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## Seismic rehabilitation of typical schools in Puerto Rico

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# SEISMIC REHABILITATION OF TYPICAL SCHOOLS IN PUERTO RICO

BY:

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A project submitted in partial fulfillment of the requirements for the degree of

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IN  
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## **ABSTRACT**

Puerto Rico is located in an earthquake-prone region. Observations from recent studies of seismic activity in Puerto Rico have proven the expectation of a major earthquake that may occur at any given time. School buildings are one of the main infrastructures targeted by major earthquakes, due to the short column effect generated by the presence of partial masonry walls besides the columns. Unfortunately, the short column effect is usually not taken into consideration during the seismic analysis of the school buildings, and most of the public school buildings in Puerto Rico are built according to a limited number of prototype plans, hence, they are facing the same issue. As non-structural elements, masonry walls have a great influence on structures, they restrict the lateral displacement of the columns of the school buildings, which might cause a sudden shear failure of the columns. Schools are commonly used as shelters and they need to be in full operation and in proper conditions right after the earthquakes. The short column effect has caused severe damage in schools in past earthquakes, which leads to the reason why an adequate rehabilitation system for typical schools in Puerto Rico must be set in place. This research is designed to provide a solution for the seismic rehabilitation of typical public educational centers in Puerto Rico for the Department of Education. After all, the results obtained will be beneficial for existing schools and will serve as a guide that can be applied to further constructions of any similar structure.

## RESUMEN

Puerto Rico se encuentra en una región sísmica altamente activa. Observaciones de estudios recientes de actividad sísmica en Puerto Rico han demostrado la expectativa de un terremoto significativo que puede ocurrir en cualquier momento dado. Los edificios escolares son una de las principales infraestructuras objetivo de estudio durante los grandes terremotos, debido al efecto de la columna corta generada por la presencia de paredes de mampostería parcial al lado de las columnas. Desafortunadamente, es normal que el efecto de la columna corta no se tome en consideración durante el análisis sísmico de los edificios escolares, y la mayoría de los edificios de las escuelas públicas en Puerto Rico se construyen de acuerdo con un número limitado de prototipos, por lo tanto, un gran número de ellas se enfrentan al mismo problema. Como elementos no estructurales, las paredes de mampostería tienen una gran influencia sobre las estructuras, restringen el desplazamiento lateral de las columnas de los edificios escolares, lo que podría causar una rotura brusca en cortante de las columnas. Es común que las escuelas se utilicen como refugios y deben estar en pleno funcionamiento y en condiciones adecuadas inmediatamente después de los terremotos. El efecto de la columna corta ha causado graves daños en las escuelas durante terremotos pasados, lo que conduce a la razón por la cual se debe establecer un sistema adecuado de rehabilitación para las escuelas típicas en Puerto Rico. Esta investigación está diseñada para proveer una solución para la rehabilitación sísmica de centros educativos públicos típicos en Puerto Rico para el Departamento de Educación. Después de todo, los resultados obtenidos serán beneficiosos para las escuelas existentes y servirán de guía que puede aplicarse a otras construcciones de cualquier estructura similar.

*This work is exclusively dedicated to my late nonbiological parents, Mrs. Théard Dominique, born Guslane Romain, and Mr. Théard Dominique.*

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# 1. INTRODUCTION

## 1.1. RESEARCH SIGNIFICANCE AND MOTIVATION

Due to its geographic location, Puerto Rico has been affected by several natural disasters throughout history and until this day its exposure remains extreme and unpredictable, which may cause severe damages to human lives and property. It is situated in an earthquake-prone region, in the limit between the North American and Caribbean tectonic plates. The Caribbean plate is formed at an oblique zone where two tectonic plates meet and the denser, North American plate, is forced beneath the lighter, Caribbean plate. Thus, Puerto Rico finds itself in a highly active seismic zone.

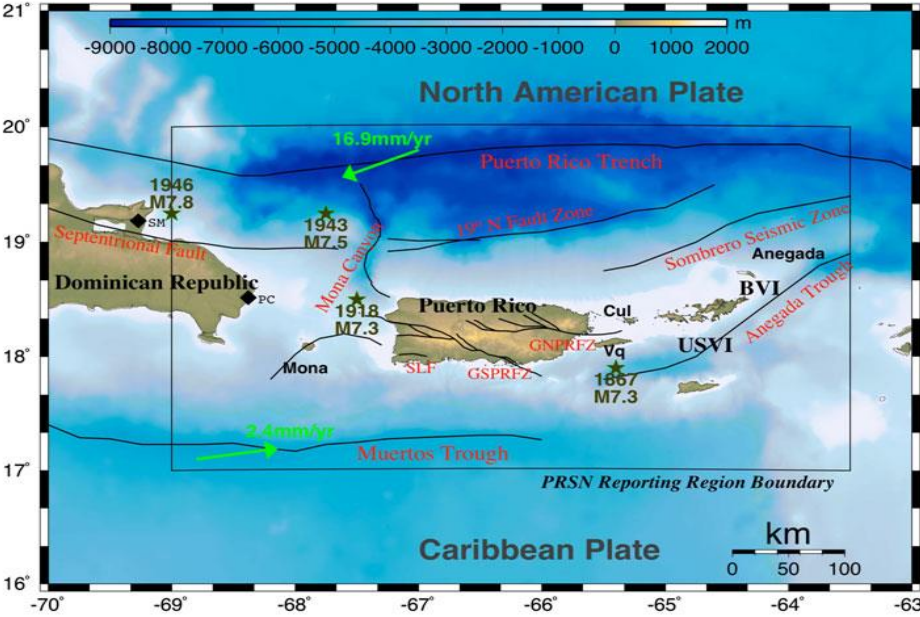


Figure 1. Map showing geological faults and approximate location of significant earthquakes from 1867 to 1946 (Seismic Network-PRSN).

According to recent studies about seismicity in Puerto Rico, it was revealed that the island can expect a significant earthquake, a so-called “big one”, which may occur at any moment due to the energy accumulated into the seismic faults. According to the Puerto Rico Seismic Network [1], a study of seismic vulnerability in Puerto Rico was conducted in 1987, showing a probability between 33 to 50% of a strong intensity earthquake (Intensity VII or more on the Modified Mercalli scale) for different areas from the island over a period of 50 years. Although, the past significant earthquakes were generated along different seismic faults, they occurred along a cycle of 51 to 117 years or an average of 83 years for destructive earthquakes. Hence, this earthquake is expected to be even stronger than past earthquakes that have affected the island since the American era (1898-present). According to the result of the increase in seismic activity in the faults of the southwest of the Island, seismic risk in Puerto Rico has increased from East to West and from North to South [2].

Listed below are the five significant earthquakes [3] that have impacted the island of Puerto Rico since the arrival of the Spaniards in 1493, causing severe damages.

- August 15, 1670: Although other major earthquakes have affected the island since the beginning of the colonization, the first significant earthquake on record in Puerto Rico occurred on this date, causing severe damages in the only two existing cities at that time, San Juan and San Germán. The specific magnitude of this earthquake could not be determined.
- May 2, 1787: The strongest earthquake struck the entire island on this day causing serious damages and the collapse of several buildings. It has been estimated as a 8.0 magnitude on the  $M_w$  scale and VIII on the Mercalli intensity scale. Its epicenter was probably located on the Puerto Rico Trench, to the north of Puerto Rico.
- November 18, 1867: The epicenter of this earthquake was in the US Virgin Islands between Saint Thomas and Saint Croix, with a 7.3 magnitude on the  $M_w$  scale and VIII on the Mercalli intensity scale. It generated severe damages, triggering a 20 feet high tsunami in the Virgin Islands and an estimated 3 to 5 feet tsunami in the municipality of Arroyo.

- October 11, 1918: The San Fermín earthquake of 1918, whose epicenter was in the *Cañon de la Mona*, 35 km to the northwest of *Aguadilla*, measured 7.3 on the Mw scale, while its intensity was estimated at VIII-IX (MM) in the West and V-VI in the East. It generated a tsunami that reached 20 feet in *Punta Agujereada, Aguadilla*, and 5 feet in *Mayagüez*. Aftershocks of this earthquake were felt for several months.
- July 28, 1943: A 7.5 magnitude earthquake on the Mw scale struck at the Northwest of Puerto Rico. Although many people felt it, there were no important damages.

According to data by the United States Census Bureau (USCB) in 2018, the population of Puerto Rico has drastically increased from 2,214,169 persons in 1950 to 3,195,153 persons in 2018, which represents a 44.30% [4]. This increase in population has been accompanied by an increase in infrastructure. Should an earthquake similar to those in the past occur today, there would be a higher risk of damage. Having to endure another similar event will come with great consequences, which can be extremely devastating and may lead to high death tolls, damaged structures, and certainly major disastrous effects on the economy of the island.

Despite the effort undertaken by authorities, as well as construction and design specialists to establish construction practices on the island that meet the requirements of the building codes for seismic designs, poor construction practices are still being executed. Hence, the structures are more vulnerable than what they should be to seismic events. On the other hand, most of the infrastructure currently in use were constructed 60 to 70 years ago. Besides the aging factor, they do not meet the requirements of the current and up to date Building Codes [5]. The American Society of Civil Engineers (ASCE) standard, ASCE-7, is used as a reference in the United States for appropriate building classification criteria. According to the ASCE-7 provisions, America's Infrastructure is classified based on the risk to human life, health, and safety associated with their damage or failure [6].

Seismic faults occurring inside the island were not considered in the 1970s and 1980s, either because they have not been discovered, or were not considered active at that time. It is not until 2000 and 2001, the United States Geological Survey (USGS) by means of the Seismic Network of Puerto Rico, conducted a study in *Cabo Rojo* and have discovered a seismic fault named *Lajas*

fault. This fault was considered in the code that is currently in Puerto Rico, also in the seismic Hazard Maps. Since then, numerous investigations have been performed around the seismic activity of the existing faults in *Juana Diaz*, between *Cañas* River and *Descalabrado* River. However, a new fault was discovered. It is suspiciously located by *Guayama*, which diagonally crosses the island, and exits in an area near *Añasco*, another branch near *Aguada*, another near *Mayagüez* and *Hormigueros*. Unfortunately, no further relevant studies could have been realized to certify this fault; it is not included in the Hazard Map of Puerto Rico, hence, it represents an immense danger to the infrastructure of the island. An earthquake generated from this fault that crosses the island can generate a strong velocity pulse and release a great amount of energy [7].

Considering how important it is to provide the Island of Puerto Rico with a design spectrum compatible with the actual building codes, the Puerto Rico Seismic Network (PRSN), a dependency of the Department of Geology of the University of Puerto Rico at Mayagüez, after its establishment in the mid-1970s, operates about 30 seismic stations in Puerto Rico and the Virgin Islands to continuously detect the movements of the earth. The Puerto Rico Strong Motion Program (PRSMMP) under the Civil Engineering and Surveying Department has 114 free field strong motion stations and 21 instrumental structures in Puerto Rico and four surroundings countries. Since no strong earthquakes have been recorded so far by the PRSN and PRSMMP, engineers are forced to either, use records of non-local earthquakes for the analysis and seismic design of structures, or extrapolate low intensity records.

School buildings in Puerto Rico are commonly used as shelters and they need to be in full operation immediately after earthquakes. Most of the public school buildings in Puerto Rico are built according to a limited number of prototype plans. Over the years, these prototypes have remained the same while the structural design requirements have been increasing with the lessons learned in past earthquakes, and so has been the increment in requirements in the current seismic design codes.

One of the most fatal causes to a structure during an earthquake is the short column effect. It is very common in public schools, not only in Puerto Rico but also other neighboring Caribbean countries. Because of the lateral flexibility in the longitudinal frame system, combined with the

mid-height masonry walls, some columns have restrictions in their lateral displacement. Most of this lateral displacement is concentrated on the last quarter height of the column, thus, the structure can suffer major damages during an earthquake. Short column effect becomes harmful as it is usually not considered in the analysis and design phases of the structure by underestimating the influence of the non-structural components. This poses a great danger to the public and great effort must be put into retrofitting and rehabilitating existing infrastructures.

Many investigations have emerged around this topic on how to address the short column effect or more effectively, how to completely avoid it. Unfortunately, many buildings in use nowadays are facing this issue, mostly, because designers and construction specialists were not aware of the influence of non-structural components of a building which give birth to the short column. From the damages observed by this phenomenon during earthquakes, it is imperative to address it.

Due to the concern of the Puerto Rico Strong Motion Program, this investigation is specifically directed to provide an alternative for the seismic rehabilitation of typical schools for the Department of Education of Puerto Rico.

## **1.2. RESEARCH OBJECTIVES AND SCOPE**

The intention of this research is to provide a seismic rehabilitation strategy for typical schools in Puerto Rico. A dynamic and nonlinear static analysis of a typical school in Puerto Rico will be performed in order to determine its actual structural condition. The architectural and structural drawings of this structure were obtained from the Public Buildings Authority of Puerto Rico. The results obtained should apply to all Puerto Rican schools that follow the same prototype design, depending on their location.

The scope of this research is to analyze the behavior of this structure by using non-local seismic records. By performing the nonlinear static and dynamic analyses, it will be possible to predict collapse mechanism and structural behavior. This will help to provide a solution for the seismic

rehabilitation and retrofitting plans for the school buildings following the prototype design for the Department of Education of Puerto Rico. The outcome of this research will benefit school buildings in use and will be available to be applied to further constructions of any similar structure.

Specifically, the objectives are as follows.

- Review the structural design and detailing provided in the drawings of a typical public school in Puerto Rico to create a 3D model of the structure using SAP2000.
- Perform nonlinear static and linear dynamic analyses to determine the collapse mechanism and the failure mode of the structure.
- Provide a strategy for seismic rehabilitation and retrofit for the Department of Education of Puerto Rico based on the results of these analyses.

### **1.3. SHORT COLUMN EFFECT**

Earthquake transfers energy to structures. Structures with adequate design should be able to properly absorb or consume the incoming energy through its members in order to prevent severe damages or worse, collapse. As for the columns, this energy is dissipated through their displacement. The ability to consume energy in the short columns is very low compared to the other columns since the displacement of the columns is proportional to their lengths. Within the same story, columns with the same cross-section will have similar horizontal displacement demand even if they have different heights (Figure 2).

Stiffness of a column is its resistance to deformation. The stiffer a column is, the larger deformation force it attracts. The stiffness of a column is inversely proportional to its height; hence, the short columns are stiffer than the other columns. Due to this fact, short columns are the most critical components because they suffer more damages during an earthquake. Short columns must be properly designed to resist such a large earthquake force. This is the "Short

Column Effect". The damage is often presented in an X-shaped crack, due to the shear failure in both horizontal directions.

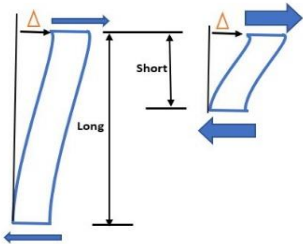


Figure 2. Short column vs long column.

**1.3.1. SHORT COLUMN EFFECT FORMATION**

The short column effect is one of the structural defects most frequently noticed after a destructive earthquake. It is commonly noticed in buildings with a mezzanine floor (Figure 3) , buildings on a sloped ground where the columns have different heights within the same story (Figure 4), or due to the presence of partial masonry walls between the columns that vertically cover the space from the floor to the threshold of a window (Figure 5). The third scenario is typically noticed in school buildings for the needs of lighting and ventilation.

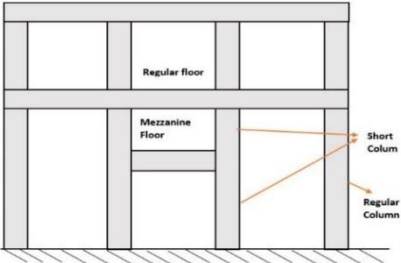


Figure 3. Building with a mezzanine floor.

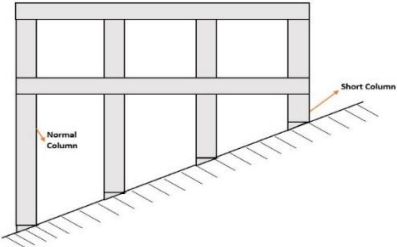


Figure 4. Building on a sloped ground.

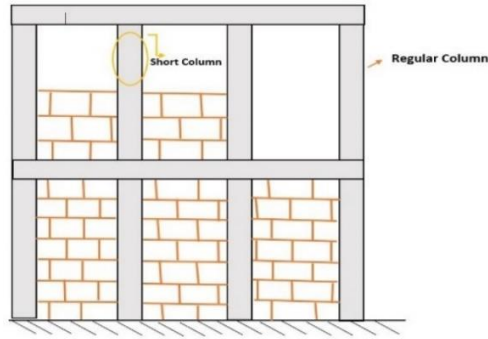


Figure 5. Building with partial masonry walls.

### 1.3.2. MATHEMATICAL ILLUSTRATION OF A SHORT COLUMN EFFECT

The illustration in Figure 6 displays an earthquake operating as a horizontal force on the slab of a building on a sloped ground. Some might assume the columns A and B would carry the same amount of the force  $F$ , which means, each column would carry half of the force  $F$ . Unfortunately, the two columns are not carrying the same force since they do not have the same stiffness.

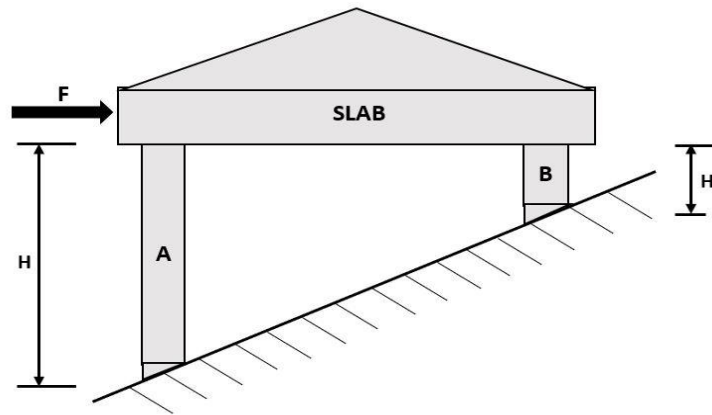


Figure 6. Building on a sloped ground.

The lateral stiffness formula of a column for a shear building is given by  $K = 12 \frac{E * I}{H^3}$  (force/length), where  $E$  is the modulus of elasticity of the material,  $I$  is the moment of inertia of the cross section, and  $H$  is the height of the column. Both columns have the same cross-section and the same

material. If the height for column A is  $H$ , then, its stiffness is  $K_A = 12 \frac{E * I}{H^3}$ . If the height for column B is  $H'$ , which is a third of  $H$ , the stiffness of the column B will be equal to:

$$K_B = 12 \frac{E * I}{(\frac{1}{3}H)^3} = 27 \times ( 12 \frac{E * I}{H^3} ) = 27 K_A$$

Therefore, column B is 27 times stiffer than column A, hence, it will attract 27 times more force than column A.

### 1.3.3. MATHEMATICAL EXAMPLE OF A BUILDING PRESENTING STRUCTURAL DAMAGES DUE TO THE SHORT COLUMN EFFECT

A building near the Olive View Hospital in Sylmar (a community in the city of Los Angeles) had significant structural damages during the January 17, 1994 Northridge earthquake. The building of one floor and two bays had the particularity that one column was shorter than the others. The failure occurred specifically in this column.

#### Data:

The structure is modeled as a “shear building” with one degree of freedom. The columns had the same cross-section. The columns’ reinforcement was 4 #8 steel bars grade 60, and #3 ties with 8-inch spacing from center to center. The compressive strength of concrete is 3 ksi.

$H_1 = 12$  ft                      (Height of a regular column)

$H_2 = 8$  ft                        (Height of the short column)

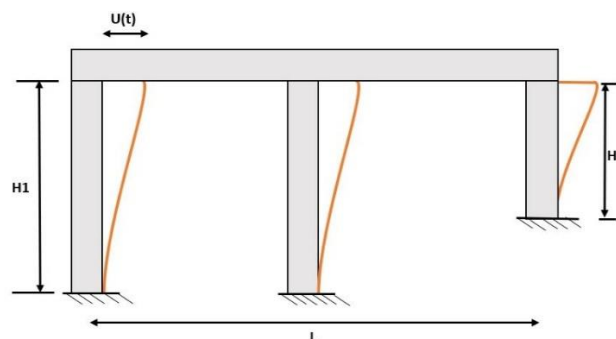


Figure 7. Side view of the building near the Olive View Hospital, in Sylmar, Los Angeles, USA.

The shear capacity of a column can be calculated from the given information using the following formula:

$V_n = V_c + V_s$ , where

$$V_c = 2 \sqrt{f'_c} * b * d = 2 \sqrt{3000} * 12 * (12 - 2.5) = 12.49 \text{ kip,}$$

$$\text{and } V_s = \frac{A_v * F_y * h * d}{s} = \frac{2 * 0.11 * 60 * (12 - 2.5)}{8} = 15.68 \text{ kip.}$$

Hence,  $V_n = 12.49 \text{ kip} + 15.68 \text{ kip} = 28.17 \text{ kip}$

**Stiffness of a column in a shear building is given by:**

$K = 12 \frac{E * I}{H^3}$ , where E is the modulus of elasticity of the material, I is the moment of inertia of the cross section, and H is the height of the element.

$E = 3,600 \text{ ksi}$  (Modulus of elasticity of concrete)

$$I = \frac{b * h^3}{12} = \frac{12 * 12^3}{12} = 1,728 \text{ in}^4$$

$$\text{Thus, } K_1 = 12 \frac{E * I}{H_1^3} = 12 \left( \frac{3,600 * 1,728}{(12 * 12)^3} \right) = 25 \text{ k/in (Stiffness of a regular column)}$$

$$\text{And } K_2 = 12 \frac{E * I}{H_2^3} = 12 \left( \frac{3,600 * 1,728}{(8 * 12)^3} \right) = 84.38 \text{ k/in (Stiffness of a short column)}$$

$$K_2 = 3.375 K_1$$

**The shear demand of a column is given by the stiffness multiplied by the maximum lateral displacement:**

$V_c = K * U_{max}$ . If the maximum displacement relative to the base of the structure, obtained from the Northridge-Sylmar earthquake response record is  $U_{max} = 0.487 \text{ in}$ , then:

The shear demand of a regular column is:

$$V_{c1} = K_1 * U_{max} = 25 \text{ k/in} * 0.487 \text{ in} = 12.175 \text{ k} < V_n$$

The shear demand of a short column is:

$$V_{c2} = K_2 * U_{max} = 84.375 \text{ k/in} * 0.487 \text{ in} = 41.09 \text{ k} > V_n$$

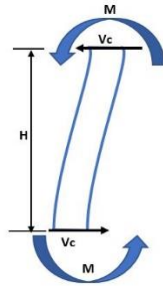


Figure 8. Free-body diagram of a column.

The moment demand of a regular column is:

$$V_{c1} * H_1 - 2 * M_1 = 0$$

$$M_1 = \frac{V_{c1} * H_1}{2} = \frac{12.175 * 12}{2} = 73.05 \text{ k-ft}$$

$$\frac{V_{c1}}{M_1} = \frac{12.175}{73.05} = \frac{1}{6}$$

The moment demand of a short column is:

$$V_{c2} * H_{12} - 2 * M_2 = 0$$

$$M_2 = \frac{V_{c1} * H_2}{2} = \frac{41.09 * 8}{2} = 164.36 \text{ k-ft}$$

$$\frac{V_{c2}}{M_2} = \frac{41.09}{164.36} = \frac{1}{4}$$

**Conclusion:**

The shear demand calculated for the short column is 41.09 kip while the shear capacity of the design is 28.17 kip. The shear capacity is independent of the height of the column. So, the shear capacity is the same for both columns. However, the shear demand is 3.375 times higher in the shorter column. Also, when the lateral load is applied, the “shear demand vs applied moment” ratio in the short column increases faster than the one in the long column. The short column will present shear failure. Thus, the damage caused is due to the short column effect.

## **1.4. DISSERTATION OVERVIEW**

The dissertation of this research initiative to undertake the short column effect featured in public schools in Puerto Rico by providing a solution for the seismic rehabilitation is divided into five chapters. A summary of the content is described as follows:

Chapter 1 introduces the topic. The purpose of this research and the accompanying motivation, the objectives and scope are described. Practical explanations followed by a mathematical illustration are presented, and finally, an example of a building presenting structural damage due to the short column effect during an earthquake is mathematically discussed for a better understanding of the topic.

Chapter 2 presents the literature review, examples of damages caused by the phenomenon studied during past earthquakes, research carried out regarding the short column effect, and finally, some alternatives proposed to reduce the effect or to avoid it.

Chapter 3 presents the procedures leading to this investigation, detailing obtained from the drawings, the assumptions for the conception of the analytical 3D model in SAP2000, and the analyses to be performed.

Chapter 4 presents the results of the analyses and the interpretation of these results.

Chapter 5 provides a summary of the methodology applied, the conclusions from the results obtained and recommendations for future works.

## **2. LITERATURE REVIEW**

---

### **2.1. INTRODUCTION**

This chapter is divided into three sections. The first section contains a summary of the behavior of short columns during earthquakes. In the second section, a summary of some investigations effectuated on short columns, as well as their proposed solutions to minimize the effect of short column are presented.

### **2.2. SHORT COLUMNS BEHAVIOR DURING EARTHQUAKES**

Short columns suffer severe similar damages during earthquakes. Considering the issue and risk associated by the phenomenon of short column effect, special attention is required to address this behavior that is called to be disastrous as they undergo brittle shear failure. It must be considered during the analysis and design phase, or better yet, it should be completely avoided in areas where there is high seismic activity. Listed below are examples of behavior of short columns during past earthquakes.

### 2.2.1. ADANA-CEYHAN EARTHQUAKE, TURKEY, 1998

An earthquake with a 6.2 moment magnitude and an IX maximum intensity occurred on June 27, 1998, at 4h56, local time, in southern Turkey [8], historically known as Cilicia near the city of Ceyhan (population 100,000). The European macro seismic scale classifies an earthquake with such intensity as destructive. Reports revealed the coordinates of the epicenter at 36.95°N and 35.31°E and the focal depth at 23 km. Followed by numerous aftershocks, the event caused severe damages as a consequence of a strong motion with a PGA of 0.28 g. According to the reports, 12 buildings collapsed, 120 others were classified as moderately damaged in Ceyhan. Also, in the nearest provincial capital, Adana (population 1.2 million) there were at least 145 deaths, 1,500-1,600 people injured, and 8,800 people displaced. The losses were estimated at US\$1 billion. Inadequate structural building designs were mostly the cause of the damages that left many thousands homeless in Adana, and Ceyhan, where the presence of short column effect was noticeable. This effect is caused by masonry walls that reduced the effective height of the columns.



Figure 9. Short Column failure in the Adana Ceyhan earthquake, Turkey, 1998.

