Objective

The objective of this first phase of the project is to get familiar with the building structure to be retrofitted with various energy-dissipating devices.

Description of Building Structure

The six-storey building structure to be retrofitted was studied by Tsai and Popov (1988) and was modified by Hall (1995). As shown in Fig. 1, the building is rectangular in shape and is braced in the North-South direction by two exterior moment-resisting frames. The design complies with the 1994 UBC code requirements (ICBO 1994) for a building located in Zone 4 on soil type S2. Design gravity loads include the roof dead load (3.8 kPa), the floor dead load (4.5 kPa), the roof live load (1.0 kPa), the floor live load (3.8 kPa), and the weight of the exterior cladding (1.7 kPa). Wind loads are based assuming a basic wind speed of 113 km/h and an exposure type B. The steel grade is assumed to be A36 (nominal $F_y=290$ MPa) for all members.

Figure 1. Building Structure to be Retrofitted.
Modelling Assumptions

All seismic/dynamic analyses are performed using the nonlinear dynamic analysis computer program RUAUMOKO (Carr 1998). A data file, representing a two-dimensional model of the building, has been distributed in class.¹

Only half of the building is modelled, as the structure is symmetrical. As shown in Fig. 1, the model then includes only one exterior frame, together with one gravity column that represents all interior frame columns. The total gravity loads acting on the interior columns are applied to the gravity column in the model and both the gravity column and the exterior frame are constrained to experience the same lateral deformation at each floor.

Only the bare steel frame is included in the analyses, i.e., the slab participation as a composite beam is not included. The inelastic response is concentrated in plastic hinges that could form at both ends of the frame members. These plastic hinges are assigned a bi-linear hysteretic behaviour with a curvature strain-hardening ratio of 0.02 (Fig. 3), and their length is set equal to 90% of the associated member depth. The plastic resistance at the hinges is based on expected yield strength of 290 MPa. An axial load-moment interaction is considered for the columns of the structure. Rigid-end offsets are specified at the end of the frame members to account for the actual size of the members at the joints. The panel zones of the beam-column connections are assumed to be stiff and strong enough to avoid any panel shear deformation and yielding under strong earthquakes. This assumption represents the most critical condition for the inelastic curvature demand on the welded beam-to-column joints, as all the hysteretic energy must be dissipated only through plastic hinging in the beams and the columns. The columns are fixed at the ground level, except the gravity column that is assumed pinned at the base and at each level.

Gravity loads acting on the frame during the earthquake are assumed equal to the roof and floor dead loads, the weight of the exterior walls, and a portion of the floor live load (0.7 kPa). P-delta effects are accounted for in the analyses, including P-delta forces generated in the interior frames. Half the weight of the building, along with a 0.5 kPa live load, is included in the reactive weights at each level. Rayleigh damping of 5% based on the first two elastic modes of vibration of the structure is assigned. All analyses are performed at a time-step increment of 0.002 s.

To capture the brittle failure of the welded beam-to-column connections, the flexural strength degradation model shown in Fig. 2 is introduced at the ends of the beam and column elements. In this model, the yield levels in the interaction diagrams may be reduced as a function of the curvature ductility in each direction. The strength degradation begins at a curvature ductility of 11.0. At a curvature ductility of 11.55, the strength reduces 1% of the yield moment. It must be noted that in RUAUMOKO it is not

¹ Alternatively, the project could be executed with the OpenSees open source platform.
possible to have the strength reduce to zero. A 1% strength is close enough to zero for engineering purposes.

Figure 2. Strength Degradation Model for Welded Beam-Colum Connections.

**Analyses Required**

The following analyses must be included in the Phase 1 of the Project:

a) **Geometry and Members Schedule**

- Draw an elevation view of the analyzed frame indicating the positions of all nodes and members.
- Draw a Table indicating for each member the following properties: member’s depth, cross-sectional area, moment of inertia around the bending axis, yields bending moment, and yield axial force.
- Draw for each column member, a graph of the axial load-moment interaction diagram.

b) **Curvature Ductility Capacity**

For this project, the failure criterion assumed for all steel beam and column elements is based on a plastic end rotation limit of 0.03 radians. Consequently, the bi-linear moment-curvature relationship shown in Fig. 3 is adopted for all beam and column ends. In this
figure, 0.2 $M_p$ represents the moment increase due to strain hardening whereas $\phi_p$ is the plastic curvature corresponding to failure of the section.

![Bi-Linear Moment-Curvature Model](image)

For each member, verify that the plastic curvature, $\phi_p$, indicated in Fig. 3 corresponds to the plastic rotation limit $\theta_p = 0.03$ rad.

For each member, determine the curvature ductility capacity at failure ($\theta_p = 0.03$ rad) and validate the strength degradation model shown in Fig. 2.

c) **Dynamic Characteristics**

- Draw a Table showing the first 5 periods of vibration of the building structure.

- For each of these 5 periods of vibration, draw the corresponding mode shape. Indicate the numerical values corresponding to the lateral displacement of each floor level.

d) **Pushover Analysis**

In a pushover analysis, a lateral monotonic load is applied to a structure until the ultimate load is approached. This static analysis, much easier to perform than a dynamic analysis, allows the evaluation of the elastic and inelastic responses of the structure under lateral loads. The results of a pushover analysis depend on the lateral load distribution considered.
Using the procedure described in page 15 of the RUAUMOKO Examples Manual (at the end of the RUAUMOKO Theory Manual), perform a pushover analysis on the structure. A proper seismic loading distribution must be selected and justified.

Present the results of the pushover analysis in graphical form, similar to that of page 17 of the RUAUMOKO Examples Manual, indicating the variation of the base shear with the top floor lateral displacement.

Clearly indicate on this graph the following points:

- The formation of the first plastic hinge in the beams, identify also its location;
- The formation of the first plastic hinge in the columns, identify also its location;
- The first failure of a beam or a column, indicate also its location.

References


