Fully Integrated Performance Based Design and Retrofit of Existing Buildings: A Case Study

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ABSTRACT: The performance based assessment and retrofit of an iconic, 14 storey, historic building in Oakland, California is presented.

The nonlinear dynamic analysis procedure was used in conjunction with FEMA-356 to assess the performance of this early 1900's, concrete encased riveted steel frame structure.

A 'fully integrated' approach was implemented, whereby the calculation of acceptability criteria and assessment of each element is performed at every time step; the performance assessment fully integrated with the analysis engine.

This method overcomes the conservatism inherent in the typical approach, which calculates the performance criteria for certain elements at the end of an analysis using peak results.

In the present case study, the use of the fully integrated approach made it possible to develop a retrofit scheme that met target performance objectives specified by the local authorities without compromising the valuable historic fabric of the building.

1 INTRODUCTION

The advantages of using a performance based design approach for the seismic assessment of existing buildings are well documented [Chambers and Kelly, 2004, Kelly, 2003]. When coupled with the nonlinear dynamic analysis procedure (NDP), this approach can be used to develop highly refined retrofit solutions for buildings with substandard seismic resistance. This is particularly advantageous in the case of buildings with historic significance, where there is a desire to minimise the impact on the historic fabric of the building of any required strengthening works.

This paper describes a recent example where this procedure was used in conjunction with the guidelines contained in FEMA-356 [FEMA, 2000]. A 'fully integrated' approach was implemented, whereby the calculation of FEMA acceptability criteria and assessment of each element is performed at every time step; the performance assessment fully integrated with the analysis engine.

Such an approach overcomes the inherent conservatism of the typical approach, whereby the performance assessment is conducted post-analysis, using the enveloped results from the analysis.

In the present case study, this made it possible to more effectively account for the existing structure's inherent capacity to resist lateral loads. The result being a retrofit solution that is considerably less intrusive and more economical than alternatives previously proposed for the building.

In addition to describing this integrated procedure, this paper highlights a number of the interesting modelling challenges encountered during the assessment.

1.1 Description of structure

The Cathedral Building, pictured in Figure 1, is an iconic 14 storey building located in Oakland, California. It was constructed around 1913.

The structural system of the building is a two-way, concrete encased, steel frame. Seven single-bay moment frames provide the seismic resistance in the transverse direction. In the longitudinal direction, two perimeter six-bay moment frames resist seismic load.

The beams are connected to the columns with clip angles top and bottom, which are in turn connected to the girder and column flanges with rivets. This type of steel frame falls into the FEMA category of a 'Partially Restrained' moment frame.

Figure 1: Cathedral Building

The owner of the building wished to convert it from offices into apartments. As a consequence of the change of use, Oakland City required its seismic performance to be assessed.

A number of critical weaknesses were identified in a preliminary seismic assessment. The main concern was a severe soft storey condition resulting from the large inter-storey height at the ground floor and an incomplete mezzanine level. It was apparent that this deficiency would severely compromise the seismic resistance of the existing structure.

A preliminary retrofit scheme was developed to overcome this issue prior to commencing the detailed assessment. This included the proposed provision of two new concrete moment resisting frames in the longitudinal direction, and two new steel eccentrically braced frames in the transverse direction, both extending up to the first floor. Transverse basement walls were also proposed to transfer the loads from the eccentrically braced frames to new foundations incorporating micropiles.

While retrofit solutions had been proposed for the building previously, these required the provision of new moment frames extending up the full height of the building in order to demonstrate satisfactory performance. The intrusiveness and cost of these solutions made them unattractive and impractical to implement.

The aim of the present assessment was to demonstrate that with the relatively minor measures in place to overcome the most severe deficiencies, the structure would have satisfactory seismic performance.

2 EVALUATION METHODOLOGY

The performance based seismic assessment and retrofit of the Cathedral Building was conducted in accordance with FEMA-356 using the nonlinear dynamic time history analysis method. FEMA-356 sets a Basic Safety Objective (BSO) which targets specified performance levels under two definitions of seismic hazard:

1. Life Safety (LS) performance level at an earthquake with a probability of occurrence of 10% in 50 years (corresponding to a 475 year return period). This is termed the Basic Safety Earthquake 1 (BSE-1).
2. Collapse Prevention (CP) performance level at an earthquake with a probability of occurrence of 2% in 50 years (corresponding to a 2,475 year return period). This is termed the Basic Safety Earthquake 2 (BSE-2).

