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THREE-DIMENSIONAL ANALYSIS OF VISCOUS DAMPER SYSTEM FOR SEISMIC RETROFIT OF BUILDING

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ABSTRACT

Current engineering practice uses three popular methods for the seismic retrofit of buildings:

1. Conventional approach for stiffening the building
2. Base Isolation of the building
3. Addition of passive energy dissipation devices to the building

This paper investigates the use of dampers as the Energy Dissipation System for the seismic retrofit of the Opera House building in San Francisco.

The viscous damper system has a longer fundamental period of vibration than the conventional retrofit utilizing braced frames or shear wall systems. As a consequence, its seismic base shear force is substantially less than that of the conventional retrofiting. Reduced seismic base shear requires substantially less structural steel (150 tons) for the Annex retrofit, and eliminates the need to modify the existing foundation. In addition, a viscous damper system reduces the lateral deflections that a building experiences in an earthquake by dissipating a large portion of the induced energy.

A detailed three-dimensional model and dynamic non-linear analysis for the viscous damper system of the building structures using the SADSAP program is introduced in this paper.

Three pairs of modified earthquake time history records of Taft, California, Tabas, Iran and Caleta de Campos, Mexico are used in the analysis.

The summary of the analysis results and discussions are included in this paper.

KEYWORDS

Seismic retrofit; viscous damper; passive energy dissipation; seismic base shear; non-linear analysis; earthquake time history.

INTRODUCTION

The San Francisco Opera House consists of two buildings, the Main House and the Annex, as shown in Figure 1-A and 1-B. There is a seismic joint of 6 inches between the buildings.

Response spectrum analysis results show that excessive deflections of the existing seismic joint are expected that falling hazards could result from severe pounding of the Annex building against the Main House when the two existing structures are subjected to the design basis earthquake (DBE) forces with a return period of 475 years.

The steel ductile moment frame system (Figure 2-A) of the existing Annex building is expected to have 12 inch drift at the top of the building in the east-west direction, and 18 inch drift in the north-south direction for the DBE event.

One of the primary purposes of seismic retrofit for the Annex building is to reduce the maximum lateral drift of the structure to an acceptable limit of 3 inches to prevent severe pounding damages and resulting falling hazards.

A conventional approach for stiffening the Annex building for this purpose was investigated, which shows that a significant amount of structural bracing as well as substantial amount of foundation work would be required for the conventional concentric braced frame retrofit.

This report presents an analysis of an alternative solution to the conventional braced frame method, the use of a viscous dampers as the energy dissipating system.

The viscous damper system is shown in Figure 2-B and 2-C for the frames in longitudinal direction (N-S), and in Figure 2-D for the frames in transverse direction (E-W).

The analysis of the viscous damper system of the Annex Building structure was performed using the SADSAP program developed by Professor Edward L. Wilson. This program incorporates the viscous damper elements and performs three-dimensional, dynamic, non-linear analysis of the damper system. The maximum design level earthquake spectrum was converted to the equivalent time history for the damper analysis.

COMPUTER MODEL

A three-dimensional model of the isometric view of the existing Annex building without the viscous damper system is shown in Figure 2-A.

The maximum design capacity for the viscous damper used in the model is 400 kips, with a damping value C of 50 K-in/sec, and a stroke of 2 inches.

There are two frames in each of the major axes. Each frame has two dampers placed between the second floor and the third floor, and two dampers between the third floor and the fourth floor. Longitudinal frames are shown in Figure 2-B and Figure 2-C. Transverse frames are shown in Figure 2-D.

The dampers in both the longitudinal and transverse directions are supported by 12 inches square steel tubular braces, restrained in the out-of-plane direction, and disengaged from the floor framing above.

Between the first and the second floors, no dampers are used due to architectural constraints. Twelve-inch square tubular diagonal concentric braces are used between the first and the second floors to increase structural stiffness and reduce lateral drift.

STRUCTURAL ANALYSIS

The existing unstiffened Annex building was checked using SAP90 for dynamic responses. The periods of the existing building are 1.45 seconds in the transverse direction and 2.04 seconds in the longitudinal direction. As stated earlier, the maximum displacements at the top of the building are 12 inches in the transverse direction and 18 inches in the longitudinal direction when the structure is subjected to the design basis earthquake forces with a return period of 475 years.

