# Preparedness for the next Earthquake of Reinforced Concrete Residential Buildings in Israel

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**SUMMARY:**
The effort to ensure the structural stability under earthquke loads, claims strategies to strengthen the existing buildings or to provide them by energy dissipation abilities. The present work deals with a range of strategies meant to ensure the stability of a existent building under seismic loads.

*Keywords: Concrete structures, Time history, Supplementary walls, Enegy dissipating devices, Base isolation*

## 1. INTRODUCTION

In their book "Earhquakes in Human History" J. Zeilinga de Boer and D.T.Sanders deal with the effects of the seismic events along the history. The hazardous areas in point of the seismic load are presented according the main seismic events that took place along centuries. The book deals, shortly, with the mechanism of the occurrence of the seismic movements due to the existing tectonic plates, the severity scales of earthquakes and the effects of he seismic events.

The first area where the seismicity is analyzed is the Middle East a zone with a long history of seismic movements. The main active area extends along the Syrian – African fault (see Figure 1.1.)

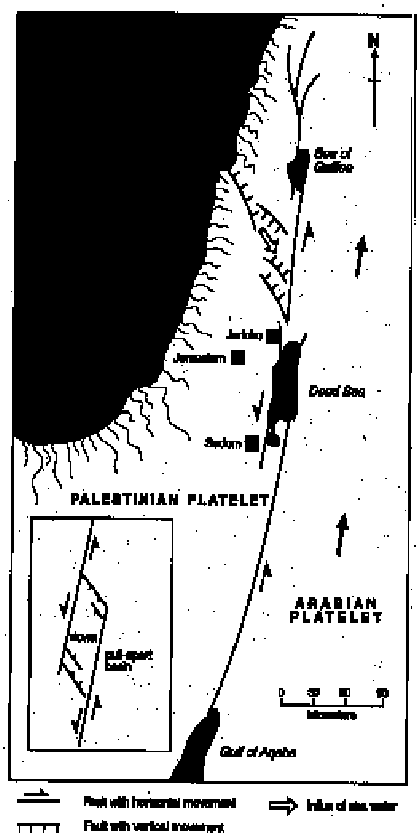


**Figure 1.1.** The Dead Sea Rift

Figure 1.1 visualizes the main fault that developed between the two tectonic plates where Israel is situated.

The last strong seismic movement, in the area, took place in 1937 and caused numerous losses. Since the density of the population in the inhabited areas augmented significantly it increased the risk of the losses which might occur as consequence of a severe seismic event.

According the data publishes by The Israeli Geophysical Institute, statistically, a severe earthquake, in the area, is due to occur each 80 year. The prognosed Magnitude of the strong earthquake is of 7 on the Richter Scale. Under these circumstances many structures have been designed and built without taking into account the dynamic horizontal loads which might develop during the seismic event.

The level of knowledge developed during the recent years. The seismic standards and abilities to enable the structures to withstand the dynamic loads improved. To this development contributed the research which was done based on the observations in areas hit by seismic events, the study of the behavior of structures, the characteristics of the materials used to build structures to stand dynamic loads and recently the implementation of control to dissipate the energy protruding the srtuctures.
This structural revolution took place in the recent years together with the development of the computers and lead to the understaning of the seismic phenomena.

The first Israeli seismic code was published in 1974 and contained very summary indications for achieving structures able to resist the seismic loads which might occur. The main goal of the designers was linked with the effort to ensure standing the loads of gravitational nature.

During the second half of the 20$^{th}$ century a lot of severe earthquakes that took place were thorougly monitored and studied by scientists using more sophisticated instrumentation and performant computers.

As consequence of the modern hardware and software the various aspects of the seismic reseach lead to improved codes and safer structures.

The first Israeli seismic code based on recommendations according the scientific development of the seismic research was published in 1995 and amended in 1998,2003 and is even these days is under revision. Since a lot of buildings in Israel are old buildings, designed and erected during the period before the publication and the implementation of the Seismic Code (SI 413), a huge effort must be accomplished to improve the behavior of these structures in the case of the occurrence of the prognosed seismic event.

The first step, in Israel, was the publication of the SI 2413 – "Guidelines for seismic resistance assessment and strengthening of existing structures" which is a tool to enable the decision if a certain building or, group of buildings possesses the ability to withstand the prognosed seismic movement. More, the ASCE/SEI 41-06 is used for the purpose to determine the strength of a certain building and to achieve the retrofitting for improving the stability of the structure under the seismic load.

Today the methodology which is the most common is the TAMA38 – The National Program No. 38 for Structural Strengthening. This program consists of adding shear walls or concrete tubular elements to increase the strength of an old building and adding one or even two stories to the building.