In the case of the Cathedral Building it was deemed essential that the Life Safety performance objective be met. However, it was agreed with the local authorities that the Collapse Prevention performance level could be relaxed slightly given that the building was of historic value, and there was a desire to avoid overly intrusive retrofit solutions. In California, A target level of 75% of code is generally considered acceptable when assessing existing buildings relative to the Uniform Building Code. The acceptance of a higher level of risk for existing buildings relative to new buildings is also a feature of New Zealand guidelines on seismic assessment [NZSEE, 2005].

A probabilistic seismic hazard analysis of the Cathedral building site was conducted in accordance with the provisions of FEMA-356. This was used to determine the site specific response spectra for the BSE-1 and BSE-2 earthquakes given in Figure 2. Following this, a set of seven two-component site specific acceleration time histories were developed for use with the NDP. These records were scaled in the frequency domain to match the target BSE-1 and BSE-2 spectra.

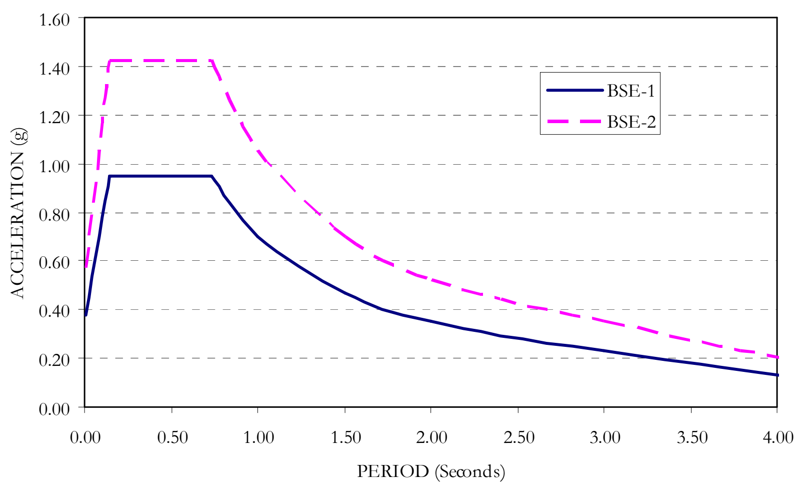

Figure 2: BSE-1 and BSE-2 5% Damped Design Spectra

3 CONSTRUCTION OF THE MODEL

The finite element model (Figure 3) was prepared using geometry obtained from a reasonably comprehensive set of existing drawings of the building. Grades of materials used in the construction of the building were not defined on the drawings so a number of representative samples were taken and tested to determine appropriate material properties for input into the model including average and lower bound steel strengths. For elements that were not sampled, default strengths as specified in FEMA-356 for construction of this vintage were assumed.

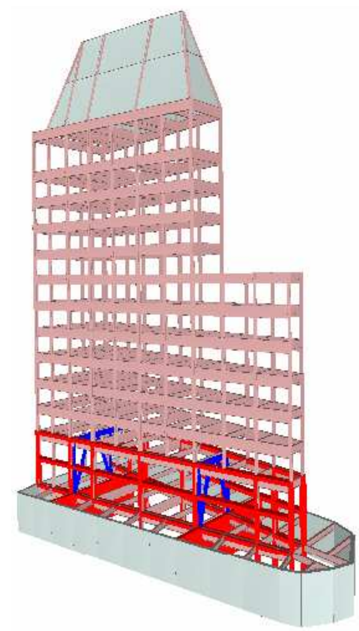

Figure 3: Cathedral Building Model. New strengthening elements shown highlighted.

