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High-performance computer-aided optimization of viscous dampers for improving the seismic performance of a tall building

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ABSTRACT

Using fluid viscous dampers (FVDs) has been demonstrated to be an effective method to improve seismic performance of new and existing buildings. In engineering applications, designs of these dampers mainly rely on trial and error, which is repetitive and labor intensive. To improve on this tedious manual process, it is beneficial to explore more formal and automated approaches that rely on recent advances in software applications for nonlinear dynamic analysis, performance-based evaluation, workflow management and the computational power of high-performance, parallel processing computers. The optimization design procedure follows the framework of Performance Based Earthquake Engineering (PBEE) and uses an automatic tool that incorporates an optimization engine and structural analysis software: Open System for Earthquake Engineering Simulation (OpenSEES). An existing 35-story steel moment frame is selected as a case-study building for verification of this procedure. The goal of the retrofit design of FVDs is to improve the building's seismic behavior to avoid collapse under a Level 2, basic safety earthquake event (BSE-2E). An objective function of building's total loss under a BSE-2E event is used and its optimal damper patterns will be proposed. The efficiency of the optimization procedure will be demonstrated and compared with the manual procedure.

1. Introduction

Fluid Viscous Dampers (FVDs) are one kind of passive energy-dissipation devices widely used in the design of structures to resist the effects of earthquakes and wind. FVDs are able to provide additional damping to reduce the overturning of a building without significantly increasing the seismic forces on existing members, making them a promising solution to accommodate the ever-increasing seismic design requirements for upgrading existing buildings [1–3].

Current design procedures for passive energy-dissipation systems generally account for the supplemental damping effects provided by these devices by modifying the design spectrum, which is able to select general damper characteristics. However, they do not prescribe specific methods to efficiently arrange dampers story-wise in a building, whereas the damper placement strategies could have a significant impact on both structural behavior and total damper cost [4]. In engineering practice, damper schemes are usually devised after a series of trials and errors. This manual approach is essentially *ad hoc,* and heavily relying on engineers' experiences. Moreover, the time-consuming process to employ sophisticated nonlinear, three-dimensional (3D) dynamic analyses and interpreter the massive amount of data would be a large impediment to gain wide applicability among practitioners.

On the other hand, a wide variety of methods have been suggested by researchers to identify optimal damper placement [5–18]. However, most of these studies focused on simplified two-dimensional (2D) models of low- and medium-rise buildings, whereas their applicability to a larger building that exhibits complex, 3D nonlinear dynamic behavior might be questionable. Most importantly, such procedures do not have an automated tool to streamline the optimal design effort that would constrain their applicability among practicing engineers. Therefore, alternative approaches by developing a numerical tool that engages the repetitive design-analysis procedure in an automated manner would be a great incentive to promote its use. These motivations become feasible nowadays with the advances and accessibility to high-performance computers and parallel processors.

In this paper, a case study building: a 35-story steel moment-resisting frame incorporating pre-Northridge welded beam-to-column connections was selected. Prior studies [19,20] based on the provisions of *Seismic Evaluation and Retrofit of Existing Buildings* [21] found that the as-built structure was unlikely to satisfy collapse prevention goals for earthquake ground motions representing Level 1 or Level 2 basic safety

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earthquake events (referred herein as BSE-1E and BSE-2E). Therefore, a two-level retrofit approach was identified to achieve the collapse prevention limit state under a BSE-2E event. While Level-1 retrofit, whereby column splice fractures were prevented and the heavy exterior cladding on the building was removed and replaced with a lightweight substitute, was found to improve the building's performance for moderate levels of shaking, they did not prevent collapse under a BSE-2E event. Consequently, FVDs was used in Level-2 retrofit to augment the measures taken in Level-1 retrofit. The focus of this study is to develop an automated tool that help identify the optimal damper arrangement schemes in the case-study building to enhance its seismic performance. Therefore, the baseline numerical model used in results presented in this paper incorporates all of the Level-1 retrofit measures. This optimization problem is formulated, and three basic ingredients for an optimization problem are selected within the framework of the Performance Based Earthquake Engineering (PBEE) [22]. One study case that used the total financial loss as the objective function is evaluated. Objective function and constraint functions are obtained based on median responses out of eleven nonlinear response history analyses (NRHA) results. The efficiency of the automated design tool is demonstrated via its comparison with a manually-designed scheme.

2. Work flow of the automated procedure

Matlab [23] is used as a mathematical tool in this study to implement the optimization procedure. The gradient-based algorithm is used to update design variables and identify an optimal solution. Given the very nonlinear nature of such a design problem, the objective function and constraint functions would be non-convex, and thus a local optimum is searched instead of the global minimum. Additionally, there are no explicit expressions for the objective function and constraints, thus their first-derivatives are estimated based on a finite difference method. The obtained gradients will be used to update the design variables for the next iteration point.