Due to the cost of such an operation, the inhabitants will be provided by a stable structure while the entrepreneur gets the suplementary apartments to sell them and to cover his investment. This method was declared to be a win-win situation but it is efficient only in the areas where there is demand for residences. The method is successful in central areas like Tel Aviv, but remote areas are not of interest for the entrepeneur. And still a lot of people live in these remote areas. The financial effort requested by this retrofitting method is beyound the means of this people and of the local authorities. Still, these people and their buildings must be prepared to resist the menace of the seismic load. The solution may require a certain ammount of money and will prove efficiency in the case of the pronosed load.

The strategies which may be used in this case comprise use of infill walls to strengthen the building, use of passive disipation devices, use of tuned mass dampers and base isolation. These solutions have been developed during the last decades, some of them have been implemented in actual structures like hospital buildings, tall buildings or bridges. Some of the upgraded buildings proved their stability during seisms that already happened. That is the reason that the present work is devoted to check the efficiency of the different strategies which developed during the last period.

The methodology is checked by means of the SAP2000 software, enabled by means of linear and non-linear analysis of the behavior of the structural model.

The analysis will be made for a typical existing four storey structure which is checked for the response spectrum of the SI 413 code and a seism chosen from existing records for PGA=0.2 g.

## 2.DESCRIPTION OF THE TYPICAL STRUCTURE AND THE ENVIRONMENT

### 2.1 The original structure

The typical structure which has been designed in the sixties to stand under gravitational loads is made of reiforced concrete frames with 20cm deep floors with storey heigt of 3.00 m. There masonry walls are taken into account as secondary elements and are not considered as involved into the stability system under the horizontal loading. According to the design of the sixties of the last century the reinforced concrete elements were not been provided by confining abilities which are now compulsory according the updated codes. The structure is visualized in Figure 2.1.

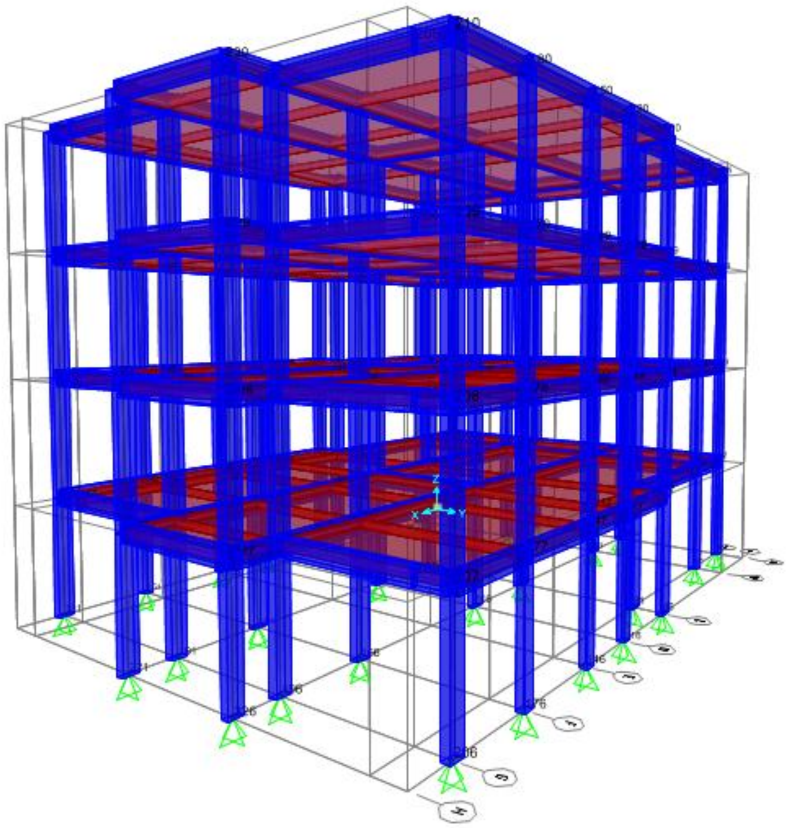

**Figure 2. 1.** The Typical Structure

The modal characteristics of the typical structure have been recorded into the Table 2.1. The structure is very flexible with its first period of oscillation, very long. The verification of the structure is accomplished according the static and dynamic nonlinear analysis for a seismic excitation from the Californian records for the Northridge Earthquake and the spectrum from the Eurocode 8. Both the seismic excitations have been calibrated for a PGA of 0.2g.

