Seismic Retrofit of Precast Panel Buildings in Eastern Europe

by

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ABSTRACT

Many countries in Eastern Europe, particularly ones from the former Soviet Bloc, are facing a potential crisis regarding their deteriorating precast panel apartment buildings. These complexes were built using industrial methods in response to the housing shortage during the 1960s, 70s and 80s. An ending life-cycle in combination with the poor design and construction quality makes these buildings extremely vulnerable to earthquakes that are frequent in the region.

This thesis addresses the need to act urgently in order to rehabilitate these structures and ensure that they meet today’s building code requirements. It is achieved through a case study that explores the effectiveness of global bracing seismic mitigation techniques on an existing precast panel building located in Sofia, Bulgaria. The in-situ building is first analyzed using SAP2000 and then again after the bracing is added to the model. A variety of parameters such as drift, floor acceleration and seismic damage are compared with cost and plausibility of the chosen options. As a final outcome, the external bracing scheme used in this study does in fact decrease both the floor accelerations and the interstory drift by at least 10% and in some cases as much as 85%.

During the thesis, several local experts and practicing structural engineers were interviewed and consulted. For this study it is assumed that the building has a close statistical representation of other buildings with similar structural system both in Bulgaria and neighboring Eastern European countries.

Thesis Supervisor: Jerome J. Connor
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1 INTRODUCTION

1.1 Overview

During the height of the Communist era in the 1960s, 70s, & 80s, manufacturing in Eastern Europe made up a substantial part of the economy. As a result, an industrial class of factory workers was established. This, in addition to the industrialization of concrete and post-war rebuilding efforts, led to the widespread development of industrial housing communities made of large precast concrete panels. These buildings could reach up to 10 stories and provide housing for over 30 families. The simple and boxy form of these complexes allowed for quick standardized construction at relatively low cost. However, the main tradeoff for the time and money saved in this process was a lower design quality (Nikolov, 2012).

Today, more than 170 million people reside in over 70 million panel buildings throughout Central and Eastern Europe (Csagoly, 2003). These prefabricated high-rise monoliths still overshadow much of the skyline of Sofia, Bulgaria, and exist in nearly all cities across the country. With an average life cycle of 30 to 40 years, many of these housing developments are now entering a stage where damages become significant and a danger to the residents. Bulgaria in particular is in a high seismic area, creating a hazard to the inhabitants of these crumbling complexes. According to M. Schumer of Berlin’s Department of Building, if renovations are made to the structures before further degradation, they have an “outstanding potential for lasting development.” (Csagoly, 2003) In recent years, an economically strong Germany has already begun to successfully repair its own deteriorating panel constructed communities, particularly in areas of the former German Democratic Republic.
This thesis will first briefly address the history behind panel construction, particularly in Bulgaria, and why specific systems were chosen over others. It will then transition into the current state of these buildings with an emphasis on structural health, thermal isolation and the social impact. Next the seismicity of the region will be presented. Finally and most importantly, the need for action will be stated. This introductory section will be followed by a case study of an existing building in the capital city of Sofia, Bulgaria.

First, Eurocode guidelines are discussed with a primary focus on maximum accelerations and serviceability criteria. Since the code does not explicitly state drift requirements, the Seismic Design Handbook’s prescribed value of $\delta = 0.005$ is used. Following this section, common local and global retrofit technologies are discussed. Several seismic mitigation methods used in the industry today include traditional strategies such as adding infill wall systems to increase stiffness, enhancing connections between elements using materials such as FRP, stiffening the building via external bracing systems, etc (Bouvier, 2003).

Once a specific technique is selected, it is applied to the building located in one of Sofia’s largest urban neighborhoods, Drujba. A structural design firm in the area, Nenplan Engineering Ltd., will provide floor plans, structural details and material parameters of this building. Finite element software will be used first to analyze the in-situ structure and then run a response spectrum analysis per Eurocode 8 after the application of the mitigation system. It should be noted that no studies have been previously published on how to analyze panel structures in SAP2000. As a secondary goal of this thesis, it will provide insight of a possible method in modeling and correctly analyzing these truly historic structures. Performance data such as drift and floor accelerations will be collected from the various models. Finally, a basic cost analysis will be performed to determine the feasibility of installation.
1.2 History of Panel Construction

1.2.1 The Industrialized Method of Building

Since the beginning of the Cold War both developing and industrialized nations across the European continent have faced the problem of a shortage of adequate housing for low-income social groups. The issue worsened towards the end of the twentieth century with the rise of a middle class. Countries facing these problems in particular have relied on mass production of residential communities using industrial methods of building. These methods primarily rely on prefabrication of structural elements as a means to provide adequate housing in a quicker and cheaper manner that also requires less skilled labor when compared to traditional methods of construction.

The general objectives of this means of building is for as much as possible of the structure be produced in a factory. Its developers sought to apply similar principles to those used for many years in the assembly line style mass production of products such as household appliances, motorcars, etc. Although prefabrication of a wide variety of building components is still popular in many regions across the world today, the bulk of home construction using industrial methods took place primarily in Eastern Europe during the second half of the twentieth century (McCutcheon, 1988).