### 2.2.2. IZMIT EARTHQUAKE(KOCAELI), TURKEY, 1999

The 1999 Kocaeli earthquake struck the Kocaeli province of northwestern Turkey on Tuesday, August 17, 1999, at 3:02 A.M., local time with a moment magnitude of 7.4 [9]. The PGA was 0.4g. Records displayed a fault rupture of 110 km, approximately. The Kocaeli province is considered as the area with the densest population of Turkey and most important area of the Turkish industry. As it is presumed to drift slowly, the Earth's crust unexpectedly ruptures along a western branch of the 1,500-km-long North Anatolian fault system, causing the 1999 Kocaeli earthquake. The Turkish government estimated 17,127 deaths, 43,953 injuries, and the displacement of more than 250,000 people because of this event. Approximately 121 tent cities were required for emergency housing, and 214,000 residential units and 30,500 business units were damaged. The affected buildings were mostly built using reinforced concrete. Poor construction practices were performed; thus, those buildings were not compliant to the local building code in vigor. The strong velocity and long duration of the pulse of the ground motion were important factors that caused damages and collapses of the buildings, but not the peak ground acceleration (PGA) registered near the fault, which were below average for an earthquake of such magnitude.



Figure 10. Damages due to the short column effect in the Izmit Earthquake, Turkey 1999.

### 2.2.3. CHI-CHI EARTHQUAKE, TAIWAN, 1999

On September 21, 1999, a disastrous earthquake of  $M_w = 7.6$  followed by large aftershocks that occurred in the city of Chi-Chi, the center of Taiwan. More than 2,400 people died, some 10,700 were injured, over 8,500 buildings were destroyed and another 6,200 were seriously damaged. The duration of the strong motion was about 30 seconds. Records at the north end of the strong ground motion rupture zone related PGAs of 0.4 g, and from 1.0 g to 2.0 g at the south end of the earthquake rupture zone. Compared to the 1999 Kocaeli earthquake in Turkey, the energy released by the Chi-Chi earthquake was 50% greater. This earthquake caused damages in economic terms and human lives in structures as schools, bridges, dams, embankments, and residences. Heavy losses to schools and public facilities were disclosed, where 43 schools in Nantou and Taichung were destroyed and 700 schools were damaged [10]. The shear column effect was one of the main anomalies noticed in schools, in the direction parallel to the exterior corridor outside the classrooms where windows above half-height infill shortened the effective length of almost all the columns [11].



Figure 11. School damaged due to the short column effect.

#### 2.2.4. TUCACAS EARTHQUAKE, VENEZUELA, 2009

On September 12, 2009, an earthquake of magnitude 6.2 Mw occurred in Tucacas, Venezuela, located about 50 km from the epicenter [12]. It severely damaged several buildings. Short column effect was the protagonist and was generated by the presence of a reinforced concrete wall in one of the structural axes, which caused the failure of all the columns of that axis. On the upper floors, the columns of that same axis showed cracks of horizontal orientation, leaving in evidence the total degradation of the ability to support the "short columns" of the lower level and that portion of the structure "hung" from the upper columns generating traction. Fortunately, the structural redundancy and the low intensity of the earthquake in the locality, allowed tensions to be redistributed towards the columns of the neighboring axes, avoiding the collapse of the structure. This building was disabled immediately after the earthquake occurred due to its precarious condition.



Figure 12. Short Column effect generated by reinforced concrete walls in a building in Tucacas, Venezuela during the September 2009 earthquake.

### 2.3. PREVIOUS WORKS

Several types of research were performed to study the short column effect. Listed below are some relevant researches to showcase proof.

In 1984, Umehara and Jirsa [13] conducted research on short columns in reinforced concrete buildings and observed their performance under seismic events. They ran 20 experimental tests to define the shear strength of the short columns under loadings using interaction curves. They concluded that when short columns are submitted to loads in both directions, failure occurs when the maximum capacity in a specified direction reaches the deflection at the maximum shear capacity of columns under loadings in one direction. They also evaluated shear strength using elliptical or circular interaction diagrams.

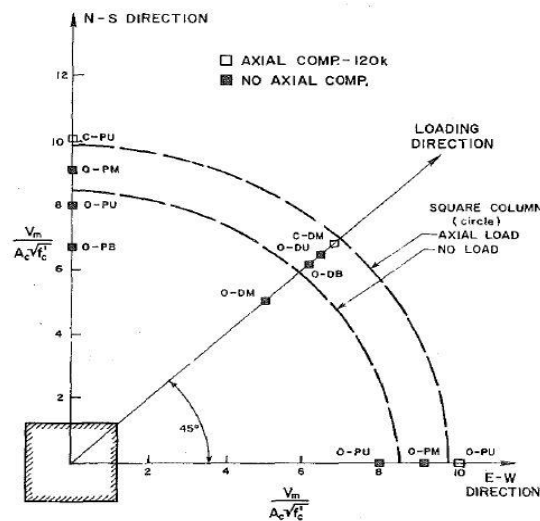


Figure 13. Shear Strength vs direction of loading - Square column.

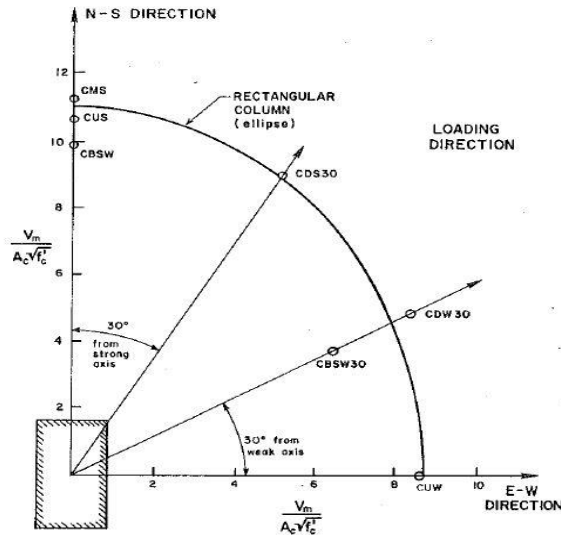


Figure 14. Shear Strength vs direction of loading - Rectangular column.

In 2005, Rojas Mercedes [14] developed easy-to-use graphs to determine the transition between shear and flexural failure in short columns. Among the 30 columns of the structure studied, the original slenderness ratio of the external columns was 4.50. When masonry walls were inserted, the current slenderness ratio of the external columns became 3.70, which is slightly lower than 4.50, leading to flexural failure. Moreover, the original slenderness ratio for the internal columns was 4, while their slenderness ratio was 2 when masonry walls were inserted, which is much smaller; hence, shear failure may occur. 13 of 15 internal columns were short, therefore, a 43% of the 30 columns were short. For a 43% percentage of short columns and 0% of walls, this school falls into the region of severe damages. Cases in which the columns are being confined by walls both in their weak and strong direction were considered. In fact, the presence of the short column effect depends on the percentage of walls in the direction of interest. Therefore, a percentage of walls (walls area over total area considered from the top view) greater than or equal to 0.45% considerably lowers the expected damage produced by short column effect from minimal to null. He later applied the same methodology to the schools found in the review of drawings, which led to the discovery that most of the schools in Puerto Rico hold a configuration that makes them vulnerable to failure due to the short column effect. He concluded that the low percentage of reinforced concrete walls in Puerto Rican schools increases the possibility of severe damage. Although the procedure used was specifically designed to study typical school columns

in Puerto Rico, some adjustments to the methodology, such as expanding the dimensions of the cross-sections of the columns can be made to make it applicable for other types of structures.

After Rojas's research in 2017, Li and Hwang [15] proceeded by proposing a unique model based on lateral load displacement curves of the short columns. The authors simulated the seismic behavior of the short columns, which failed in shear with a height-to-depth proportion of less than 2. A trilinear relation was proposed based on the observations obtained from the experimental results to remove the complexities from the analytical model. A report was published to compare the experimental models and verify the relevance of the suggested model. They mentioned the significance of stiffness to reach the shear capacity of the short columns, meaning that short column stiffness varies before and after shear cracking. Thus, the failure mode in the short column is ruled by fragile shear failure.

In 2018, Beauperthuy and Urich [16] also studied the behavior of short columns, one of the most common causes of catastrophic damage in seismic events. According to the authors, the best and the most economical solution is to avoid the short column effect by conveniently separating the columns from other components (Figure 15), which limits the free deformation of the columns. The other solution is to incorporate the non-structural components in the analysis and design since the real problem is not that the columns are "short", but that the influence of non-structural components has not been considered in the project. When realistic conditions are considered in the models, non-structural and structural components can be properly designed. Hence, deformation compatibility can be verified for the overall performance of the structural system and the rest of the building components. They proposed to incorporate infill wall segments on both sides of the short columns for the rehabilitation and reinforcement of the structure (Figure 16).

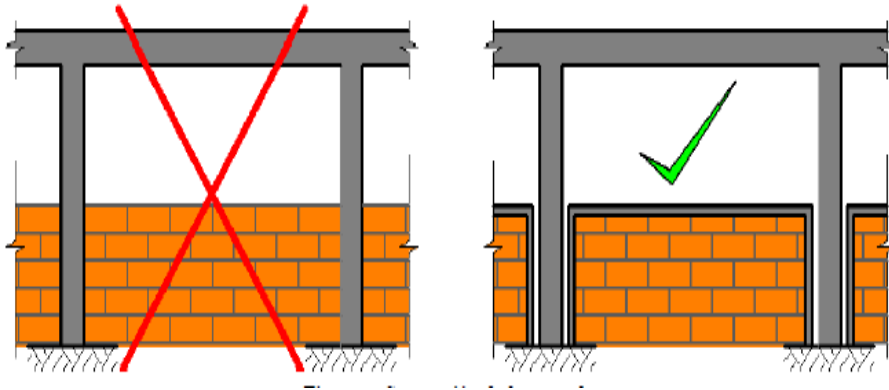


Figure 15. Separation of the columns from the masonry walls.



Figure 16. Incorporation of infill wall segments on both side of the short columns.

All the above-mentioned studies have examined the short column effect and all have found it is necessary to improve the performance of these elements. Because they were not considered at the time of the design and analysis, the short column effect represents a great danger. It is the construction and design specialists' responsibility to identify buildings that are already in use and exposed to these problems and to ensure that the construction rules are followed in the future. Evidence suggests the application of adequate design and construction methods to rehabilitate these buildings, and thus, improve their structural behavior.

## 2.4. PREVENTION OF THE FORMATION OF THE SHORT COLUMN EFFECT

Damages observed by the short column effect during past earthquakes and the researches generated on this topic clearly stipulate that this anomaly should be avoided at all costs. When referring to new buildings, it can be avoided from the architectural design, and as for the existing buildings, different solutions of seismic rehabilitation and retrofit can be used to improve the resilience of the building.

Nowadays, many building codes have considered the formation of the short column effect and have provided specified design requirements so that the short columns, if present, are properly designed to be compliant to the building codes in vigor. Special requirements within those codes are established to address the issue. For instance, the building code of India (India IS: 13920-1993) belongs to that list. Instead of the usual stirrups at the level of the confinement zone for regular columns, the Indian Building Code establishes special confining reinforcement requirements for short columns. This special reinforcement must be applied throughout the entire height of the column, and must be extended into the column vertically above, as it is shown in Figure 17 [17].

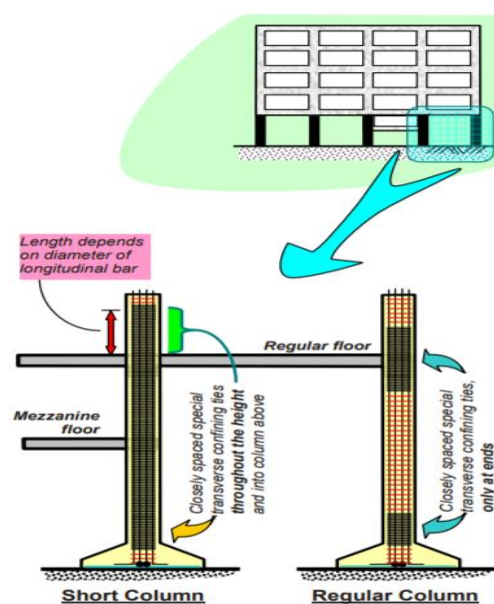


Figure 17. Details of reinforcement of a short column versus a regular column.

Besides the Indian Code, a study conducted by Muñoz, et al. [18] on Peruvian school buildings, in 2004, states that the educational infrastructure built before 1997 has suffered considerable damages during recent earthquakes, while those designed and built under the 1997 Peruvian Seismic Code have not suffered any sort of structural damages in the 8.4 Mw 2001 earthquake. One of the important reforms established in the 1997 Peruvian Seismic Code is the increment in stiffness in the longitudinal direction. In fact, the short column effect in school buildings arises from a lack of stiffness in the longitudinal direction, and Peruvian school buildings were, traditionally, flexible in the longitudinal direction. Hence, the 1997 Peruvian Seismic Code takes into consideration the formation of the short column effect in school buildings by increasing the demand in stiffness in the longitudinal direction while the transverse direction remains the same since the structural arrangement is compliant to the seismic requirements of the new code. The June 2001 earthquake has extensively damaged traditional school buildings in Peru while the modern school buildings performance has been excellent.



Figure 18. School building before 1997 damaged in the Atico earthquake, Peru, 2001.



Figure 19. Modern school building, after 1997, undamaged in the Atico earthquake, Peru, 2001.

## 3. METHODOLOGY

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### 3.1. INTRODUCTION

In this chapter, the procedures that lead to the modeling of the analytical prototype are discussed, considering the modeling of its different components, basic assumptions, material properties, design loads, earthquake records selection, and the analyses to be performed.

### 3.2. GENERAL CHARACTERISTICS OF THE PROTOTYPE MODEL

As a brief description of the school building prototype provided by the Public Buildings Authority of Puerto Rico, the Monserrate De León Irrizary High School, also known as the *Boquerón* High School, consists of four modules that work independently. It is a public school that belongs to the Department of Education of Puerto Rico. This school, considered as the best in the town of *Cabo Rojo*, was established in 1988 and the first graduating class was in 1990. The school is located on highway 101, Km. 18 Hm. 2, in *Cabo Rojo*, Puerto Rico. The two-level school has offices, classrooms, laboratories, lunchroom, cafeteria, library, food storage, bathrooms, and corridors. Based on the information obtained in the architectural and structural drawings of the typical Puerto Rican school, a 3D model of the structure was created using SAP2000.

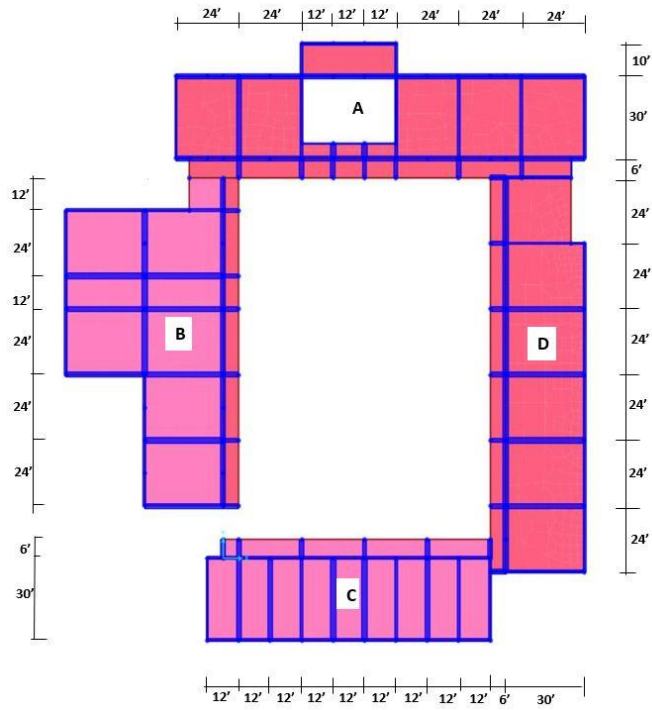


Figure 20. First floor top view on SAP2000.

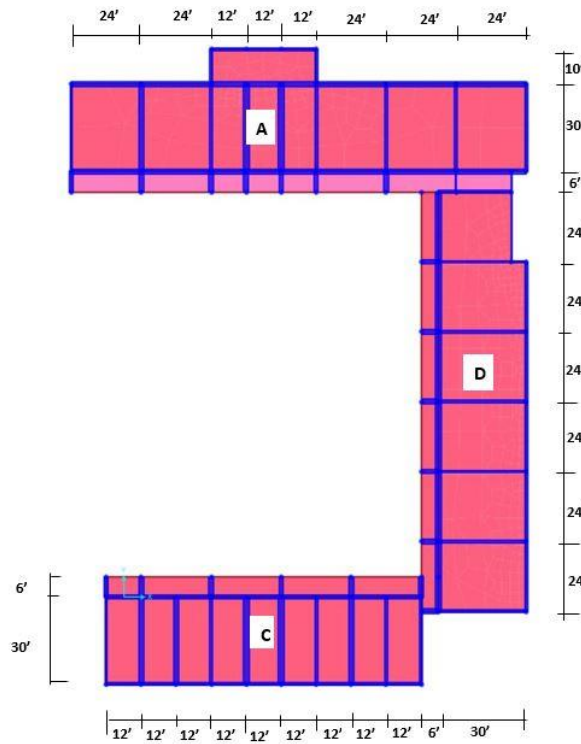


Figure 21. Second floor top view on SAP2000.

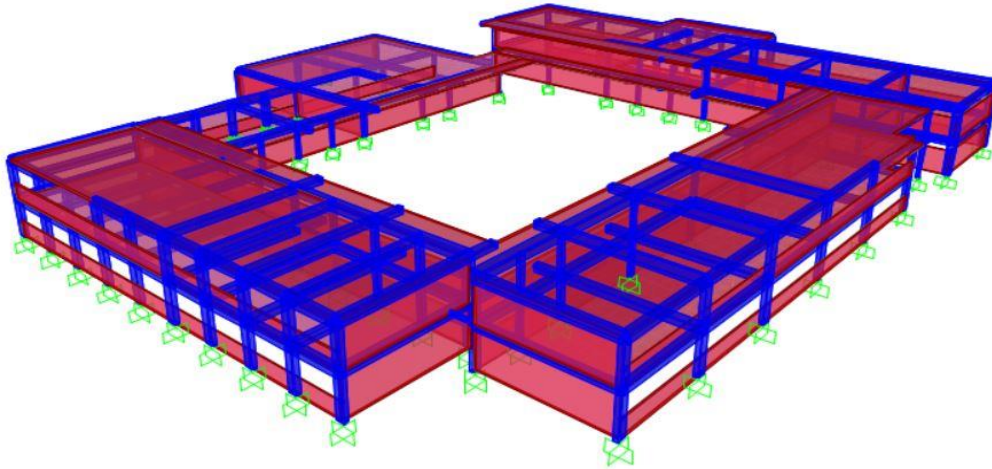


Figure 22. 3D model on SAP2000.

### 3.2.1. SELECTED MODULE FOR THE ANALYSES

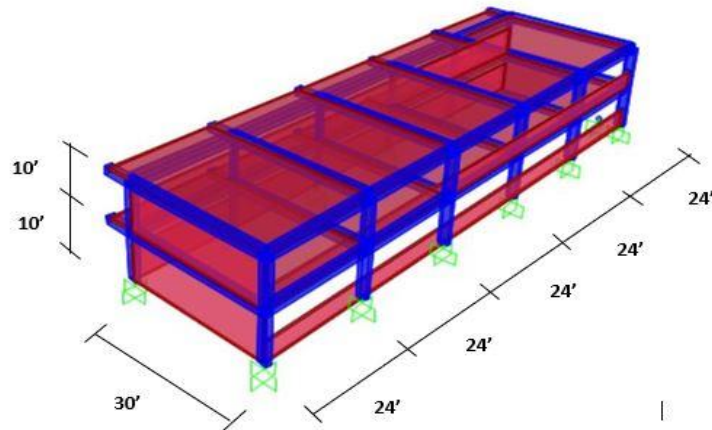


Figure 23. Module D.

The selected module carries no walls in its longitudinal direction, and it contains two longitudinal frames. Each frame will be studied individually. The frame in front of the classrooms, which also carries the corridor slab is labeled as “Frame I”, and the other one in the back of the classrooms is labeled as “Frame II”. The slab thickness is 6” for level I and II. The columns were assumed fixed at the base. Soil interaction was not considered.

**FRAME I**

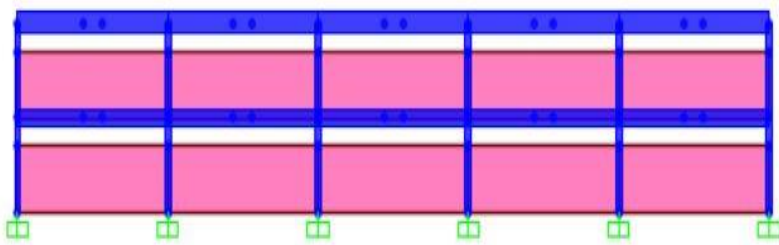


Figure 24. Frame I 2D model in SAP2000.

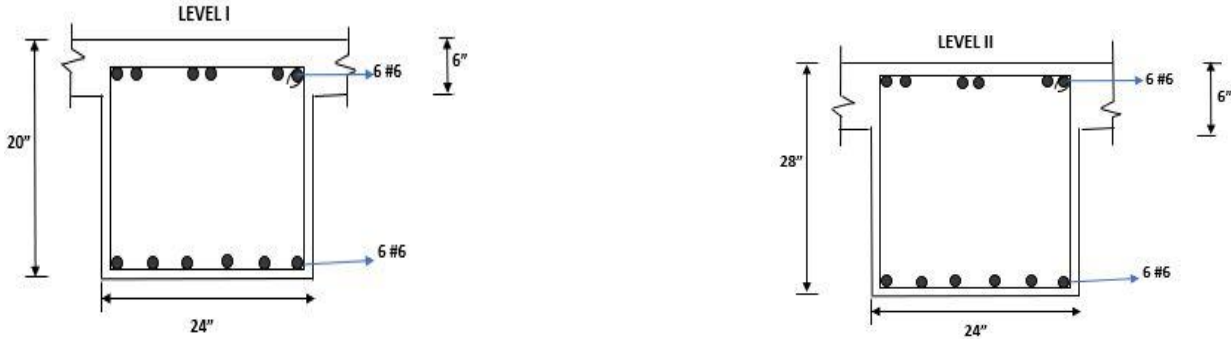


Figure 25. Longitudinal beam section and reinforcement of the Frame I.



Figure 26. Interior and exterior columns section and reinforcement.

**FRAME II**

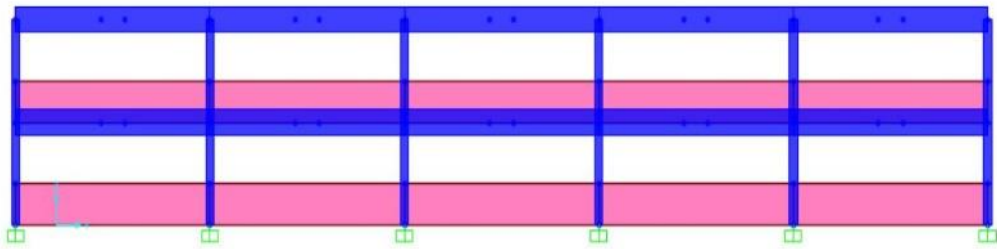


Figure 27. Frame II 2D model in SAP2000.

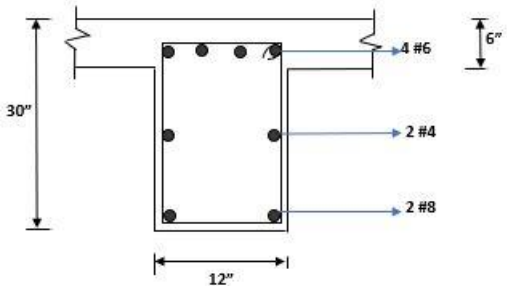


Figure 28. Longitudinal beam section and reinforcement of the Frame II.

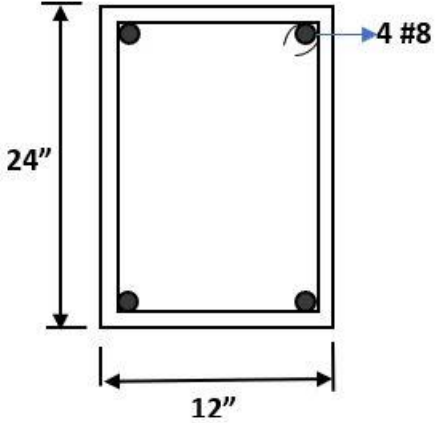


Figure 29. Interior and exterior columns section and reinforcement.

### 3.2.2. MATERIAL PROPERTIES

From the structural notes of the drawings, the reinforcement steel bar details and specifications used are ruled by the ASTM A-615 Grade 60, hooks bent and laps as required by the ACI 318-77, and the compressive strength of concrete ( $f'_c$ ) is 3,000 psi, except for columns that shall be 4,000 psi at 28 days.

### 3.2.3. MODELING MASONRY WALLS AS SHELL ELEMENTS

Considered as non-structural walls, masonry walls are commonly used as partitions in the structure of the school buildings in Puerto Rico. Although they are not playing any structural role in the structure's performance, they have a significant influence and need to be considered in the structural analysis. In fact, masonry walls have a substantial stiffness and strength, and by not reaching the complete height of the frame level, they restrict the lateral displacement of some columns and lead to the formation of the short column effect in school buildings.

The masonry walls will be modeled as shell elements for this research. The Building Code requirements and specification for Masonry Structures (ACI530-11) [19] establish the modulus of elasticity for concrete masonry can be taken as  $900 \cdot f'_m$ , where  $f'_m$  is the masonry compressive strength that can range from 1,500 to 4,000 psi for concrete masonry. The thickness of the masonry wall is initially 6 in. However, masonry walls in Puerto Rican school buildings are usually ungrouted. The National Concrete Masonry Association (NCMA) presents section properties of grouted and ungrouted concrete masonry walls in NCMA TEK Manual section 14-1B [20], from 4 to 16-in wide. The corresponding net and average cross-sectional properties for a 6-in wall, ungrouted, is presented in Table 3.1. It is recommended to use net cross-sectional properties to determine strain and stress following applied load.