**Table 2. 1.** The Modal Characteristics of the Typical Structure

| OutputCase | StepType | StepNum | Period | Frequency | CircFreq | Eigenvalue |
|---|---|---|---|---|---|---|
| Text | Text | Unitless | Sec | Cyc/sec | rad/sec | rad2/sec2 |
| MODAL | Mode | 1 | 1.407201 | 0.71063 | 4.465 | 19.936 |
| MODAL | Mode | 2 | 0.946946 | 1.056 | 6.6352 | 44.026 |
| MODAL | Mode | 3 | 0.80563 | 1.2413 | 7.7991 | 60.826 |
| MODAL | Mode | 4 | 0.388475 | 2.5742 | 16.174 | 261.6 |
| MODAL | Mode | 5 | 0.262716 | 3.8064 | 23.916 | 571.99 |
| MODAL | Mode | 6 | 0.227187 | 4.4017 | 27.656 | 764.88 |
| MODAL | Mode | 7 | 0.217125 | 4.6056 | 28.938 | 837.41 |
| MODAL | Mode | 8 | 0.162657 | 6.1479 | 38.628 | 1492.2 |
| MODAL | Mode | 9 | 0.144321 | 6.929 | 43.536 | 1895.4 |
| MODAL | Mode | 10 | 0.122579 | 8.158 | 51.258 | 2627.4 |
| MODAL | Mode | 11 | 0.10517 | 9.5084 | 59.743 | 3569.2 |
| MODAL | Mode | 12 | 0.089289 | 11.2 | 70.369 | 4951.8 |

Figure 2.2 shows the accelerogram used to check the abilities of the structure proposed to be enabled by retrofitting solutions.

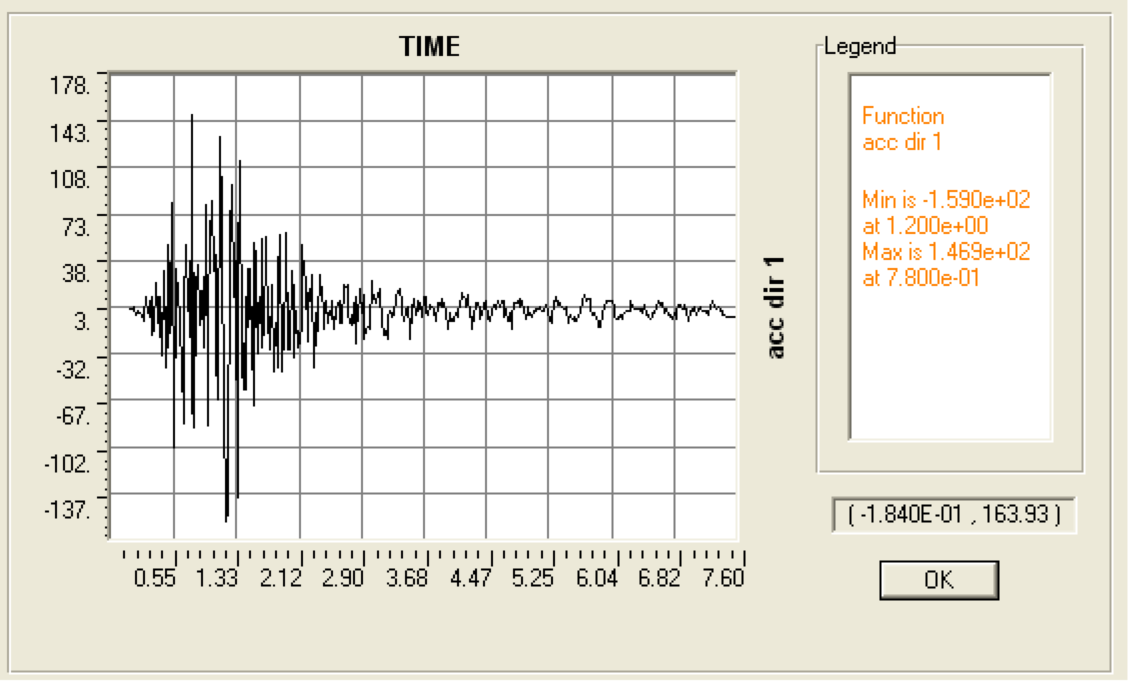


**Figure 2. 2.** The accelerogram for the structural seismic load

The response spectrum for the spectral analysis is shown in Figure 2.3. The PGA to be enforced upon the structural models will be of 0.2g. The soil characteristics are considered of D for considering the SSI (Soil Structure Interaction) during the analysis. The performance analysis will be applied by means of comparison of the structural capacity and the demand. The comparison will lead to the definition of the performance point. According the graphs and the performance point one can appreciate the stability of the structure or the degree of the damage which might occur under the considered seismic movement. The performance analysis is fully described in the ASCE/SEI 41-06 Code and is provided in the Eurocode 8 2004 as its annex.

