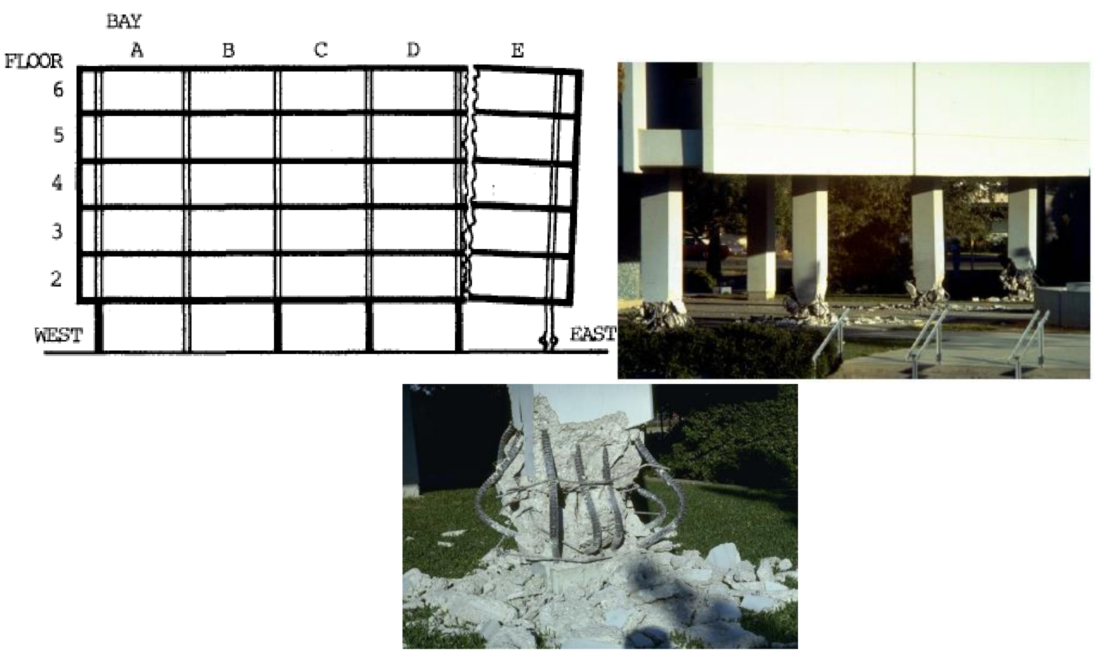


Fig.25: Imperial Valley structural damage to the East end of the structure, *after Whitaker* [1] *and NISEE*

Preliminary direct loss analysis results and comparison with observed damage

The building response to the Imperial Valley earthquake is studied considering as input for the direct loss analysis both the peak floor accelerations and the peak story drift ratios at the geometric center of the building (**GC** analysis) and at the location of each moment frame and shear wall (**SF** analysis). Evaluation of EDPs (Figure 26) at instrumented and non-instrumented floors and loss analysis were performed under the same assumptions adopted for the 7-story hotel.

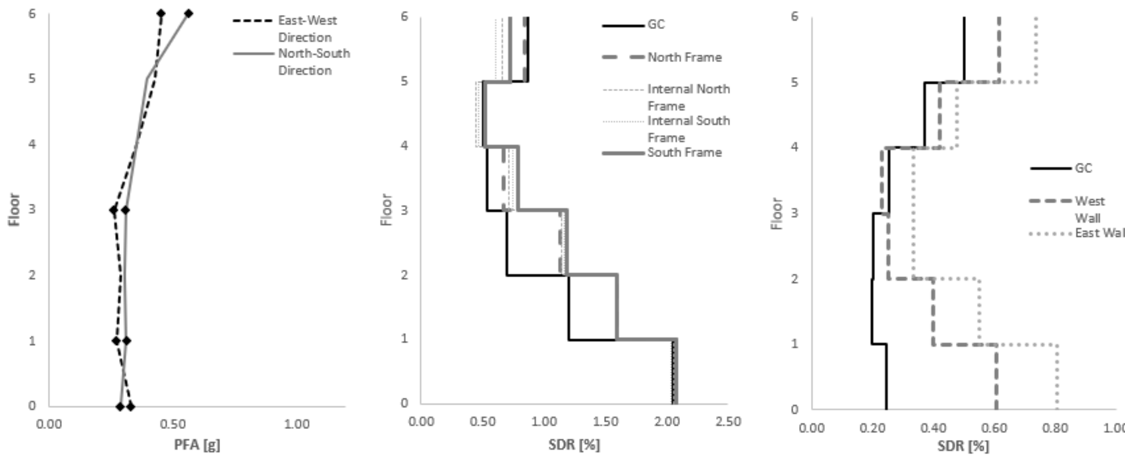


Fig.26: Calculated Engineering Demand Parameters for the building

For damage and loss analysis, the building non-structural content population was defined using FEMA P58 “Normative Quantitative Estimation Tool”, while structural components were defined based on specific knowledge of the structural system. Preliminary results, not considering interpolation errors, are presented comparing damage predictions through **SF** analysis to post-earthquake damage observations [19]. The evaluation of damage along the structural height is performed through the Matlab routine IBLA.

