Seismic Vulnerability and Retrofitting of Municipal Theatre of Lima, Peru

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SUMMARY:
The building of Municipal Theatre of Lima started in 1920. Since then, the structure has experienced at least four major earthquakes and a fire in 1998 had damaged roof of the building. Since 2008 a restoration project that includes a structural retrofitting had been carried out. The Lima´s municipal theatre has a plan area of 1190m^2. It is a concrete structure constructed with a theatre purposes. It has a Hall and Reception salon in front and a Pit area and stage. This five story building has a 17m high. The roof structure was made of steel in area of 396m^2. The most vulnerable components of the structure were front wall in the upper part and the lateral concrete frames with infill blocks of plaster masonry existing in the second and third floor. Some parts of the concrete frames structure show damage in steel bars due to humidity and poor maintenance.

Keywords: Seismic Vulnerability, Historical Building, Retrofitting.

1. INTRODUCTION

The lot assigned to the Municipal Theatre of Lima was built with theatre purposes in the early 1920´s. It is a five story concrete building. Since then, at least four major earthquakes and a fire occurred 1998 have affected the roof of the structure. After 1998 the building is closed because the fire badly damages the steel roof structure and the stage.

Before the retrofitting the concrete structure presented cracks and exposed steel bars in the third floor of the gallery for the audience. There is also some damage due to past earthquakes and structural deficiencies which were never completely corrected. The whole structure seems to be rigid whereas the concrete elements were not accorded with the quantities of steel recommended in ours actual seismic standards. Also there is some deterioration mainly as a result of environmental effects, humidity and poor maintenance.

The roof structure of the theatre were composed by five steel truss that were severe badly damage due to the fire. Above this structure were two wood stories that had been removed. Numerical simulations on the old building show irregularities that affect its seismic response. The retrofitting carried out on the building pretend to correct these irregularities trying to enhance its seismic behavior.

Development of an effective methodology for future repairs and strengthening of upon important historical constructions require an integration of knowledge at least in the areas of seismology, geotechnics, structural engineering, material science, architecture, art as well as social, cultural and economic aspects. This paper presents the results of the earthquake response analysis for the original building and the retrofitted one. Some details in the retrofitting process will be discussed.

2. ABOUT THE THEATRE

The building of the theatre was built under the influence of French later neoclassic style. It has two important main areas: In front, the span of the three levels with the entrance, the principal hall and cafeteria (basement); inside the building, the five´s levels gallery for the audience with a common shape plan view of horse-shoe. The imperial ladder is a singular element that connects both main areas. Architectonically, the main chamber presents a great number of blaster ornaments cover with gold sheets.