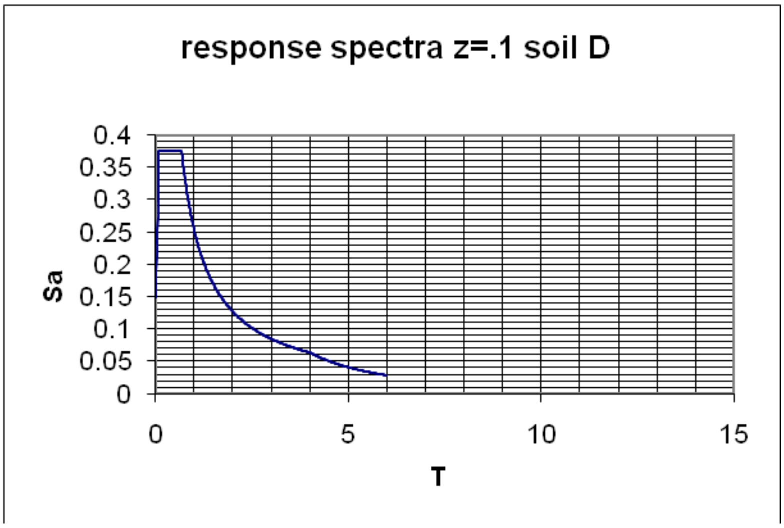


**Figure 2. 3.** The response spectrum used for the structures load

## 2.2. The Analysis of the Original Structure

*2.2.1 The Static Nonlinear Analysis*

The original structure is achieved from a space frame, the columns and the beams being rather flexible without confining abilities. The floor is made of a 20 cm thick plate of reiforced concrete. The strength of the concrete used did not exceed 30MPa.

**Table 2. 2.** Pushover Curve Capacity Demand

| Step | Teff | Beff | SdCapacity | SaCapacity | SdDemand | SaDemand | Alpha | PFPhi |
|---|---|---|---|---|---|---|---|---|
| | | | m | | m | | | |
| 0 | 0.795387 | 0.05 | 0 | 0 | 0.053057 | 0.337615 | 1 | 1 |
| 1 | 0.795387 | 0.05 | 0.011043 | 0.070269 | 0.053057 | 0.337615 | 0.972146 | 1.184547 |
| 2 | 0.817666 | 0.069612 | 0.013651 | 0.082196 | 0.050551 | 0.304382 | 0.973667 | 1.179371 |
| 3 | 0.881309 | 0.119571 | 0.01711 | 0.088684 | 0.046879 | 0.242972 | 0.978989 | 1.161983 |
| 4 | 0.89132 | 0.126999 | 0.017568 | 0.089022 | 0.04652 | 0.23573 | 0.979383 | 1.160689 |
| 5 | 0.914159 | 0.143874 | 0.018522 | 0.089227 | 0.045781 | 0.220535 | 0.980595 | 1.156296 |
| 6 | 1.677796 | 0.293858 | 0.066063 | 0.094476 | 0.06324 | 0.090438 | 0.997643 | 1.050772 |
| 7 | 1.752353 | 0.297091 | 0.072769 | 0.095398 | 0.065678 | 0.086103 | 0.997954 | 1.046677 |
| 11 | 1.952178 | 0.441713 | 0.072789 | 0.065212 | 0.071036 | 0.063641 | 0.995767 | 1.052236 |

The masonry walls were not taken into account leading to a very flexible structure with high value of the first oscillation eigenvalue. The performance point occurs for a period of 1.606 sec. at horizontal displacement of D=0.065m. The base shear is of 788 kN according the analysis. The loads determined according the analyses lead to ovestrengthening of the columns leading to damages followed by the collapse of the structure according the analysis. The joints become plastic hinges leading to structural failure. This behavior must be changed since the building will not resist the prognosed seismic load.

The strategies shown further comprise the achievement of reiforced concrete shear walls for the staircase area, the achievement of a base isolated structure by introducing rubber isolators over the foudations and achievement of a controlled structure by means of a supplementary mass and a supplementary viscous damper.

