"ESTE DOCUMENTO CONTIENE IMÁGENES EN MAL ESTADO"

Experiences on the Use of Supplementary Energy Dissipators on Building Structures

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ABSTRACT

Ductility, or deformation energy, is by far the largest source of energy dissipation of structures, since normal levels of internal damping represent only a small portion of energy dissipation. However, large material deformations such as those required in building components to perform in a ductile manner, are often associated with cracking and degradation of its strength, particularly in concrete structures. The installation of some manufactured devices to critical regions of structural systems, specifically engineered to concentrate on them the largest part of the dissipated energy during an earthquake, increases the structure's overall thoroughness and improves its performance and reliability during major seismic events.

This paper describes the retrofit of three buildings in Mexico City using damping devices. The size and number of these added elements are a function of the dynamic characteristics of the specific structure, the amount of previous damage, the anticipated earthquake motion imposed to the structure and the design performance level intended.

INTRODUCTION

Retrofitting earthquake damaged buildings or upgrading existing buildings to meet higher code demands is a task difficult to accomplish and of high professional risk.. On one side, the engineer who takes on either job automatically assumes the full responsibility of the structural integrity of a building designed by others, and meeting older and less stringent codes (as compared to the ones currently in use), but hopefully built according to good standard practices and with quality materials. On the other side, the analytical methods and tools used in the study of the problem have to take into consideration certain characteristics of the structure and its materials that are difficult to evaluate and ascertain, such as the amount of

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degradation of the strength of the structure, the actual mechanical properties of the materials throughout the structure, structural and nonstructural weights, significance of the nonstructural materials in the structural response of the building, and the condition of the foundation and influence of soil interaction in the dynamic behavior of the building, to mention just a few.

This situation is aggravated by the difficulties normally encountered in the physical implementation of the required reinforcement or strengthening of members, which involve some local demolition, replacement of materials and finishes and delicate construction operations, especially when all of the above have to be carried out while the building is in operation, causing substantial discomfort to its occupants, and altering their schedules.,

The present paper describes some of the author's personal experiences in the retrofit of three buildings in Mexico City in which energy dissipative devices were installed to improve their earthquake response. One of these buildings had suffered damage in the 1985 Mexico earthquake and was not adequately retrofitted, resulting in damage from the earthquake of 1989. The two others were cases of building upgrading; one practically undamaged whereas the other one had a retrofit after the 1957 Mexico earthquake, the extent of which was almost impossible to ascertain. The particular type of retrofit used on each building will be discussed in each case after describing the building's characteristics and its particular problem.

The author's intentions in writing this paper are only to share his experiences with others. It is not intended to reflect the state-of-the-art and his authority on this matter, nor to discuss the professional responsibility he has undertaken by participating in these interesting projects, as this issue varies from country to country. He welcomes any observation or comment on his work, as enhancing the very necessary communication among professionals engaged in this particular area of work, and hopefully improving our engineering efforts, design practices and procedures in behalf of society, in order to produce better and safer structures.

CODE ISSUES

The Federal District Building Code of 1987 was issued as a result of the substantial number of casualties, building collapses and structural damages caused by the 1985 Mexico earthquake. It represents the so-called state-of-the-art document prepared by experts in the profession that represents the latest knowledge in the subject. It responds to the technological advances of the writing body and incorporates the experiences of practicing engineers, so it is a document perceived as a legal obligation that must be complied with in professional work. It is also used as a yardstick to measure the adequacy of existing structures, and as a professional tool , which, when used with skill and care, allows the practicing engineer to complete a safe and economical design or retrofit of a building.

However, every building code accepts that there is some probability of risk which which cannot be foreseen. Depending on their use, certain types of building structures have less tolerance for the acceptance of the risks than others, and therefore can absorb the cost of incorporating more stringent code provisions than others. However, no society is wealthy enough to force existing buildings to comply with a new and more stringent code, beyond certain feasible cost of its retrofit, which implies a tolerance of risk.

Passive dissipation of energy is a promising trend in upgrading and retrofitting existing structures, since codes very seldom deal with detailed procedures to attain safe retrofits, as they strongly depend on the actual condition of the particular structure in each case.