The two most popular methods include both the large panel and the prefabricated component systems. Their role in industrial building especially in public sector residential apartments was considerably greater in Eastern than in Western Europe. Towards the end of the century, when the role of industrialized building decreased in Western Europe, it steadily increased in Eastern Europe. McCutcheon also notes in his research that the greater the role of industrial methods, the greater the use of large panel systems. This is particularly evident in almost all former Soviet Bloc countries. He concludes that the regions with the most housing shortage were under socialist regimes.
1.2.2 Housing in Socialist Bulgaria

As discussed in the previous section, Europe and socialist countries in the 1950s and 1960s faced a massive housing shortage as a result of the war, the raise of industrialization, and urbanization. Similar to other European countries recovering from the war, the socialist regime in Bulgaria used almost all of its financial resources and labor force for the development of heavy industry. The post-war housing problems of its people were not of high importance to the regime. Instead the state remained close to tradition and exploited the demographic and economic structure of its domestic society. In other words, since the rural population still dominated it was not the responsibility of the state to provide housing to the population that lived outside the city.

According to the data presented in Table 1.1 from a paper published by the Bulgarian Dept. of Economics in 1974, it is evident that the vast majority of the Bulgarian population did not live in an urban society before 1950. Furthermore, only 8% of the population lived in cities whose limits encompassed more than 100,000 inhabitants. Similar to the other Eastern European states at the end of the war, Bulgaria became a part of the Soviet Bloc and in doing so it adopted a totalitarian regime. This gave way to the Communist Party that under the Constitution of 1947 assured for the care of the state and its people. In brief, it meant total control over economical, political and social life. Embedded in social life is the states involvement for the provision of providing adequate housing for its citizens.

As a result of rapid urbanization and the lack of public housing in major cities, the typical socialist large panel buildings began to appear not only in Bulgaria but also across other neighboring countries. Again referring to Table 1.1 and comparing the data from 1970 with the previous it is evident that there was an influx of people from the countryside into the urban environment.
Industrialized housing in Bulgaria started in the country’s capital with the construction of the Tolstoy complex in the late 1950s and early 1960s. The new complex provided apartments for over 216 families and consisted of nine blocks of flats no more than four stories in height (Mirtchev, 1977). This landmark project marked the beginning of large panel construction and paved the road for decades of mass development across the country. The rise of industrialization in both manufacturing of goods and homes was directly linked to urbanization and the reason why urban populations more than doubled in less than twenty years (Table 1-1).

Since the state is in control of social life in the totalitarian system, it significantly restricted the choice one had in respect of where to live. Through administrative means, the state attempted to solve social problems. This was directly connected to its desire to control both economic and social life. In order to successfully achieve this the state needed a system to regulate current urban populations and future planning. Due to its limited finances available to deal with the housing problems, it relied heavily on industry to fund the housing for its workers. What resulted are large urban communities that housed workers with similar professional backgrounds.

Table 1-2 represents the state vs. non-state investments in housing data amongst several socialist countries. By these results it is evident that the achievements in Bulgaria were quite impressive by the end of the 1960s, which was termed the first experimental stage of development. It also clearly illustrates that in fact there was not so much state care but rather state control of housing. This time also marked the adoption of the USSR’s “20 Year
Program for Accelerated Development” in Bulgaria. The program gave industrialization and urbanization a real push. Its priority in housing expansion launched a mass industrialization of housing construction in which the large-panel industrial methods took after those already implemented in Germany, France and the USSR.

Table 1-2: 1960s state vs. non-state housing investments in socialist countries, (Yaremenko, 1981).

<table>
<thead>
<tr>
<th>Country</th>
<th>Housing built with state investments</th>
<th>Housing built with other sources of investment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulgaria</td>
<td>8.7%</td>
<td>91.3%</td>
</tr>
<tr>
<td>Hungary</td>
<td>26.6%</td>
<td>73.2%</td>
</tr>
<tr>
<td>East Germany</td>
<td>32.5%</td>
<td>67.5%</td>
</tr>
<tr>
<td>Poland</td>
<td>39.5%</td>
<td>60.5%</td>
</tr>
<tr>
<td>USSR</td>
<td>50.6%</td>
<td>49.4%</td>
</tr>
<tr>
<td>Czechoslovakia</td>
<td>60.0%</td>
<td>40.0%</td>
</tr>
</tbody>
</table>

The mass production of housing continued through the decades while state involvement in the housing construction also gradually increased. In the second half of the 1970s, 49% of all housing investments in Bulgaria came from state resources while in other socialist countries it was around 30% to 40% of the total housing budget (Mirtchev, 1977). The USSR was the only one ahead of Bulgaria with more than 73% state investment.