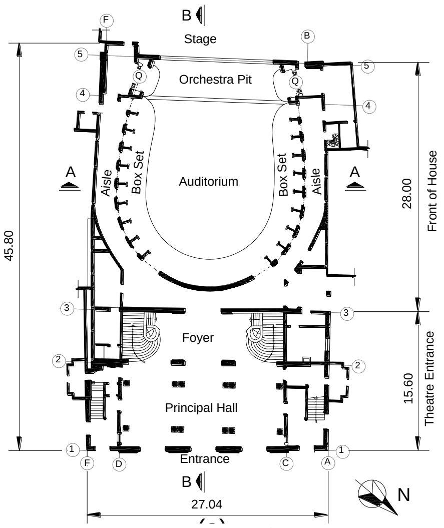

Figure 1. Plan view of the structure

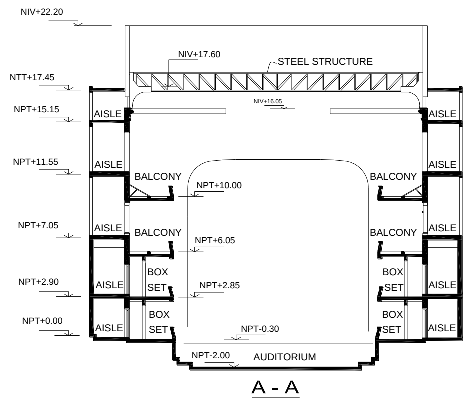

Figure 2. Cross view of the building

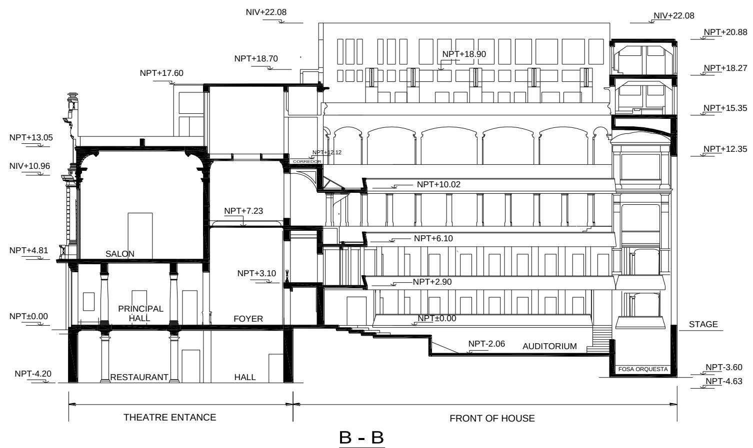

Figure 3. Lateral elevation of the building.

The structural configuration of the theatre is irregular in plan and elevation. It is made on reinforced concrete frames and concrete walls in the first two floors. In the third and fourth floor of the auditorium presents a concrete frame system with some masonry infill walls made of plaster and clay blocks. The density of concrete walls on the first floor of the building is about 9%.

The main entrance hall is 4.65m height and structurally consists of walls of concrete, masonry and 12 circular columns; the roof slab is supported on reinforced concrete beams. The second floor reference area for the reception hall is 8.3m height and has walls of concrete and masonry, the roof slab is supported by inverted beams. On the other hand, auditorium zone of the building have five levels. Each level has an area of corridors and flown slabs corresponding private rooms (first and second floors) and area galleries (third and fourth floor). The old roof structure of the theatre was composed by five 20.0m steel truss. This steel truss was simply supported at the ends.

3. STATE OF THE STRUCTURE

The structure of the theater shows structural damage in specific areas of it. The steel structure of the roof of the auditorium has obvious structural damage by fire (see Figure 12).

Figure 4. Internal view of the building.

Figure 5. Principal Hall

Figure 6. Back view of the building

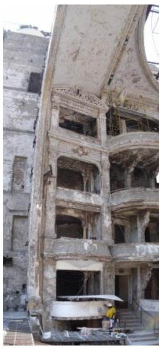

Figure 7. Back view of the building

Figure 8. Floor system of fourth level of galleries

Figure 9. Lateral view

Figure 10. Lateral view

Figure 11. Third floor column

Figure 12. Old steel roof structure of the building.

The front area of the building is better preserved and the one with less damage, however, the 4m height no confined front masonry wall above the upper story presents little horizontal cracks at the bottom showing its seismic vulnerability

The frames of the axes A and F of the second, third & fourth floor have 10cm infill walls made of brick units of plaster. Some of the beams and columns of these frames have exposed reinforcing steel and an advanced grade of corrosion. (see Figure 9 and 10).

Some columns located at the third and fourth floor of the auditorium area presents shear reinforcement distribution (stirrups) does not reach the standards for concrete in seismic regions (see Figure 11).

The 20cm concrete shear wall have a steel ratio of about 0.12%, which is much smaller than specified in current codes of reinforced concrete (see Figure 6).

Defects such as cracks and material deterioration due to steel corrosion were often repaired aiming only at aesthetics, rather than correcting the structural causes of damage. It is then important to make an exhaustive review of the structural elements and contrast this information with the analysis results. There is no evidence of differential settlement in the walls. There is no evidence of humidity.

4. SOIL CONDITIONS

The theatre stands on firm soil formed from well graded sediments like gravel, sands and pebbles of alluvial nature, coming from resistant rocks like granodiorite, gabbrodiorite, diorite and granite. The thickness of this dry and compacted material is estimated as several hundred of meters. For the purpose of the seismic analysis the soil profile is qualified as rigid, with a characteristic period less than 0.4s.