Table 3-1. Section properties for 6 in concrete masonry wall

2a: Horizontal Section Properties (Masonry Spanning Vertically)									
Unit	Grout spacing (in.)	Mortar bedding	Net cross-sectional properties <sup>a</sup>			Average cross-sectional properties <sup>b</sup>			
			$A_n$ (in. <sup>2</sup> /ft)	$I_x$ (in. <sup>4</sup> /ft)	$S_x$ (in. <sup>3</sup> /ft)	$A_{avg}$ (in. <sup>2</sup> /ft)	$I_{avg}$ (in. <sup>4</sup> /ft)	$S_{avg}$ (in. <sup>3</sup> /ft)	$r_{avg}$ (in.)
Hollow	No grout	Face shell	24.0	130.3	46.3	32.2	139.3	49.5	2.08
Hollow	No grout	Full	32.2	139.3	49.5	32.2	139.3	49.5	2.08
100% solid/solidly grouted		Full	67.5	178.0	63.3	67.5	178.0	63.3	1.62
Hollow	16	Face shell	46.6	155.1	55.1	49.3	158.1	56.2	1.79
Hollow	24	Face shell	39.1	146.8	52.2	43.6	151.8	54.0	1.87
Hollow	32	Face shell	35.3	142.7	50.7	40.7	148.7	52.9	1.91
Hollow	40	Face shell	33.0	140.2	49.9	39.0	146.8	52.2	1.94
Hollow	48	Face shell	31.5	138.6	49.3	37.9	145.5	51.7	1.96
Hollow	72	Face shell	29.0	135.8	48.3	36.0	143.5	51.0	2.00
Hollow	96	Face shell	27.8	134.5	47.8	35.0	142.4	50.6	2.02
Hollow	120	Face shell	27.0	133.6	47.5	34.4	141.8	50.4	2.03
2b: Vertical Section Properties (Masonry Spanning Horizontally)									
Hollow	No grout	Face shell	24.0	130.3	46.3	31.4	137.7	49.0	2.09
Hollow	No grout	Full	24.0	130.3	46.3	32.2	139.3	49.5	2.08
100% solid/solidly grouted		Full	67.5	178.0	63.3	67.5	178.0	63.3	1.62
Hollow	16	Face shell	45.8	154.2	54.8	53.1	161.5	57.4	1.74
Hollow	24	Face shell	38.5	146.2	52.0	45.9	153.6	54.6	1.83
Hollow	32	Face shell	34.9	142.3	50.6	42.3	149.6	53.2	1.88
Hollow	40	Face shell	32.7	139.9	49.7	40.1	147.2	52.4	1.92
Hollow	48	Face shell	31.3	138.3	49.2	38.6	145.7	51.8	1.94
Hollow	96	Face shell	27.6	134.3	47.8	35.0	141.7	50.4	2.01
Hollow	120	Face shell	26.9	133.5	47.5	34.3	140.9	50.1	2.03

The net area of a 6-in wall, ungrouted, face shell bedding is 24 in<sup>2</sup>/ft. The corresponding equivalent thickness is:

$$t \times 12 \text{ in/ft} = 24 \text{ in}^2/\text{ft} \quad \Rightarrow \quad t = 2''$$

The material properties of the masonry walls used in SAP2000 is summarized in Table 3-2.

Table 3-2. Masonry wall material properties in SAP2000

DESCRIPTION	VALUE
<b>Compressive strength (<math>f'_m</math>)</b>	1,500 psi
<b>Modulus of elasticity (<math>E_m</math>)</b>	1,350 ksi
<b>Specific Weight (<math>\gamma</math>)</b>	130 lb/ft <sup>3</sup> (0.075 lb/in <sup>3</sup> )
<b>Thickness (t)</b>	2 in

### 3.2.4. GRAVITY LOADS

The dead loads are typically the weight of partitions, slab, beams and columns and are assigned using the self-weight option that is commonly used in SAP2000. The live loads used for the different areas of the structure can be found in the specification details of the drawings and are

displayed in Table 3-3. Live loads are assigned by tributary areas as uniformly distributed loads on the beams for the Pushover and time history analyses.

Table 3-3. Live loads

AREA	VALUE
Roof	20 psf
Classrooms	40 psf
Laboratories	50 psf
Corridors and stairs	100 psf

### 3.2.5. SHEAR CAPACITY CALCULATION OF THE COLUMNS

The calculation of the shear capacity  $V_n$  of the column cross-section is given by the summation of the shear capacity of concrete  $V_c$  and the shear capacity of the ties of transversal reinforcement  $V_s$ .

$$V_n = V_c + V_s$$

The formula for  $V_c$  is established by the Building Code Requirements for Structural Concrete (ACI318M-14) [21], and is equal to:

$$V_c = 2 \left( 1 + \frac{Nu}{2000 \cdot Ag} \right) \sqrt{f'_c} \cdot b \cdot d$$

where  $Nu$  is the design axial load,  $d$  is the distance from the external fiber in compression to the centroid of the reinforcement in tension,  $Ag$  is the concrete gross area ( $Ag = b \cdot d$ ), and  $f'_c$  is the compressive strength of concrete.

$V_s$  is calculated using the formula of the Building Code Requirements for Structural Concrete (ACI318M-14), and is equal to:

$$V_s = \frac{A_v * F_y h * d}{s}$$

where  $A_v$  is the area of the steel of the transversal reinforcement,  $s$  is the maximum spacing of the ties center-to-center, and  $F_y h$  is the yielding strength of the ties.

Table 3-4 shows the shear capacity of the columns. Although it is part of the equation, the design axial load  $N_u$ , is considered equal to 0, in other to be more conservative. The additional capacity of the section due to design axial load will be later displayed.

Table 3-4. Shear capacity calculation of the columns.

Column	b(in)	h(in)	$f'_c$ (psi)	$f'_y$ (psi)	d(in)	$A_v$ (in <sup>2</sup> )	s(in)	Vc(kip)	Vs(kip)	Vn(kip)
Exterior	12	24	4,000	60,000	21.5	0.22	10	32.6	28.4	61.0
Interior	12	12	4,000	60,000	9.5	0.22	10	14.4	12.5	26.9

### **3.3. ANALYSIS PROCEDURES**

A nonlinear static analysis, known as Pushover Analysis, and a linear dynamic analysis, namely the time history analysis, will be performed for this research.

#### **3.3.1. PUSHOVER ANALYSIS**

Analysis and design methods of structures have evolved through the history of engineering. From the lessons learned due to the occurrence of natural events that have had major impacts on human life and structures, new methodologies have been implemented, considering new investigation criteria that approximates the results obtained in reality. Building Codes requirements commonly consider linear-elastic behavior for the structural analysis and design of buildings. However, this scenario differs from reality. Structural components of the buildings that either collapsed or presented significant damages were induced to an inelastic range. Then, the Pushover Analysis has emerged as an alternative to the complicated methods of structural analysis to assess the damage of buildings that experience inelastic behavior. The Pushover Analysis significantly facilitates the step-by-step dynamic procedures and simplifies certain criteria, which makes it relatively simpler compared to the nonlinear dynamic analysis, and it is one of the most used techniques for the evaluation of seismic performance of structures and in structural analysis for design purposes.

The nonlinear static analysis, or "Pushover" method, consists of applying a vertical distribution of lateral loads, after the gravity load is applied to the structure. The load is increased monotonously until the structure reaches a maximum displacement. Generally, loads with triangular height distribution are applied, because they represent forces which will deform the structure in the first vibration mode. The structural response is then examined through the base shear versus displacement of the top of the structure graph. This method allows determining the performance sequence, member failure, and structural damage levels, as well as the general

progress that defines the capacity curve. When applying loads, SAP2000 provides two different types of methods:

- **Force-controlled:** When the magnitude of the load is known, the full load combination is applied, and the performance of the load is measured depending on the displacement or deformation obtained.
- **Displacement-controlled:** When the magnitude of the load is not known, the load combination is adjusted until the target displacement is reached.

By performing a Pushover Analysis, it is possible to estimate possible occurrence of damages and the limit state of the structure being evaluated. At first, the structure deforms elastically, and after exceeding the limit state of elasticity, then, it begins to behave in what it is known as non-linear behavior, which approximates to the reality and makes the method more effective. In the 1960s and 1970s, many researchers proposed criteria to be considered in creating the parameters that today constitute the standards of the Federal Emergency Management Agency (FEMA) and Applied Technology Council (ATC).

### **3.3.2. NONLINEAR BEHAVIOR AND PERFORMANCE OF STRUCTURES**

The structure performance levels can be described by the level of damage the structure experienced after seismic events. The FEMA356 pre-standard and ASCE41 standard, through the periodic publications of their updated editions, provide information about the performance levels that may be considered acceptable for a given structure with specific ground motion. The performance levels based on the parameters defined in the FEMA356 pre-standard and ASCE41 standard are:

**Operational (non- structural):** Damages mostly appear on non-structural components. Original strength and stiffness system preserved. Structural system fully operational.

**Immediate occupancy (IO):** Damages very limited. Original strength and stiffness practically retained. Minor concrete cracking, non-structural damage, minor yielding of steel.

**Life safety (LS):** Damages limited. Significant damages but no collapse risk of the structure. Significant loss of original stiffness. Temporary or permanent structural repairs required.

**Collapse prevention (CP):** Extreme structural and non-structural damages. Risk of instability and collapse.

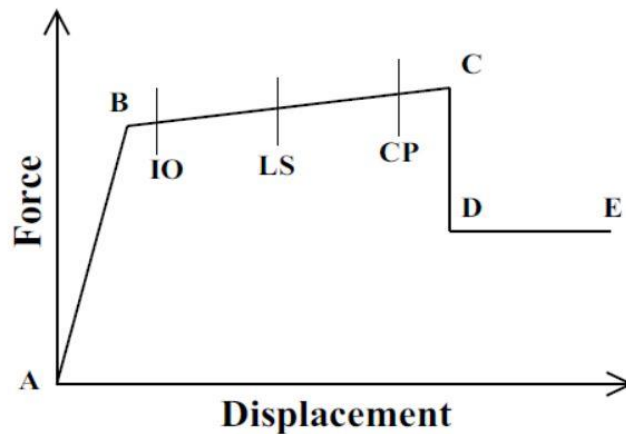


Figure 30. Idealized Pushover curves.

### 3.3.3. TIME HISTORY ANALYSIS

A linear dynamic analysis, by means of the Time History analysis, is an important technique for structural seismic assessments. To perform such an analysis, a representative earthquake time history is required for the structure being evaluated. It is a step-by-step analysis of the dynamic response of a structure to a specified load that will vary with time. It is used to determine the seismic response of a structure under dynamic loads of the representative earthquake [22].

#### 3.3.3.1. EARTHQUAKES RECORDS SELECTION

As mentioned in the first chapter, no strong earthquakes have been recorded in Puerto Rico in recent times. Imported records are used for the seismic resistant analysis and design of

structures. In 1999, Irizarry, Martinez and Portela [2] performed a study to develop an elastic design spectrum for San Juan, Ponce and Mayagüez, considered as the three main metropolitan cities of Puerto Rico. The assessment was based originally on earthquakes records from all around the world from a data base of more than 15,000 records, where finally 7 records that meet the parameters defined by the authors, such as, geological formations, faults' length, among others, were selected for the three areas mentioned earlier. Lately, in 2018, Alicea [5] used those records in his research to perform a dynamic seismic assessment of the Ponce Highway Bridge and the PRILA Condominium. Those records have been mathematically transformed and modified to meet the expected soil accelerations and amplifications specified in ASCE7-16 for Ponce, and San Juan, Puerto Rico and will be used to assess the linear dynamic analysis for this research. Information regarding these earthquakes are displayed in Table 3-5 and Table 3-6.

Table 3-5. Earthquake records selected for San Juan, PR.

<b>Earthquake name</b>	<b>Date</b>	<b>Depth [km]</b>	<b>Magnitude</b>	<b>Station name</b>	<b>Epicenter [km]</b>	<b>PGA [g]</b>	<b>Filename</b>
Japanese Earthquake	06-12-68	31	6.4 M <sub>w</sub>	Miyako Harbor Works, Ground	103	0.167	JAP03.104
Miyagi Prefecture	06-12-78	40	6.3 M <sub>w</sub>	Ofunato Harbor, Jetty	103	0.226	JAP03.173
Michoacán Aftershock	09-23-85	20	7.6 M <sub>s</sub>	Papanao	83	0.226	MEX03.112
Northridge	01-17-94	18	6.7 M <sub>w</sub>	Pacoima, Kagel Canyon	43	0.301	PACOIMAW.V2
Northridge	01-17-94	18	6.7 M <sub>w</sub>	Rolling Hills Estates Rancho Vista School	50	0.116	ROLLHILL.V2
ADAK	05-02-71	38	6.8 M <sub>s</sub>	Adak Naval Base (Hand Dig.)	67	0.187	USAAK02.067
Loma Prieta	10-18-89	11	7.1 M <sub>w</sub>	Apeel Array #9, Crystal Springs Res.	62	0.108	USACA66.001

Table 3-6. Earthquake records selected for Ponce, PR.

Earthquake name	Date	Depth [km]	Magnitude	Station name	Epicenter [km]	PGA [g]	Filename
San Salvador	10-10-86	8	5.4 M <sub>s</sub>	Geotechnical Investigation Center	4.3	0.420	CIG.EW
San Salvador	10-10-86	8	5.4 M <sub>s</sub>	Hotel Sheraton	7	0.301	HSH.EW
San Salvador	10-10-86	8	5.4 M <sub>s</sub>	National Geographical Institute	5.7	0.535	IGN.EW
San Salvador	10-10-86	8	5.4 M <sub>s</sub>	Institute of Urban Construction	6	0.387	IVU.EW
San Salvador	10-10-86	8	5.4 M <sub>s</sub>	Universidad Centro Americana	4	0.417	UCA.EW
Coyote Lake	08-06-79	6	5.8 M <sub>L</sub>	Gilroy Array #6, San Ysidro	10	0.357	USACA18.033
Morgan Hill	04-24-84	9	6.2 M <sub>L</sub>	Gilroy Array #6, San Ysidro	37	0.286	USACA36.011

### 3.3.3.2. SOIL TYPE SELECTION PROCEDURE

The ASCE-7 building code establishes the site classification procedure for seismic design. Site class types are presented in Table 3-7.

Table 3-7. Site classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
Any profile with more than 10 ft of soil having the following characteristics: —Plasticity index $PI > 20$ , —Moisture content $w \geq 40\%$ , —Undrained shear strength $\bar{s}_u < 500$ psf			
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1 ft/s = 0.3048 m/s; 1 lb/ft<sup>2</sup> = 0.0479 kN/m<sup>2</sup>.

According to the ASCE-7 specifications, it is recommended to use soil type D when the soil properties are unknown. Soil type D is selected for this research because there is no available data of the soil properties where the evaluated building school is located.

**3.3.3.3. PROBABLE EARTHQUAKES COMPATIBLE WITH ASCE7-16 FOR SAN JUAN**

**Probable earthquake for San Juan, PR in soil Type D: JAP03.104**

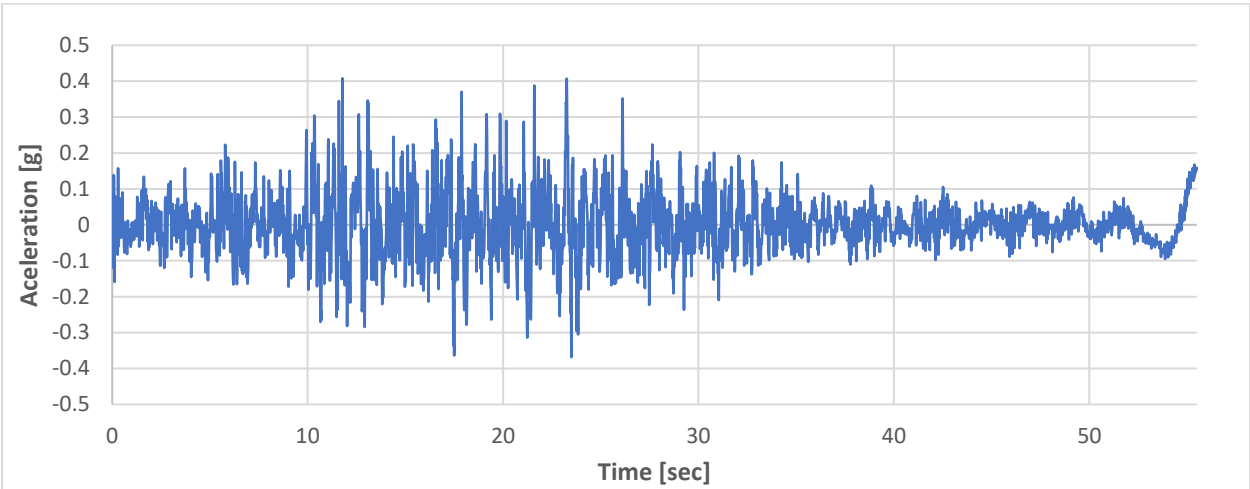


Figure 31. Probable earthquake JAP03.104 for San Juan PR for soil type D.

**Probable earthquake for San Juan, PR in soil Type D: JAP03.173**

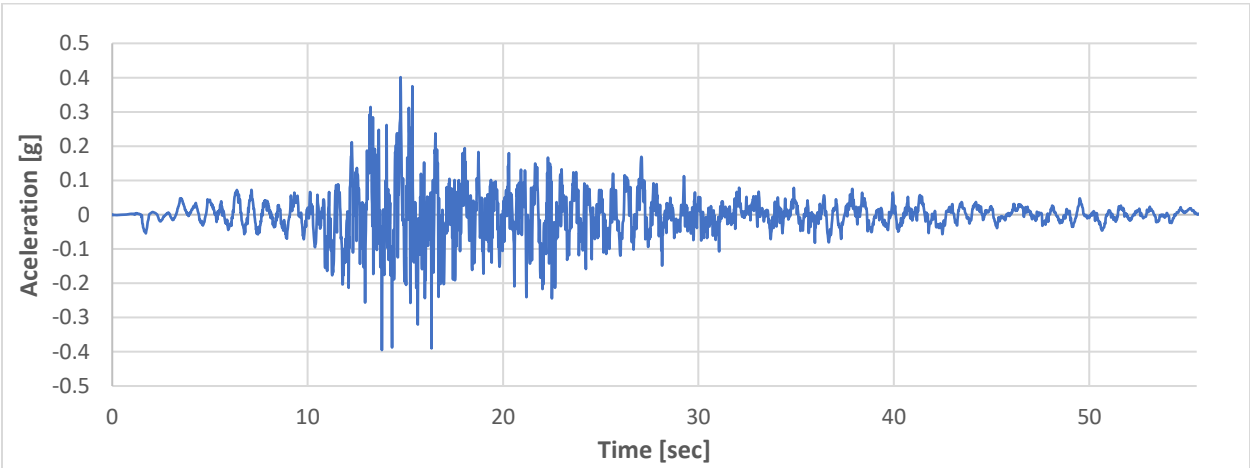


Figure 32. Probable earthquake JAP03.173 for San Juan PR for soil type D.

**Probable earthquake for San Juan, PR in soil Type D: MEX03.112**

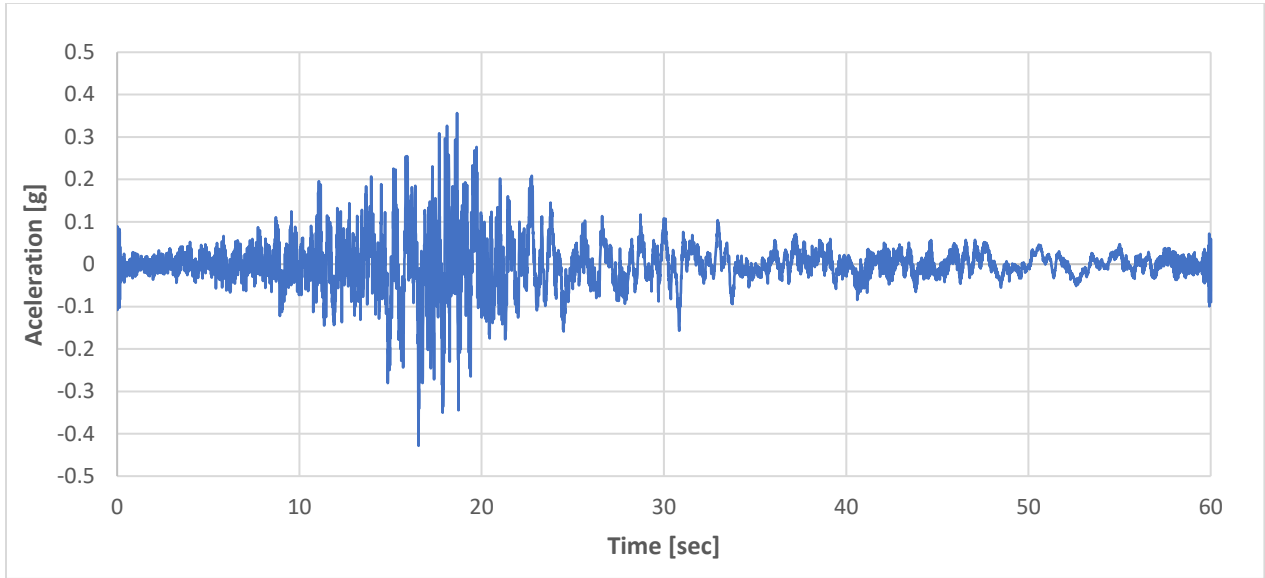


Figure 33. Probable earthquake MEX03 for San Juan PR for soil type D.

**Probable earthquake for San Juan, PR in soil Type D: PACOIMAW.V2**

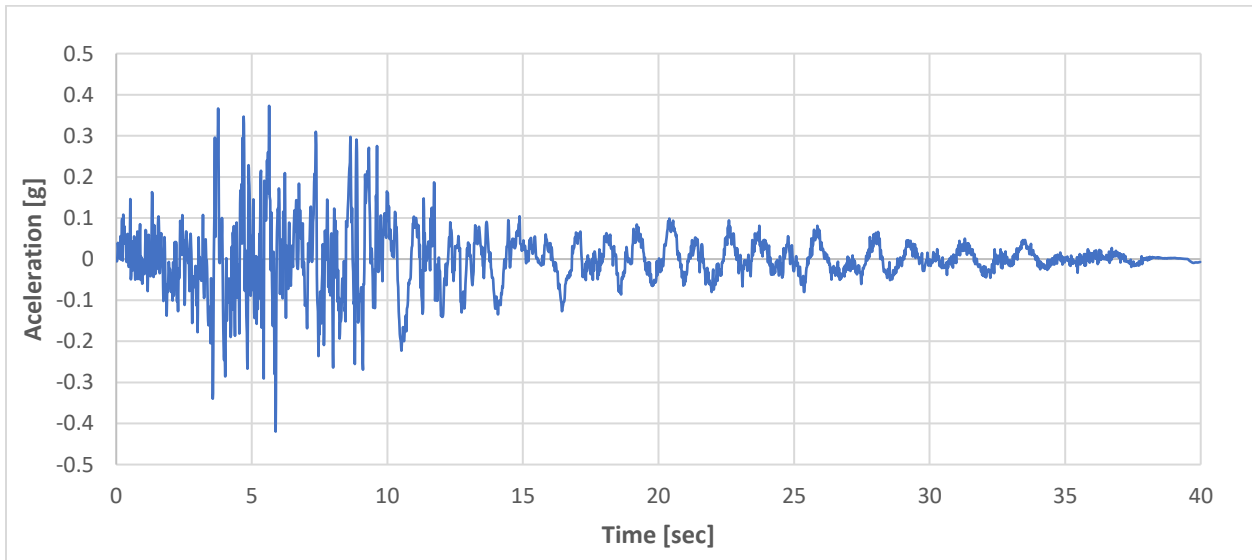


Figure 34. Probable earthquake PACOIMAW for San Juan PR for soil type D.

**Probable earthquake for San Juan, PR in soil Type D: ROLLHILL.V2**

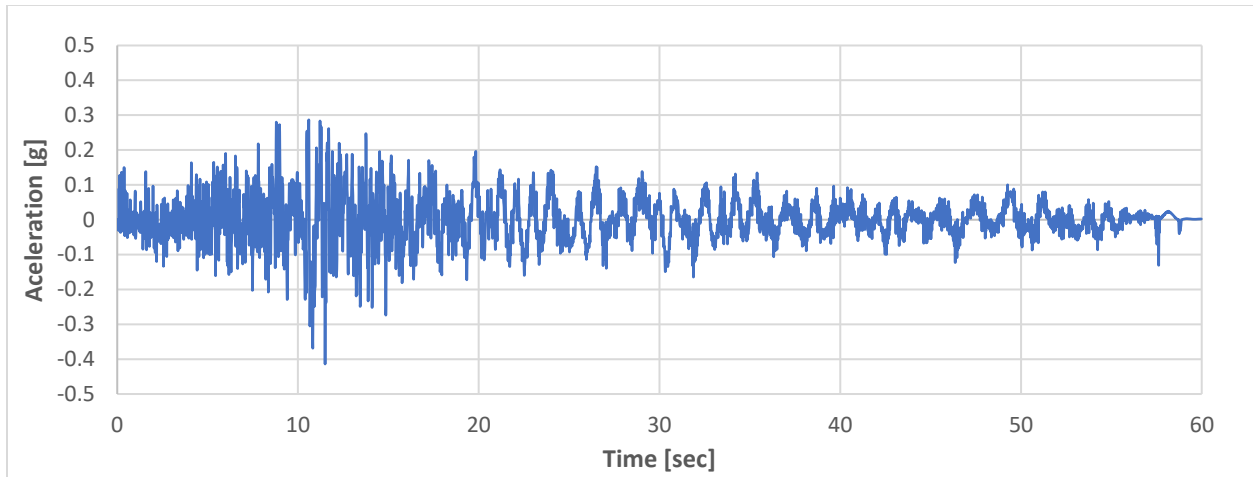


Figure 35. Probable earthquake Rollhill for San Juan PR for soil type D.

**Probable earthquake for San Juan, PR in soil Type D: USAKA02.067**

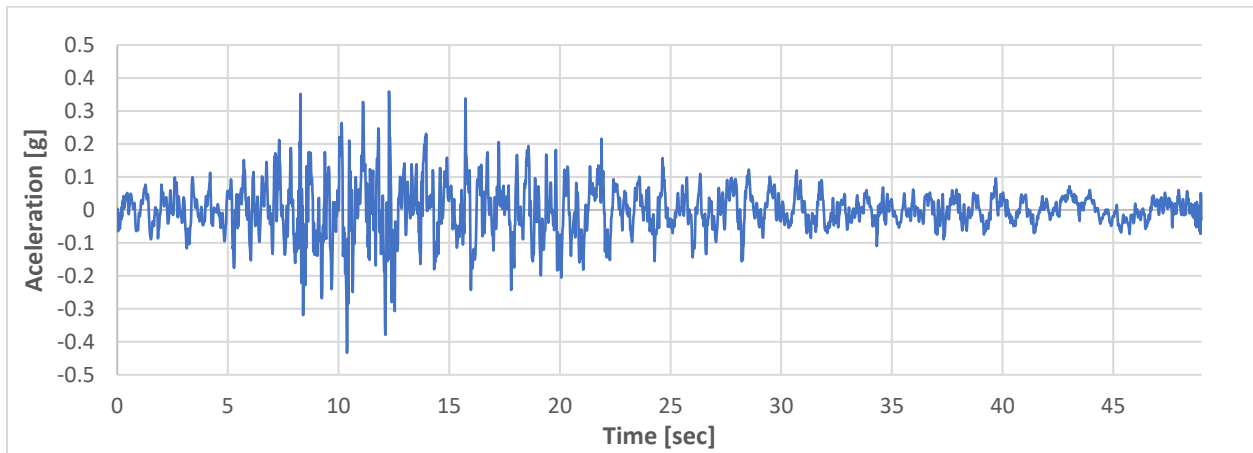


Figure 36. Probable earthquake USAKA02 for San Juan PR for soil type D.