It is intended that the energy dissipative devices installed in the original structure serve as a first defense line against seismic forces, leaving the structure itself as a second defense line. If structures are retrofitted according to these criteria, the structures only receive part of the seismic forces calculated according to the new code. In many instances these forces are of the same order of magnitude for which the structure was originally designed. Consequently, only those portions of the structure and its foundation where the devices are installed need to be strengthened.

The hysteretic cycles of steel plate devices (ADAS*) provide substantial amounts of supplemental energy dissipation to the original structure, which can reduce the spectral accelerations and therefore, the earthquake forces.

The 1987 Mexico Building Code contains no specific provisions regarding the use of energy dissipative devices of any type, and although it doesn't specifically admit their use, it doesn't forbid it either. Even though the code, as it relates to seismic design, is usually characterized as being empirical, the adopted empirical approach does have a rational basis and this rational basis must be expanded to consider the design criteria for the energy dissipative and supplemental damping systems.

In order to do that, the professional practitioner has to rely on analytical procedures not required by the code to find what he believes to be the most appropriate procedure to a rational solution to the problem. He thus exposes himself to a certain professional risk, as he cannot protect his particular practice by the judicial shield of the code.

For instance, in order to extend the rational base of the code, identified as the dynamic behavior of a single degree of freedom system, to the development of a criteria for the design of an inelastic multi-degree of freedom system, it is necessary to identify the objective levels of system strength, stiffness and ductility, and be consistent with the selection of the input design earthquake. However, we know that system ductility is subjectively developed, and its basis comes largely from engineering judgment and intuition, converting our otherwise seemingly scientific approach to only an educated guess.

The author has agreed to accept the challenge of producing what he believes is a conscious, educated guess rather than a truly scientific approach in the case of the three

* ADAS--Added Damping And Stiffness elements are patented devices developed jointly by Bechtel Power Co. and Counterquake Corporation.

buildings whose retrofit is discussed. He acknowledges this to be not only the basic difference between an engineer and a scientist, but the only way to do what society expects from the engineering profession

THE THREE BUILDINGS STUDIED

I) Izazaga #38-40

The Izazaga #38-40 building has twelve floors above grade, a basement and a small two-story machine room situated above the roof level of the building. Figures 1 and 2 are photos of the building with views from Izazaga Street and from the rear of the building, respectively. The building was constructed in the late 1970s and the basic structural system is reinforced concrete frame, with substantive structural walls on column lines 1 and 5, and minor structural walls on column lines 4, C and E (see figures 3 and 4). In addition, the end walls in the transverse direction are brick infilled, and function as stiff, but weak, shear walls. The floor area has 1218 m^2 (13,110 sq. ft).

This building experienced moderate structural damage during the earthquakes of 1985 (Ms=8.1 and 7.3), basically due to the high flexibility demonstrated by most of the concrete moment resisting frame systems incorporating waffle slabs. As a result, the building was repaired by increasing the concrete shear walls on column lines 1 and 5 and adding small interior concrete columns and beams to the brick end-walls at the third points of the spans and at mid-floor height, respectively, to increase the building's lateral stiffness.

Further cracking of the end-walls in the earthquakes of 1986 (Ms=7.0) and 1989 (Ms=6.9)demonstrated the inadequacy of the this retrofit, and very possibly the building's strength degradation. This situation motivated the feasibility study for implementing energy dissipative elements, ADAS, for the seismic retrofit and upgrade of the building in accordance to the 1987 Mexico City Building Code. After finding it appropriate, the design of the ADAS devices and the construction drawings for the retrofit were made.

The overall criteria used for the ADAS design analyses were the 1987 Mexico City seismic regulations (Mexico Code, 1987), supplemented with site-specific response spectrum studies performed by a geotechnical consultant (Ellstein, 1990, 1991) (Figures 5A, 5B and 5C).

Extensive non-linear response time-domain analyses were essential for final verification of ADAS element designs. The use of shortened synthetic time-history records for the postulated earthquake ground motion for the site, generated by acceleration, velocity and displacements with various damping ratios (Figures 6 and 7), considerably reduced the execution time of the computer simulation runs, made with the DRAIN 2DX program (Ellstein, 1990, 1991). Without impairing the overall accuracy of the results soil-structure interaction effects were included in the analyzed models using the modeling guidelines presented in the Mexico City's Seismic Regulations.