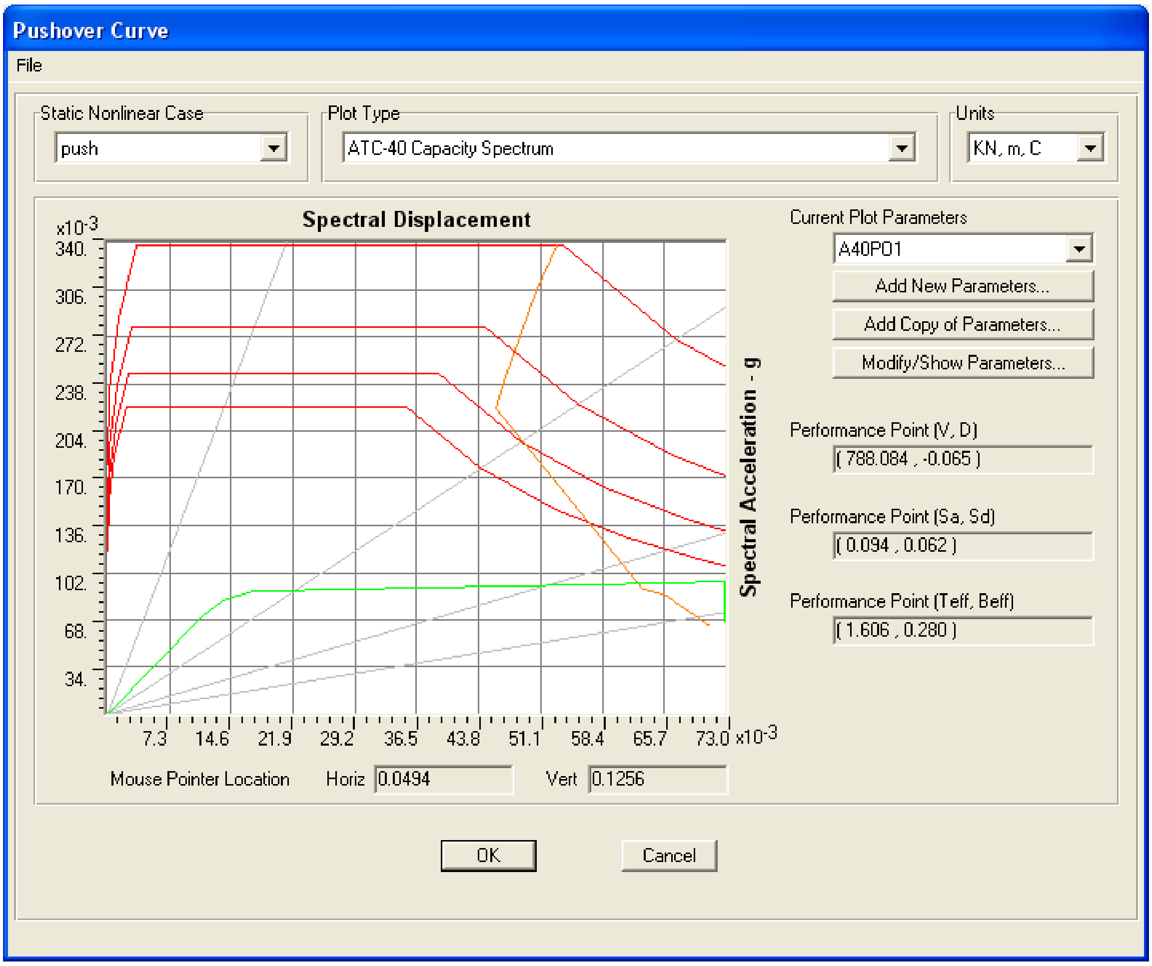


**Figure 2. 4.** The Performance Point of The Original Structure

*2.2.2 The Dynamic Nonlinear Analysis*

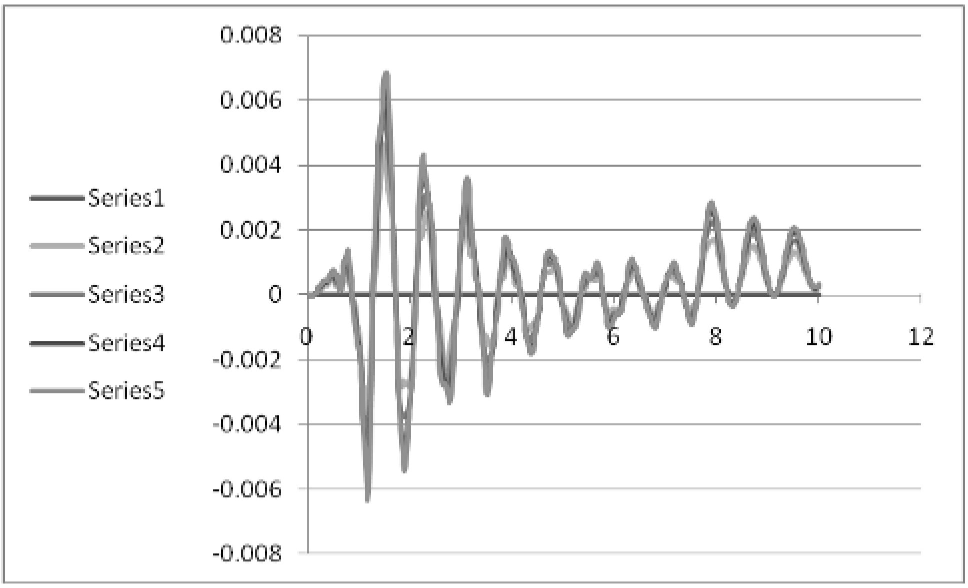


**Figure 2. 5.** The Timehistory of the story displacements

## 3. THE STRUCTURE STRENGTHENED BY SHEAR WALLS AROUND THE STAIRCASE

The strengthening of the structure may be achieved by implanting shear walls in the staircase area.
The supplementary walls increase the rigidity of the structure. The performance point reveals a period of T= 0.155 sec and augment the base share force to 1937 kN. The capacity of the structure is represented by an elastic line with a top deformation at the top of the building of 0.0025m. The supplementary walls avoid the development of the plastic hinges in the reinforced concrete frame.
The strengthening of the structure enables the addition of one or two suplementary stories. The suplementation of stories leads to increased height and increased weight. Special attention must be accorded to the foundations and to the suplementary inertial loads.