The second step of the analysis is to analyze the modified structure with only the additional 12"x12"x1" steel braces between the fist floor and the second floor. The structure periods are shortened to 0.81 seconds in the transverse direction and 1.01 seconds in the longitudinal direction. The displacements at the top of the building modified by diagonal braces are reduced to 6 inches in the transverse direction, and 8 inches in the longitudinal direction when subjected to the same design basis earthquake forces.

Finally, the viscous dampers are added to the second model at the third and fourth floor level, which reduce the displacements at the top of the building from 6 inches to 3 inches in the transverse direction, and from 8 inches to 2.8 inches in the longitudinal direction when subjected to the required design basis earthquake forces.

The three-inch displacement at the top of the Annex building is the maximum lateral drift allowed for the structure to reduce pounding damages and falling hazards.

The design level earthquake spectrum for a return period of 475 years is shown in Figure 3-A.

In addition, three pairs of modified earthquake time history records of California, Tabas, Iran and Caleta de Campos, Mexico as shown in Figure 3-B to Figure 3-D to envelop the design basis earthquake spectrum are used for the time history analyses of the Annex building.

A comparison between the original spectrum and the calculated spectra from the modified equivalent time histories are shown in Figure 3-E.

The Peak Ground Acceleration (PGA) of the design basis earthquake is 0.46g.

SUMMARY OF THE ANALYSIS RESULTS

Displacements, Velocities and Damper Forces

The maximum time history response of floor lateral displacements, velocities and maximum damper forces are summarized in Table 4-A for the transverse direction and in Table 4-B for the longitudinal direction. The description of time history records in Table 4-A and 4-B are as follows:

AA = Taft, California BB = Tabas, Iran and CC = Caleta de Campos, Mexico

The maximum lateral displacements on the fourth floor are 3.3 inches in the transverse direction and 2.4 inches in the longitudinal direction.

The maximum axial forces of the dampers between the third floor and the fourth floor are 372 kips in the transverse direction and 335 kips in the longitudinal direction, while the damper forces between the second and the third floor are 387 kips in the transverse direction and 405 kips in the longitudinal direction.

Stress Check for All Existing Members

All new damper frame members are designed for the maximum forces per the time history responses. To check all existing members by time history responses would be very time-consuming. Instead of doing that, we have chosen to estimate the equivalent damping ratio of the system (modified by new diagonal braces but without dampers), which will result in similar maximum lateral displacements for the designed damper system. Through trial and error, we determined that the corresponding equivalent damping ratio to be approximately 30%. The existing members are then checked by the SAP90 program for the structural model with diagonal braces but without dampers and a damping ratio of 30% in the spectrum analysis.

The SAP90 results show that all existing members remain in the elastic range of the material capacity except the offset columns and their supporting W36 beams on lines D and 2, and lines D and 8, at the third floor, which need to be strengthened. Subsequently, time history responses are obtained for the members in need of strengthening.

Finally, the members are designed for the maximum forces obtained from time history responses, plus unreduced dead load, and reduced live loads.

Conclusion

From the analysis, the Annex structure with viscous dampers subjected to the three time history records enveloped by the DBE with 5% damping is shown to be approximately equivalent to the same structure without dampers subjected to the DBE with 30% damping ratio.

Comparisons between the existing structure, the conventional braced frame system and the viscous damper system are shown in Figure 5-A and 5-B.

Figure 5-A shows the spectral acceleration demands for the retrofit systems as required to limit the maximum building drift to three (3) inches. The capacity of the existing structure with a structure period of 1.45 seconds was designed for UBC-level earthquake, with a base shear of 0.04 g, which is substantially less than the DBE spectral demand of 0.5 g.

Figure 5-B shows the retrofit system capacity v.s. the corresponding spectrum. In order to limit the total building drift to three (3) inches, the conventional retrofit with braced frames has to be substantially stiffer than the original structure. The additional braces (stiffness) shortens the fundamental period of the structure significantly from 1.45 seconds to 0.5 seconds. Consequently, the corresponding spectral acceleration for the design basis earthquake increases to more than 1.0 g which requires substantial amount of foundation work in addition to new massive diagonal members.