Based on functional and earthquake resistant design considerations, a total of ten frame bays were identified in the building retrofit for installation of the ADAS elements.

These are shown in Figure 8. Figure 9 shows a typical arrangement of braced bays in another building, with one, two, or three ADAS per bay. It is important to note that since this building was to be retrofitted while in operation, all braced frames were located on the external column lines, to minimize the interference with its occupants.

Table 1 compares the elastic fundamental periods of the "as is" Bare Frame with those for the building with ADAS elements for both the longitudinal (X) and transverse (Y) major response directions. In addition, the table shows the influence that the flexible soil foundation has on the building's periods. The table shows that the ADAS retrofit effectively increases stiffness, and this shortens the building periods in both response directions. For this building and earthquake ground motions like that in Mexico City, increasing stiffness reduces building response deformations and accordingly, damage.

TABLE 1

Fundamental Periods
(Gross Section Properties Used)

	Fundamental Period (sec.)	
	X (Longitudinal)	Y (Transverse)
Bare Frame		
Rigid Base	2.04	2.82
Flexible Base	2.33	3.82
With ADAS		
Rigid Base	1.67	1.78
Flexible Base	2.01	2.24

Figures 10 and 11 show the substantially increased stiffness and strength that result from the ADAS elements for the seismic retrofit of the building, for column lines 1 and 3. These are plots of base shear vs. roof displacement. The lateral loading used to generate these figures is a static, inverted triangle-shaped force. The flattening of the curves show that ADAS elements yield, as expected.

An important indicator of the earthquake response performance of buildings is the magnitude of the interstory lateral deformation. Small interstory deformation will produce little damage in buildings. Conversely, large interstory deformations will produce substantial damage. Increasing the strength and stiffness of a building typically come together (as a pair) with conventional structural strengthening strategies. Because ADAS elements yield, it is possible to increase initial stiffness without increasing strength an equal amount.

TABLE 2

Global Earthquake Response Parameters for Column Line 3

	Condition of Building	
	As Is Bare Frame	ADAS Retrofit
Maximum Base shear Coefficient (unitless)	0.18	0.18
Maximum Roof Displacement (inches)	20.2	17.5
Maximum Building Lateral Deformation (inches)	18.2	13.7
Average Interstory Drift Angle (unitless)	0.011	0.0081
Maximum Interstory Drift Angle (unitless)	0.019	0.012

Table 2 compares several earthquake response parameters for column line 3 when considered as the "as is base frame" and for ADAS retrofit design. The first row shows that the base shear coefficient is 0.18 for both building conditions. The second and third rows in the table show the "maximum roof displacement" and "maximum building lateral deformation," respectively. The difference between these values is the deformation of the soil foun-

TABLE 3

Global Earthquake Parameters for Column Lines 1, I and G

	Column Line		
	1	I	G
Maximum Base Shear Coefficient (unitless)	0.15	0.12	0.15
Maximum Roof Displacement (inches)	17.7	16.1	18.9
Maximum Building Lateral Deformation (inches)	14.8	12.9	14.0
Maximum Interstory Drift Angle (unitless)	0.0088	0.0076	0.0083
Maximum Interstory Drift Angle (unitless)	0.011	0.012	0.012

dation. Note that the soil deformation for the ADAS design is greater than it is for the "as is" frame, which reflects the increased strength and stiffness of the ADAS retrofit.

The fourth and fifth rows of Table 2 show the average and maximum interstory drift angles, respectively. These reveal that: (1) the "as is" condition is unacceptable because the maximum interstory drift of 0.019 exceeds the Mexico City Building Code's maximum permitted drift of 0.012, and (2) the ADAS retrofit design produced a smaller interstory drift which satisfies the code. It is important to note that the ADAS design solution has produced smaller interstory drifts than those for the "as is" bare frame without increasing the base shear coefficient, which reflects the energy dissipation characteristics of the ADAS devices. If a conventional (non-energy dissipating) retrofit had been implemented, the base shear and foundation loads could have been increased significantly. This, to the author's judgment, is one of the most important engineering goals achieved with this innovative technique of retrofitting earthquake damaged structures, as the strengthening of foundations in the difficult soil conditions of Mexico City is both costly and extremely complicated.