3.1 Steel beams

The steel beams are typically fabricated from four angles and a web plate, riveted together to form an I-section. The stiffness of the concrete encasement of the girders was incorporated in the model as recommended in FEMA-356. With this procedure, the stiffness of the girders is factored up to account for the area of concrete between the flanges of the beam. No strength increase due to the encasement was included as there was no reliable mechanism for ensuring the beam and the concrete act compositely.

FEMA-356 contains guidelines for modelling the nonlinear behaviour of the 'Partially Restrained' riveted steel beam-column joints. These consider the hierarchy of failure of the various components that make up the joint, and base the overall capacity of the joint on that of the critical element.

The beams are spliced close to the column face to a haunched plate. This plate is riveted to the column flange/web using the clip angles. The capacity of the joint was evaluated for each of the possible failure mechanisms including the beam capacity, the splice capacity (including shear failure of web plate, tension failure of angles, shear failure of rivets) and failure of the haunched plate (including tension failure of rivets connecting angles to the column, shear failure of rivets connecting angles to haunch plate and bending failure of angles). Shear failure of the rivets connecting the haunch plate to the column was typically found to govern the behaviour of the connections.

Joint behaviour was incorporated in the overall building model using lumped plasticity models for the beams rather than by explicitly modelling the joints with torsional springs. With this method, the effective stiffness of the beams is calculated based on beam bay dimensions and an assumed connection rotation

FEMA-356 does not provide explicit guidance on appropriate moment-rotation properties for the minor axis joints in the transverse direction, where the beams frame into the webs of the columns (refer Figure 4). With this arrangement, a moment at the end of the beam will impose a twist on the column web before it is transferred to the column flanges. This adds flexibility to the joint and also provides an additional inelastic mechanism (web yield) which can affect the overall response of the joint.

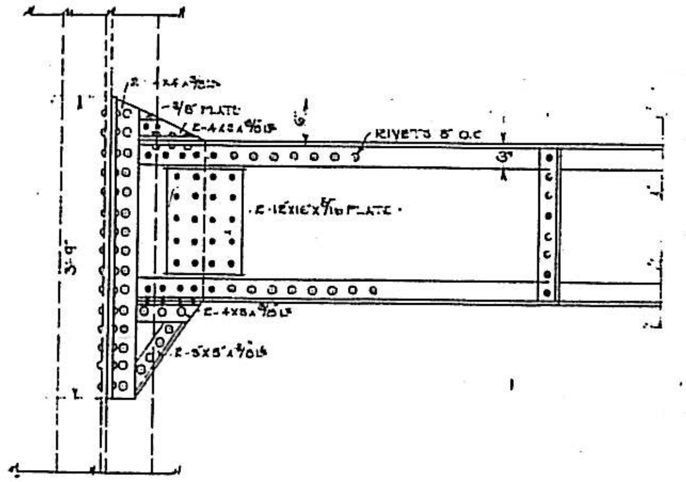

Figure 4: Minor Axis Beam-Column Connection

In order to determine the nature of this effect, a series of finite element models of typical minor axis joints were developed in LUSAS. Triangular thick shell elements with quadratic formulation were used to construct these models and nonlinearity was incorporated by way of Von-Mises yield criteria. The effect of the concrete encasement in preventing inward movement of the column flanges was incorporated in the model by way of compression only strut elements.

These models were used to determine nonlinear moment-rotation curves representing the isolated effect of web flexibility. The moment-rotation curves were then incorporated in the definition of the overall moment-rotation of the minor axis joints determined using the FEMA procedure.

3.2 Steel columns

The existing column sections are typically hot rolled steel H-sections, concrete encased for fire protection. A lightweight mesh exists to hold the concrete encasement together but was not considered sufficient to confine the concrete. As a result, the concrete encasement was not considered to add to the strength of the columns. The concrete was however considered to add to the flexural stiffness of the columns in a similar fashion to the beams.

The concrete encasement was also considered to prevent local buckling of the steel sections. As a result all column sections were considered compact. The concrete encasement was not considered to have any contribution to resisting lateral buckling of the columns.

In accordance with FEMA-356, the axial compression capacity of the columns is based on the lower bound yield strength of the steel, with the flexural capacity based on the average yield strength.