The viscous damper system on the other hand, is not as stiff as the conventional retrofit, and has a fundamental period of 0.81 seconds. Its corresponding spectral acceleration for the design basis earthquake with an equivalent damping of 30% is only 0.40 g. As a result, almost all of the existing structural elements remain in elastic capacity range when a less substantial diagonal bracing system is added, and no foundation work is required.

From this comparative study, we have concluded that an energy dissipation system with viscous dampers can be used effectively to reduce building drift as well as base shear demands. The benefits of eliminating the need to retrofit the foundations is particularly appealing for existing structures supported by deep foundations with high groundwater table. We believe that the viscous damper system is cost effective in this application.

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Wilson, Edward L.. *The Static and Dynamic Analysis of Structures with a Limited Number of Nonlinear Elements.* The SADSAP Computer Program.

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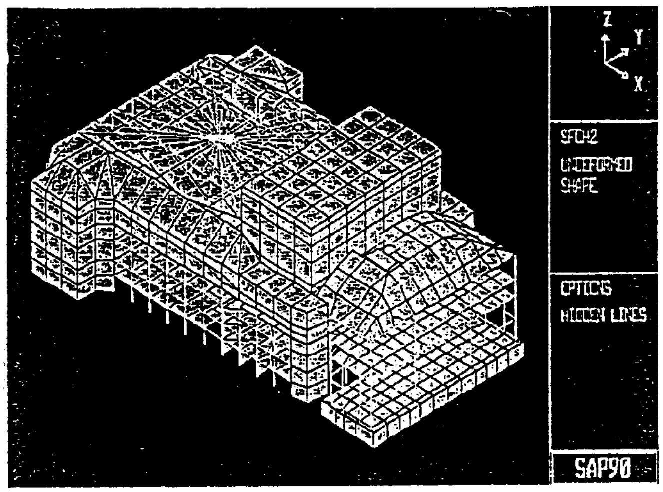