Table 3 shows similar response performance characteristics for column lines 1, I and G with the ADAS retrofit. This table shows that the interstory drift limit requirements of the Mexico City have been met for these column lines as well.

The ADAS retrofit of the Izazaga #38-40 Building has produced several important improvements in its overall seismic response performance. The increased strength, stiffness and energy dissipative characteristics of the ADAS retrofit have reduced the maximum interstory drift by nearly 40 percent. As stated previously, reductions in interstory drift correspond directly with reduced structural and nonstructural damage. Because of the resistant ductility characteristics of the ADAS devices they will not experience strength degradation (Bergman, 1987), and thus the improved response performance characteristics of the building are expected even if the building is subjected to larger earthquakes than the ones used for earthquake simulation analyses.

Because of the substantial stiffness of Column Line 1 in the longitudinal (X) direction of motion relative to the frame lines, the "as is" building exhibits significant torsional response. Torsional response behavior has been determined to be particularly damaging in the past earthquakes. The ADAS retrofit design for this building has been specially engineered to reduce this torsional response behavior.

II) <u>Cardiology Hospital Building</u>

The Cardiology Hospital Building has five floors above grade, a basement and a one-story machine room situated above the roof level of the building. Figures 12 and 13 are photos of the building with views of the front and back sides. The building belongs to the largest hospital complex in Mexico City: the Century XXI Hospital Center of the Mexican Institute of Social Security (IMSS), formerly identified by the name of "Medical Center," and sadly remembered by the numerous casualties that resulted from the severe damage and collapse of some of its hospital buildings during the September 19-20, 1985 earthquakes (Martinez-Romero, 1986).

The building was constructed in the 1970s and consists of three parallel continuous semi-lightweight concrete (1,800 Kg/m²) (112 psf) reinforced concrete frames, containing eight 9.00 m. (29.5 ft.) bays each in its longitudinal (X) direction. Each frame is spaced at 13.05 m. (42.8 ft.) from each other, in its transverse (Y) direction, and are connected together by continuous beams, at every column line. The building plan dimensions are 9 x 9.00 m. = 81.00 m. (265.7 ft.) in length, by 2 X 13.05 m. = 26.10 m. (85.6 ft.), and floor heights are 4.23 m. (13.9 ft.) from the base ground to the street level and four floors of 3.90 m. (12.8 ft.) each, with a total height of 19.93 m. (65.3 ft.).

The floor system consists of double-tee post-tensioned concrete beams, sitting on the bottom flange of the longitudinal beams with semi-lightweight concrete topping slabs of 6.4 cm (2 1/2"). Column and beam sizes are 0.90 m. x 0.50 m. (36" x 20") and 0.70 m. x 0.50 m. (27.5" x 20"), respectively.

This building was interconnected with an adjacent service building, through ramps at different locations (see Figure. 13). Moderate structural and nonstructural damage was observed on this building, as a result of the 1985 earthquakes, reflecting mostly large inter-story drifts. The building was quickly re-conditioned to be operative due to the emergency. Simultaneously the IMSS authorities ordered a thorough structural revision in accordance with the 1987 Building Code.

Two different schemes of retrofit were visualized by two independent firms. The one proposed by the author, featuring a series of external buttresses linked to each of the building's floors at certain column lines, through ADAS energy dissipative elements, was selected for its simplicity of execution and minimum interference with the day-to-day functioning of the hospital.

Figures 14 and 15 show the basic retrofit scheme studied for this building (which features tubular steel-trussed buttresses externally located to the building along its two principal directions), and a typical connection detail between the building and each buttress through ADAS devices.

All three longitudinal column lines and six of the nine transverse column lines had external buttresses at each side of the building due to the low diaphragm capacity of the double-tee floor system. A total of 18 external buttresses were designed to install the dampers; each buttress supported five ADAS in total; i.e., one device every floor location, with a total of 90 devices.

Extensive non-linear response analyses in the time-domain were executed to verify the design of the ADAS devices, the buttresses and to verify the global seismic behavior of the building with the external buttresses. Soil interaction effects were found to be highly significative in building behavior, particularly if the buttresses were to be installed on isolated foundations, independent from the building. This required extending the building's foundation to receive the buttresses as shown on Figure 16.