Columns were represented by a biaxial flexural element. An interaction diagram is calculated based on nominal material strengths. The interaction between bending moments and axial load is a function of the axial load and is defined by the following four equations (FEMA equations 5-10 to 5-13):

$$\left[\frac{P_c}{2P_{uc}} + \frac{M_x}{M_{ux}} + \frac{M_z}{M_{uz}}\right] \leq 1.0 \quad for\ \frac{P_c}{P_{uc}} \leq 0.2 \tag{2}$$

$$\frac{P_c}{P_{uc}} + \frac{8}{9}\left[\frac{M_x}{M_{ux}} + \frac{M_z}{M_{uz}}\right] \leq 1.0 \quad for\ 0.2 \leq \frac{P_c}{P_{uc}} \leq 0.5 \tag{3}$$

$$\left[\frac{P_c}{P_{uc}} + \frac{M_x}{M_{ux}} + \frac{M_z}{M_{uz}}\right] \leq 1.0 \quad for\ 0.5 \leq \frac{P_c}{P_{uc}} \tag{4}$$

$$\left[\frac{P_t}{P_{ut}} + \frac{M_x}{M_{ux}} + \frac{M_z}{M_{uz}}\right] \leq 1.0 \quad for\,TensionLoads \tag{5}$$

M_y, M_z, P_c and P_t denote bending moments about the element y and z axes, axial compression and axial tension respectively. Subscript *u* denotes ultimate. P_{ut} and P_{uc} are axial ultimate strengths in tension and compression. As for the beams, a bilinear strain hardening yield function was used.

Steel columns with axial compressive forces exceeding 50% of the lower-bound compressive strength are considered force-controlled for both axial load and flexure. This requires that columns remain elastic under maximum seismic load.

3.3 Performance criteria

FEMA-356 defines acceptable plastic deformation levels for the beams and columns for the various performance objectives, as listed in Table 1. For beams the plastic rotations are compared directly with the output plastic rotations in each element from the analysis. For columns, the allowable plastic rotations are a function of the yield rotation, θ_y, which is in turn a function of the axial load ratio as described by the following equation (FEMA equation 5-2):

$$\theta_y = \frac{ZF_y l_c}{6EI_c}(1 - \frac{P}{P_y}) \tag{6}$$

With this formulation, the allowable deformations reduce as the axial load in the column increases.

Table 1: Plastic Rotation Limits for Primary Components in Steel Frame with Partial Restraint

		Allowable Plastic Rotation	
Component	**Conditions**	**LS**	**CP**
Steel Girders	Partially restrained moment connection (clip angle governed by rivet shear failure)	*0.020*	*0.030*
Steel Columns	$P/P_{CL} < 0.2$	$6\theta_y$	$8\theta_y$
	$0.2 < P/P_{CL} < 0.5$	$8(1\text{-}1.7P/P_{CL})\theta_y$	$11(1\text{-}1.7P/P_{CL})\theta_y$

4 AN INTEGRATED APPROACH

The coupling of modelling parameters and acceptance criteria to the axial load in the column makes it difficult to apply FEMA-356 effectively unless the relationships are integrated within the analysis engine. If one attempts to use a generic analysis package to conduct the time history analyses and then assess the performance of elements by comparing FEMA limits to peak results obtained during the analysis, one will find there is a high degree of inherent conservatism in this approach. For example, unless time histories of element forces are recorded for each element, calculation of the acceptable plastic rotations in the columns would have to be performed using the envelope axial loads from the analysis, which may not have in fact occurred simultaneously with the peak rotations to which they are being compared.

In the case of the Cathedral Building, attempts to use the envelope approach resulted in the prediction of unsatisfactory performance, even at relatively low lateral load levels. To get around this issue, an integrated approach was developed whereby the FEMA relationships described above were integrated with the ANSRII analysis engine [Mondkar and Powell, 1979]. Modelling parameters and acceptability criteria were then able to be calculated at every time step as a function of the current axial load, P.