In general, more walls are added to the structure such that the supplementary weight should be supported by the added new structure and not by the original structure.

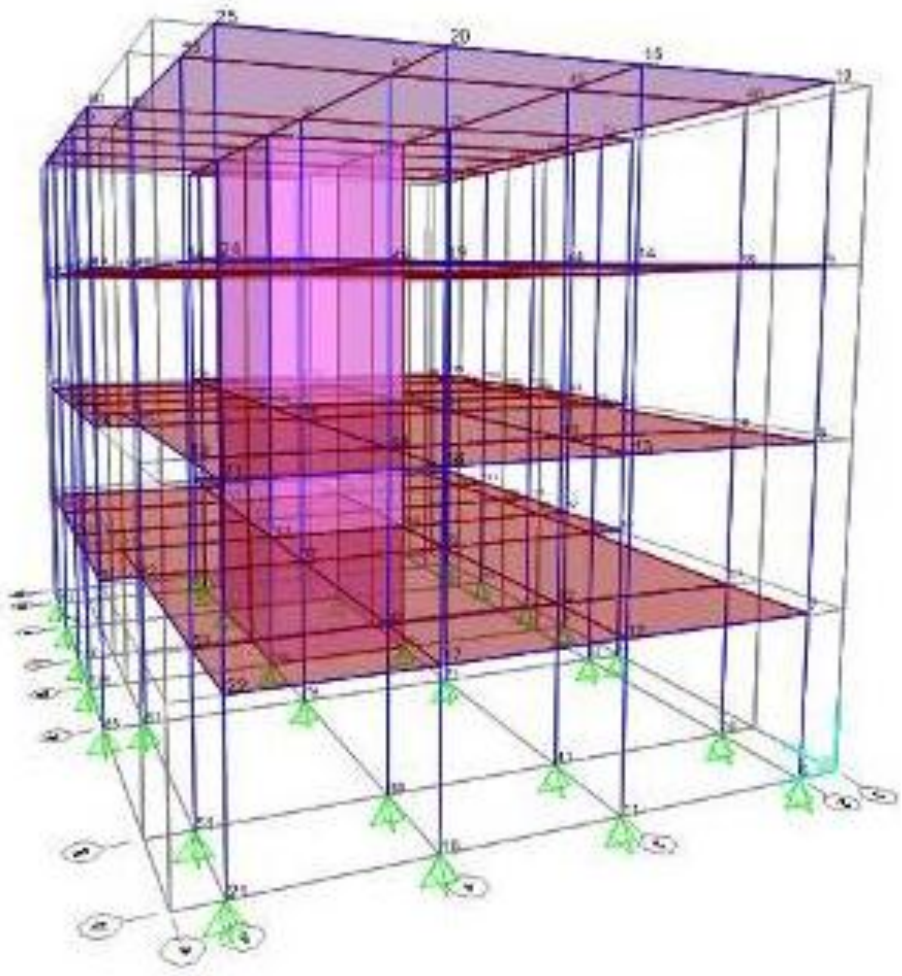

**Figure 3. 1.** The structure strengthened by means of supplementary shearwalls

Figure 3.2 presents the performance of the structure provided by shearwalls in the staircase area. The performance point is situated at the period T = 0.155 sec, a significat strengthening compared to the original structure. The base sharing force augmented to $F_H = 1936kN$ while the top displacement is limited to $D = 2.53mm$.