The final results show a substantial reduction of both the base shear and the lateral deflections and drift of the original building produced by the combined effect of the external stiffening action from the buttresses and the additional damping from the ADAS. The shear force that the building transmitted to the buttresses demonstrated the effectiveness of this retrofit scheme. Importantly, the sum of the horizontal shears in each direction calculated from the horizontal reactions on the building's columns and the buttresses was considerably less base shear than the base shear of the building alone, without this retrofit. This demonstrates the benefits of energy dissipation. Table 4 shows reduction in base shear on the building with the ADAS retrofit of around 50 percent, as compared to the original building. Thus, it was possible to upgrade the building to the 1987 Building Code requirements for seismic force, without additional structural reinforcement, i.e., avoiding alterations to the hospital which would alter its activities. Other important goals were attained with this retrofit: namely around 50 percent reduction in the top displacement and average drift ratios, at 0.006 or lower as required by the code for this important type of building; and the roof accelerations were also reduced from 4 to 32 percent, the largest reductions being in the transverse direction.

Interestingly, both deformation of the building and the buttress deformation resemble those of a shear-type building, due to the number of floors and the height-to-width ratio of the building. This situation explains the moderate reduction in column axial forces, as compared to the reduction in bending moments, which is very significant. It also explains why the ADAS yield forces, $P_{y,}$ and yield defléction, $A_{y,}$ are very similar from floor to floor, at each buttress.

If the retrofit had been made by the external buttress rigidly linked to the building at every floor, instead of having used the ADAS to connect them, the forces on the buttress and its foundations would have resulted in considerably larger values to produce similar structural displacement response. This situation would have imposed very substantial increases in the foundation, as well as in the buttress sections and materials, and large forces in the buttress chords.

This seismic retrofit was the most economical and beneficial, and the only one that allowed the continuous operation of the hospital, essential for the purposes of the owner, the IMSS.

III) <u>The Reforma #476 Building</u>

The Reforma #476 building is a complex of three buildings. Building 2 is in the center of the complex. Buildings 1 and 3 are located on each side of Building 2 and are mirror-image symmetrical in plan and elevation. Because of the symmetry, only Building 1 and 2 were analyzed. Each of the three buildings has a basement, a ground floor level (Planta Baja), and nine numbered floor levels. Buildings 1 and 3 have an additional mezzanine floor level immediately above Planta Baja. The height of the lower level in Building 2 is approxi-

TABLE 4

Comparison of Results of the ADAS Retrofit for the Cardiology Hospital Building

DIRECTION	LONGITUDINAL				TRANSVERSE					
Colum Line	5 (Ext. Frame)		7 (Int. Frame)		M (Ext Frame)		O (Int. Frame w/ Butress)		P (Int. Frame)	
Building Condition	Bare Frame	Frame with ADAS	Bare Frame	Frame with ADAS	Bare Frame	Frame with ADAS	Bare Frame	Frame with ADAS	Bare Frame	Frame with ADAS
Base Shear (Ton)	600	514	1174	837	295	179	234	138	266	134
Coefficient (g)	0.20	0.17	0.19	0.14	0.39	0.24	0.15	0.09	0.18	0.08
Reduction in Base Shear %	--	14	--	29	--	40	--	41	--	50
Top Displacement (cm)	24	12	25	12	27	7	29	7	28	12
Average Story Drift Unitless	0.120	0.060	0.125	0.060	0.135	0.035	0.145	0.035	0.141	0.060
Reduction %	--	50	--	52	--	74	--	76	--	57
Roof Acceleration (gals)	260	233	270	229	254	192	276	189	276	267
Critical ColumnAxial Force (Ton)	263	241	471	409	278	236	454	416	467	425
Critical Column Moment (Ton-m)	139	110	192	99	236	146	260	144	264	179

stories. The height of the roof for all three buildings is the same. In addition, a two-story penthouse above the roof at portions of all three buildings which serve as machinery rooms. The building is supported on piles and is located in the Seismic Zone of the Mexico, D.F. basin (Zone II).