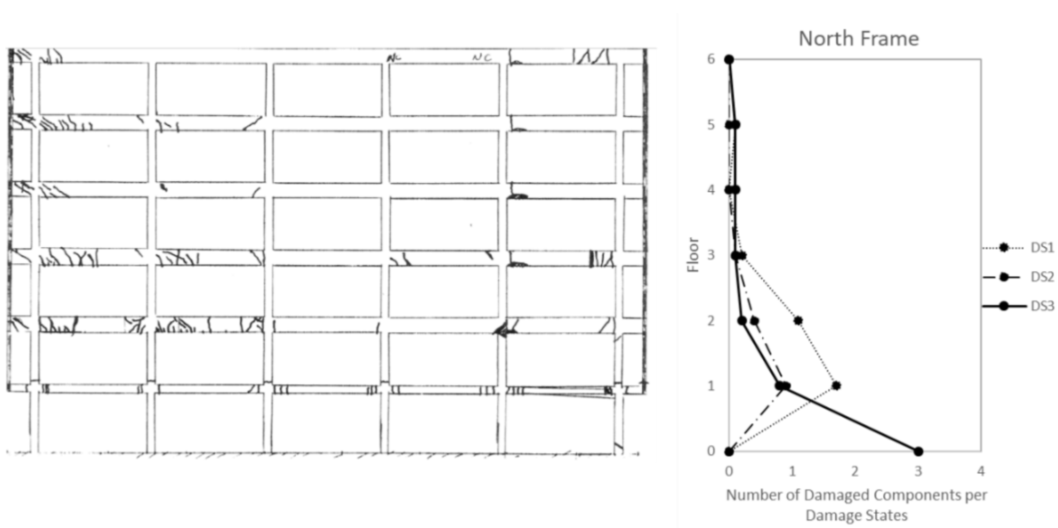


Fig.27: Calculated vs. observed damage to the building north frame

The comparison of calculated and observed damage shows a considerable underestimation of the damage distribution for the north frame. It is to be noted that damage to the east bay beam (right side of the frame in Figure 27), induced by a specific failure mechanism which is the shortening of the columns, is not represented through the FEMA P58 fragility functions, as they only consider peak horizontal story drift ratio, not vertical drift due to column collapse. Notwithstanding this consideration, the direct damage analysis is not identifying any damage above the second floor. Comparison of calculated and observed damage of the other frames and walls shows a similar underestimating trend. We continue to study the discrepancy between calculated and observed damage at the time of this writing.

Figure 28 compares calculated losses with a post-earthquake repair cost estimate prepared by county engineers in the aftermath of Imperial Valley Earthquake, as reported by *Whitaker et al.* [1]. The significant discrepancy is consistent with the discrepancy in results shown in Figure 27.

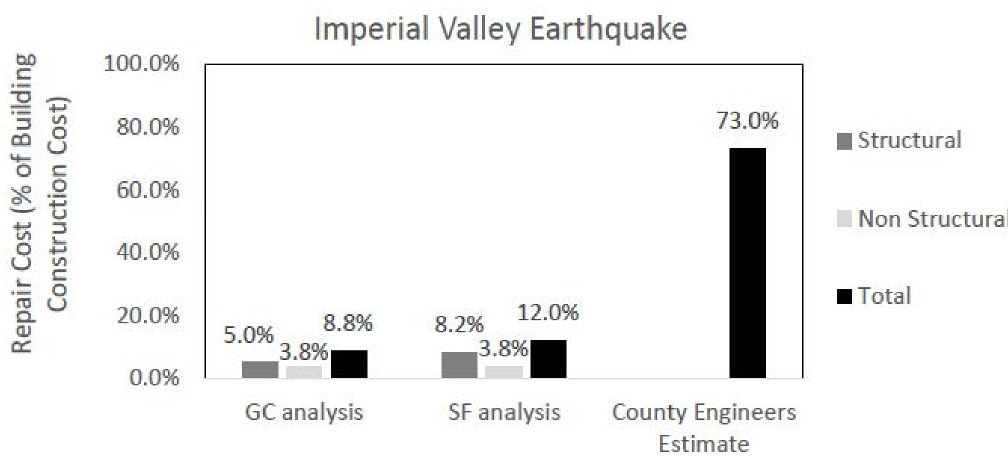


Fig.28: Predicted repair costs vs. repair cost estimate

Current work

The work presented in the previous sections represents preliminary findings to an ongoing study. For this reason, conclusions are not presented herein. Instead a brief description of the current work and anticipated results is presented.

We continue work on the archetype buildings database such that we will be able to draw more general conclusions about the ability of different interpolation methods to reconstruct unrecorded EDPs in an instrumented building. The prediction error for different interpolation techniques is only tested with respect to absolute peak story drift ratio and peak floor acceleration, which are the components of interest for loss analysis. The goal of this archetype study is to characterize interpolation prediction uncertainty that will be then used to define probability distributions of unrecorded EDPs. Once the archetypes study is completed, the direct loss analyses of the buildings presented in this paper will be re-evaluated through a probabilistic approach.

A third case-study building, the Sylmar County Hospital [CSMIP Station #: 24514], that suffered significant non-structural damage during the Northridge event, is currently being analyzed using the direct loss assessment methods described above. The main intent is to evaluate the ability to estimate non-structural components performance and to make an observation of the reasonableness of output from FEMA P58 relative to acceleration-sensitive components.

In parallel to the above-mentioned studies, an industry partner, Interprogetti Engineering Consulting, is currently pursuing the analysis of the first two buildings through the entire FEMA P58 probabilistic loss assessment methodology. Results from the direct loss analysis and the application of the entire FEMA P58 will be compared and their ability to estimate damage will be tested against earthquake damage observations. In particular, the reasonableness of direct loss assessment results is going to be investigated with the intent of evaluating the feasibility of implementing a rapid post-earthquake loss estimation methodology.

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