**Probable earthquake for San Juan, PR in soil Type D: USAKA66.001**

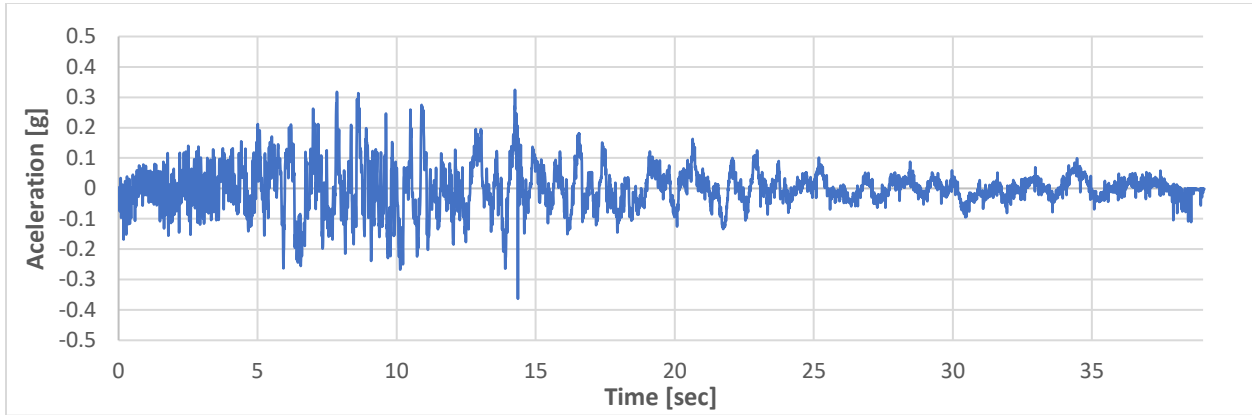


Figure 37. Probable earthquake USAKA66 for San Juan PR for soil type D.

**Probable earthquake for Ponce, PR in soil Type D: CIG.EW**

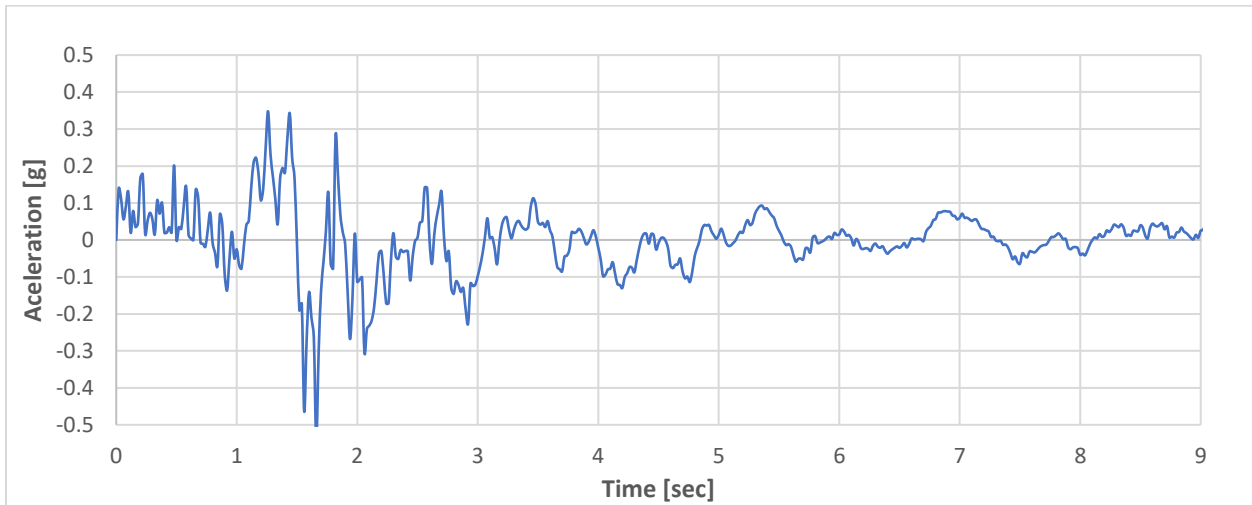


Figure 38. Probable earthquake CIG for Ponce, PR for soil type D.

**Probable earthquake for Ponce, PR in soil Type D: HSH.EW**

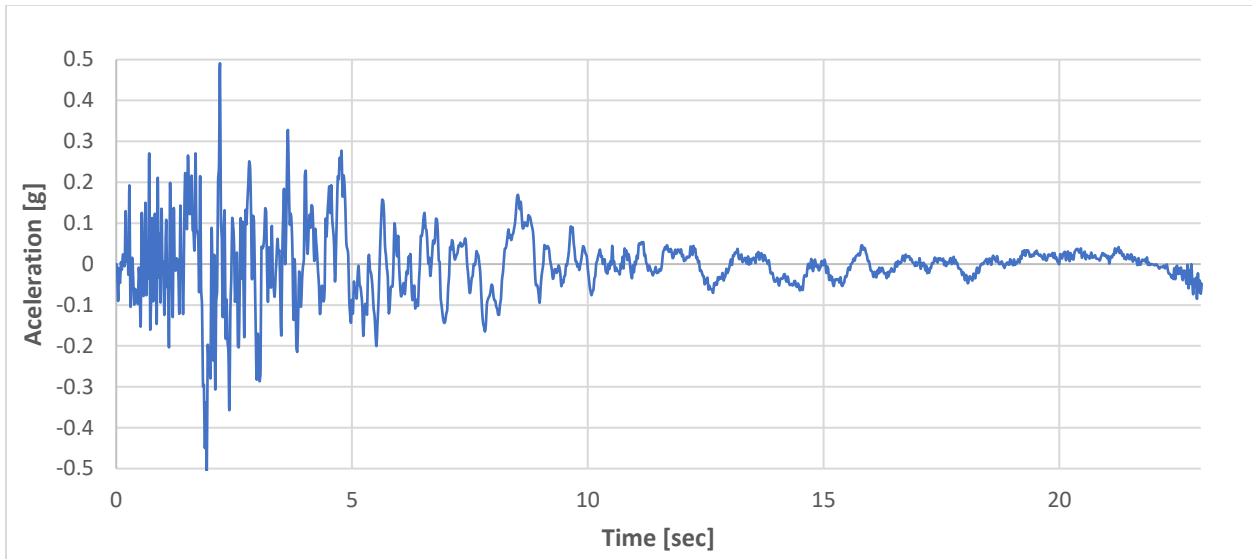


Figure 39. Probable earthquake HSH for Ponce, PR for soil type D.

**Probable earthquake for Ponce, PR in soil Type D: IGN.EW**

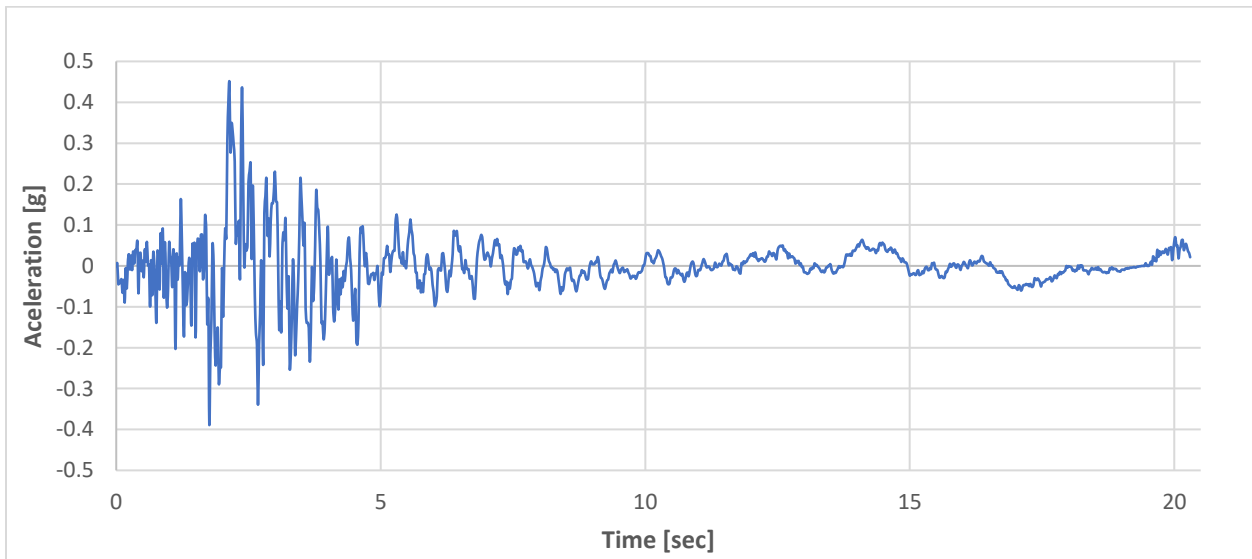


Figure 40. Probable earthquake IGN for Ponce, PR for soil type D.

**Probable earthquake for Ponce PR in soil Type D: IVU.EW**

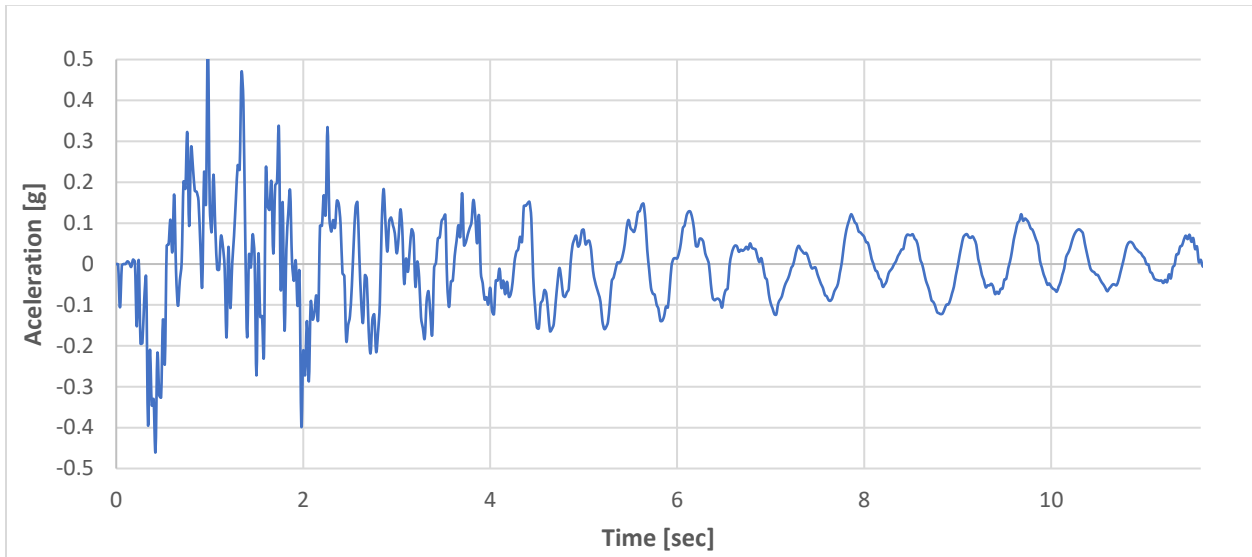


Figure 41. Probable earthquake IVU for Ponce, PR for soil type D.

**Probable earthquake for Ponce, PR in soil Type D: UCA.EW**

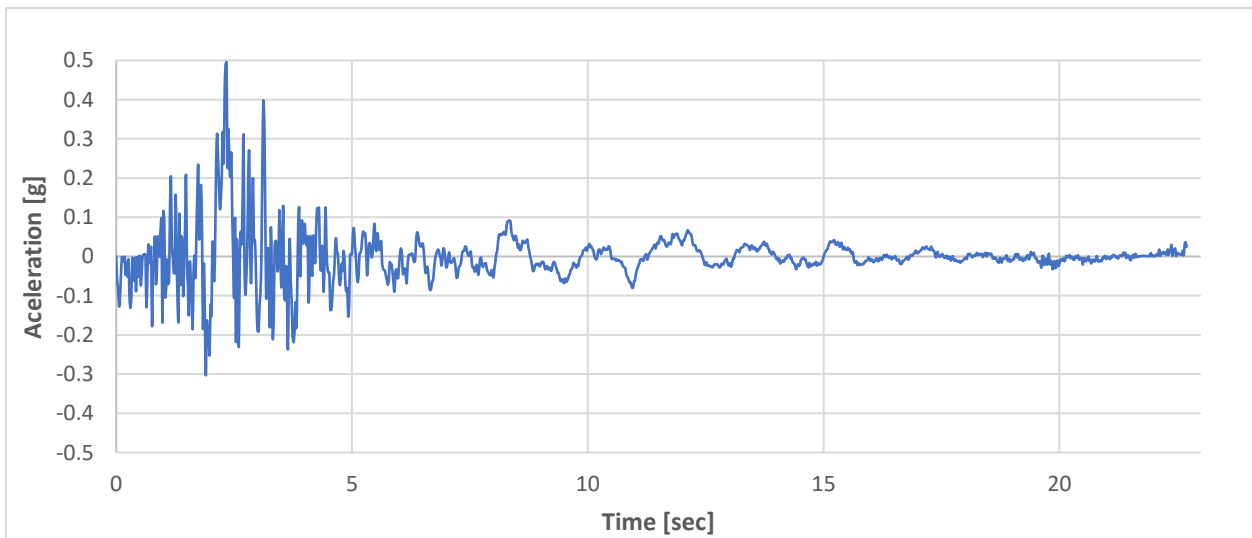


Figure 42. Probable earthquake UCA for Ponce, PR for soil type D.

**Probable earthquake for Ponce, PR in soil Type D: USACA18.033**

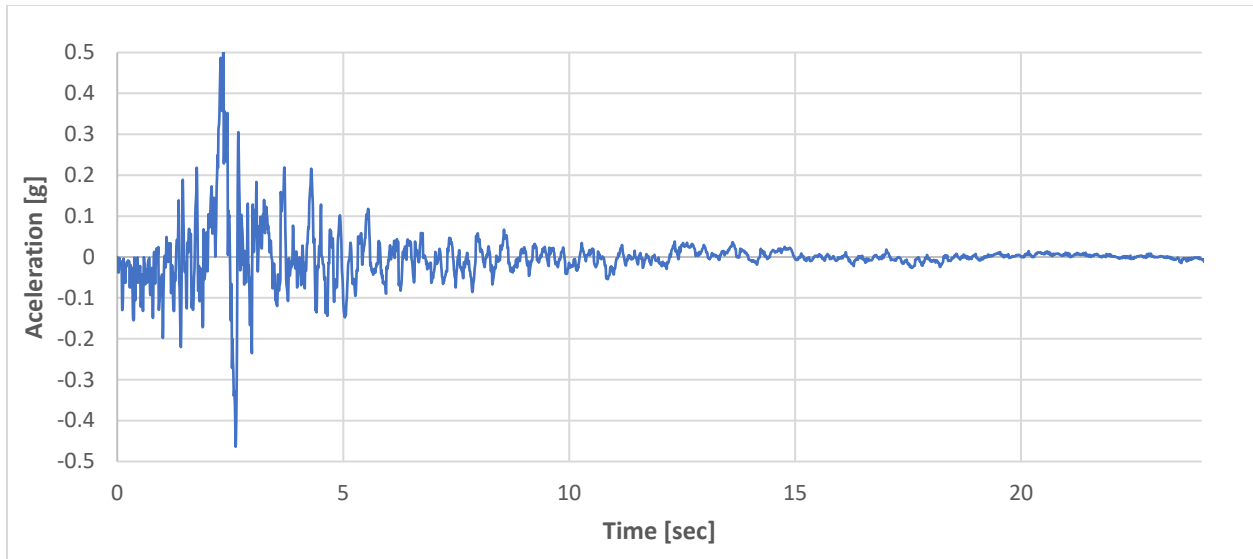


Figure 43. Probable earthquake USACA18 for Ponce, PR for soil type D.

**Probable earthquake for Ponce, PR in soil Type D: USACA36.011**

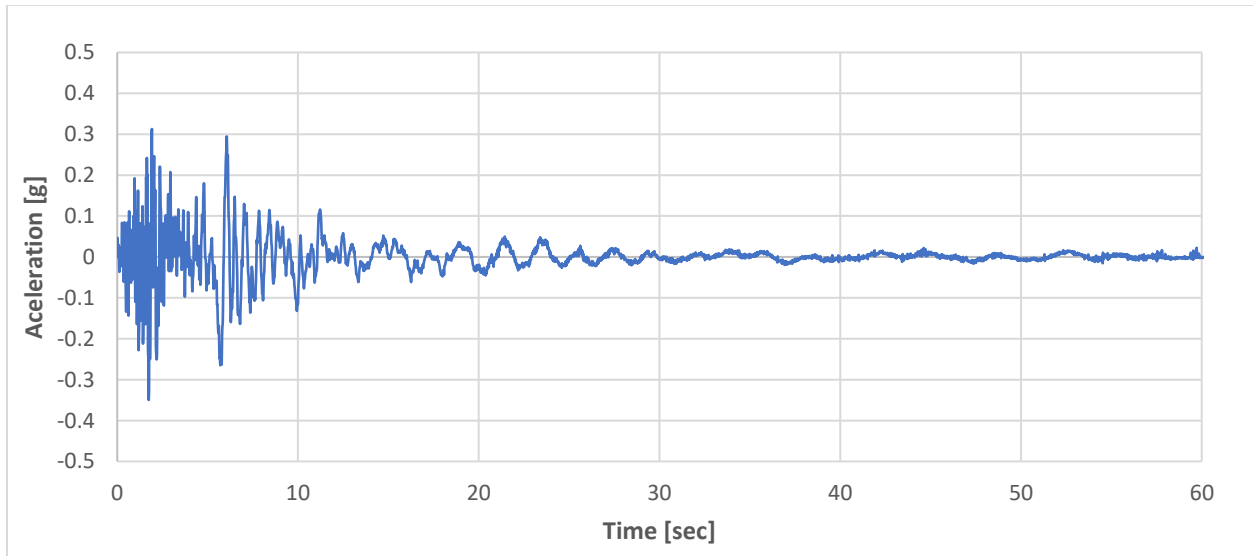


Figure 44. Probable earthquake USACA36 for Ponce, PR for soil type D.

The plan for this research is briefly described here:

1. Acquire representative architectural and structural plans of a typical school in Puerto Rico from *the Public Buildings Authority (AEP) of Puerto Rico* that contain details about size and structural design of beams, columns, wall sections, and slabs; story height; and number of stories and spans, among others.
2. Create a 3D model of the structure using SAP2000 not considering masonry walls, a general-purpose finite element program which performs the static or dynamic, and linear or nonlinear analysis of structural systems.
3. Submit the structure to a non-linear Static Pushover analysis to determine its capacity and mode of failure.
4. Incorporate masonry walls into the computational model which creates the short column.
5. Perform the Pushover analysis to determine the capacity of the structure and failure mechanism taking into consideration shear failures.
6. Develop one rehabilitation strategy and apply it to the structure.
7. Perform the previous analysis for the rehabilitation strategy considered to determine the lateral load capacity and the mechanism of failure of the structure.
8. Compare the results obtained in step 3, 5 and 7.
9. Acquire non-local earthquake data used in previous research for locations like San Juan, Ponce, and Mayagüez, Puerto Rico. These seismic records must be compatible with the elastic design spectrum of ASCE7-16.
10. Perform Time History analysis by submitting the structure to the non-local earthquake records compatible with the elastic design spectrum of ASCE7-16 to study the structure behavior.
11. Compare the results of the analysis to develop conclusions for the Department of Education of Puerto Rico.

## 4. RESULTS

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### 4.1. INTRODUCTION

This chapter presents the results and the interpretation of the analyses performed on the prototype model. The information on the prototype model is extracted from the review of drawings of a typical school in Puerto Rico, where a module without walls in its longitudinal direction is selected to study the nonlinear behavior of the structure. The sequence of plastic hinge is displayed according to the idealized Pushover curves (Figure 30). Later, a rehabilitation strategy is selected and is applied to the structure, and results are displayed and compared to evaluate the efficacy of the selected rehabilitation system.

### 4.2. NONLINEAR STATIC ANALYSIS RESULTS

#### FRAME I

Case I: Frame I without masonry walls in the longitudinal direction.

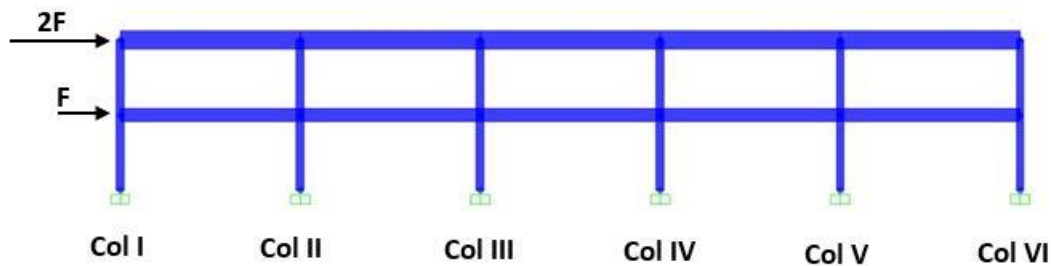


Figure 45. Frame I 2D model without masonry walls in the longitudinal direction in SAP2000.

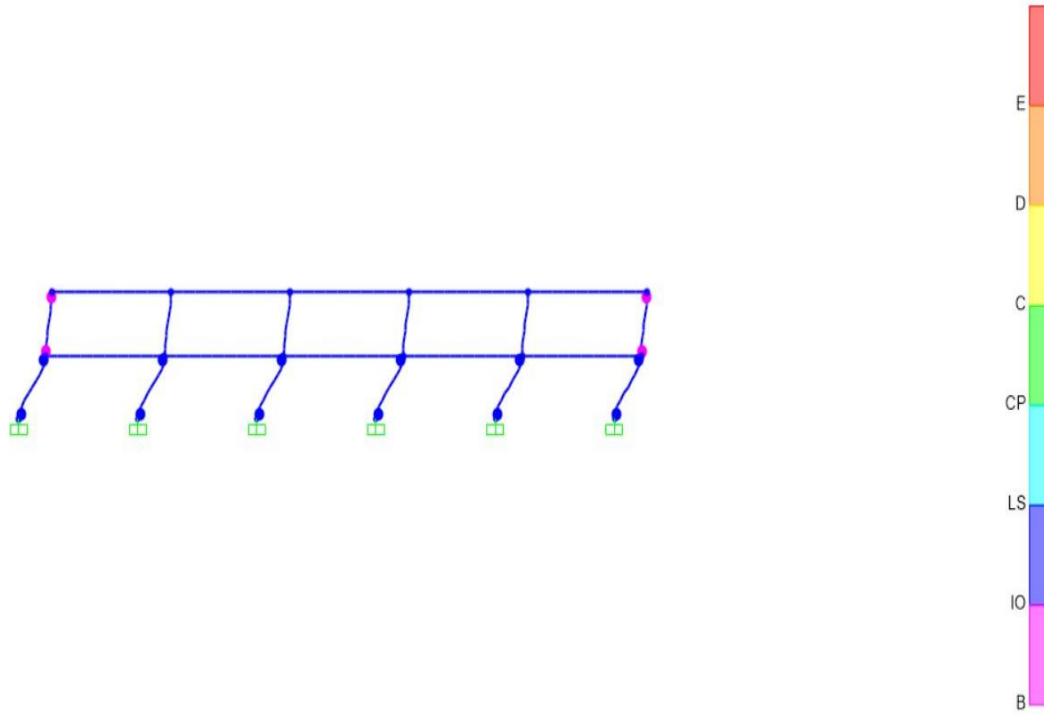


Figure 46. Sequence of plastic hinge formation – Case I.

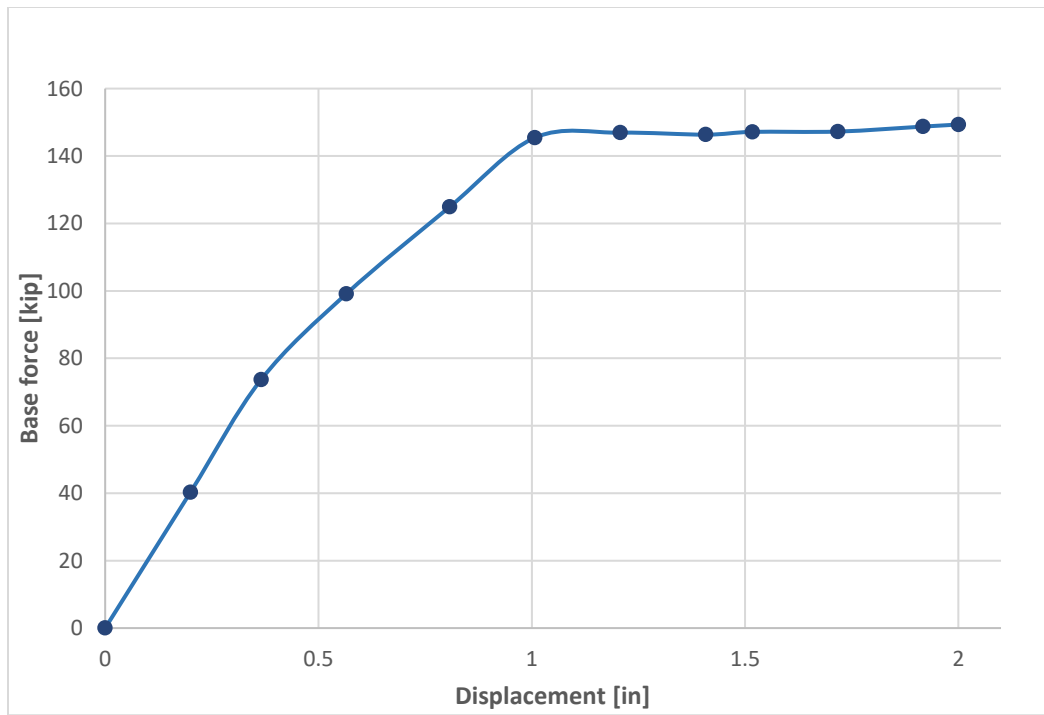


Figure 47. Pushover capacity curve of Frame I – Case I.

Table 4-1. Shear capacity of columns vs shear demand during the analysis – Case I.