The building is owned by the Mexican Institute of Social Security (IMSS) and was constructed in the 1940s as its central headquarters. The basic structural system for the ... cast-in-place reinforced concrete frame. The concrete solid-slab floors are ... reinforced concrete beams and either round columns with closely spaced spiral ...cement or rectangular columns with moderately to largely spaced reinforcing ties. ... and 18 show photos of the building from the front and the back. Figure 19 shows ... view of the main lobby of Building 2, displaying the double height ground floor.

Through its almost 50 years of service, the building has survived at least 11 major earthquakes Ms=7.0 or larger in magnitude; the July 28, 1957, Ms=7.5 earthquake, produced moderate to significant structural and nonstructural damage which was repaired by strengthening parts of its structural system. Interestingly, the September 19, 1985, Ms=8.1 earthquake, being larger in magnitude, did not produce the same type of damage as the 1957 earthquake, due, in the author's point of view, to two main reasons: (i) the strongest direction of the seismic waves of the 1957 earthquake were south-north, taking the building in its weakest direction, as compared with the 1985 earthquakes, whose strongest direction of its seismic waves was almost west-east (which happens to be the strongest direction of the building), and (ii) the 1985 earthquakes found a structure and foundation stronger than the original.

The building foundation consists of a partially compensated foundation box, 5.5 m deep (18 ft.), with about 1,600 wooden piles bearing on the first hard soil layer, located about 26 m. under the ground level. The compressibility of the underlaying soil deposits originated significant relative settlements of the three buildings and tilting of one of the buildings (building 1), which required in 1954 the addition of 135 concrete bearing piles on control mechanisms, designed to stop the tilting and to restore the building's verticality. The continuing extraction of water from the subsoil produced further settlement of the building, which demanded a major revision and strengthening of the building's foundation in 1967, when about 493 concrete piles, also on control mechanisms, were added. Importantly, no signs of structural distress or damage were found then on the foundation box, nor on the building structure (Hermosillo-Martinez, 1971). The building at present has practically recovered its verticality due to the effective control by the pile mechanisms.

The architectural significance of this building in the urban context of Mexico City has it cataloged as part of the National Register of Classical Buildings. Likewise, it houses the strategic and logistic operations of the IMSS, who has classified it as an "essential" building. These important factors lead the IMSS to request the author to conduct a careful study of the structural safety of the building, according to the upgraded 1987 Building Code. It was found to be hazardous, not only according to the new code requirements (which was expected), but also from the fact that there is a serious scientific possibility that strong earthquakes originating in the so-called Guerrero Gap, which has the highest seismic potential, might occur soon (Singh, 1986). These would produce about the same seismic wave directions as the 1957 earthquake, to which this building could be specially vulnerable.

Based on functional, economical, architectural and earthquake resistant design considerations, the alternative of implementing a series of energy dissipators strategically located in the building structure was found to be both feasible and the most convenient way to provide retrofitting which conforms to the current code and guarantees the building's structural safety. The retrofit was to be made while the building remained in operation, a situation which required serious consideration and agreement among the IMSS engineers, architects and authorities and the technical team of Counterquake Corporation and Bechtel Power Co. lead by the author's consulting engineering firm, who proposed and designed the retrofit scheme. The IMSS authorities submitted the proposed retrofit scheme for the opinion of the Institute of Engineering of the National Autonomous University of Mexico (UNAM) and the Fundacion Javier Barros Sierra, A.C., who reviewed and endorsed the project as feasible.

A total of 40 frame bays in the three-building complex were identified as appropriate for installing ADAS elements in connection with the seismic retrofit design. These are shown in Figures 20, 21 and 22 at the bold arrow locations. Figures 20 and 21 are plans of Buildings 1 and 3 that show the ADAS bays for each floor from the lower level (Planta Baja) through the roof (AZO). Figure 22 is a plan view of Building 2 showing the ADAS bays from level N1 through AZO.