FIG. 1-A ISOMETRIC VIEW OF THE MAIN HOUSE AND THE ANNEX

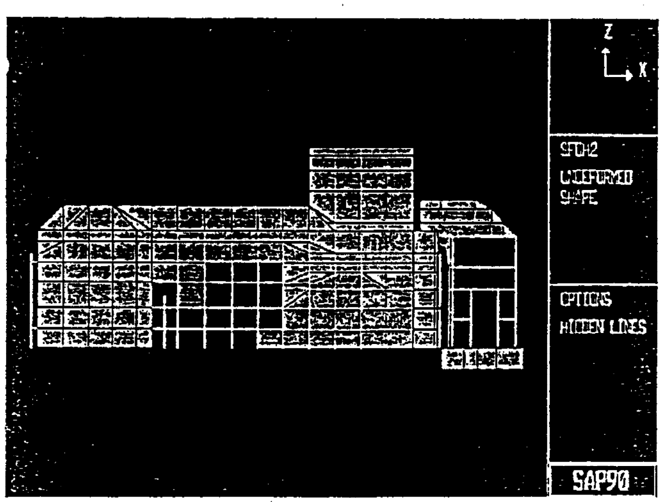


FIG. 1-B SECTION OF MAIN HOUSE AND ANNEX

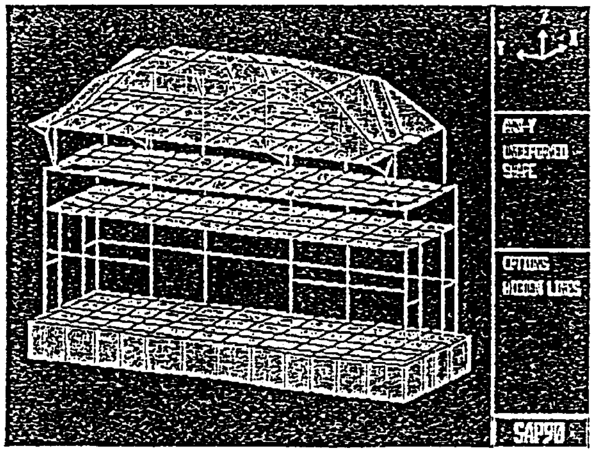


FIG. 2-A THE EXISTING ANNEX BUILDING

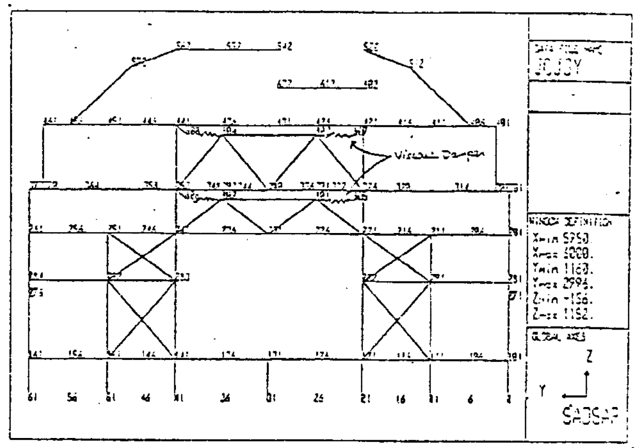


FIG. 2-B LONGITUDINAL FRAME DAMPER LOCATION

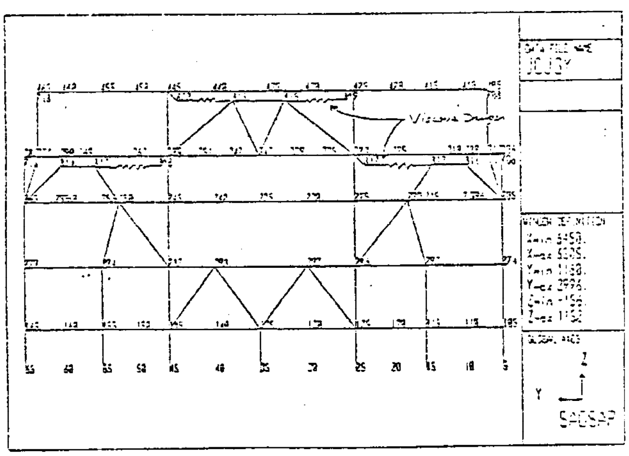


FIG. 2-C LONGITUDINAL FRAME DAMPER LOCATION

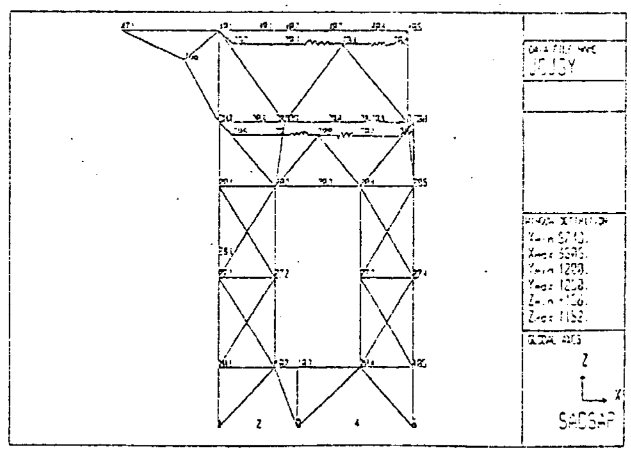


FIG. 2-D TRANSVERSE FRAME DAMPER LOCATION

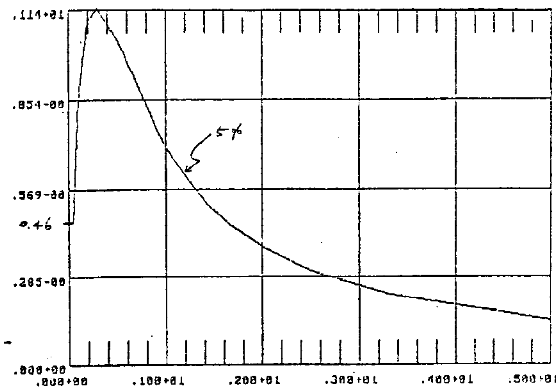


FIG. 3-A 475 YEARS RETURN PERIOD EARTHQUAKE SPECTRUM

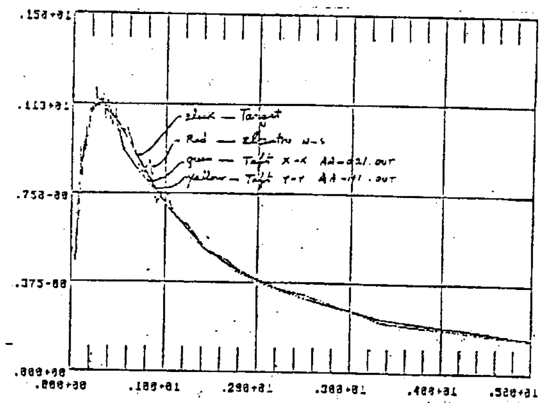


FIG. 3-E SPECTRA FROM THE MODIFIED EQUIVALENT TIME HISTORY

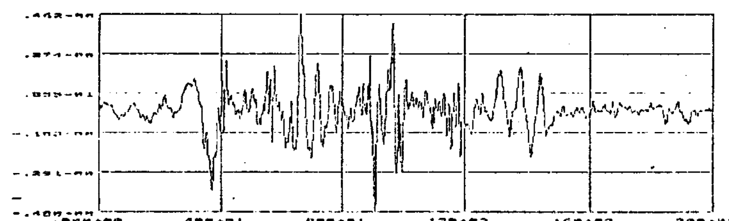


FIG. 3-B MODIFIED EARTHQUAKE TIME HISTORY RECORD OF TAFT, CALIFORNIA