The procedure is comprised of the following steps:

1. Define a baseline yield rotation about each axis assuming $P/P_{ye} = 0.20$.
2. At each step, calculate the yield function using equations (2) to (4) depending on the ratio P/P_{ye}. If the yield function is > 1.0, adjust the stiffness to incorporate the plastic hinge.
3. Calculate the modified yield rotation at this step using equation (6) and the current axial load. Adjust the acceptance criteria (as a plastic rotation limit) and calculate the ratio of demand to capacity. Repeat for minor axis bending.
4. If the axial load ratio P/P_{ye} is greater than 0.50, the column is force controlled and the acceptance criteria for plastic rotations is set to zero.

This procedure is considered to be as rigorous an implementation of FEMA-356 requirements as is practical for nonlinear analysis. With the implementation of this procedure, the undue conservatism inherent in the envelope approach was avoided and it became possible to demonstrate that the building could achieve acceptable performance.

5 BUILDING PERFORMANCE

With the provision of the new concrete moment resisting frames and steel eccentrically braced frames at the lower levels, the building was demonstrated to have sufficient capacity to resist the full BSE-1 type event with regards to life safety criteria. Drifts of around 2.0% were predicted at this level of load.

During a number of the BSE-2 level analyses, the building underwent a progressive collapse as a result of excessive displacements and P-Delta. The BSE-2 level event was subsequently scaled down until satisfactory performance of the building with regard to collapse prevention criteria was achieved.

At 85% of BSE-2, the majority of the elements performed within acceptable limits for collapse prevention and all the analysis runs completed without signs of collapse. At this level of load, drifts of around 2.7% are anticipated.

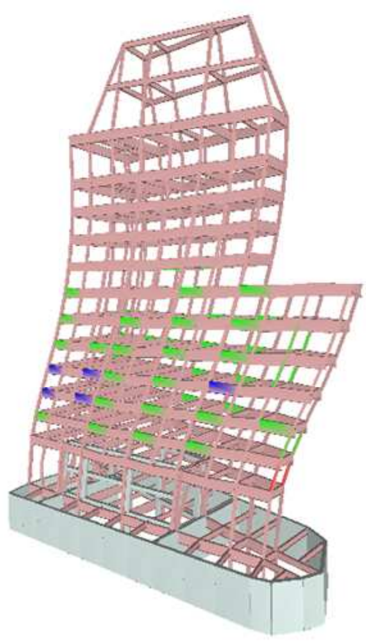

Figure 5: Building Performance at 85% BSE-2.

A number of existing steel column elements did however exceed the CP limits at this level of load due to the simultaneous occurrence of small plastic rotations and axial load ratios exceeding 0.5. It was found that in these cases, the deficiency could be easily overcome by providing a new post in parallel with the existing column. This effectively reduces the axial load ratio in the existing elements so that they are no longer force controlled.

With these measures in place, it was demonstrated that the Cathedral Building can likely resist an earthquake equivalent to 85% of BSE-2 without collapse. This is approximately equivalent to a risk of collapse under full BSE-2 of 4% in 50 years.

Figure 5 shows the exaggerated deformed shape of the structure at the time of maximum displacement for one of the analysis runs at for 85% BSE-2. The colour code used to indicate damage levels is:

1. Green indicates deformations beyond IO but less than LS.
2. Blue indicates deformations beyond LS but less than CP.
3. Red indicates deformations beyond the CP limit.

Due to the historic nature of the building and the impracticalities of strengthening to meet the full BSE-2 criteria, agreement was reached with Oakland City that the performance of the building with the proposed strengthening works was satisfactory.

6 CONCLUSIONS

This paper has presented a recent application of a 'fully integrated' performance based design procedure, whereby the FEMA modelling and assessment relationships are incorporated directly in the nonlinear time history analysis computational engine and calculated at every time step of the analysis.

The dependence of a number of FEMA relationships on the axial load ratio in steel columns makes this integrated approach necessary in order to avoid a the seismic assessment being unduly conservative.

The case study has shown that this procedure is practical to implement in a design office and is very useful in helping to quantify and communicate the relative seismic risk of existing buildings. This in turn helps all involved to make pragmatic, informed decisions about the acceptability of that risk.

This enabled a retrofit solution to be developed that is comparatively economic and unintrusive, assisting to preserve the valuable historic fabric of the building.

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