Parameters	Column I	Column II	Column III	Column IV	Column V	Column VI
Shear capacity [kip]	64.3	31.9	32.0	32.0	31.9	64.3
Shear demand [kip]	19.0	27.7	27.7	27.7	27.7	27.7

A displacement-controlled of 2 inch Pushover analysis is performed where no masonry walls were considered in the longitudinal direction of the Frame I. The capacity of the structure, presented in Figure 47, is 146.9 kip, and no shear failure is noticed when the shear capacity of each column is calculated and compared to the maximum shear produced by the lateral loads applied during the analysis for each corresponding column. From the results of this analysis, Frame I does not suffer any shear failure when performing the Pushover Analysis.

**Case II: Frame I with masonry walls in the longitudinal direction when shear failures were not considered by SAP2000.**

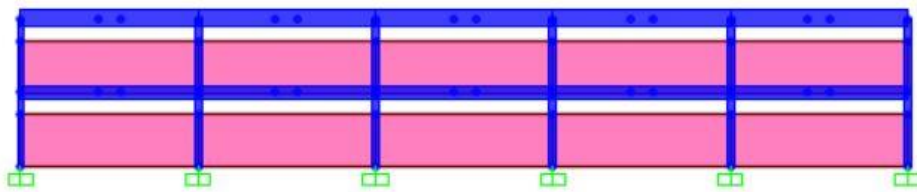


Figure 48. Frame I 2D model with masonry walls in the longitudinal direction in SAP2000.

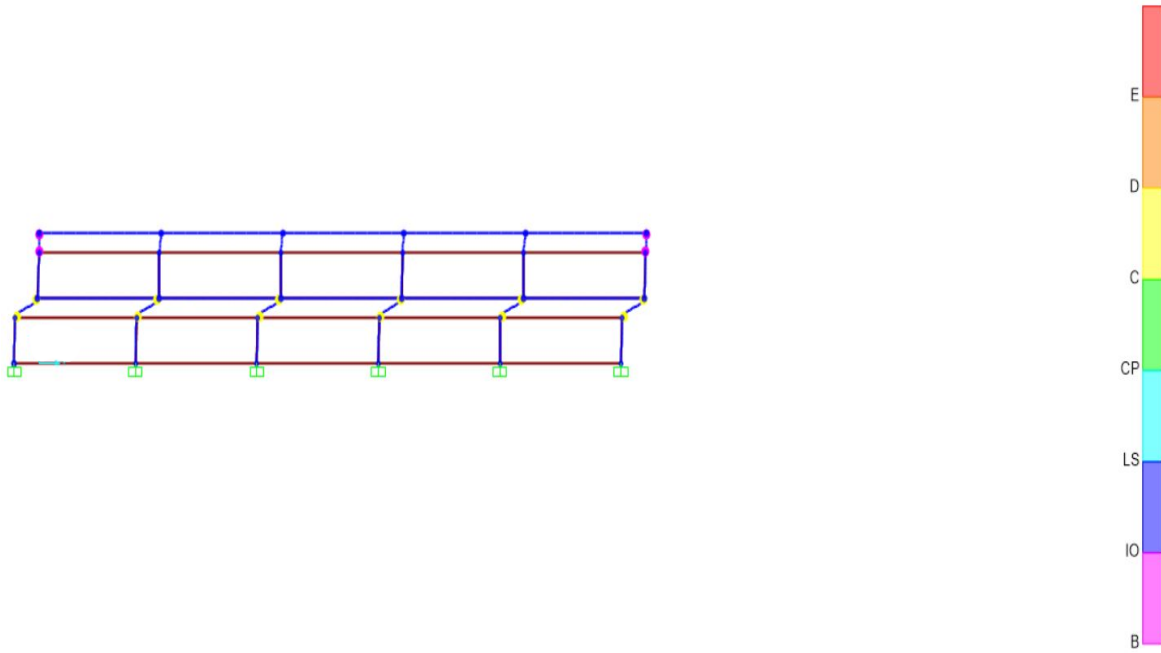


Figure 49. Sequence of plastic hinge formation – Case II.

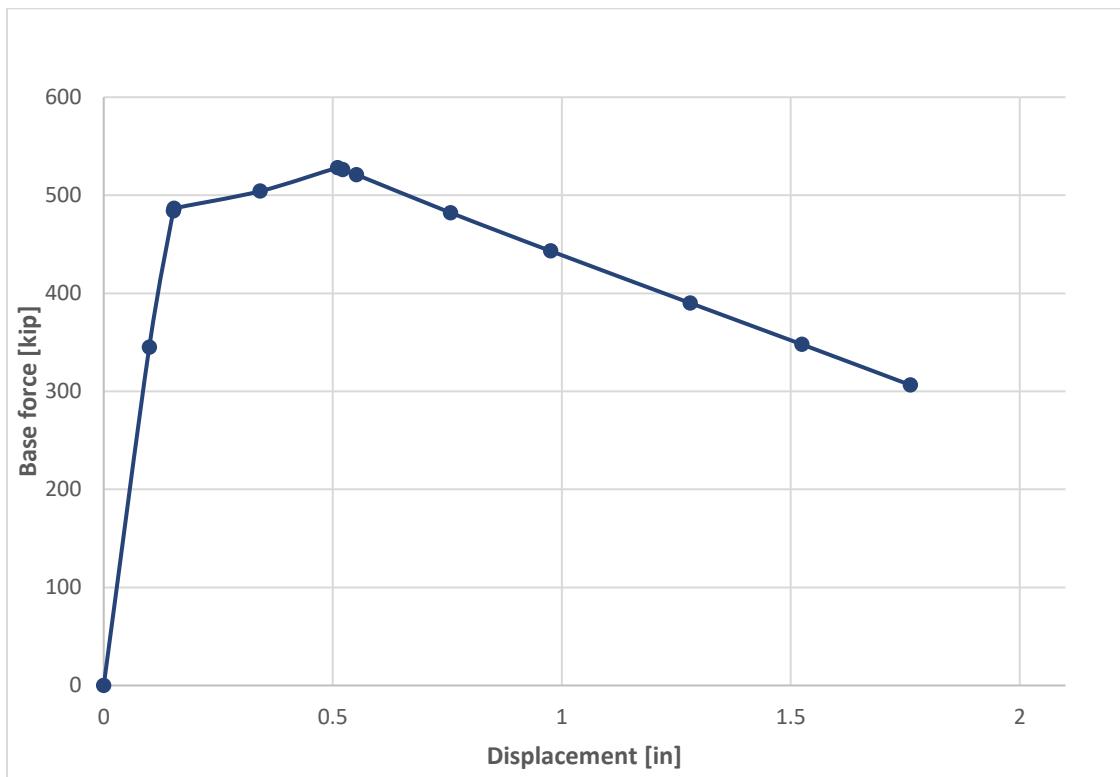


Figure 50. Pushover capacity curve of Frame I – Case II.

Table 4-2. Shear capacity of columns vs shear demand during the analysis – Case II.

Parameters	Column I	Column II	Column III	Column IV	Column V	Column VI
Shear capacity [kip]	64.4	32.0	32.2	32.2	32.0	64.4
Shear demand [kip]	61.9	99.5	98.6	98.6	97.8	72.6

The displacement-controlled analysis is performed after the introduction of the masonry walls in the longitudinal direction of Frame I where no shear failures were taken into consideration in SAP2000. The capacity of the structure, which was 146.9 kip with no masonry walls, has increased to 528.1 kip. From Table 4-2, the results show that the local shear experimented when lateral loads are applied exceed the shear capacity of the columns; hence, the columns suffer shear failure during the analysis.

**Case III: Frame I with masonry walls in the longitudinal direction with shear failures considered in SAP2000.**

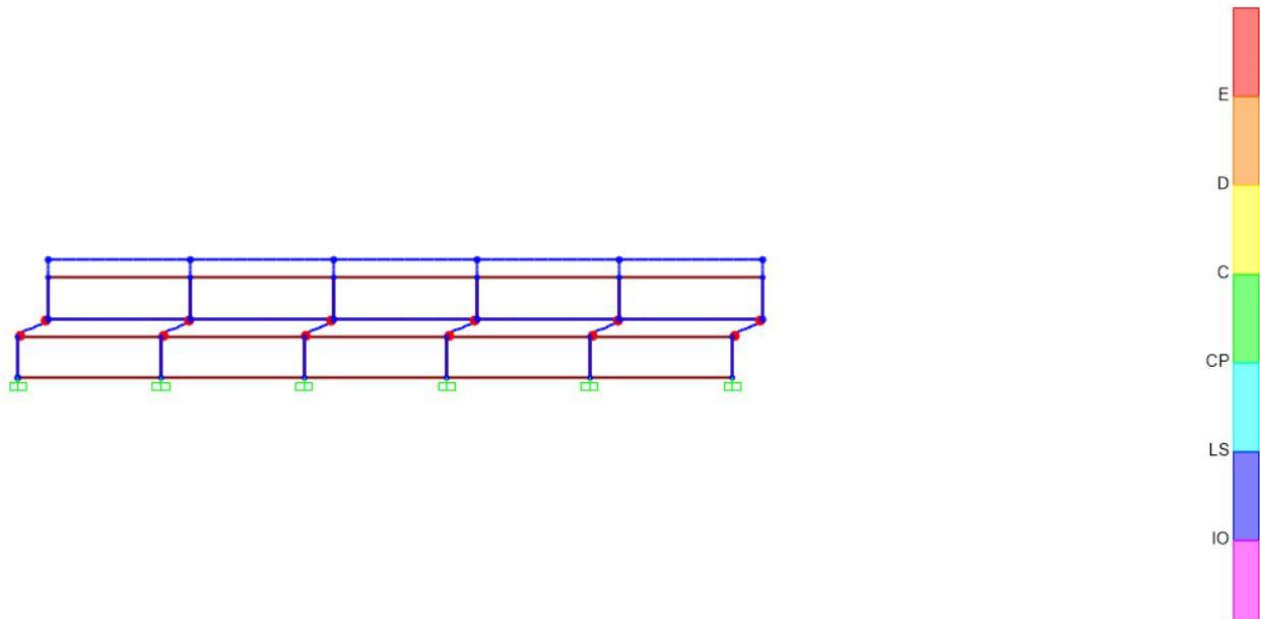


Figure 51. Sequence of plastic hinge formation – Case III.

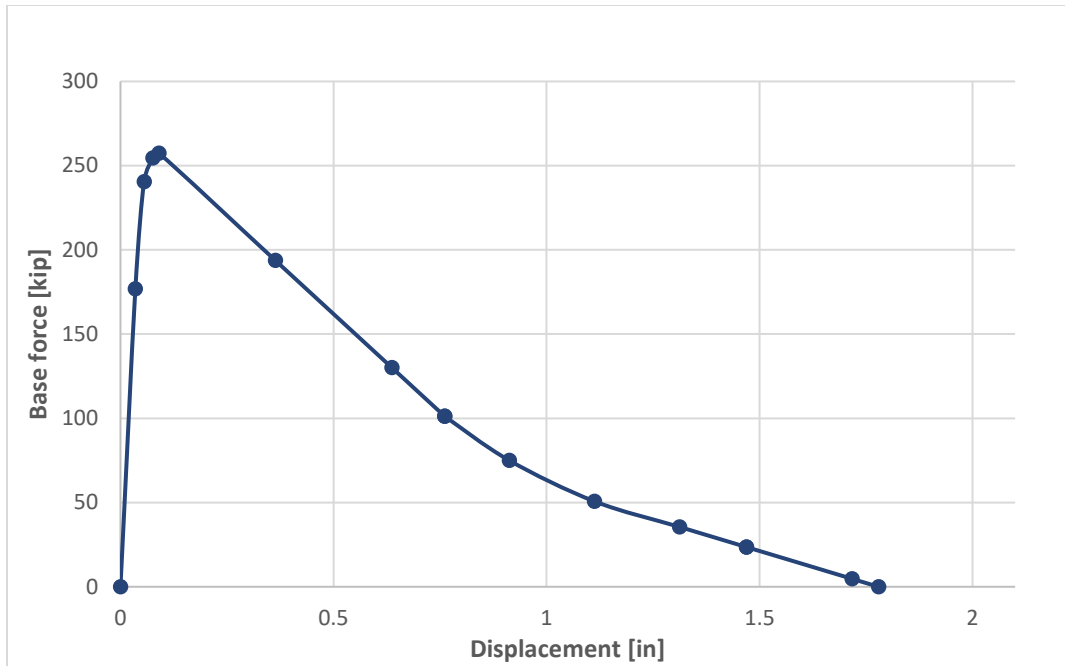


Figure 52. Pushover capacity curve of Frame I – Case III.

Table 4-3. Shear capacity of columns vs shear demand during the analysis – Case III.

Parameters	Column I	Column II	Column III	Column IV	Column V	Column VI
Shear capacity [kip]	64.4	32.0	32.2	32.2	32.0	64.4
Shear demand [kip]	64.9	32.4	32.4	32.4	32.4	64.9

The previous analysis is performed, but taking shear failures into consideration in SAP2000; in other words, when the applied shear reaches the shear capacity of the column, a hinge is formed. The Pushover Analysis will proceed until the structure becomes unstable. The capacity of the structure is now 257.4 kip when shear failures are taken into consideration. Once this value is reached, the graph starts decreasing until the structure is not able to carry further load. The results of this analysis are displayed in Table 4-3.

**Case IV: Comparison of the three past analyses.**

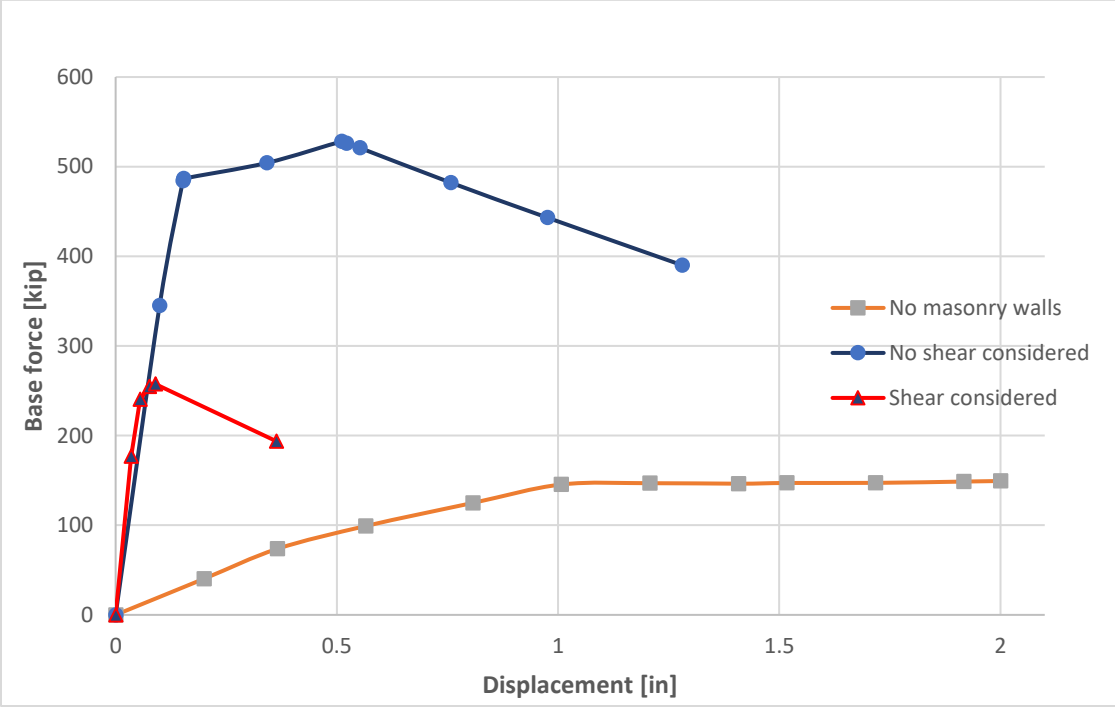


Figure 53. Pushover capacity curves of Frame I with and without shear - Case IV.

Figure 53 presents a comparison of the Pushover results from the past three analyses. The base force vs displacement graph for those analyses state that the capacity of the frame is 257.4 kip when shear failures are taken into consideration, while the actual capacity of the frame is 528.1 kip when shear failure is not considered. Inserting masonry walls significantly reduces the lateral displacement of the structure and attracts higher lateral load. Hence, the structure fails by shear, way before reaching its capacity. Frame I requires a rehabilitation system that is adequate to lower or completely eliminate the shear carried by the columns due to the short column effect created by the introduction of the masonry walls.

### 4.3. REHABILITATION STRATEGY PROPOSED

In 1997, Hassan and Sozen [23] studied the seismic vulnerability of low-rise buildings in regions with infrequent earthquakes based on observations made on a group of damaged buildings during the 1992 Erzincan, Turkey earthquake. According to the authors, buildings with a 0.5% or more wall percentage (walls area over total area from the top view) undergo negligible or no damages during earthquakes. Reinforced concrete walls are quite effective in resisting lateral load, also in terms of costs and application. These walls are commonly placed to stiffen the structure in its longitudinal direction, thus, attract more lateral load, and provide more capacity to the structure as they are reinforced, and therefore reduce the shear force that would be transmitted to the columns, which would reduce the possible effect of short column. Then, the reinforced concrete wall for the seismic rehabilitation system is 3 feet long and 8 inches wide. A RC wall segment will be placed at a selected position of each classroom. The RC wall must be extended from the foundation to the roof of the structure.

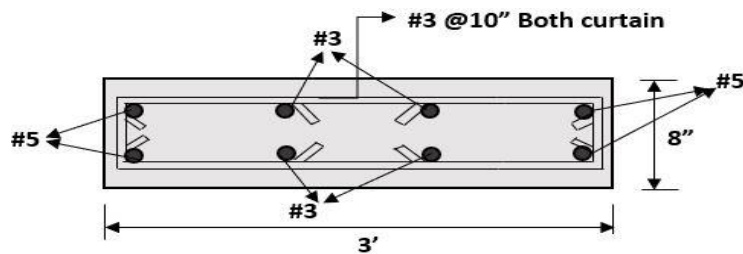


Figure 54. Reinforced concrete wall suggested for the rehabilitation strategy.

**Area of a classroom:**  $A = 24 \text{ ft} * 30 \text{ ft} = 720 \text{ ft}^2$

**Wall area required:**  $0.5\% * A = 3.6 \text{ ft}^2$

**Wall area provided per classroom:**  $2 (8 \text{ in} * 36 \text{ in}) = 288 \text{ in}^2 = 4 \text{ ft}^2$ .

The two segments of RC wall area supplied to each classroom satisfy the 0.5% wall percentage requirement to reduce damages during an earthquake.

### 4.3.1. SHEAR CAPACITY CALCULATION OF THE RC WALL

$l_w = 36$  in (wall length)

$h_w = 20$  ft (total wall height)

$t_w = 8$  in (wall thickness)

The shear capacity of the RC wall can be calculated according to:

$$V_n = A_{cv} (\alpha_c \lambda \sqrt{f'_c} + \rho_n f_y);$$

Where:

$$\frac{h_w}{l_w} = \frac{20 \times 12}{36} = 6.67 \text{ then, } \alpha_c = 2.$$

$$\rho_n = \frac{A_s}{A_c} = \frac{2 \times 0.11 \times 12 / 10}{8 \times 12} = 0.00275.$$

Table 4-4. Shear capacity calculation of the RC wall.

$l_w$ (in)	$t_w$ (in)	$A_{cv} = t_w * l_w$ (in <sup>2</sup> )	$\alpha_c$	$\lambda$	$f'_c$ (psi)	$f_y$ (psi)	$\rho_n$	$V_n$ (kip)
36	8	288	2	1	4,000	60,000	0.00275	84

**$V_n = 84$  Kip**

**Case V: Rehabilitation strategy: introduction of reinforced concrete walls to the structure.**

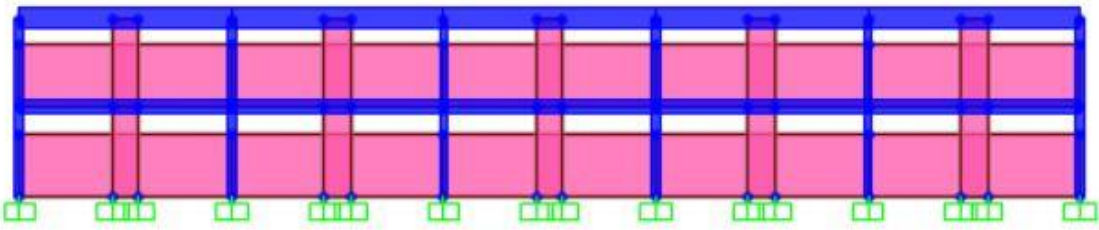


Figure 55. Frame I 2D model with the proposed rehabilitation strategy in SAP2000.

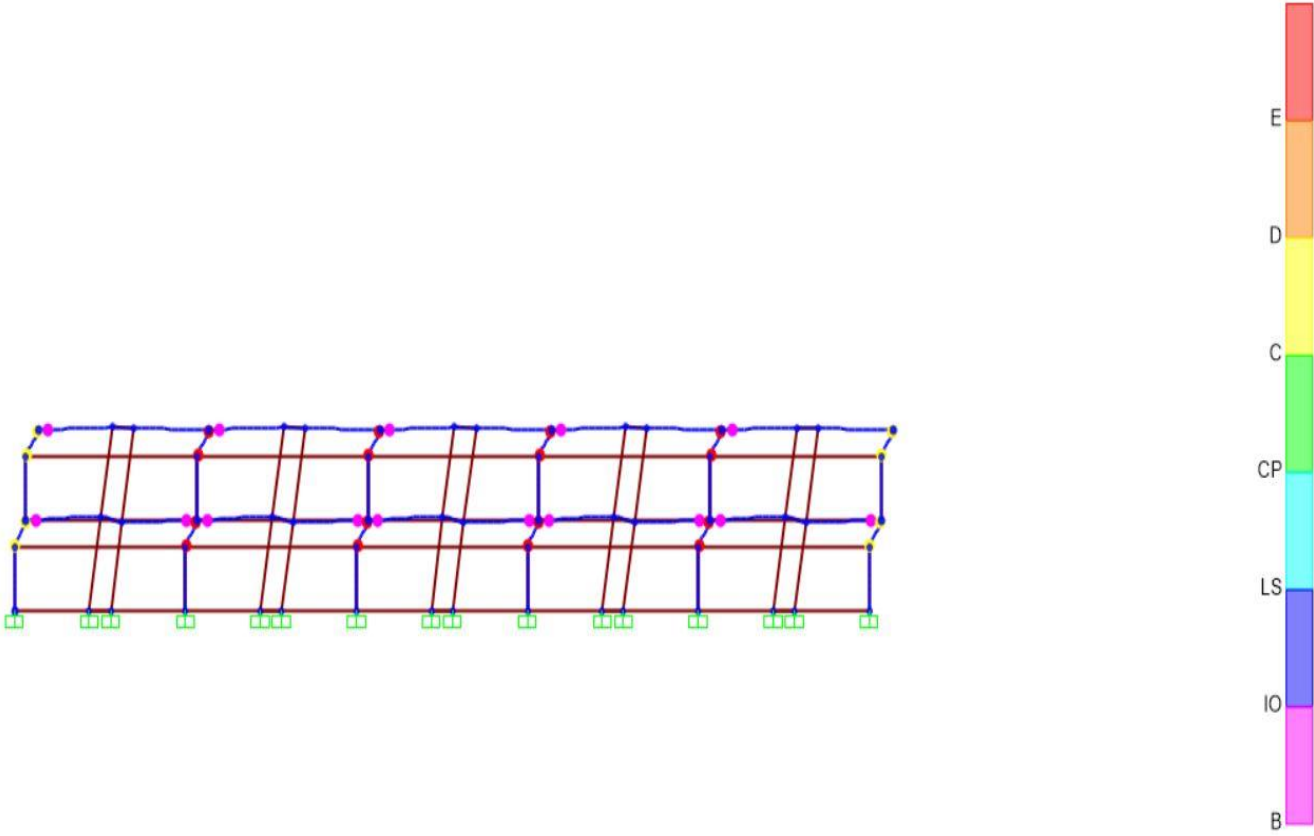


Figure 56. Sequence of plastic hinge formation – Case V.

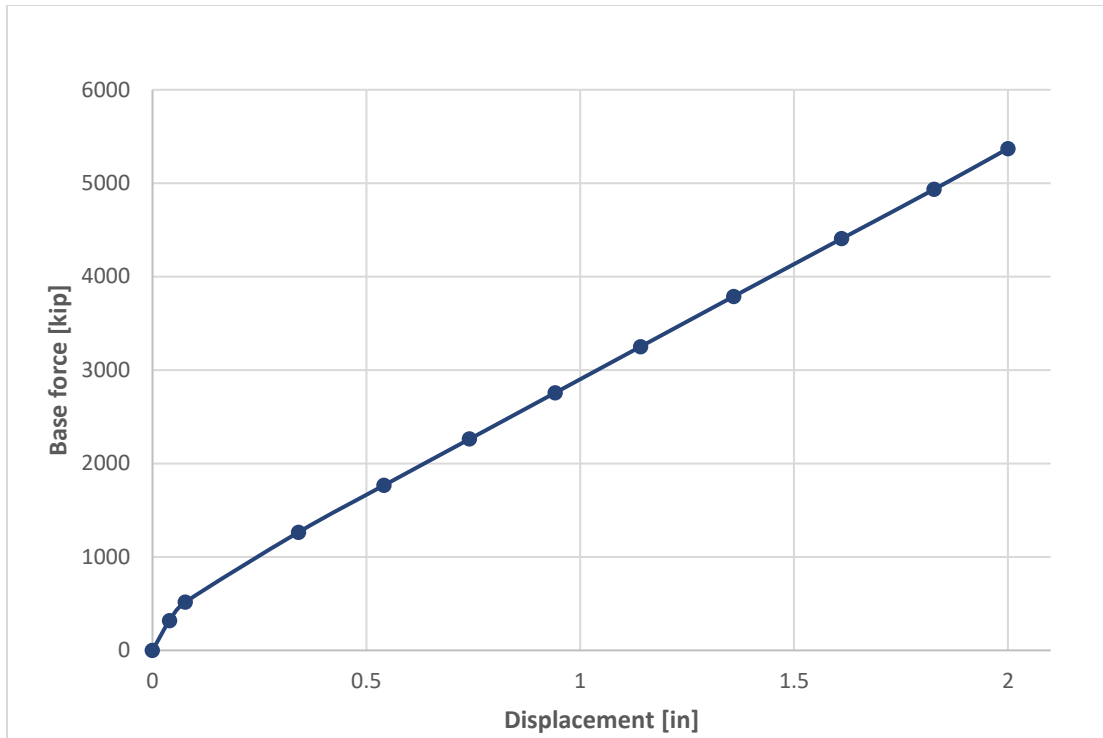


Figure 57. Pushover capacity curve of Frame I with the rehabilitation strategy inserted - Case V.

Table 4-5. Shear capacity of columns vs shear demand during the analysis – Case V.