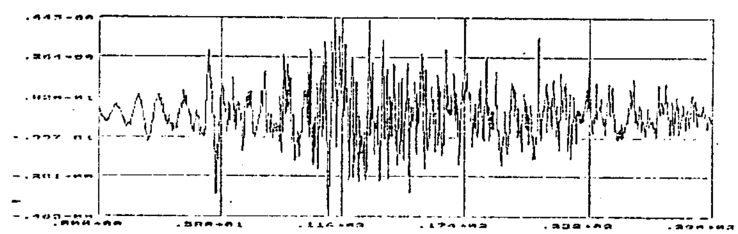


FIG. 3-C MODIFIED EARTHQUAKE TIME HISTORY RECORD OF TABS, IRAN

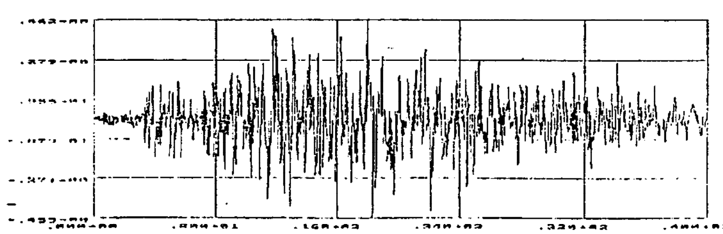
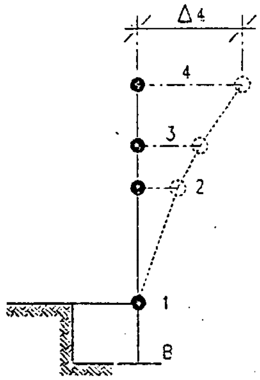


FIG. 3-D MODIFIED EARTHQUAKE TIME HISTORY RECORD OF CALETA DE CAMPOS, MEXICO



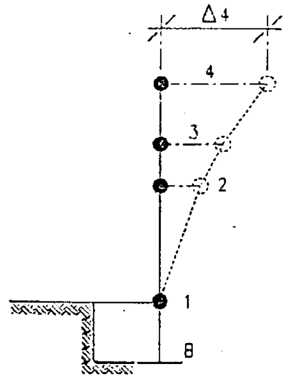
FLOOR	TIME HISTORY	AA X–X SOUTH	BB X–X SOUTH	CC X–X SOUTH	AA X–X NORTH	BB X–X NORTH	CC X–X NORTH
4TH	DISPLACEMENT	3.3	3.3	3.0	2.5	2.5	2.3
	VELOCITY	29.3	28.8	25.2	21.6	25.4	24.1
	DAMPER FORCE	367	372	321	298	355	312
3RD	DISPLACEMENT	2.3	2.5	2.2	1.9	1.9	2.1
	VELOCITY	22.6	22.5	21.1	16.2	21.7	17.3
	DAMPER FORCE	387	356	331	258	347	293
2ND	DISPLACEMENT	1.7	1.8	1.6	1.4	1.4	1.5
	VELOCITY	16.3	16.7	15.6	11.8	15.8	12.8
	DAMPER FORCE	---	---	---	---	---	---