### 1.2.3 The Introduction of Typification

The rapid expansion and industrialization in the housing sector that followed in the 1970s and 1980s led to an assembly line production method. Not only were almost all of the building elements mass-produced off-site but also with the introduction of a new principle the production could now start all the way from conceptual design. This principle is called typification. In general, typification is the process of reducing a population of selected technologies based on a set of valid criteria until the model with best potential is chosen.
Typification, in other words, is a form of optimization by standardizing designs and determining the most suitable for the given scenario. Specifically in construction, the building designs are *typified* based on the projected use of parts and structural elements. The principle is best applied to mass production in order to facilitate a lower overall cost and construction time. This is achieved by having the fewest number of standard parts such as wall panels, beams, floor slabs, and so on. Factors such as loads, constructability and economics determine the number of standard parts or structural elements. As a result, buildings become divided into simple modules that make construction a repetitive process.

This idea really took off and dominated residential construction in socialist countries in the late 1970s and the 1980s. During this time two main construction systems governed – a large-panel construction system and a mixed construction system using bearing walls with large-area formwork and floor structures of pre-fabricated elements. In the large-panel system, after the architecture is chosen, the building is cut into panels (horizontal and vertical) and a nomenclature is drawn up. In this case, every element of the building has an exact address and no other type section or building of a different nomenclature can be built.

In the closed system of typification, there is no single system of unification of the building elements. Instead, the main influence is exerted on the technological peculiarities of production such as the nominal capacities for the building elements that are required to release a stereotype of buildings (with slight modifications for the specific site). This however has a negative impact on the architectural appearance and urban planning of the residential complexes. It forms entire communities of similar looking structures with low aesthetic appeal. Still, to create some diversity, normally, every building element fabricator uses its own strict nomenclature (closed system) with a full
set of elements for the buildings. This form of closed technology is not only difficult in terms of the architecture and aesthetics of the buildings, but also with regards to the process in the production plant and the organization of construction.

Contrary to the closed system of typification there is the method of typifying of building elements or the open system of typification shown in Fig.1-1. This method is best applied to large-panel buildings, which is the main focus of this thesis. According to Eng. Ivanova’s publication on residential buildings from the standpoint of the design organizations, the following is a brief outline of open typification:

- A standard module is selected. Normally, this is the main module which is equal to M = 100 mm
- An enlarged structural modular grid is selected. This is the modular grid on which the vertical bearing elements (walls and columns) lie. The enlarged modular grids can be 3M/3M, 6M/6M, and 12M/12M. They may also be rectangular 12M/6M
- A series of architectural solutions are developed for each individual enlarged modular grid
- Depending on the results of the analysis of the series of composition solutions (economy, functional solution, etc.), an optimal enlarged structural grid is selected
- Once the optimal enlarged structural grid is selected, unification of the building elements and connections is carried out
- In the process of unification, first, the building elements are grouped by their intended use and organized in families such as bearing wall panels, floor panels, façade panels, light partition panels, elements for stair-case cells and roof elements
- For each family of elements unification is carried out primarily for the constant and variable parameters: for example, in bearing wall panels the height and thickness are constant, and the length is variable
The smallest length of the building element is selected: again for example, in the bearing wall panels the smallest length is 24M. The greatest length is also selected, for example 60M, 72M or 84M. This is primarily based on a sequence of architectural iterations.

In the end, a catalogue of building elements is drawn up.

Finally, a check is made to see whether the building elements of the catalogue meet the criteria of the particular scope, site, living conditions and so on, making the selection of the building an iterative approach.

The catalogue of building elements, Fig.1-2, obtained by this method of typification contains elements that are not specific to a particular location in the structure. They are unified in a way that they can be used in different locations in different buildings. This makes it possible for the design of virtually any building as long as a proper modular grid is developed. It practically means that individual design of these buildings is now as simple as selecting parts from a catalogue largely reducing the amount of time and effort spent in the design. Fig. 1-3 & 1-4 illustrate how once the elements are unified different combinations of structural cells are possible.

Figure 1-1: Open typification of large-panel construction system. A combination of different panels that can be used to construct a particular building type. (Mirtchev, 1977)
It is also possible to develop exemplary and repeating designs of residential buildings. The typified flats and sections can serve both for development of designs of current buildings and future models for development of individual designs. Thanks to the profound work on typification of flats and sections, each can serve as a benchmark based on which the architect sets off to look for his solution. Similarly, the exemplary designs can serve as models for the fabrication of members. Naturally, the exemplary designs can be applied directly in construction as repeatable, especially in the cases when design personnel are scarce.

Figure 1-2: Catalogue of building elements for large-panel buildings used in open typification (Mirtchev, 1977).
Figure 1.3: Different configurations of structural cells (Mirtchev, 1977).

Figure 1.4: Positioning of floor panels in different structural cells (Mirtchev, 1977).
1.2.4 Communist Style Residential Building Types

To a large degree, the optimization of the everyday life activity of families and individuals depends on their interaction with the space around them along with its organization and furnishing. Hence, the quality of the functional space in the apartment and building itself has a vital importance for establishing comfort and a positive standard of living. This forms the need for the design of these flats to have operational flexibility. These changes to the flats pose a number of problems to the structural solution of large-panel buildings. Above all, the more open functional space of the flat requires greater support distances between the vertical bearing elements, in this case, wall panels. This also allows for the possible use of light partition walls for creating boundaries between the main bearing walls. Consequently, in order to incorporate flexibility, many different architectural building types exist.