5. MATERIAL PROPERTIES

Most structural elements in the building were made of reinforced concrete. In order to quantify the strength and stiffness of concrete, compression tests were performed in CISMID. The maximum compressive strength of concrete were 9.07MPa (99kgf/cm^2) in 1st floor plate, 11.82MPa (113kgf/cm^2) in the 2nd floor plate, 25.1MPa (256kgf/cm^2) on the 2nd floor beams, 12.06MPa (123kgf/cm^2) on the 3rd floor beams, 6.96MPa (71kgf/cm^2) and 13.04MPa (133kgf/cm^2) in the columns of the 3rd and 4th floor, and 18.73MPa (191kgf/cm^2) in the columns of the structure above portal of the stage.

Figure 13. Concrete sample from concrete Wall in 1st floor.

Figure 14. Concrete sample from column of 4th floor.

Figure 15. Concrete sample from inverted beam of 4th floor.

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Table 5.1. Results of concrete sample in compression test

ID - SAMPLE	ELEMENT	LOCATION	COMPRESION RESISTANCE	
			(Mpa)	(kgf/cm^2)
D-1	Wall	1st Floor	18.14	185
D-2	Wall	1st Floor	7.94	81
D-3	Wall	2nd Floor	11.08	113
D-4	Beam	3rd Floor	12.65	129
D-5	Beam	2nd Floor	25.11	256
D-6	Column	3rd Floor	6.96	71
D-7	Column	4th Floor	13.04	133
D-8	Column	5th floor	18.73	191
D-9	Wall	1st Floor	12.94	132
D-10	Beam	4th Floor	12.16	124

The steels bar used in the building has a modulus of elasticity of 2.0E+6 GPa (2.1x10^{10} kgf/cm^2) and the yield stress is 274MPa (2800kgf/cm^2).

6. MICROTREMOR MEASUREMENTS

We performed a study of environmental microtremors purpose of obtaining fundamental modes of vibration of the structure at various points and levels of the building. The Fourier transforms obtained from measurements in the building have peaks at 3.3 – 3.5 Hertz (0.28s). This value of fundamental period has been the basis for calibration of the period of vibration of corroborating mathematical model of mass and stiffness ratios referred to in it.

Figure 16. Microtremor test of building

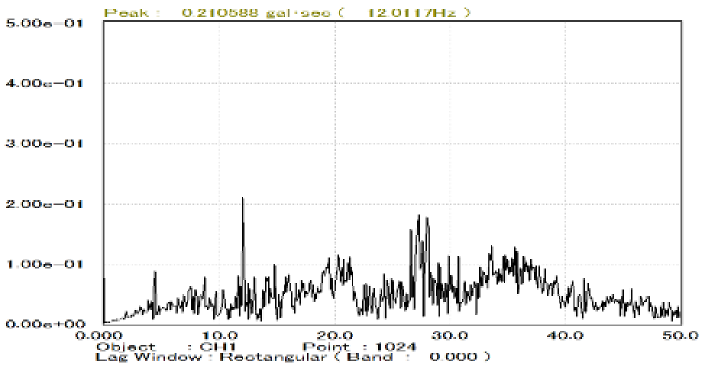

Figure 17. Fourier transform from measurements

7. STRUCTURAL MODEL

A special effort was made for the development of an appropriate mathematical model for the analysis of this building of complex geometry. The building mainly consists of walls of concrete and masonry, and concrete frames. A linear elastic analysis was considered a reasonable tool for the structural investigation, providing at least a basic understanding of the existing building response.

Figure 18 depicts the mathematical model for the analysis. 5,808 frame elements were used to represent beams and columns. The walls and the roof covers were modeled with 26,197 two dimensional shell elements.