The flowchart of optimization procedure is illustrated in Fig. 1. It starts once the initial conditions are identified, including: 1). initial design variables $DV^{(1)}$; 2). the lower and upper bounds of DV; and 3). optimal criteria determining the end of automated procedure (e.g., minimal change of function values between two successive iterations, maximum number of iterations etc.). The objective function and constraint functions are evaluated at the initial point $DV^{(1)}$, and loops over several iterations. At each iteration, the functions will be evaluated at a few neighbourhood points: $DV_i^{(k)}$ $(i = 1,2,\ldots,n_k)$ around the designated design points $DV^{(k)}$ $(k = 1,2,\ldots)$, where k is the iteration sequence, n_k is the number of function evaluations at k^{th} iteration. At each $DV_i^{(k)}$ $(i = 1,2,\ldots,n_k)$, function values are obtained from NRHA results, which is conducted using the computer program: Open System for Earthquake Engineering Simulation (OpenSees) [24], and return the results to optimization loop in Matlab. Once the function values at all these points are known, the gradients of objective or constraint functions are estimated to instruct the algorithm to identify next "best" design point: $DV^{(k+1)}$. However, if the difference of design variables or function values between $DV^{(k)}$ and $DV^{(k+1)}$ is smaller than the minimal value defined in the optimality criteria, the optimization procedure will stop. Otherwise, the above procedure repeats at iteration $(k + 1)$. Alternatively, the procedure could be stopped if reaching the maximum number of iterations or function evaluations.

This procedure takes several trials at each iteration point to solve the quadratic programming subproblem, and the number of trials depends on the number of design variables and constraints [25].

3. Case-study building and ground motions used

The existing building considered is a 35-story tall steel office building located in San Francisco, California; construction began in 1968. The tower is about 56 m × 41 m in-plan and 150 m in height. The structural system consists of complete 3D moment-resisting space frames in both horizontal directions. Fig. 2 shows the building model; dimensions of typical floor height and bay widths are illustrated. Beam-to-column moment connections used typical pre-Northridge details, and the column slice details utilized only partial penetration welds. Both types of details are considered quite brittle.

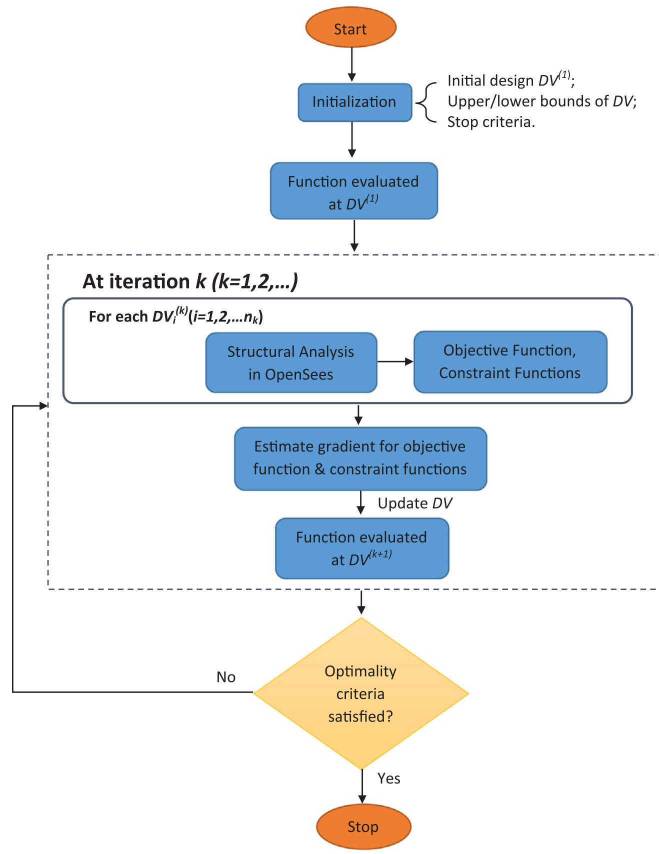

Fig. 1. Flowchart of automated optimization procedure.

To investigate the building's dynamic behavior, a 3D numerical model was generated using OpenSees; see Fig. 2. The numerical model included all above-ground framing members and for the purpose of this investigation, a fixed-base boundary condition was assumed at the ground level. Columns were modeled using displacement-based nonlinear beam–column elements with fiber sections and the Giuffré–Menegotto-Pinto material (*Steel02*) model. The analysis assumed that the partial joint penetration welds used at the column splices in the as-built structure were replaced by complete joint-penetration welds. Beams were modeled using force-based nonlinear beam–column elements with finite-length plastic hinges at both ends. The "brittle" moment-curvature relation of the beams was captured by using a hysteretic material model with constraints on the maximum and minimum rotation capacities set according to the recommendations in ASCE 41. The panel zones, perimeter precast concrete façade, and non-structural components (such as the concrete elevator core walls and moveable interior partition walls) were not explicitly modeled. The first three elastic modal periods of the building incorporating Level-1 retrofits were: 4.33 s (X-direction translation), 4.18 s (Y-direction translation), and 3.59 s (rotation); see [19] for more information regarding the building and its numerical models.

Nonlinear response history analyses were used to examine the structural response of the retrofitted building. Eleven records were selected for the automated design (Fig. 3), and their median responses are used to calculate the objective function and constraints, as permitted by ASCE 41. Detailed information of ground motion selection and scaling