The years between 1965 and 1971 were designated as the experimental stage of construction via industrial methods. In that period large-panel building types dominated 75-80% of the total volume of residential housing in the People’s Republic of Bulgaria (Popov, 1974). After applying them in practice, some of these designs revealed substantial architectural and construction deficiencies, as well as low technical and economical indicators, which imposed reassessment. This primarily led to the introduction of new more modern types of houses in the period of 1971-1975. They feature a considerably large diversity of the room sizes, the number of rooms and the functional space as a whole.

Development of increasingly modern socialist forms of living required drastic improvement of the base design, diversification of the types of houses, their planning and their furnishing. In particular, construction, planning and selection of the types of buildings need to take into account the demographic of the population and the multiple requirements of the different social groups that previous nomenclatures, or types, left out. The next generation flats, which
were also being built in the USSR, Poland, Hungary, Yugoslavia, still kept the same sectional type as before. They feature groups of 2, 3 or 4 flats on one floor, united vertically by one staircase-elevator shaft. Although these modern flats now took into account the needs of the individuals in the apartment, their design knowingly leaves out the opportunity for a public environment where neighbors or members of a building can come together and interact. It should be mentioned that these building styles were not limited to only residential. Office buildings, factories and various other public structures were also designed in this manner.

In the period that followed 1975, the People’s Republic of Bulgaria established the mass application of large-panel residential buildings under the following nomenclatures:

- Bs, n VI-2 – 63  (Fig.1-5)
- Bs VIII-2 – 64 Zemlyane  (Fig.1-6)
- Bs VIII-4 – 64 - Sf  (Fig.1-7)
- Bs, n IV-VIII-Gl – 63  (Fig.1-8)
- Bs V – Pd – type “Al. Tolstoy“  (Fig.1-9)

The catalogues for these nomenclatures contain a different number of designs for specific blocks – Bs, n VI-2 – 63: 18 blocks, Bs VIII-2 – 64 Zemlyane: 4 blocks, Bs VIII-4 – 64: 28 blocks, Bs, n IV-VIII-Gl: 10 blocks and Bs V – Pd: 2 blocks. In practice, the 62 total blocks of each nomenclature found a wide variety of applications, mainly with two-three room flats. Thus, the possibility of buildings different in type, size and in accordance with demographic needs of the population was drastically increased.
Figure 1-5: Bs, n VI-2 – 63 [Ivanova, 1968].

Top – perspective of residential building in the district of Borovo – Sofia; Bottom – section view
Figure 1-6: Nomenclature Bs VIII-2 – 64 “Zemlyane” (Ivanova, 1974).

Top – Façade  Bottom – Section view
Figure 1-7: Bs VIII-4 – 64 – SF (Ivanova, 1974).

Top – Façade  Bottom – Section view
Figure 1.8: Bs, n IV-VIII-Gl – 63 (Ivanova, 1974).

Top – Façade  Bottom – Section view
Figure 1-9: Nomenclature Bs V-Pd type “Al. Tolstoy” – improved, (Ivanova, 1974).
1.2.5 Building Description

As discussed previously, the dominant method used in residential construction throughout Bulgaria and other socialist countries in the 60s, 70s and 80s was the large-panel system. Within this scheme there are two main structural systems; framed panel construction and frameless panel construction. In the framed panel technique, a precast concrete frame is first assembled after which individual panels and hollow core planks are placed to form the floors and walls. Fig.1-10 illustrates this method. In the frameless panel system, the wall and floor panels completely bear on each other and transfer all gravity and lateral forces down to the foundations. They combine to form a cellular module that essentially makes up the living space (Fig.1-11). The preferred system of choice at that time was the frameless method since it takes less time and labor. For the building considered in this thesis, the frameless large-panel system is used.

Figure 1-10: Components of a precast concrete frame system (Brzev).
In both schemes all major elements such as the wall and floor panels are prefabricated off site in a factory. Elements that are constructed on site include the foundation, joints/connections, topping, and partitions. To fabricate the precast elements, concrete is poured into horizontal flatbed forms that have been prepared with the proper reinforcing steel arrangement and if applicable the required door and window openings (Fig. 1-12). Once poured the elements are placed outside where they cure.
Upon completion of the core building structure, the partitions and floor topping close out the apartment and core interior spaces. These components are applied on site after the structural framework of wall and floor panels has been constructed. Typically there are two common thicknesses of partitions corresponding to 10 cm and 15 cm clay masonry units. For leveling a 20 mm topping covers the precast floor panels (Burns, 1981). Usually after the connections between wall and floor elements have been made they are solidly grouted to prevent any infiltration that can cause corrosion. In the modernized stages of residential construction the bathroom and toilet areas were also outsourced to the factory. Completed bathroom modules were fabricated off site and lifted into place so that workers only have to make connections between the pipes to the rest of the building.
The foundations of these mammoth structures are one key building element that cannot be prefabricated off site. Therefore, determining the proper structural design is key to the overall structural integrity of the building. Sofia, the location of the building being studied, is located where the Danubian Plain meets the lower North ridge of the Balkan Mountains. This area is known to have good physical soil characteristics that are low in organic matter and phosphorus (Stoyanov, 1996). After corresponding with local structural engineers, it was determined that there are relatively low risks of finding soil with inadequate bearing capacity. The majority of residential and other building structures constructed in this period generally have two types of foundations; a single thick mat or a combination of strip/wall footings. Complexes built around the coastal cities such as Varna and Bourgas, experience sandy soils with lower bearing capacities. In these cases, a deeper pile foundation is preferred since it increases the stiffness of the soil-foundation system.