Because this building has been declared a National Landmark, it was deemed inappropriate to seismically retrofit the lower level (PB-N1 with exposed bracing crossing the interior space of the lobby in Building 2 (Figure 19). Accordingly, strengthening in the longitudinal direction is achieved by moving the ADAS bays from Column Lines M and O to Column Lines L and P. In the transverse direction, the only ADAS bays are at Column Lines 11 and 26, between M and O (Figure 23). Additional strength and stiffness is being developed in the PB-N1 levels by increasing the sizes of nearly all interior columns. Finally, the N1 floor diaphragm is being strengthened to provide the required lateral load transfer.

Computer simulation modeling of each of the two buildings was made using the DRAIN 2DX program, in order to take into consideration the highly non-linear ADAS elements. Thirteen computer models of plane frame analyses were created in total; specifically, column lines 1,4,6,10,I,K,L,M and O for Building 1, and column lines M, 11,13 and 16 for Building 2 were modeled because each has distinct structural features. Column Lines O, 21,23 and 26 of Building 2 were not modeled because of symmetry with the four column lines in Building 2 that were modeled. In addition, DRAIN-2D computer models for column lines without ADAS elements were constructed to complete the determination of the relative stiffness of the various frame lines. Soil-structure interaction effects were considered based on the modeling guidelines presented in the Mexico City Building Code 1987 (RCDF' 87), but their effects were found to be insignificant in the building behavior.

As in the case of the other two buildings discussed in this paper, the overall criteria used for the ADAS design analyses performed consisted of the Mexico City Building Code seismic regulations. Supplemental to this, four distinct earthquake ground motion records were also used for performing the computer simulation ADAS design evaluations (Scholl, 1990).

The four earthquake ground motion records used in the ADAS design evaluation are listed in Table 5. The RF-1 record is a site-specific estimate of the maximum earthquake ground motion that the IMSS Reforma buildings might experience, and was prepared by Laboratorios Tlalli in Mexico, D.F. (Ellstein, 1990, 1991).

The SM-1 ground motion is a synthetic record prepared by the author's office (Juarez-Ortega, 1990). It was created to match the Transition Zone (Type II) response spectrum in Mexico City, according to the RCDF'87 for an essential, Class A, facility. The DT-1 ground motion is a synthetic record prepared for a different building site in a deep clay region of Mexico City. These records were prepared by Bechtel Corporation (Scholl, 1990). The TY2 is the record obtained from the September 19, 1985 Mexico earthquake at a stiff soil, and is located approximately 3 km west of the IMSS Reforma building site.

Accelerogram plots for each of the four basic records (not amplified) are shown in Figure 24. Plots of the 5 percent and 25 percent damped response from each of the four basic records are shown in Figure 25.

TABLE 5

Earthquake Ground Motion Records Used for
ADAS Design Evaluation

Record Designation	Record Peak Acceleration	Amplification Factor Used In Analyses	Amplified Record Peak Acceleration	Amplified Response Spectrum Acceleration @ T=1.7 SEC Damp=5%	Damp=25%
RF1	0.127 g	1.5	0.190 g	0.39	0.24
SM1	0.225 g	1.0	0.225 g	0.47	0.27
DT1	0.100 g	1.0	0.100 g	0.25	0.17
TY2	0.035 g	6.5	0.228 g	0.36	0.21

Table 5 shows that the RF-1 and TY2 records were amplified for performing the computer simulation ADAS design evaluations. The amplification factor of 1.5 for the RF-1 record was established to account for: (1) the IMSS Reforma building as an important (essential) facility and (2) accidental torsion. The TY2 ground motion record has a low peak acceleration of only 0.035 g and the 5 percent-damped response spectrum amplitude at 1.7 sec. is only 0.055 g. The amplification factor of 6.5 was established to make the building response amplitude for this record about equal to the building response for the other records.

The ADAS design analyses for the IMSS Reforma building were performed using the amplified RF-1 record. The other three records were subsequently used to check the response of the buildings. The designation RF-2 was used to distinguish the RF-1 record amplified by a factor of 1.5.

The results presented herein summarizes the results of the earthquake response analyses performed in connection with designing the ADAS elements for the seismic retrofit of this building. They show the overall impact of the ADAS retrofit response performance as compared with the "as is" base frame building, and with a postulated retrofit involving strong steel bracings that would not yield during earthquake response.