The structural walls of the building are composed of at least two materials: brick and cement mortar. Although they are strictly non-homogeneous and anisotropic, they were modeled as homogeneous and isotropic, with equivalent linear properties based on tests. The walls were supposed fixed at their base. This is consistent with the observation that; even for extreme conditions, there are not vertical tensile stresses at the base of the walls

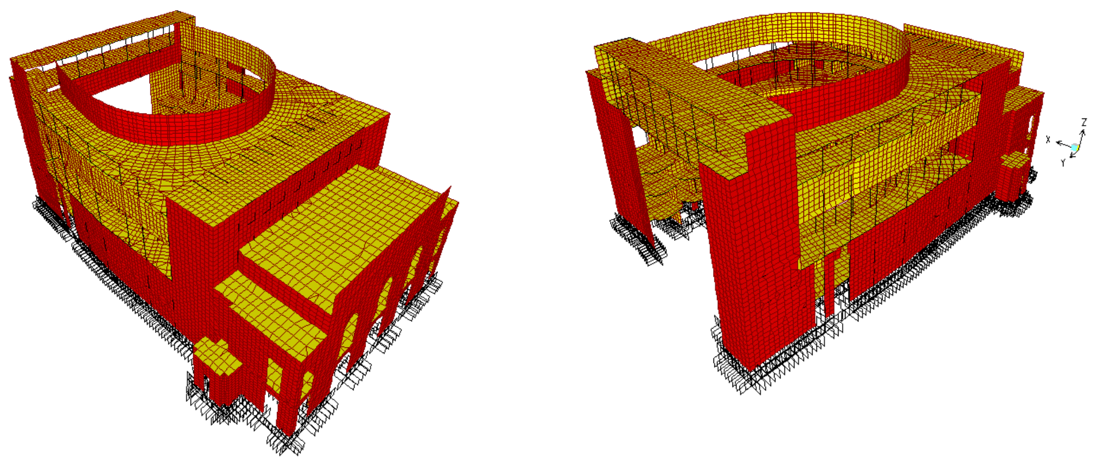

Figure 18. Mathematical model views of theatre structure

The plaster masonry walls of the third and four floors were no performed in the present analysis due to its architectonic more than structural nature.

The adequacy of the model was corroborated by its ability to predict not only the results of the microtremor tests but also to identify the portions of the structure which may be more severely stressed by earthquake loads, which are in agreement with the damage observed in past events.

8. NUMERICAL ANALYSIS AND RESULTS

An elastic analysis of the building was carried out for dead and live loads and earthquake effects. Wind loading is not important in this case. The analysis provided essential information about stress distributions and an estimate of the seismic response of the structure, helping to identify the most vulnerable zones in the building and to interpret the existent damage. The seismic analysis was based on a seudo acceleration spectrum as defined in the Peruvian seismic code, with the following parameters: Z=0.4 (zone factor), U=1.3 (importance factor), S=1.0 (soil factor), C=2.5Tp/T=2.5 (dynamic amplification factor), R=3/4*(4) (for reduction factor).

The fundamental periods obtained for the structure ware 0.36seg having coupled masses in both directions. The maximum displacements in the top of the building were 9.13cm in x direction analysis and 8.0cm; both of displacements occurred near to the stage of the theatre. In the front of the building the displacements were 2.6cm, greatly reduced due to the wall density and less story levels. The drift in the fifth level were 5.8/1000 at the back of the building and correspond to an intermediate damage level due to the low ductility of the structural elements.

Most of the concrete walls of the building have earthquake shear stresses which are well below their allowable capacity. However, there are some exceptions, described in what follows: the shear stress of the laterals walls in the first and second floor reaches 1.47MPa (15kgf/cm2) and 1.079MPa (11kgf/cm2) for the second floor. The shear capacity of wall in axis 5 results below than the seismic demand.

Some shear concrete shear walls form first and second floors have levels of stress because of earthquake effects that were over its capacity due to the low compressive strength of existing concrete with a poor reinforcement.

The flexural and shear capacity of the third floor columns in the area of audience no reach the seismic demand because of the less shear reinforced and poor concrete quality.

The front non confined masonry wall in the upper part of the building have a displacement that reach 4.5cm out of the plane and have no capacity to resist out of plane excitations.

9. STRUCTURAL RETROFITTING

The process of retrofitting the building pass over the correct the deficiencies found in the analysis of seismic vulnerability considering the building with a new roof steel structure at the top of the building described in what follows:

a) External retrofitting of concrete existing columns: to enhance the capacity of the columns it has been added external steel plates.

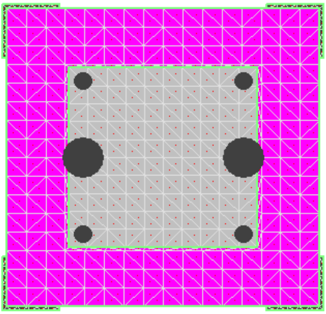

Figure 19. Cross section of column.