To get this entire assembly of structural and non-structural elements erected in the correct order, on time and in the most efficient manner possible takes careful planning and organization of worker crews. Since the frameless panel structure is essentially self-supporting, it is vital to correctly sequence floor panel placement so that the structure can properly distribute gravity load. Fig.1-13 illustrates how the wall elements are lifted into place by a crane, leveled, welded and grouted. Once these elements are in place the floor panels are placed and connected horizontally on top (Fig.1-14).
The floor slabs themselves can span in one direction and distribute load uniformly to the walls or, if the module dimensions allow, a slab can act in two-way action by being restrained at all four sides. It is important to note that all elements at a single node or joint must but secure in place before any grouting can be done. Finally, just as a side note, assembly of the framed panel system is shown in Fig.1-15. The concrete space frame is first erected then individual floor planks are lifted in place by crane.

Figure 1-14: Frameless Large-Panel Assembly (Burns, 1981).
Figure 1-15: Framed Panel System Assembly (Nikolov, 2012).
1.3 Earthquake Hazard

1.3.1 Seismicity of the Region

The country of Bulgaria is situated in the Balkan seismic belt that is a subset of the larger Alpine-Himalayan belt. Cities that lie in this belt are characteristically known for their exposure to high seismic risk. From a plate-tectonic point of view, the Balkan Region lies on the continental plate of Eurasia, see Fig.1-16. There are two major characteristics of this plate. The first is that the northern part of the European continent (i.e. the northwestern part of the plate) is relatively stable regarding seismic activity. However, the southern Mediterranean regions of Bulgaria, Greece & Turkey lie close to a fault line that generally sees an above average amount of seismic activity. (Zagorchev, 1992).

Figure 1-16: Map of major tectonic plates with their respective movements [www.schoolphysics.co.uk].

From a building design point of view this area is of high risk and therefore of high concern for structural engineers. This region is composed of relatively old nations that are home to a variety of historic and fragile structures with residential buildings inclusive.
Earthquakes of magnitudes 6.0 to 7.8 have previously been reported in the area. According to professor and seismologist Zoran Milutinovic, although earthquakes of magnitudes greater than 6.0 are rather infrequent, when they do occur, the structural weakness of prevailing traditional urban and rural building typology constructed prior to 1964 can cause widespread devastation in regions affected. During the last 100 years few destructive, even catastrophic earthquakes, have been affecting the country (Zagorchev, 1992). Fig.1-17 visually illustrates the seismicity of the region since 1990. This map is a collection of all recorded seismic activity.

Figure 1-17: Seismicity of Balkan Peninsula since 1990 (USGS, 2012).
In the past two decades several major earthquakes have been recorded across the Balkan region. Fig.1-18 depicts all of the seismic events with magnitude 7.0 or greater since 1990. The last major earthquake to hit Bulgaria occurred on May 22, 2012 in the town of Pernik (USGS, 2012). It had a magnitude of 5.6 and resulted in significant structural damage however no loss of life. Pictures of the aftermath and a discussion of this earthquake with its effects on large-panel buildings will be presented in the following section.

From the data presented in this section it can be concluded that there is a relatively high earthquake hazard for Bulgaria, especially cities located on the southern boarder with Greece and Turkey.

Figure 1-18: Earthquakes of magnitude 7.0 or greater (USGS, 2012).
1.4 Current State of Large-Panel Buildings

1.4.1 Structural Health

The following section will give a sense of the structural health for large-panel buildings using field data and investigations conducted after the 2012 Pernik earthquake. The conclusions drawn from this study will be assumed for the overall population of this building type across Bulgaria constructed in the 1970s and 1980s. This will be based on the data collected by two current faculty members, Prof. Zdravko Petkov & Assoc. Prof. Atanas Nikolov, from the Sofia Technical University of Architecture, Civil Engineering and Geodesy.

For the past year Nikolov has focused his research on analyzing the data from the Pernik earthquake. In that time, he made several visits to two panel suburbs, or areas that predominantly have panel construction, in the town. From a personal interview, he states that as construction technology of large-panel buildings was imported from the former Soviet Union, the design methods and the methods for numerical modeling were not quite clear but should be connected with plastic and failure mechanism.