MAXIMUM DIPLACEMENTS IN INCHES

MAXIMUM VELOCITIES IN INCHES PER SECOND

MAXIMUM DAMPER FORCES IN KIPS

FIG. 4-A MAXIMUM DEFLECTION AND RESPONSES IN LONGITUDINAL DIRECTION



FLOOR	TIME HISTORY	AA Y–Y EAST	BB Y–Y EAST	CC Y–Y EAST	AA Y–Y WEST	BB Y–Y WEST	CC Y–Y WEST
4TH	DISPLACEMENT	2.4	2.3	2.2	2.1	2.1	1.8
	VELOCITY	21.5	20.9	18.3	19.5	17.0	15.7
	DAMPER FORCE	335	257	255	330	232	242
3RD	DISPLACEMENT	1.9	1.6	1.5	1.6	1.4	1.4
	VELOCITY	15.9	16.2	14.5	14.3	14.9	14.2
	DAMPER FORCE	405	387	355	330	284	250
2ND	DISPLACEMENT	1.1	0.9	0.8	0.8	0.7	0.3
	VELOCITY	8.5	10.3	8.8	7.4	9.1	9.4
	DAMPER FORCE	---	---	---	---	---	---

MAXIMUM DIPLACEMENTS IN INCHES

MAXIMUM VELOCITIES IN INCHES PER SECOND

MAXIMUM DAMPER FORCES IN KIPS

FIG. 4-B MAXIMUM DEFLECTIONS AND RESPONSES IN TRANSVERSE DIRECTION

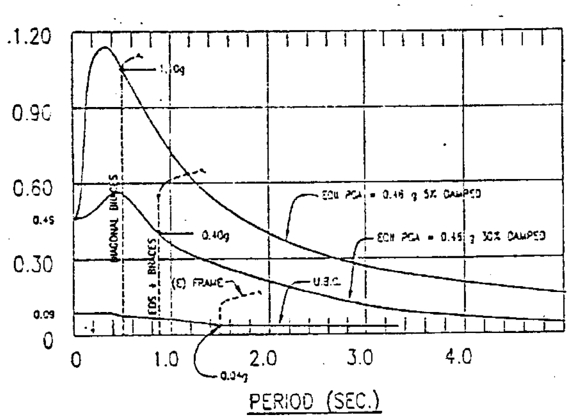


FIG. 5-A

DEMAND VS. SPECTRUM (DEMAND FOR Δ max = 3'-0")

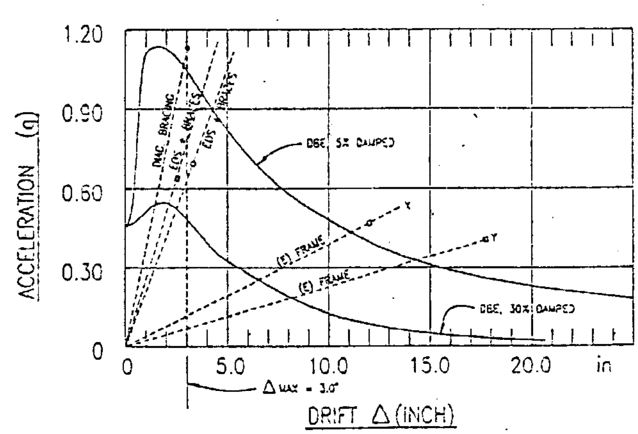


FIG. 5-B

CAPACITY VS. SPECTRUM (Sa vs. Sd) Sa = w^2Sd