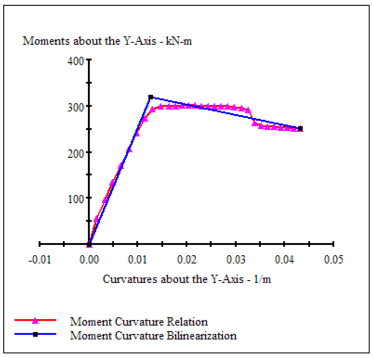

Figure 20. Moment Curvature of retrofitted concrete column.

Figure 21. View of column after retrofitting.

b) External retrofitting of existing concrete shear walls: It has been added steel mesh to the shear walls.

Figure 22. Retrofitting wall view.

Figure 23. Retrofitting external wall view.

c) Addition of new infill concrete shear walls in existing concrete frames in building in Y direction.

Figure 24. Infill shear wall Steel mesh in between frame elements.

Figure 25. Anchorage of steel bars in concrete frame elements.

Figure 26. New concrete infill shear walls in the structure.

d) Union of some floor slabs in the building

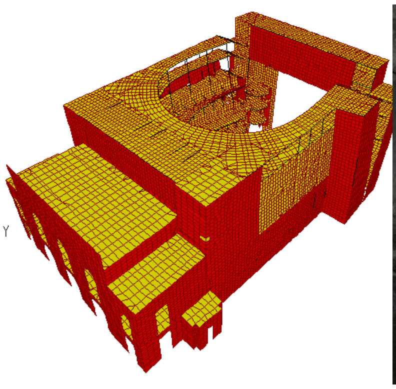

Figure 27. Numerical model to understand the effect of some different slabs in the structure.

Figure 28. Union of some slabs through anchors.

Figure 29. Union of some slabs through anchors.

e) Reinforcement of front wall of the building

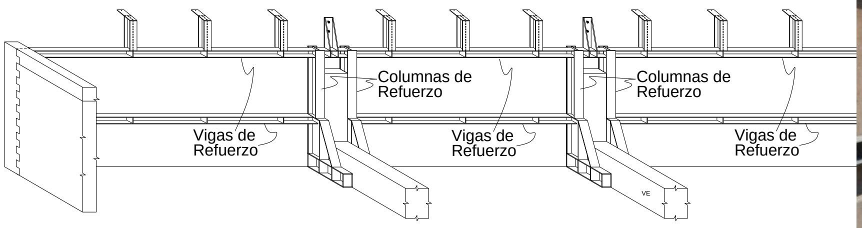

Figure 30. Isometric Draw of the Retrofitting project for the upper part of the front wall of the building.

Figure 31. View of the retrofitted wall.

10. CONCLUSIONS

Structural analysis showed that the former theater building had some deficiencies in seismic behavior due to their configuration and poor maintenance. A deficiency is in concrete frames of the third and fourth floors parallel Y direction on the axes A and F which in some cases have exposed reinforcing the presence of moisture. The no confined front wall above last level is vulnerable to earthquakes by large out of plane displacements. The columns inside audience room in third and four floors present seismic deficiencies in shear and bending demand. Some existing concrete walls in first and second floors of the building have poor steel ratios that not agree with nowadays standard requirements. The fire damage occurred in 1998 full coverage of the former steel, but not concrete structures.

The strengthening over the building tries to introduce some capacity to supply its earthquake deficiencies. The concrete walls of the first and second floors have more steel ratio. The columns of the third and fourth floor have been reinforced by external steel plates. Also, there have been some concrete walls between the frames fill perimeter in the third and fourth floors. It has also reinforced the front wall by a steel structure attached to it. The old steel roof has been replaced by a new one.

The reinforcement has tried to correct the major structural deficiencies found in the building and does not imply no need for further structural located intervention in case of an earthquake. However, adequate monitoring and maintenance of the structures could ensure the continued operation of the theater for many years, safeguarding the lives of people and historical value that holds the building.

ACKNOWLEDGEMENT

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