For him it was quite important to see how the large-panel buildings behave subjected to severe earthquakes and what kind of mechanisms are observed. In the past, he studied these types of buildings in the former USSR, East Germany and Romania. This was done because there is currently nobody capable of explaining the motivation of the choice for design parameters such as strength reduction factors for buildings constructed in Bulgaria. In other words, the information from the time of construction is so limited that assumptions made for these type buildings in Bulgaria must come from data of similar buildings in other countries.
From a personal interview with Nikolov, the work to assess current buildings is divided into four main stages. His conclusions are the following:

1. **Data Collection:**
   There is no known data of the free vibration periods for large-panel buildings and their respective damping (Nikolov, 2012). Since no documentation or projects are available, modal shapes and modal periods should be measured in-situ (experimentally), using ambient vibration methodology. Nobody in Bulgaria has such interest and the funding to conduct this study. However, it is very important to identify the dynamic properties of these structures. In addition, there is no lab data from the testing of single large-panel subjected to shear action. They only have visual data from the aftermath of the Pernik earthquake, which gives an idea for possible failure mechanisms of the panels.

2. **Data Analysis and Failure Mechanism Identification:**
   Nikolov and his team tried to analyze the failure mode of a single wall panel and came to conclusion that there are two stages before failure of a single wall system:
   
   **Stage a):** Failure of the concrete large-panel element (lightly reinforced). The panel is separated into different parts with crack lines where the principal tensile stresses of concrete were exceeded (Nikolov, 2012).

   **Stage b):** Dowel connections between panels exhibit pure nonlinear behavior. Bearing capacity of the dowel connection is greater as the axial load on friction surfaces is greater. The failure of the dowel connection happens when the shear capacity is exceeded (Nikolov, 2012).
3. **Compose a Model:**

It is important to compose a simple but reliable enough numerical model to simulate the dynamic behavior of a single panel system, single wall system and the dynamic behavior of overall building as well. As shown in Fig.1-20, the dynamic model of a large panel building consists of truss elements (elastic), n-link elements (inelastic) used for dowel connections and floor diaphragms with proven diaphragmatic action for floors.

![Figure 1-20: Numerical Model (Nikolov, 2012)](image)

It should be dynamically equivalent to large panel buildings, but there is no data for such structures (modal periods and corresponding modal vectors). The model should be statically equivalent to existing large panel structures, which means that hypothetically it allows finding the redistribution of internal forces due to lateral static loads (pushover curve). This data is also not available and stops the development of the numerical model. Following Eurocode 8 there is no data indicating how much energy is dissipated in large-panel buildings (Nikolov, 2012). It means that strength reduction factor (behavior factor in Europe) will be not known. To overcome this there is a need for lab testing in order to study the energy dissipation in dowel connections. In the former USSR the behavior factor was
accepted to be $q = 4$ without any explanations. The same value was accepted in Bulgaria, but now it should bring motivation in the viewpoint of European and American engineers of understanding of the problem. In general, Dr. Nikolov says that in this situation large-panel buildings in Bulgaria are not designed according to EU standards (capacity design rules and performance requirements are not satisfied). Thus they need strengthening. How much strengthening depends on the data, which is missing at the moment in Bulgaria. A lot of responsible people in Bulgaria do not want to pay money for the collection and selection of data, for instrumentation facilities and for advanced software. They simply want to skip this process because they do not understand the importance of the problem. It is a problem that cannot be solved by one, two or three professionals.

4. Study of a variety of numerical examples:

Making attempts to predict numerically the dynamic and seismic response of large panel buildings. This activity can be done if we have success in the previously mentioned items. A data base system containing the results for different systems of large panel buildings should be created.

The following pages contain images taken by Prof. Bonev and his research team. This investigation proved to be one of the first of its kind in the short history of the Republic of Bulgaria (founded in 1990). Several key assumptions can be made regarding the behavior of large-panel buildings, the sources of concern and potential ways to fix them.
The first set of images was taken from a local grade school building dating back to 1970. From Fig.1-21 it can be noted that the earthquake caused large shear cracks to form between adjacent vertical panel elements up the entire building height. Essentially it means a global weak vertical connection that allows panels to move relative to each other (Fig.1-22). This now leaves the building behaving drastically different from what was originally intended. Instead of monolithic behavior similar to the bundled tube structural system this damage allows for inelastic behavior of the vertical connections.

Figure 1-21: Shear cracks between vertical panels, (Bonev, 2012).

Figure 1-22: Weak vertical connection (Burns, 1981)
The next set of images was taken inside a residential panel building constructed in the late 1970s. Fig.1-23 contains several photographs that depict global and local damage of wall elements. In all of the images the toping and paint has fallen off. Large cracks form an X in the middle of the wall element where the principle shear stresses exceed capacity. Locally, connection damage is also present. One can notice the crushing of the concrete that has occurred due to high compressive stresses. Bonev’s conclusion is that the possible failures of these buildings can be associated to connection failure (Bonev, 2012).

Figure 1-23: Local and global damage of wall panels, (Bonev, 2012).
1.4.2 Thermal Health

As discussed in the previous sections, the construction of residential complexes across Central and Eastern Europe using industrial methods was very effective in satisfying the demand for housing during the 1960s, 70s & 80s. They were very efficient in saving material and providing the lowest cost. However, from a thermal and energy consumption point of view they were very inefficient. According to Zsebik of the Solonova initiative, these residential buildings are far from today’s requirements (A. Zsebik, 2005). They have huge maintenance costs, obsolete heating systems, moldy walls and drafty windows.