Parameters	Column I	Column II	Column III	Column IV	Column V	Column VI
Shear capacity [kip]	62.9	29.5	29.6	29.6	29.5	62.9
Shear demand [kip]	61.2	30.9	30.9	30.9	30.9	61.2

A rehabilitation strategy was selected and implemented to the structure, which consisted of introducing reinforced concrete walls into the frame. The displacement-controlled Pushover Analysis of 2 inch was performed, and showed a substantial increase in the capacity of the structure from 528.1 kip to 5370.3 kip. Shear failures were considered for this analysis. Although the columns reached their shear capacity, the base force vs displacement graph remained in a linear range, and the capacity kept increasing. By introducing the reinforced concrete walls to the structure in the longitudinal direction, precisely a segment per classroom, the stiffness considerably increased, allowing the structure to carry more loads in this direction.

### Case VI: Comparison of the past analyses.

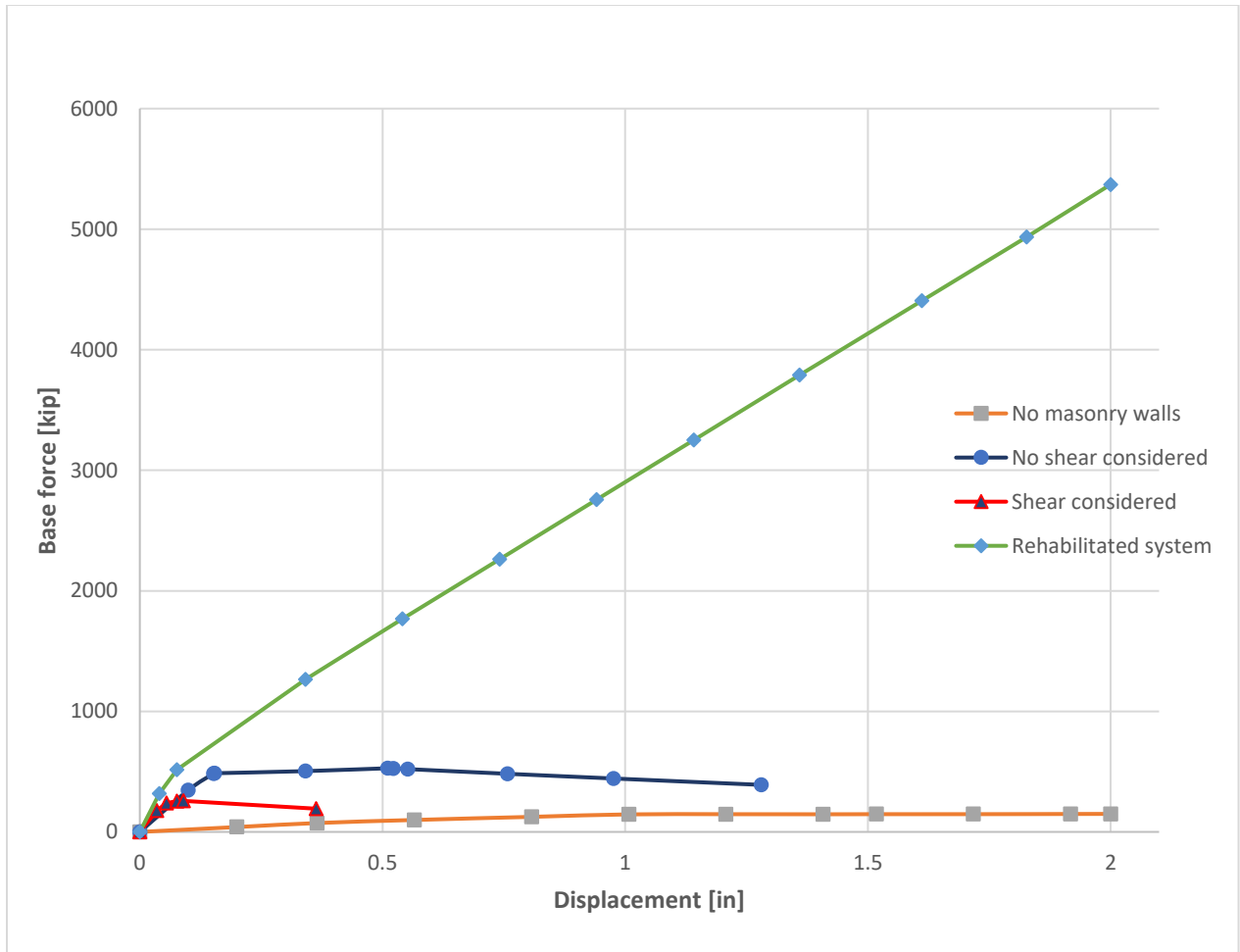


Figure 58. Pushover capacity curves for all the analyses performed on Frame I - Step VI

Figure 58 displays the base force vs displacement graph for all the Pushover analyses performed on Frame I, that includes the results of the Pushover performed on the frame with masonry walls with and without shear failures considered, and the results of the Pushover for the rehabilitation strategy selected. The reinforced concrete walls introduced to the system considerably increased the capacity of the structure and will carry most of the lateral load applied to the system. The results of the Pushover analysis with the rehabilitation strategy selected were quite satisfactory for the purpose of this research, as it allowed the structure to remain linear, and essentially, no shear failures of the columns was registered during the analysis.

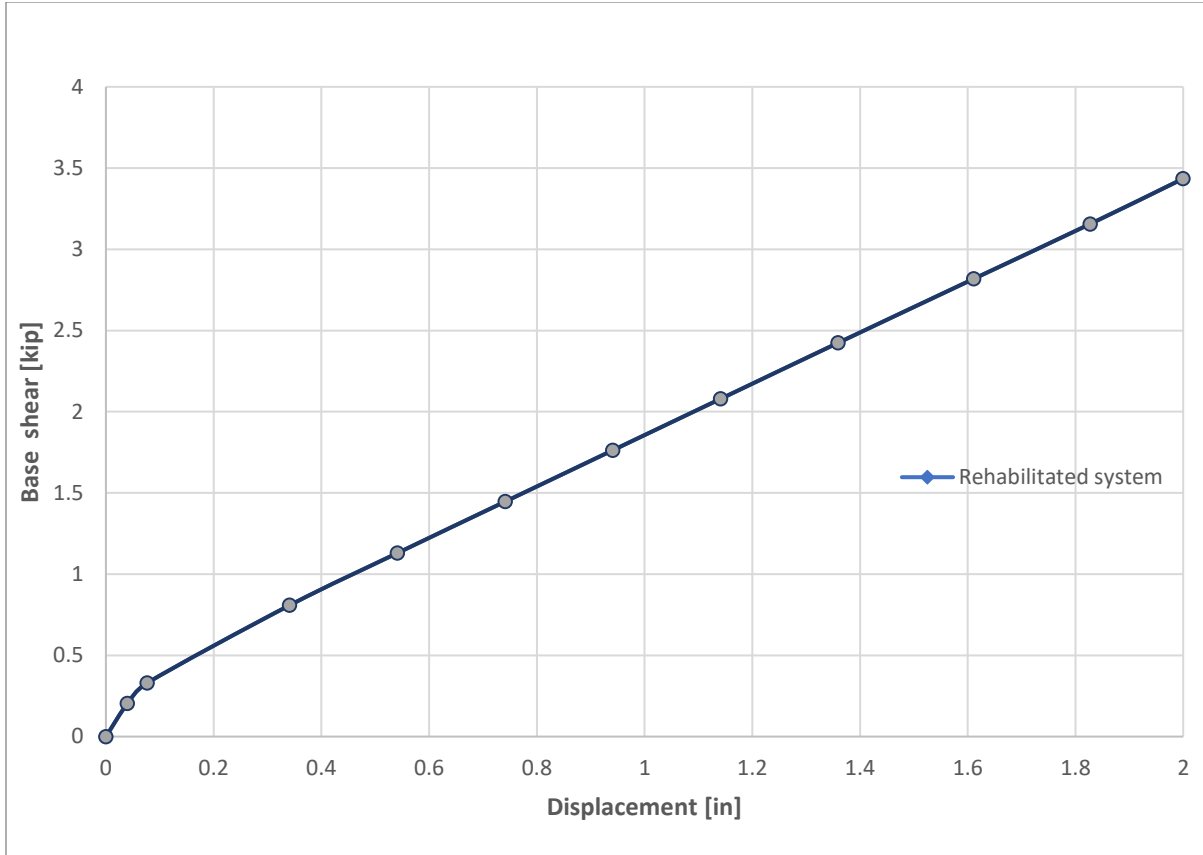


Figure 59. Pushover capacity curve of Frame I with the rehabilitation strategy inserted, when the base shear is divided by the total weight of module D.

Figure 59 presents the Pushover capacity curve when the base shear of Frame I with the rehabilitation strategy inserted is divided by the total weight of the module D. The total weight obtained in SAP2000 is 1563.4 kip.

## FRAME II

Case I: Frame II without masonry walls in the longitudinal direction

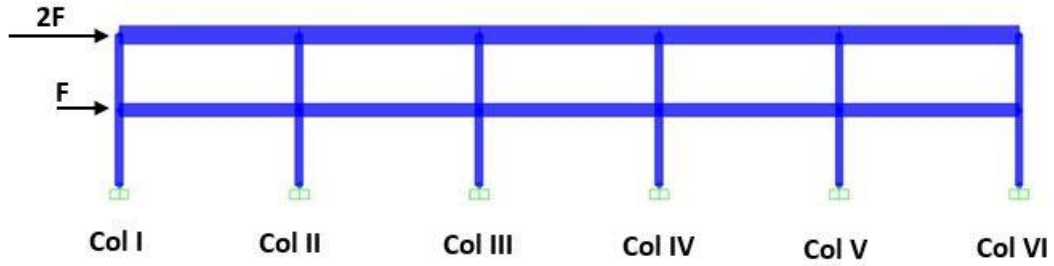


Figure 60. Frame II 2D model without masonry walls in the longitudinal direction in SAP2000.

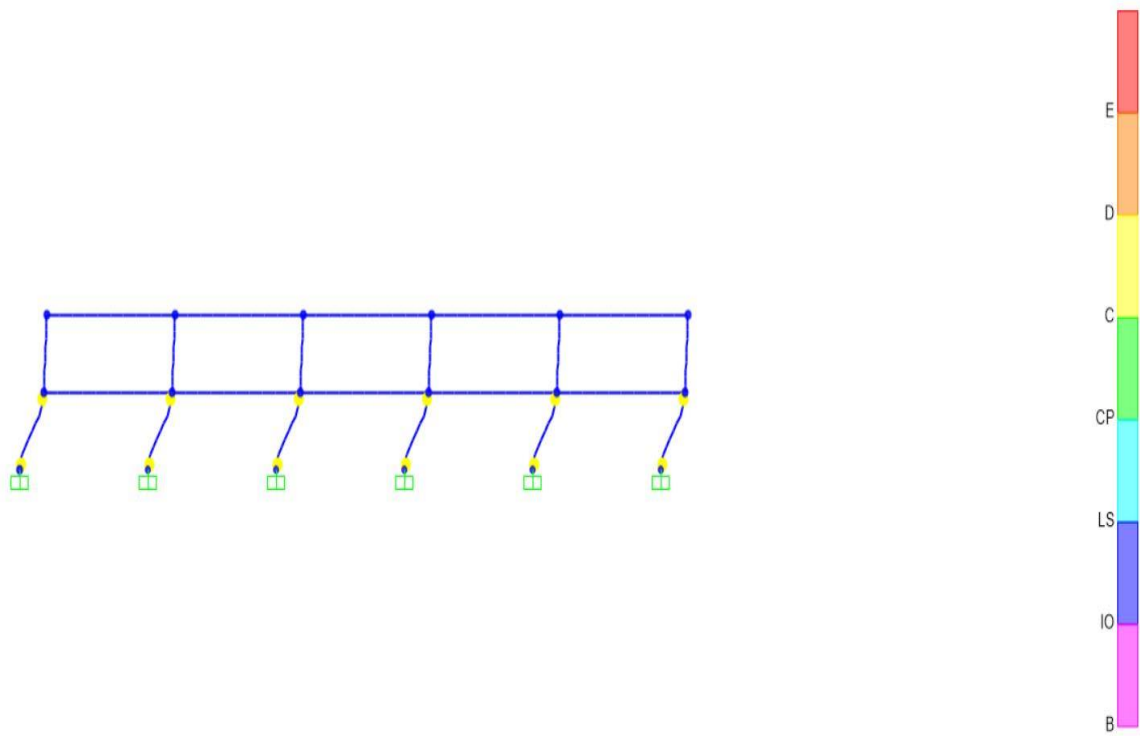


Figure 61. Sequence of plastic hinge formation – Case I.

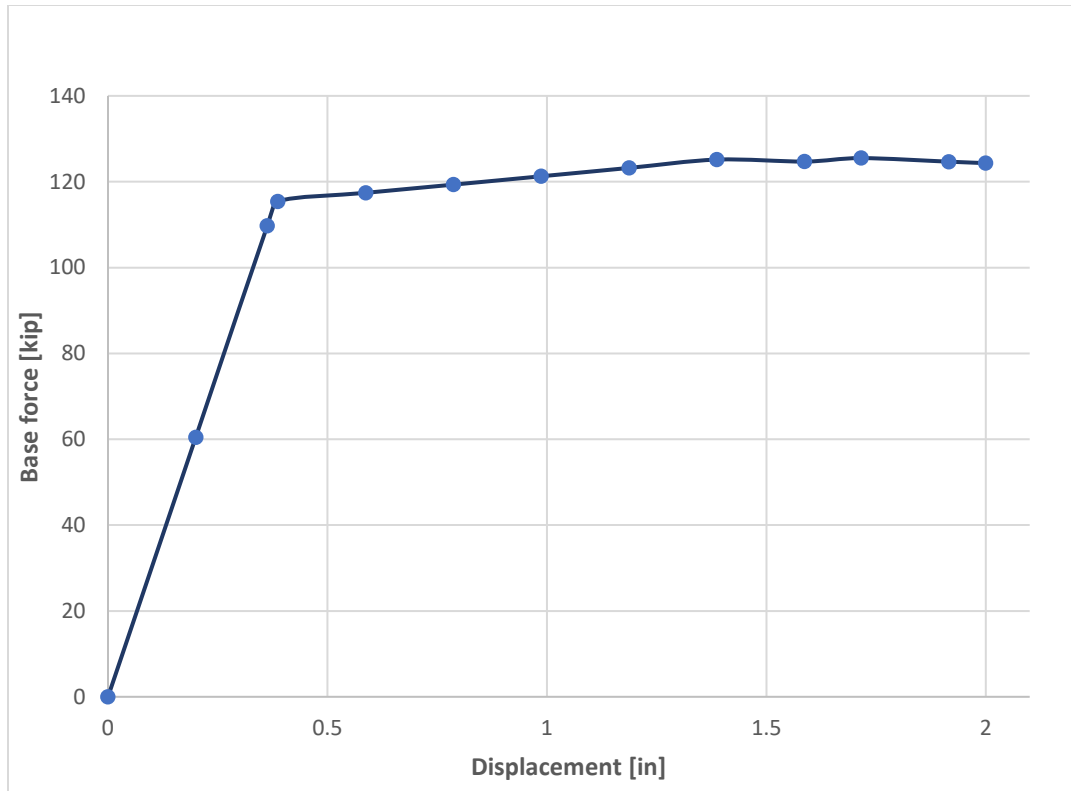


Figure 62. Pushover capacity curve of Frame II – Case I.

Table 4-6. Shear capacity of columns vs shear demand during the analysis – Case I.

Parameters	Column I	Column II	Column III	Column IV	Column V	Column VI
Shear capacity [kip]	63.3	65.2	65.2	65.2	65.2	63.3
Shear demand [kip]	20.0	21.4	21.4	21.4	21.4	20.0

Similarly, a 2 inch displacement-controlled Pushover analysis was performed on Frame II, where no masonry walls were considered in the longitudinal direction of the structure. The maximum applied shear was 125.5 kip, and no shear failure was noticed when the shear capacity of each column was calculated and compared to the maximum shear produced by the lateral loads applied for each corresponding column during the analysis. From the results of this analysis, Frame II did not undergo any shear failure when performing the Pushover analysis.

**Case II: Frame II with masonry walls in the longitudinal direction when shear failures were not considered by SAP2000.**

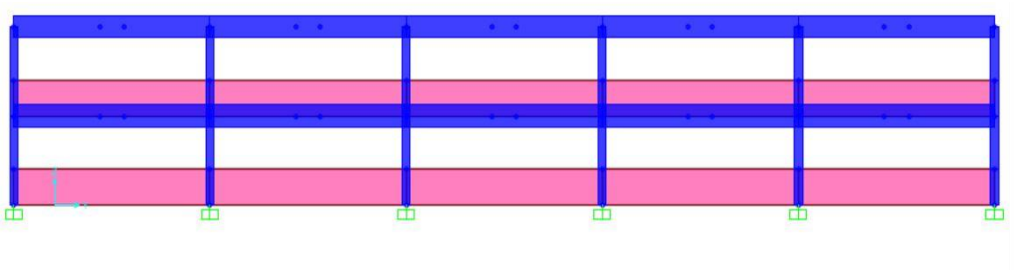


Figure 63. Frame II 2D model without masonry walls in the longitudinal direction in SAP2000.

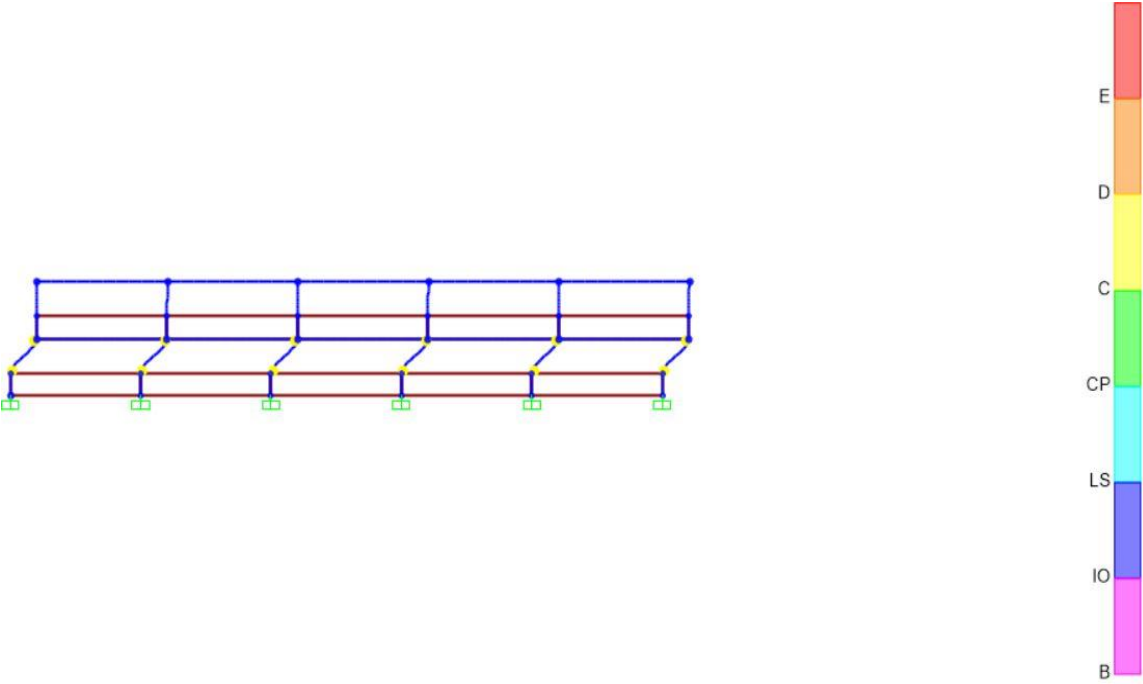


Figure 64. Sequence of plastic hinge formation – Case II.

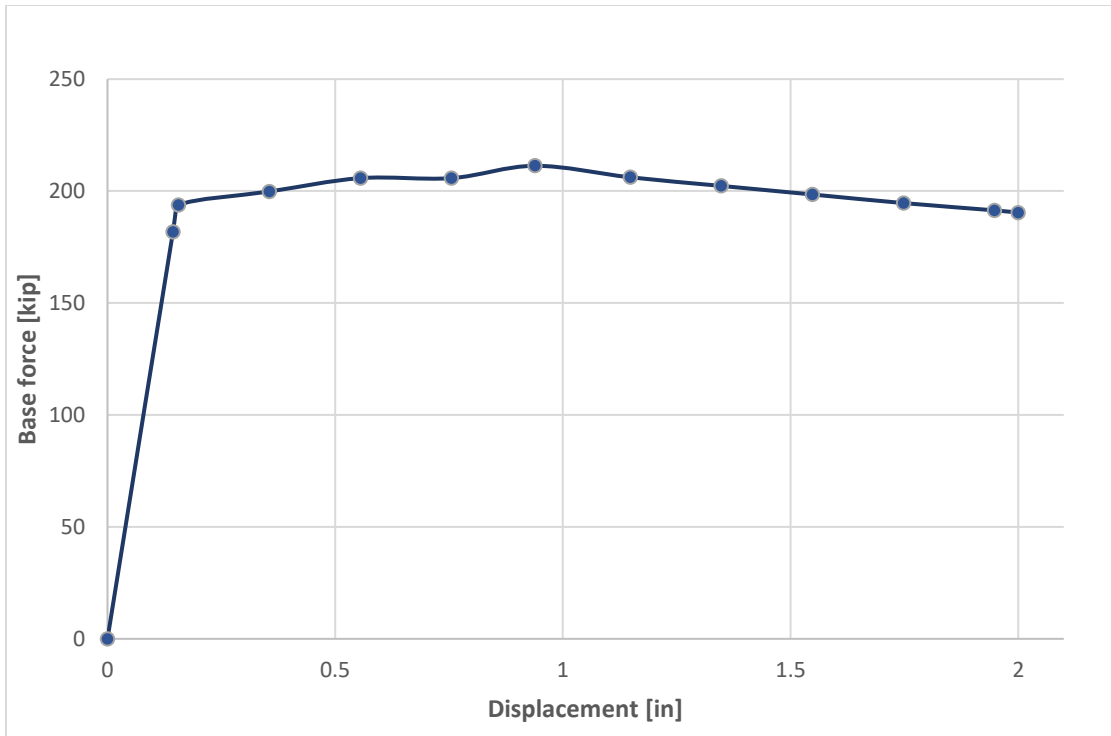


Figure 65. Pushover capacity curve of Frame II – Case II

Table 4-7. Shear capacity of columns vs shear demand during the analysis – Case II

Parameters	Column I	Column II	Column III	Column IV	Column V	Column VI
Shear capacity [kip]	63.4	65.5	65.3	65.3	65.3	63.4
Shear demand [kip]	33.0	35.9	36.0	36.0	36.0	33.0

The 2 inch displacement-controlled Pushover analysis for Frame II displays quite better results because the shear applied did not reach the shear capacity of the structure. Although the introduction of the masonry walls increased the capacity from 125.5 kip to 211.3 kip, the columns did not experience any shear failure. Frame II does not necessarily need a rehabilitation system, but since it is a part of the module selected, the reinforced concrete walls will be inserted in the longitudinal direction to reduce torsional effects.

**Case III: Rehabilitation strategy: introduction of reinforced concrete walls to the structure.**

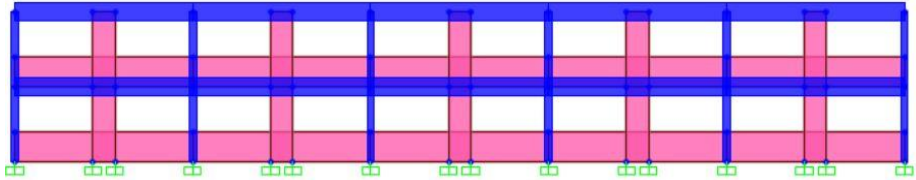


Figure 66. Frame I 2D model with the proposed rehabilitation strategy in SAP2000.

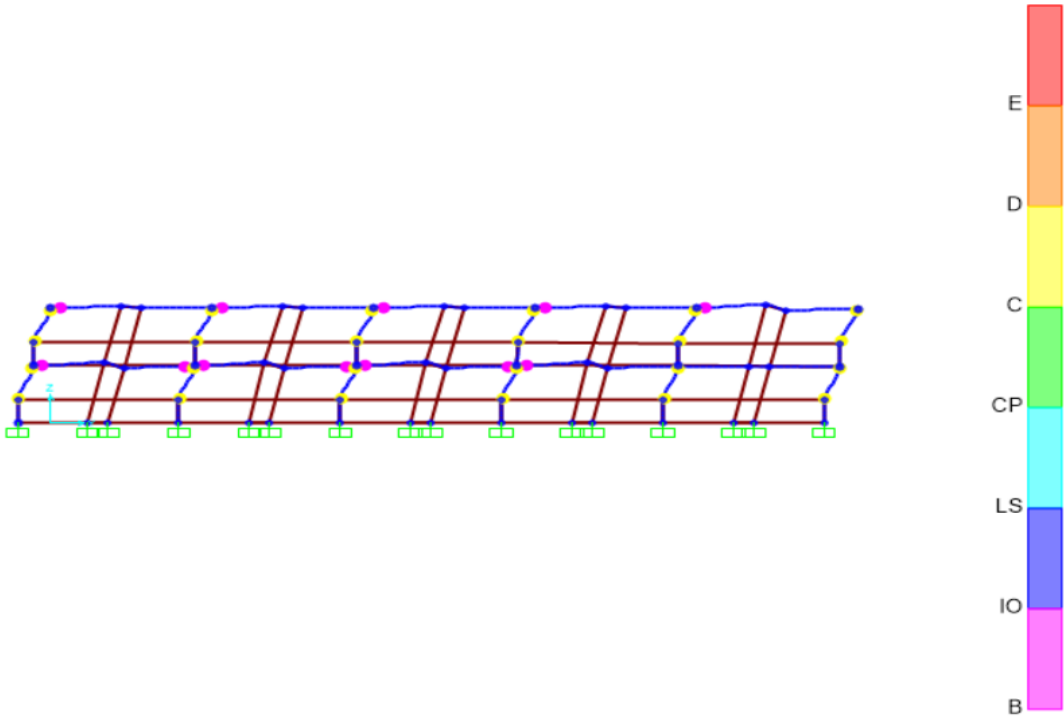


Figure 67. Sequence of plastic hinge formation – Case III.