Table 6 compares the elastic fundamental periods of the "as is" bare frames with those for the building with ADAS devices for both the longitudinal (X) and transverse (Y) major response directions. The table shows that the ADAS device retrofit effects an increase in stiffness, and thus shortens the building periods in both directions. For these buildings, and earthquake ground motion like that in Mexico City, increasing stiffness reduces building response deformations, and accordingly, damage. The bare frame analyses were performed with three-dimensional models using the ETABS program (Habibullah, 1992).

Table 7 compares the response of the two column lines for each of the four earthquake records used in the ADAS design evaluation. The roof displacements for Building 1, Column Line 4 (X, Longitudinal Direction), and Building 1, Column Line L (Y, Transverse Direction) are shown. As mentioned previously, the RF-2 record was used for designing the ADAS devices for the buildings. The RF-2 record produces essentially the maximum building response from among the four records.

Although the peak ground acceleration (PGAs) and response amplitudes at T=1.7 seconds vary somewhat for the four records (see Table 5), the roof displacements for the four records for each column line are quite similar. The consistency of the roof displacement results from the shape of the response spectrum at periods greater than 1.7 seconds because the period of the building lengthens as it goes nonlinear. For example, the DT1 response spectrum at T=1.7 seconds is low, but increases significantly for periods longer than 1.7 seconds. The roof displacement of Column Line L (Longitudinal Direction) is about 5 cm (2 in.) less than the roof displacement for Column Line 4 (Transverse Direction) because the basic elastic period of the building is shorter in the longitudinal direction (see Table 7).

High floor accelerations in a building will impose high forces on partitions, furniture and equipment anchored to the floors. Thus, minimizing response accelerations in buildings also reduces damage to its contents. Minimizing accelerations in buildings also reduces the earthquake forces that must be resisted by the building's major structural components, e.g., beams, columns and foundation. Thus, the goal in any seismic design (or retrofit) is to mini-

TABLE 6

Fundamental Periods

Building Condition	Fundamental Period (sec.) X (Longitudinal)	Fundamental Period (sec.) Y (Transverse)
Buildings 1& 3		
Bare Frame *	1.95	2.05
With ADAS	1.5	1.7
Building 2		
Bare Frame *	1.9	2.25
With ADAS	1.45	1.8

* Periods from ETABS Model

TABLE 7

Example Roof Displacements for Earthquake Records
Used in ADAS Design Evaluation

Earthquake Record	Building 1 Column Line 4 (Y Transverse)	Building 1 Column Line L (X Longitudinal)
RF1 x 1.5 = RF2	33.2	28.0
SM1	32.4	27.7
DT1	32.0	25.3
TY2 x 6.5 = TY6	33.6	28.0

mize both story drift and acceleration. ADAS devices were found ideally suited for making this goal a reality.

Tables 8, 9 and 10 compare the earthquake response performance of three column lines of the IMSS Reforma building for three conditions: (1) the bare frame, (2) frame with ADAS, and (3) frame with elastic bracing. The frame with elastic bracing condition would be the case if the building were retrofitted with strong steel bracing that does not buckle in compression or yield in tension. Column Line L is representative of the Building 1 response in the Longitudinal direction and Column Line 4 is representative of the Building 1 response in the transverse direction. Table 10 summarizes important response parameters of Building 2, Column Line M and is representative of the longitudinal motion for the building.

TABLE 8

Earthquake Response Results
IMSS Reforma Building 1, Column Line L

(RF2 Ground Motion Record: October 1991)

FORCES

Building Condition	Base Shear (Tons)	Base Shear Coefficient	Force in Critical Column (Tons) Comp.	Force in Critical Column (Tons) Ten.	Roof Acceleration (cm/sec^2)
Bare Frame	273	0.11	265	65	158
Frame W/ADAS	281	0.13	417	106	203
Frame W/Elastic Bracing	296	0.14	507	214	250

DEFORMATIONS

Building Condition	Displacement At Roof (cm)	Story Drift (cm) Max.	Story Drift (cm) Ave.	Story Drift Ratio Max.	Story Drift Ratio Ave.
Bare Frame	34.6	5.1	3.2	0.0128	0.0079
Frame W/ADAS	28.0	3.7	2.6	0.0093	0.0064
Frame W/Elastic Bracing	28.7	3.8	2.6	0.0095	0.0065