The low physical quality combines with social problems. Poorer classes of society replaced those who could afford to leave the old apartment block. All this caused a poorer value of large-panel residential buildings. In Bulgaria, roughly 98% of existing large-panel buildings are privately owned (S. Zaimova, 1999). This makes it very difficult to renovate these buildings because it requires that all tenants agree on the renovation type.

1.4.3 Resident Health

In 2003, the World Health Organization undertook a field survey on large-panel housing across Eastern European countries with aims at a preliminary assessment of housing conditions and their potential health consequences. Based on empirical data collected from 259 dwellings and 601 residents, it can be concluded that several housing conditions do have an impact on the health perception of their residents (M. Braubach, 2003).

Noise annoyance was recognized as one of the most prevalent problems affecting residential health and well-being. The most important health effects that were identified are respiratory diseases. According to Braubach, there is a strong association between housing conditions such as tightness of windows, perception of indoor climate (temperature, indoor air quality), and the prevalence of people suffering from respiratory diseases.
1.5 The Need for Retrofit

In conclusion, this first chapter introduced a construction methodology implemented across Eastern Europe in the 1960s, 70s & 80s. By using industrial methods the State was able to effectively meet the growing housing demand while also keeping the cost and construction time to a minimum. The downside of these gains was that the design quality was relatively poor. The vast majority of these structures still provide housing for hundreds of thousands of people across Europe today. Unfortunately, as building codes progressed and got more advanced these buildings did not. They remain essentially untouched since their assembly nearly 30-40 years ago.

Since the fall of the USSR in 1990, there are little to no existing articles or research conducted on the behavior of large-panel buildings especially due to seismic activity. The only way to make proper assumptions of their behavior is through investigations and the development of accurate dynamic models. From the structural properties listed in section 1.2 and the high seismicity associated with the region in section 1.4 it is evident that these buildings possess a risk to human safety. The underlying conclusion from this chapter is that large-panel buildings in Bulgaria are not designed according to EU standards (capacity design rules and performance requirements are not satisfied). Thus they need strengthening.

In the remaining chapters, the thesis will focus on which parts of the buildings are deficient in regard to the EU standards, the modeling of an actual building (before and after retrofit), and the evaluation of the effectiveness of this retrofit technique. A cost feasibility study is performed and suggestions for future research are made.
2 METHOD & PROCEDURE

This chapter first goes into detail of the current building code requirements used in seismic design across Europe. General design rules as well as basic building performance requirements and compliance criteria will be presented. Special importance will be placed on provisions for seismic assessment and retrofitting of existing buildings. Then a series of different local and global retrofit techniques will be discussed. Finally, the specific building on which the case study is focused will be presented.

2.1 Eurocode 8

In 1975, the Commission of the European Community decided to take action in the construction industry. Its goals were to establish a set of harmonized technical specifications designed to eliminate obstacles associated with construction. These rules would be implemented across all of the Member States and replace current national building policies. Fifteen years went by until the Commission finally introduced the first generation of the Eurocodes in the 1980’s.

At the time, Bulgaria specifically, was still under the rule of a communist regime and used the same codes as the USSR. After the collapse of the USSR in 1990, the new republic established its own national set of rules and design criteria. Finally, after joining the European Union in 2007, Bulgaria adopted the Eurocodes. Since it is in a high seismic zone, it must specifically comply with Eurocode 8: Design of structures for earthquake resistance.

From the most recent edition of the code, released in 2004, there are two fundamental requirements that every structure should meet: a no collapse requirement and a damage limitation requirement. In order to comply with these requirements a set of limit states must be satisfied: an ultimate limit state and a serviceability limit state. In general, when designing a new building an
elastic response spectrum is used. In this paper only a response spectrum analysis will be performed with a peak ground acceleration of $0.38g$ (see graph). Since the large panel system closely resembles that of a shear wall system it can be intuitively expected that these building types are generally very stiff. Using the $T = \frac{n}{10}$ rule of thumb quickly suggests that panel buildings will fall in the peak ground acceleration region of the spectrum.

It is important to monitor the drift of a structure for several reasons including structural stability, damage to (non)structural elements, and human comfort. Since the Eurocodes do not specify a control requirement for drift other sources need to be consulted. From Seismic Design Handbook and the 2008 World Conference on Earthquake Engineering the proposed interstory drift index levels are: $\delta = 0.002$ (nonstructural damage likely), $\delta = 0.005$ (headaches & dizziness, structural damage probable), $\delta = 0.015$ (nonstructural damage certain, structural damage likely). (Naeim, 2001 & McCormick, 2008).
2.2 Rehabilitation Strategies

Different buildings can require different rehabilitation strategies. These strategies can range from the local to the global scale or both. After an effective analysis is performed on the existing structure, it can be determined its deficiencies, and then an appropriate strategy can be selected.

2.2.1 Local Strategies

Local rehabilitation is usually applied to buildings that have a sufficient overall load capacity but have certain individual members with inadequate strength or deformation capacities. Techniques for improving these deficiencies include enhancement of connections and member strength or deflection. This method is preferred when there are only a limited number of a building’s components that are deficient. It is also results in the most economical choice of retrofit scheme (Bouvier, 2003).