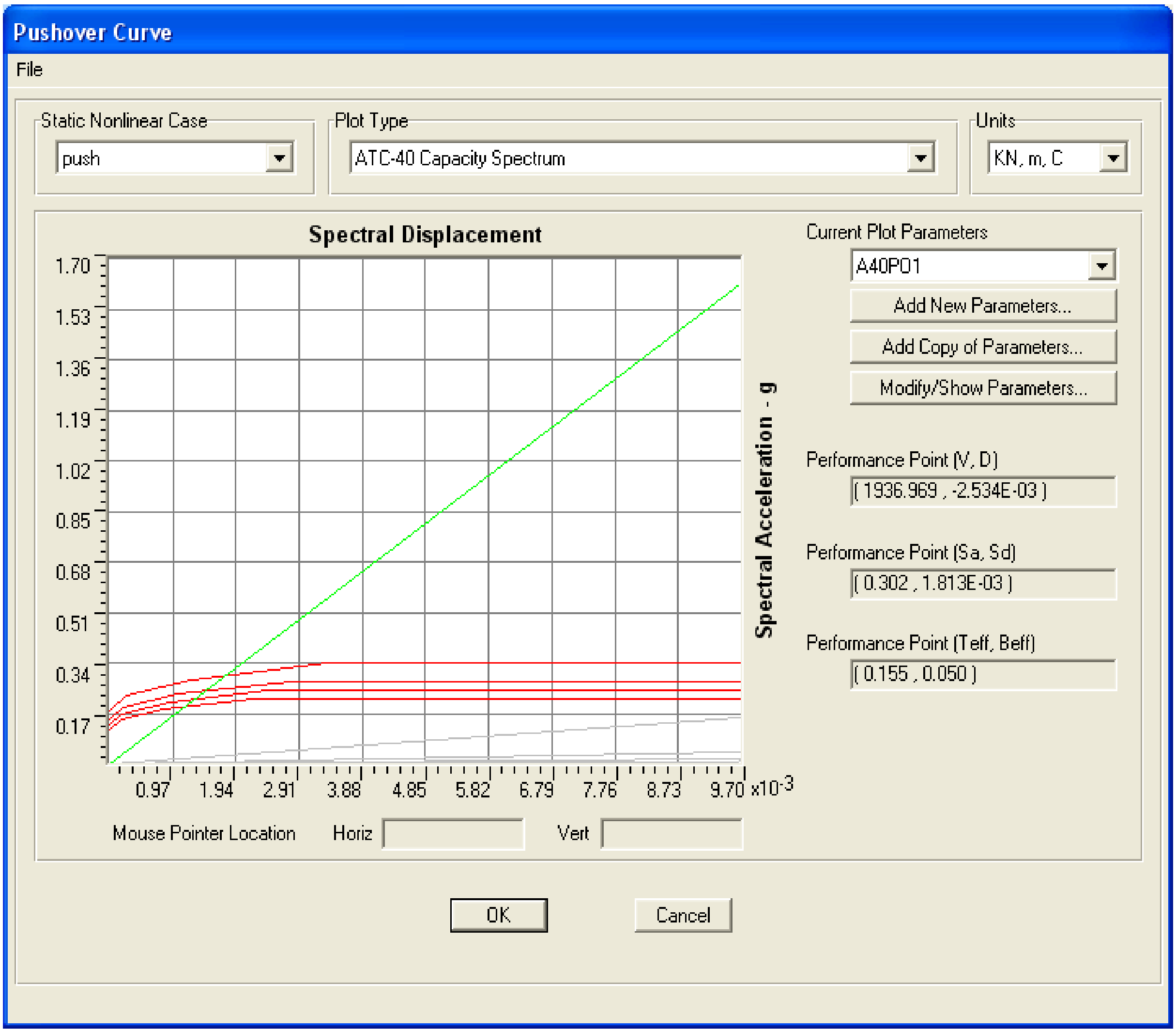


**Figure 3. 2.** The peformance of the structure strengthened by means of supplimentry shearwalls

## 4. RUBBER BASE ISOLATION CONTROLLED STRUCTURE

Base isolation is one of the solutions to diminish the seismic energy protrusion into the structure. There are a lot of devices which can be used to provide the base isolation such as: layer of teflon between the structure and its foundation, spheric elements which permit easy displacement, inverse pendulum and ruber isolators acting in shear.

The model which was used in the present work was provided by the rubber isolators of SAP2000 under each foundation having a shear capacity of 100 kN/m. Equiped with these isolators the structure had a response visualized in Figure 4.1 with almost similar behavior at each level.