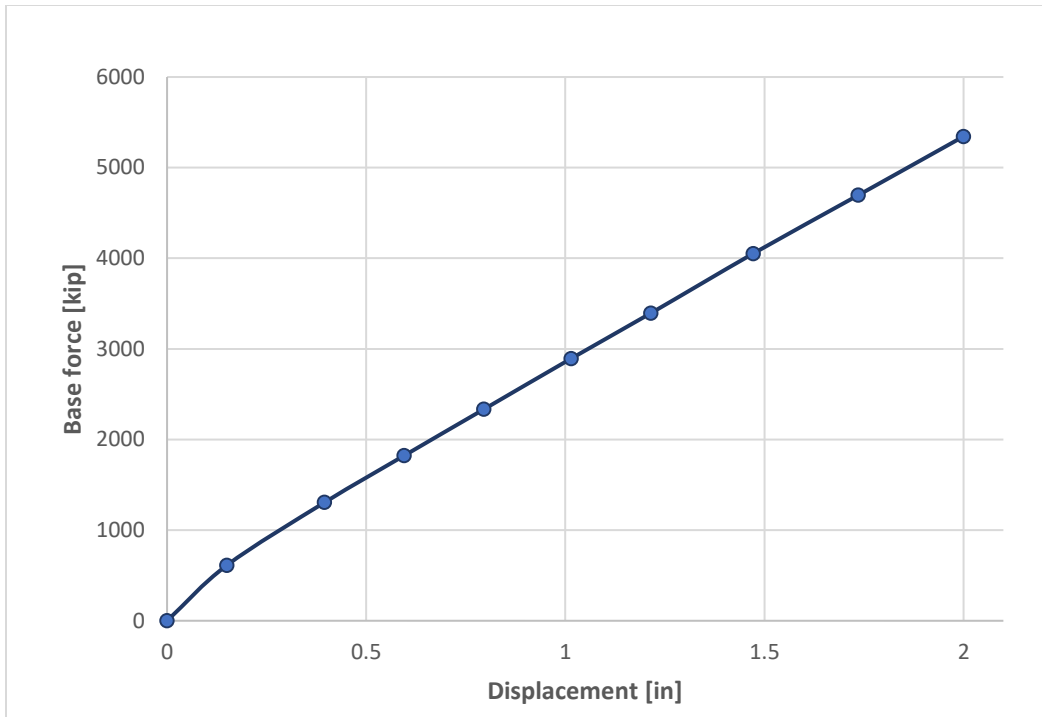


Figure 68. Pushover capacity curve of Frame I with the rehabilitation strategy inserted - Case III.

**Case IV: Comparison of the past analyses.**

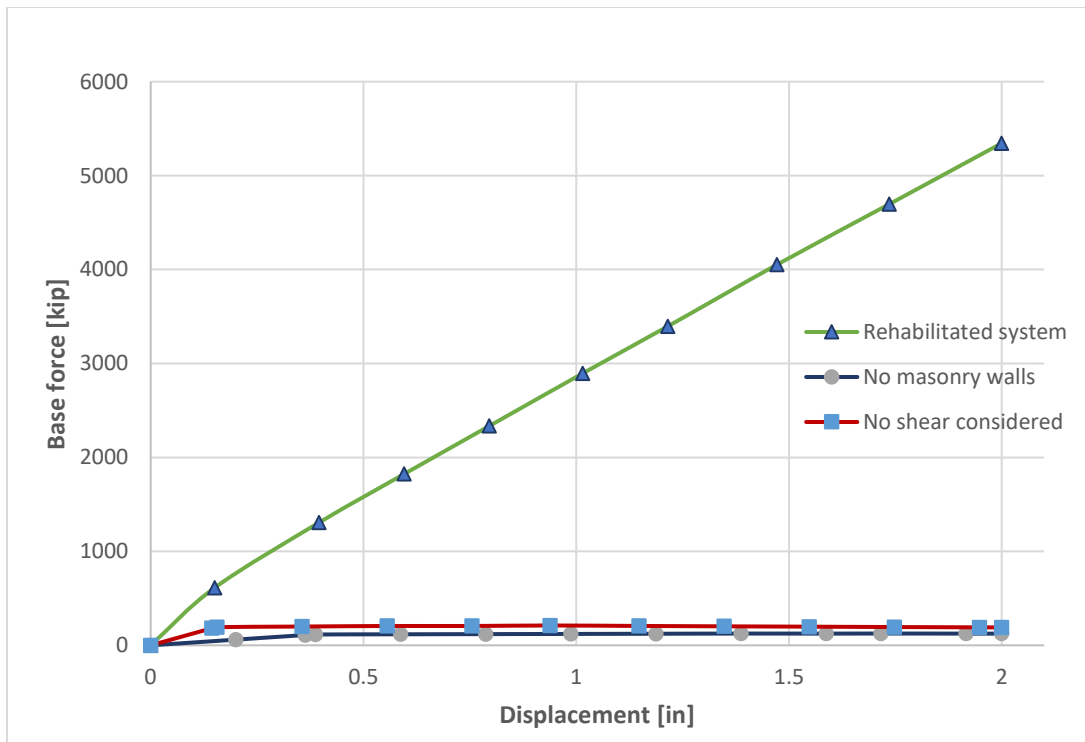


Figure 69. Pushover capacity curves for all the analyses performed on Frame II – Case IV

Since the rehabilitation strategy was not required for Frame II to assess the short column effect, the procedure became less complicated. The reinforced concrete walls will be inserted to preserve the geometric pattern to reduce the torsional effect. The capacity of the structure substantially increased from 211.3 kip to 5344.5 kip.

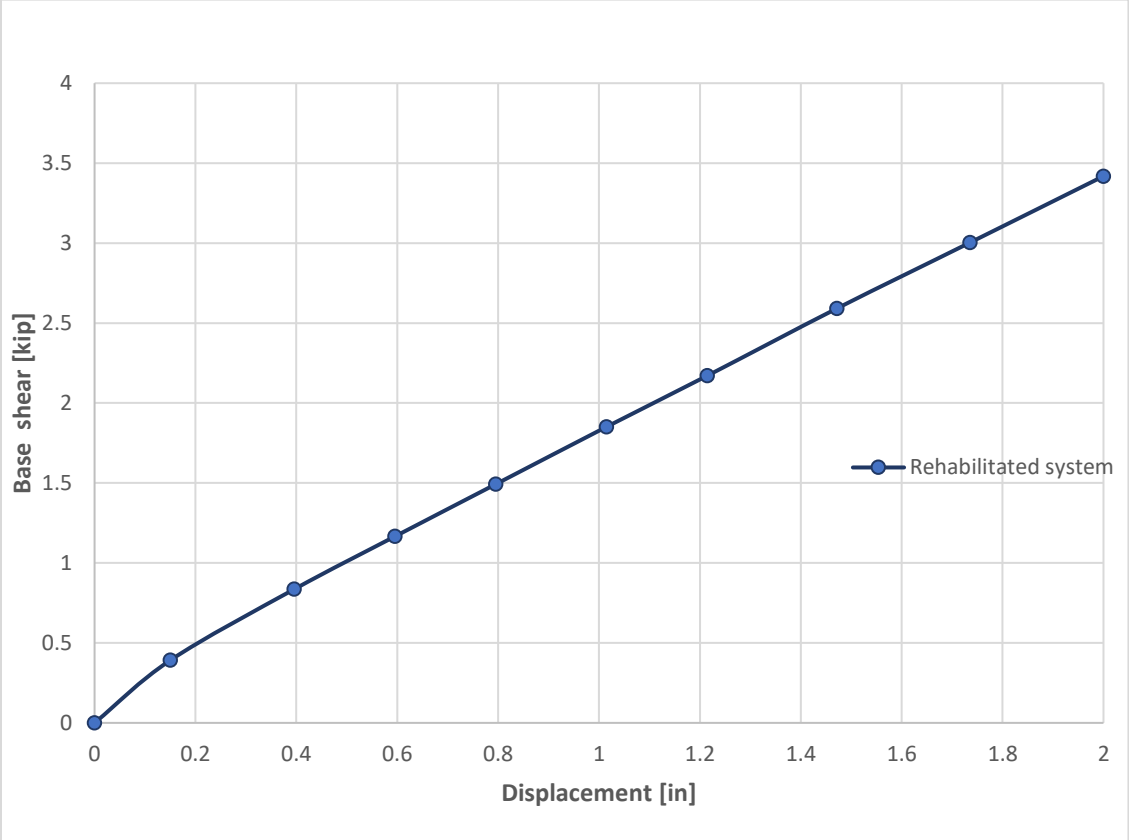


Figure 70. Pushover capacity curve of Frame II with the rehabilitation strategy inserted, when the base shear is divided by the total weight of module D.

Figure 70 presents the Pushover capacity curve when the base shear of Frame II with the rehabilitation strategy inserted is divided by the total weight of the module D.

#### 4.4. TIME HISTORY ANALYSIS RESULTS

A linear time history analysis was performed on the selected module using the probable earthquakes that may occur in Ponce, Mayagüez and San Juan, compatible with the elastic design spectrum of ASCE7-16 to study the structure behavior. More details about those earthquakes are presented in Chapter 4. The results of these analyses are displayed in Table 4-8 and Table 4-9.

Table 4-8. Peak values of demand parameters of the selected module for PONCE, PR.

Parameters	CIG.EW	HSB.EW	IGN.EW	IVU.EW	UCA.EW	USACA18.033	USACA36.011
Peak velocity [in/s]	1.47	1.65	1.10	1.84	1.71	1.60	1.95
Peak displacement [in]	0.021	0.018	0.018	0.018	0.017	0.017	0.019
Base shear [kip]	192.8	162.8	170.3	166.8	151.4	151.9	172.1

Table 4-9. Peak values of demand parameters of the selected module for SAN JUAN, PR.

Parameters	JAP03.104	JAP03.173	MEX03.112	PACOIMAW.V2	ROLLHILL.V2	USAAAK02.067	USACA66.001
Peak velocity [in/s]	1.95	1.84	1.54	1.743	1.95	1.87	1.78
Peak displacement [in]	0.017	0.016	0.016	0.018	0.017	0.019	0.015
Base shear [kip]	150.3	141.7	149.6	158.0	146.5	169.3	125.1

The results for the general response of the module when Time History analysis was performed using the 7 probable earthquakes selected for the Ponce and San Juan areas was quite satisfactory. The maximum base shear generated, 238.8 kip, did not exceed the capacity of the rehabilitated system which was 5370.3 kip. The Time History analysis shows that the selected strategy helped carry the lateral load applied and strengthen the structure enough to avoid shear failure of the columns. Thus, the introduction of reinforced concrete walls method was adequate to rehabilitate the structure.

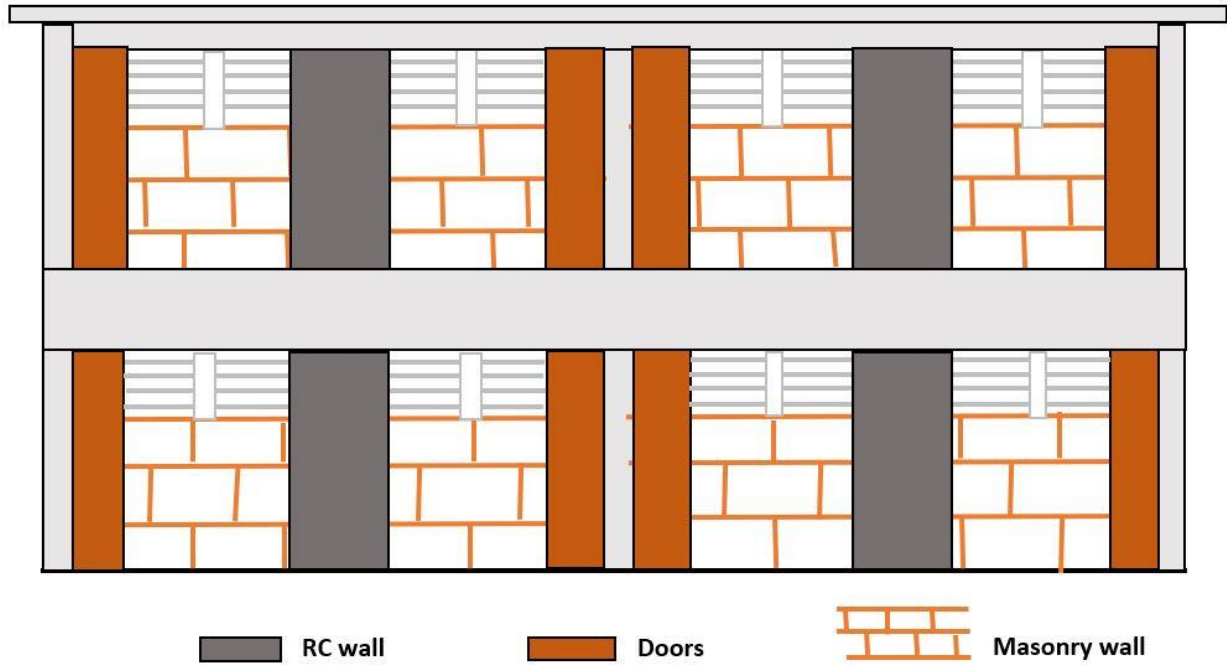


Figure 71. Front view of the selected module of typical school building in Puerto Rico with the suggested RC walls – Frame I.

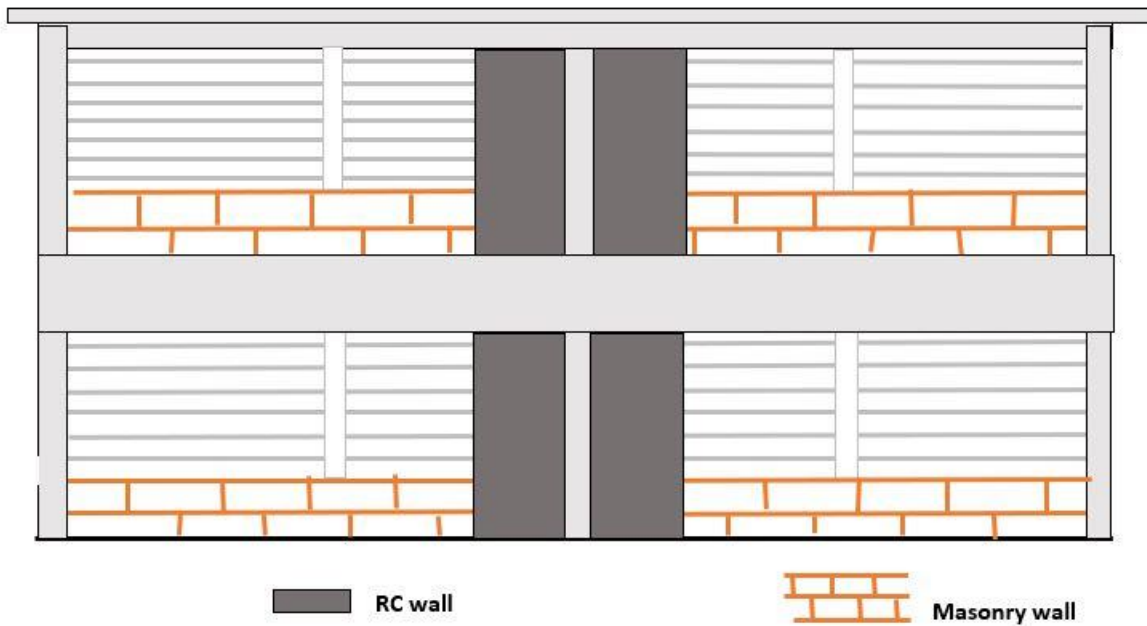


Figure 72. Back view of the selected module of typical school building in Puerto Rico with the suggested RC walls – Frame II.

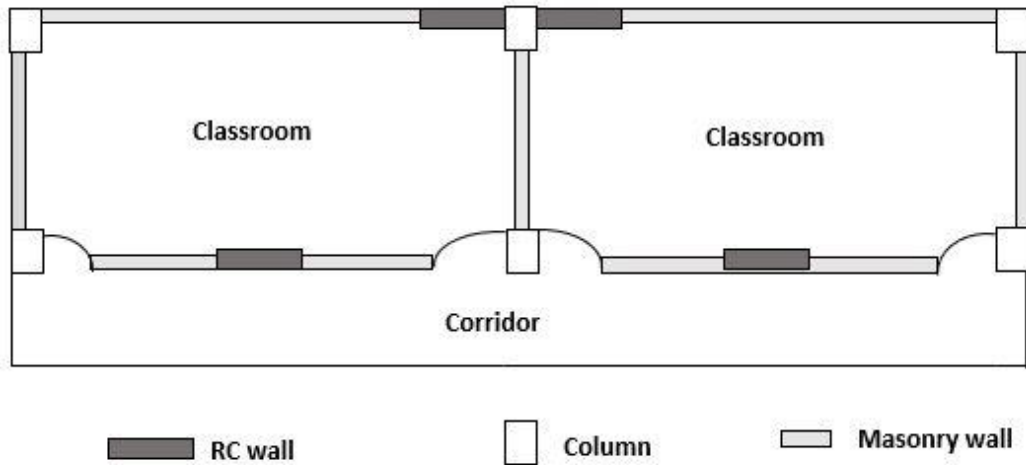


Figure 73. Top view of the selected module of a typical school building in Puerto Rico with the suggested RC walls.

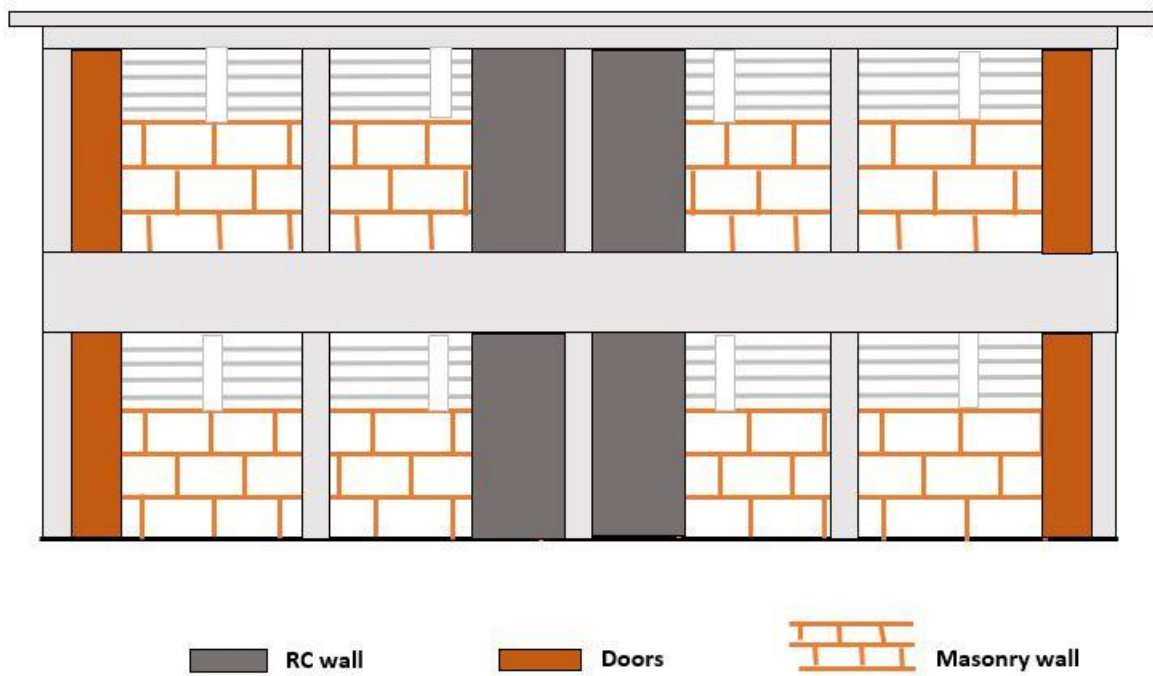


Figure 74. Front view of another module of a typical school building in Puerto Rico with the suggested RC walls – Frame I.

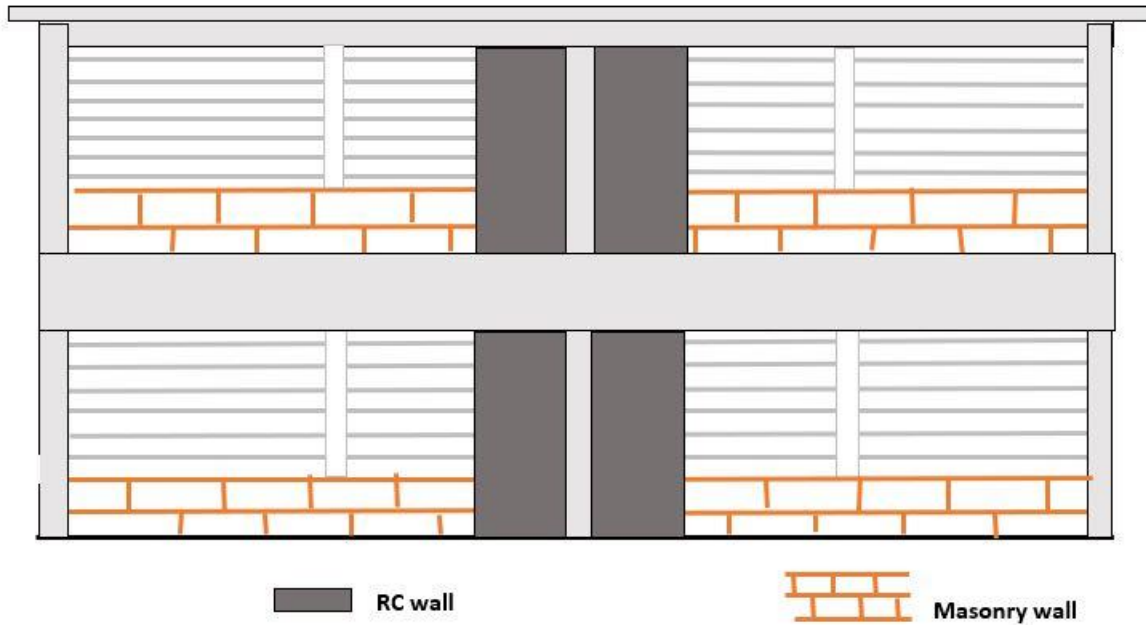


Figure 75. Back view of another module of a typical school building in Puerto Rico with the suggested RC walls – Frame II.

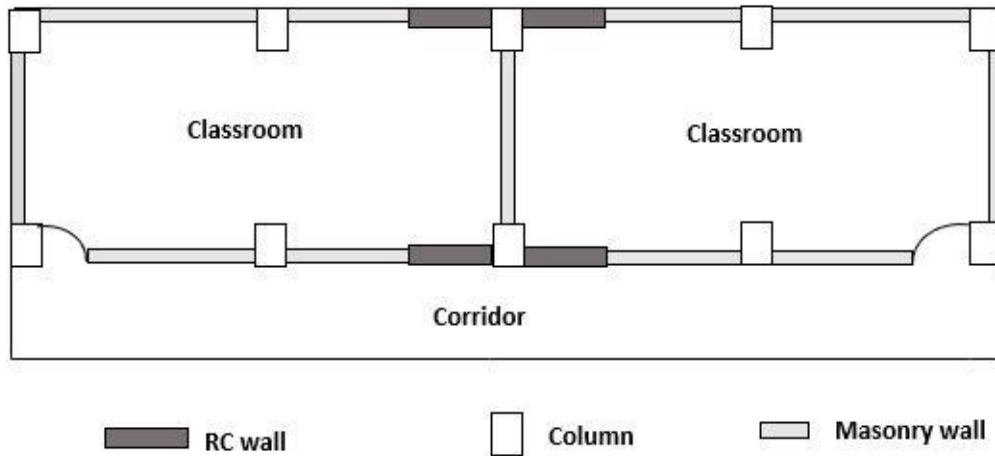


Figure 76. Top view of another module of a typical school building in Puerto Rico with the suggested RC walls.

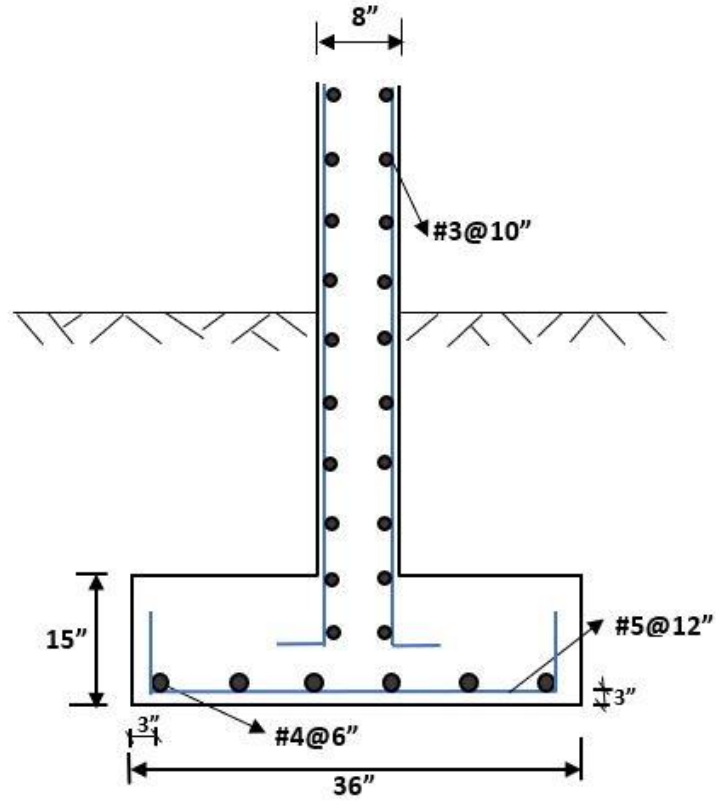


Figure 77. Recommended foundation for the suggested RC walls

## 4.5. OTHER ANALYSIS

The previous assessments were performed on a module without walls in the longitudinal direction of the structure. This scenario, however, provided a better understanding of the influence of the walls when inserting them in the structure. In fact, the short column effect is due to the presence of the mid-height masonry walls whose influence was not considered in the analysis and design phases of the structure. Reviewing the drawing plans, some modules have bathrooms placed in their longitudinal direction. Reinforced concrete walls are now placed in the longitudinal direction to set up the bathroom's walls. This assessment was conducted to evaluate the influence of the bathroom's walls, and if or if not, a rehabilitation strategy is required for this module.

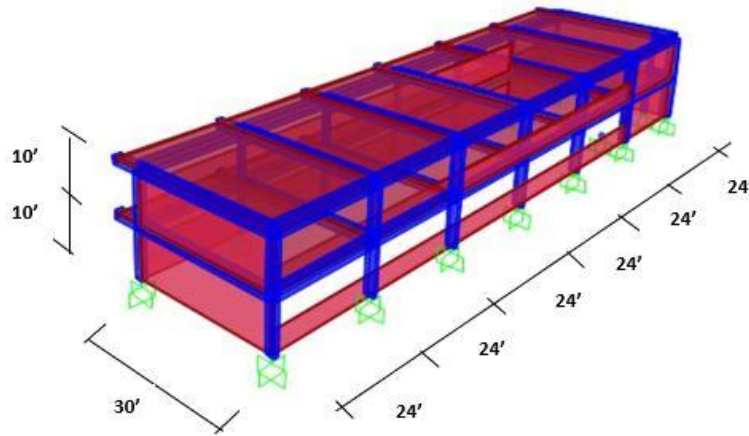


Figure 78. Module D with bathrooms inserted in SAP2000.