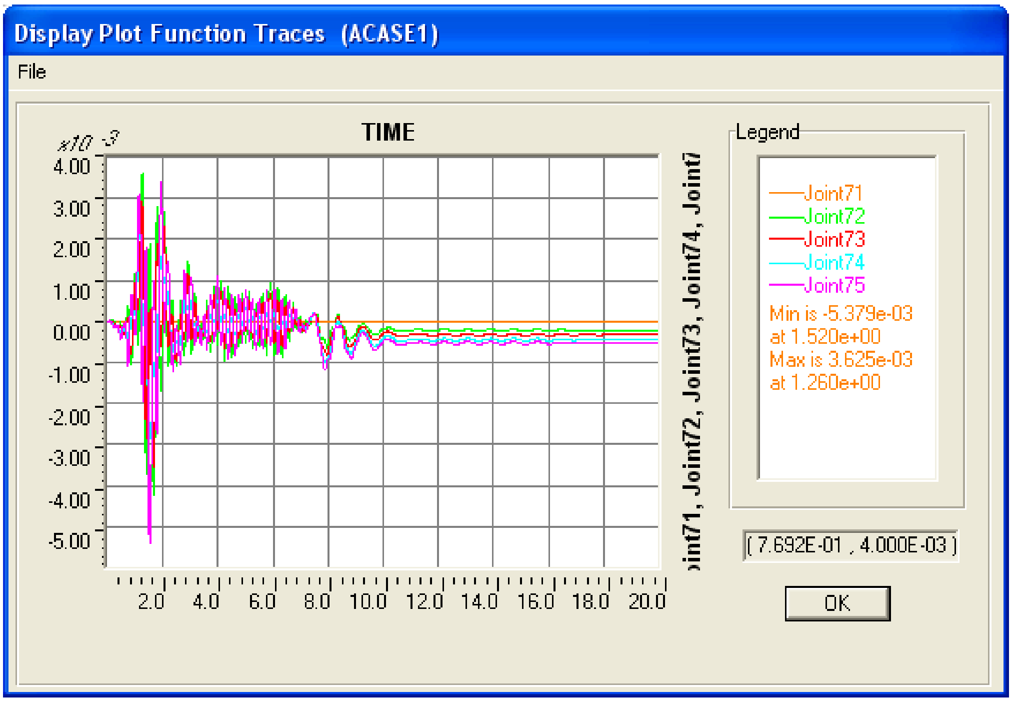


**Figure 4. 1.** The structural displacement at each story

## 5. THE USE OF THE TUNNED MASS DAMPER

The tunned mass damper is a solution of energy dissipation for buildings subjected to horizontal dynamic excitation, mainly earthquake or wind. It is achieved by adding a supplementary mass, spring and damper at the top of the building. It is enough to add a mass which is 2% of the mass of the entire structure. One must care that the eigenfrequency of the suplemental device should be equal to the basic frequency of the original structure. As consequence of this structural upgrading the resonance will be avoided and the supplementary mass will move significantly while the original structure will hardly move. The scheme used for modeling is shown in Figure 5.1.

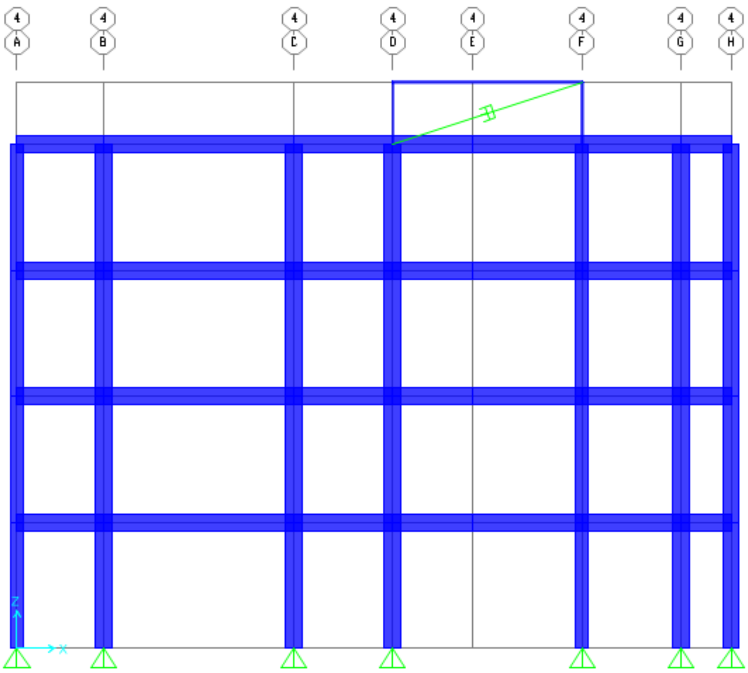


**Figure 5. 1.** One of the frames provided by the TMD

All these solutions, which have been proposed, require an optimization analysis to provide the proper energy dissipation device to reduce the loading effects.

The second factor is the financial investment since the energy dissipation solutions are meant to reduce the investment of the retrofitting process. In the price it is included the cost of the repairs after the seismic event took place.

## 6. CONCLUSIONS

The seismic risk is high and its mechanism does not permit to elaborate a precise prognose of the instant and of the intensity of the menacing seismic event.

In general, most of the seismic faults have been located but there is no certain information about the development of a new or of a still not active fault.

The amplification of the seismic oscillations for certain locations must be known.

All buildings must be checked and, if necessary, upgraded to stand the prognosed seismic intensity.
The codes must offer solutions to upgrade the structures according the optimal methods which will ensure stability and a minimal financial effort.

The behavior of the secondary elements should be followed thoroughly to avoid losses which may be very high.

The performance analysis must provide the desired solutions to strengthen the structure according the prognosed intensity at least.

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