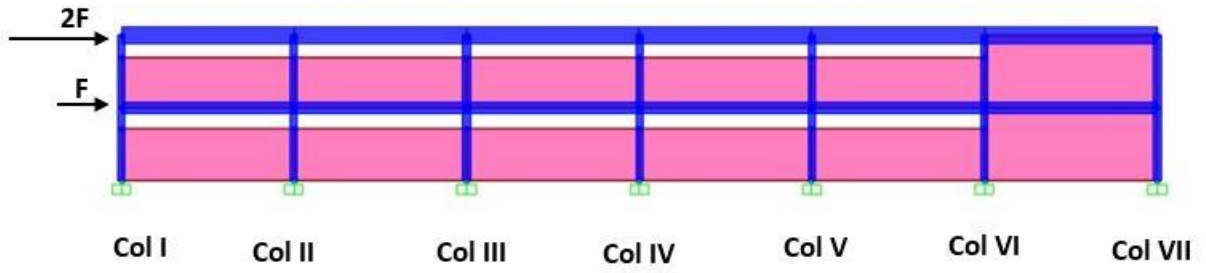


Figure 79. Frame I 2D model in SAP2000.

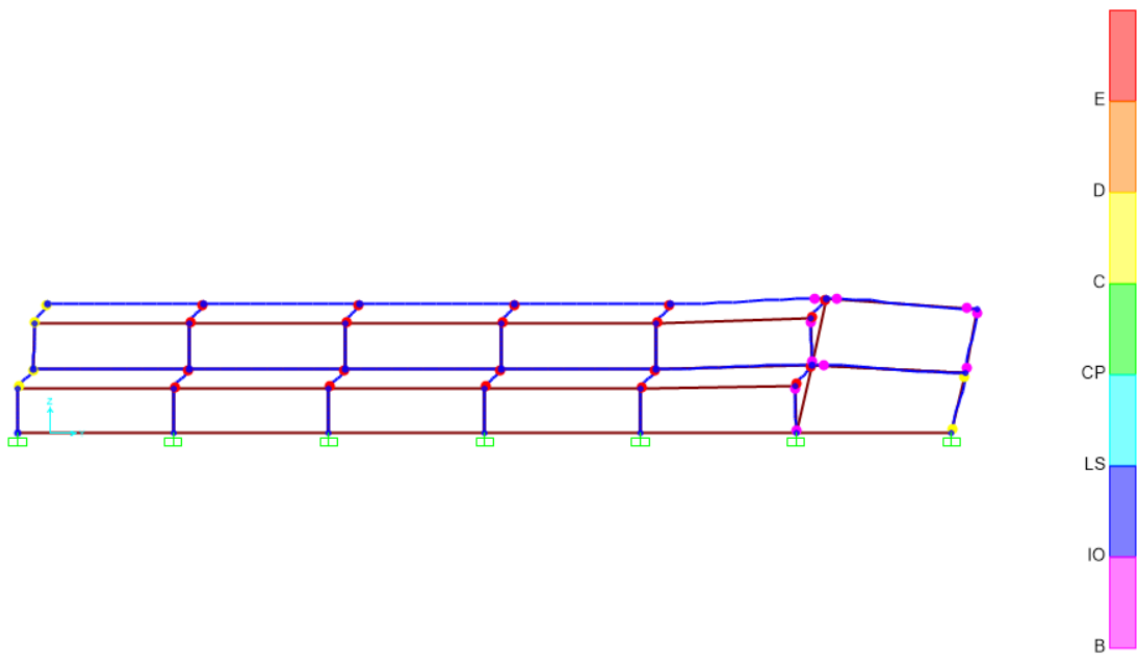


Figure 80. Sequence of plastic hinge formation.

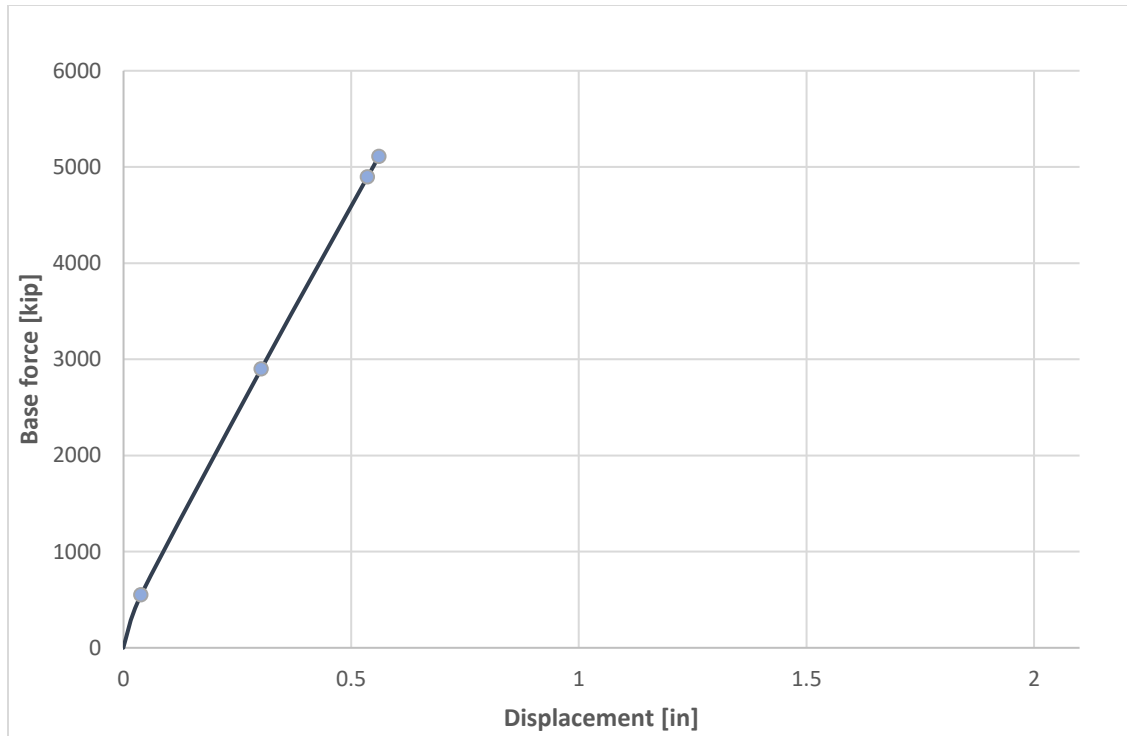


Figure 81. Pushover capacity curve of Frame I Module D with bathrooms inserted in SAP2000.

Table 4-10. Shear capacity of columns vs shear demand during the analysis

Parameters	Column I	Column II	Column III	Column IV	Column V	Column VI	Column VII
Shear capacity [kip]	64.4	32.1	32.3	32.3	32.1	28.8	61.7
Shear demand [kip]	61.5	29.5	29.5	29.5	29.5	29.5	18.2

A 2 inch displacement-controlled Pushover analysis performed on Frame I revealed that the base force vs displacement graph remained linear during the analysis. The capacity of the structure was 5109.9 kip when shear failures of the columns were considered. The presence of the masonry walls of the bathrooms increased the stiffness of the frame and helped carry the lateral load applied. From the results of this analysis, the rehabilitation strategy of inserting reinforced concrete walls to the structure is not required when bathrooms are placed in the longitudinal direction of the frame.

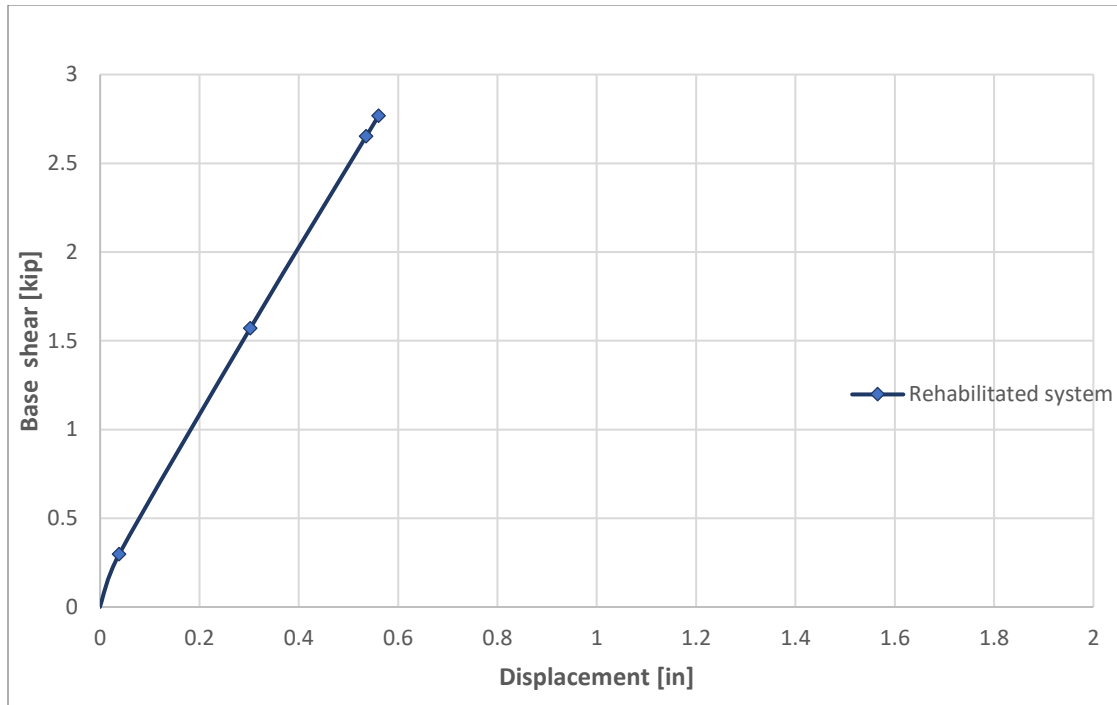


Figure 82. Pushover capacity curve of Frame I with bathrooms inserted in SAP2000, when the base shear is divided by the total weight of module D.

Figure 82 presents the Pushover capacity curve when the base shear of Frame I with bathrooms inserted in SAP2000, with the rehabilitation strategy is divided by the total weight of the module D with bathrooms. The total weight obtained in SAP2000 is 1845.7 kip.

## 5. CONCLUSIONS AND RECOMMENDATIONS

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### 5.1. SUMMARY

The introductory section of this research presents the major earthquakes that has struck Puerto Rico, the seismic threats and risks the island is facing due to seismic activity along the seismic faults, and due to the increase in infrastructure and population. Then, a definition of the short column effect followed by a mathematical illustration to better understand the phenomenon was drawn up, as well as some typical formation scenarios of the short column effect. The objective of this research is to propose a solution for the seismic rehabilitation of existing public schools in Puerto Rico for the Department of Education. In order to proceed further, it was imperative to deeply investigate the phenomenon; therefore, an exhaustive literature review was conducted and presented in the second chapter, presenting various examples of several earthquakes that have taking place around the world, and the corresponding damages caused by the presence of the short column effect. Furthermore, research performed regarding the subject are presented, as well as the approaches of different investigators to provide solutions that help mitigate the short column effect. A typical school in tropical countries was selected to perform the analyses. Architectural and structural drawings were provided by the Public Buildings Authority. Using the information obtained from the review of drawings, the next step was to prepare a 3D computer model of the structure. The considerations and assumptions made around the edification of the model in SAP2000 are well explained in the Chapter 3, as well as the procedures leading to the execution of the analyses. Two analyses were performed: a nonlinear static Pushover and a linear dynamic analysis. The Chapter 4 presented the results and the discussion of the analyses executed. The first analysis consisted in submitting the structure to a nonlinear static Pushover analysis not considering non-structural elements, to determine its capacity and mode of failure. Even if it can be considered hypothetical, it was used to assess the influence of the non-structural elements of the building and how the short column effect is created. Later, the non-structural

elements, by means the masonry walls, were included to the model. The second analysis consisted in performing the nonlinear static Pushover analysis to determine the new capacity of the structure and the failure mechanism with the non-structural components included. From this new model, an initial analysis was carried out to determine the capacity of the structure by considering only flexural failures. Subsequently, another analysis was performed to evaluate the capacity of the structure when shear failures of the columns were taken into consideration. Once a member reached its shear capacity, hinge was formed. The shear capacity of the cross-section of the columns was obtained using the 11.5.4.4 formula from the ACI318-14 which consists on summarizing the input of the shear capacity of the concrete and the input of the shear capacity of the ties of transversal reinforcement. Then, a rehabilitation solution was selected and proposed to address the issue. A strategy considered to be effective and economical was selected and applied to this investigation. This rehabilitation strategy was the introduction of the reinforced concreted walls to the structure. Once the reinforced concrete walls were introduced to the models, the nonlinear static Pushover analysis was performed, and the results of all the previous analyses were presented and compared. Finally, the efficacy of the selected rehabilitation solution was verified by submitting the structure to a linear dynamic analysis which consisted of obtaining earthquakes records compatible with the elastic design spectrum of ASCE7-16 from past investigations, and submit the structure to these records to study its behavior.

## 5.2. CONCLUSIONS

Observations from past earthquakes have proven how destructive the shear column effect can be to infrastructure when it is not properly addressed. As for school buildings that are designed to be operational after seismic events, it is imperative to address this issue. The methodology developed for this research has brought some major uncovering around the topic. The first nonlinear static analysis performed on the module selected highlights the influence of the masonry walls. This hypothetical scenario presents the structural analysis and design real considerations. Two hypotheses can be considered: first, the engineer of engineer staff who drew the structural components is not the same as the one who drew the non-structural components; second, an engineer or engineer staff drew the entire structure but did not considered the influence of the non-structural components. Consequently, according to the result of this analysis, the structure has a good structural behavior, the columns resist perfectly well presenting only flexural failures, no shear failures are noticed, but unfortunately, the reality is different. The second analysis performed considered the insertion of the masonry walls into the structure before performing the nonlinear static Pushover analysis. The capacity of the structure drastically increased due to the presence of the masonry walls. The sequence of plastic hinge formation of the second analysis showed early precipitous failures of the short columns when shear failures are taken into consideration. The results of this analysis show that shear brittle failure occurs in the columns before the structure reaches its flexural capacity. The shear column effect is created by the introduction of the mid-height masonry walls.

The insertion of the reinforced concrete walls, as a method of rehabilitation to the structure, substantially increased its capacity. In fact, the reinforced concrete walls increased the stiffness of the structure, hence, the lateral force required is harshly higher. The reinforced concrete walls carry out most of the lateral force, reducing to a minimum the vulnerability of the short column to fail in shear.

Later, a time history analysis was performed to validate the rehabilitation strategy selected. Results of this analysis show that the local responses of the structure considering the short

column effect, remain within the established limits, hence, if an earthquake were to happen, assuming that the conditions of its occurrence will remain the same, the seismic rehabilitation solution selected will be quite satisfactory for the structure to adequately behave. The rehabilitation system must include the foundation design of the new wall. A relevant aspect of introducing the RC walls is to significantly reduce the lateral displacement of the structure, reducing or even avoiding damage to non-structural elements. This in turn, helps with the psychological aspect of human beings by avoiding panic during the telluric motion.

For the module selected, the two frames were independently studied. Results show that the frame between the classrooms and corridor is the most vulnerable. Since the columns of this frame are shorter, they are stiffer, and consequently attract larger amount of load. The other frame does not necessarily require a rehabilitation system, as the corresponding columns of this frame does not suffer any shear failure during the analysis. But the reinforced concrete walls can still be inserted to this frame to minimize torsional effects.

Finally, a module with walls in the longitudinal direction was selected due to the presence of the masonry walls of the bathroom to perform the same previous analyses. The masonry walls of the bathrooms can strengthen the structure by increasing its capacity. Since the columns will not have to endure any shear failure during an earthquake, the short column effect will be reduced by the presence of the masonry walls of the bathrooms. Thus, a module with a bathroom in its longitudinal direction does not require a rehabilitation strategy to assess short column effect issue. However, by inserting the masonry walls of the bathrooms, the structure attracts a larger lateral force and needs to properly distribute this force through the foundation of the structure. The existing foundation system was not designed for a such large force, so, special attention is required to adequately design the foundation to distribute the force to the ground.

### 5.3. RECOMMENDATIONS

- It is recommended to validate the proposed rehabilitation system of this research for all similar typical public schools in Puerto Rico, which consist in inserting RC walls in each frame of the structure, two segments by classrooms, considering various alternatives available to insert the RC walls. These walls will attract more lateral load, but they have the capacity to resist the applied load, hence, that implies that they will require a larger foundation. The RC wall design must include an adequate foundation design.
- As the modules are connected through the corridor slab and work individually, it is recommended to study the response of the modules as they collide with each other. This assessment is beyond the scope of this work, hence was not considered.
- Finally, it is recommended to use local earthquakes of smaller magnitudes, and properly modify the accelerograms, so that the seismic scenario can be more realistic.

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## APPENDIX

Shell element was used in SAP2000 to model the masonry wall. However, there are multiple methods available to model a masonry wall in SAP2000, such as diagonal strut, flexible diaphragm analysis, rigid diaphragm analysis, and so on. Rigid diaphragm analysis will be used to validate the shell element method used to model the masonry wall. The rigid diaphragm analysis consists of distributing the lateral loads to the walls, considering the relative stiffness of the individual walls. The masonry walls will be modeled using a rigid link in SAP2000, by assigning the value of the relative stiffness of each wall to a rigid link.

Shear walls are either considered cantilevered (Figure 83) or fixed (Figure 84).

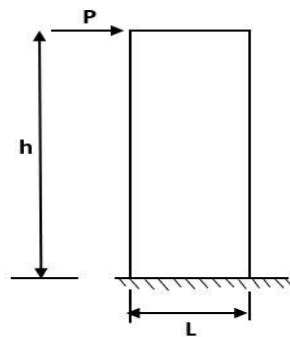


Figure 83. Cantilevered wall

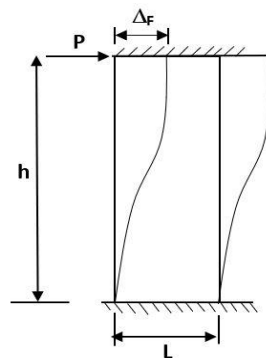


Figure 84. Fixed wall

The deformation formula for a cantilevered wall is given by:

$\Delta_C = \frac{P h^3}{3 E m I} + \frac{1.2 P h}{G A}$ , where  $E m$  is the masonry elastic modulus,  $I$  is the cross-sectional moment of inertia of the wall ( $I = \frac{t L^3}{12}$ ),  $G$  is the masonry shear modulus ( $G = 0.4 E m$ ), and  $t$  is the wall thickness.

Then,  $\Delta_C = \frac{P}{E m t} \left[ 4 \left( \frac{h}{L} \right)^3 + 3 \left( \frac{h}{L} \right) \right]$

The stiffness of a cantilevered wall is equal to:

$$K_C = P / \Delta_C = \frac{E m t}{4 \left( \frac{h}{L} \right)^3 + 3 \left( \frac{h}{L} \right)}$$

The deformation formula for a fixed wall is given by:

$$\Delta_F = \frac{P}{E m t} \left[ \left( \frac{h}{L} \right)^3 + 3 \left( \frac{h}{L} \right) \right]$$

The stiffness of a fixed wall is equal to:

$$K_C = P / \Delta_C = \frac{E m t}{\left( \frac{h}{L} \right)^3 + 3 \left( \frac{h}{L} \right)}$$

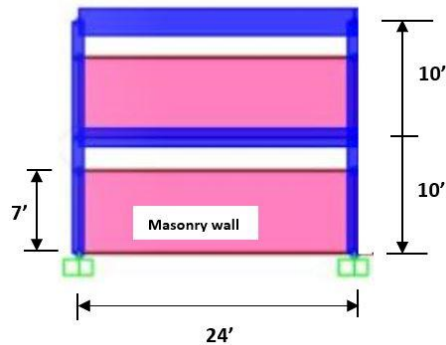


Figure 85. Front view of a classroom in Frame I

Masonry wall properties are presented in Table 3.2(Chapter 3).

Equivalent stiffness of the masonry wall in 1<sup>st</sup> floor:

$$K1 = \frac{Em t}{\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)} = \frac{1350*2}{\left(\frac{7}{24}\right)^3 + 3\left(\frac{7}{24}\right)} = 3001\text{k/in}$$

Equivalent stiffness of the masonry wall in 2<sup>nd</sup> floor:

$$K2 = \frac{Em t}{4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)} = \frac{1350*2}{4\left(\frac{7}{24}\right)^3 + 3\left(\frac{7}{24}\right)} = 2771 \text{ k/in}$$

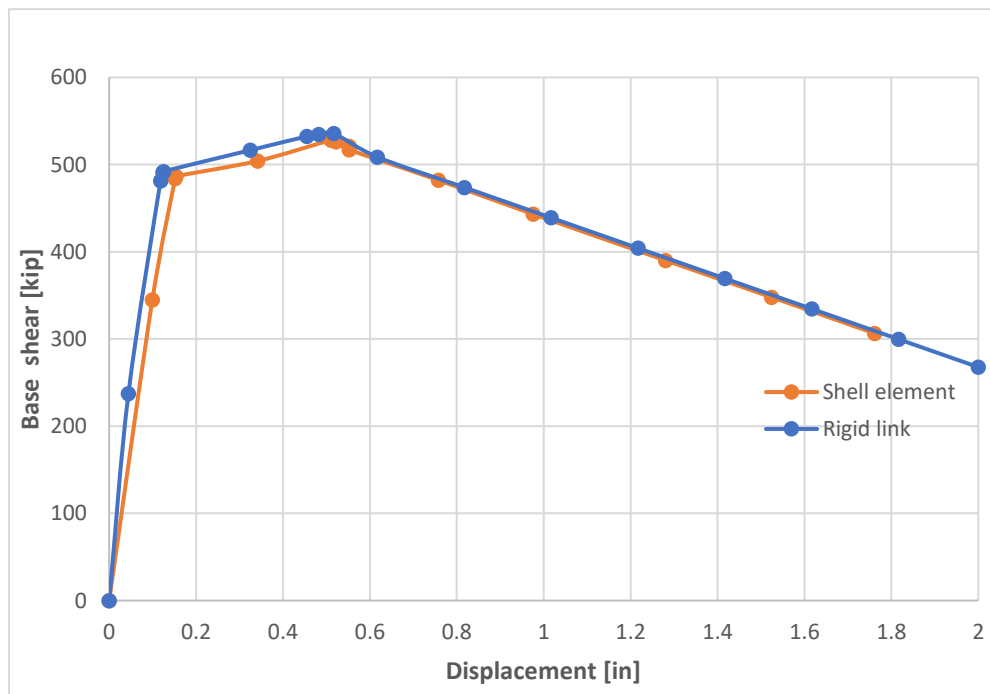


Figure 86. Pushover capacity curves using shell element vs rigid link of Frame I – Case I

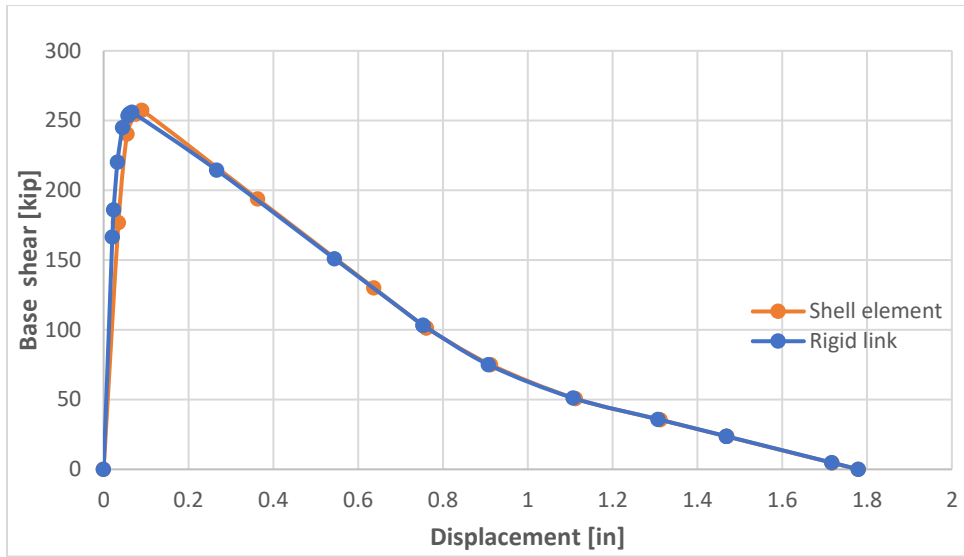


Figure 87. Pushover capacity curves using shell element vs rigid link of Frame I – Case II

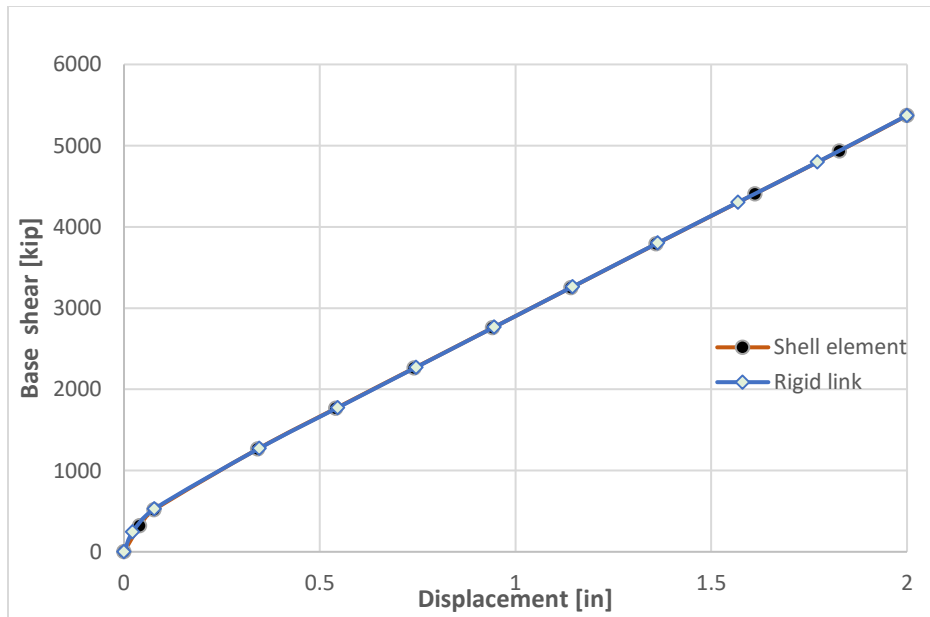


Figure 88. Pushover capacity curves using shell element vs rigid link of Frame I – Case III

Results of the Pushover analyses performed on the structure are quite similar using shell element and rigid link from the rigid diaphragm analysis to model the masonry wall in SAP2000.