The intent of local strengthening is to improve the performance of structural members at such locations that it will enable them to overcome strength demands determined from the analysis. This is all done without changing the structure’s response from a global perspective. Some popular local strengthening techniques include the confinement of columns using plates or CFRP, regrouting connections with mortar or CFRP, installing clip angles to strengthen joints between adjacent concrete elements etc. It is important to note that these measures can be applied to components without affecting their strength capacity but instead can be used to increase deformation in order to mitigate damage. Fig.2-1 shows the use of a clip angle and CFRP in a precast concrete building located in the US (Dumas, 2012).
Figure 2-1: CFRP strips used for deflection control (top)
Clip angle at wall/hollow core plank interface (bottom)
2.2.2 Global Strategies

When local rehabilitation strategies are not enough global measures are generally used to improve the buildings behavior as a whole. During seismic events especially, it is important the building behaves properly. There are three broad areas that passive global seismic improvements fall under. They are global mass reduction, structural stiffening or increase in damping (Bouvier, 2003). Other passive systems such as base isolation exist although they are hard to apply to existing buildings.

When a structure is too soft it can sometimes perform poorly during an earthquake because of large lateral deformations induced by the ground motion. Global stiffening of the structure is a good technique for rehabilitation in such buildings. Several ways of stiffening the structure include adding infill or shear walls and adding new external or internal bracing systems. The addition of such systems not only stiffens the structure but also provides different load paths hence decreasing the demand on certain members. Illustrated in Fig.2-2 is how the infill walls work to increase stiffness and a building with an external bracing system (Bouvier, 2003).

Figure 2-2: Infill wall system (top)  
External bracing system (bottom)
2.3 Case Study: Existing Large-Panel Building in Sofia

The following section introduces the building on which the case study is performed. All material parameters, assumptions, models and procedures used in the analysis will also be presented. Structural drawings, plans and material properties were provided by Sofia based structural consultants, Nenplan Engineering Ltd.

2.3.1 Building Description

In the 1980s, government owned Sofproject designed a series of different building nomenclatures across the city of Sofia. The vast majority of these projects were built in urban housing complexes. The building that is the focus of this investigation is an 8-story precast large-panel apartment block located in one of Sofia’s largest urban neighborhoods by the name of Drujba (see Fig.2-3). It was built in 1983 in response to the need for low-cost housing and employs industrial methods of construction that have been used for several decades.

The structural organization of the building consists of precast concrete wall and floor panels. These panels form a cellular module with walls at 3.60 meters on center. The structural system idealized for this structure is a series of closed tubes. This basic form fundamentally provides good earthquake resistance. Compared to other systems, the closed form gives a building a relatively high torsional stiffness.

A typical floor plan, shown in Fig.2-4, consists cellular modules that are organized in a pattern around a common core where the stairwell and elevator is located. The specific nomenclature of this building is Bs VIII – 69 Sof. However, this specific floor plan can be applied to buildings 4 – 9 stories in height. One of the main advantages of industrial methods was utilized in this buildings planning and construction. Outside the individual modules, entire building sections (outlined in the box), 18 meters in length, were essentially stuck together, forming a large elongated row with individual entrances.
Figure 2-3: Case study subject 8 story building section (top), Larger complex of same structural nomenclature (bottom) 
Courtesy: Nenplan Engineering Ltd.
Figure 2-4: Original floor plans of 8-story building. Top represents floor elements. Bottom represents wall elements. (Courtesy: Nenplan Eng. Ltd.)
After studying the original floor plan, it was replicated using modern AutoCAD software. Fig. 2-5 illustrates the organization of the apartments and core within the structural framework defined by the wall and floor panels. Within each building section, there are three different apartment types: a 2, 3 and 4 room option. All apartments are organized with living space arranged between the entrance from the core and the kitchen. Every option comes with a balcony and one bathroom. An overall building elevation is on the next page.

Figure 2-5: AutoCAD layout of original floor plan. (Courtesy: Nenplan Engineering Ltd.)
Fig.2-6 illustrates typical wall and floor panels. All wall panels are 2.80 meters in height, 0.14 meters thick, and 8 meters long. Floor panels have the same span 3.60 meters x 8 meters and are 0.10 meters thick. All elements are reinforced with two layers of 200 mm x 200 mm welded wire cages as a minimum temperature steel requirement. The cage provides a horizontal reinforcement ratio of 0.0007 and a vertical reinforcement ratio of 0.0014. Reinforcement that is used to connect and splice two members is made up of two layers of $\varnothing 12$ bars.

Figure 2.6a: Typical floor panels (Nikolov, 2012).
Figure 2-7b: Typical wall panels (Nikolov, 2012).
The connections between the precast panels are a very important part of the building assembly. They are achieved through welding the extending dowels in the splice region that remains after wall and/or floor panels are positioned. The connection regions contain lateral reinforcement embedded in the wall and floor edges, and longitudinal reinforcement for continuity along the vertical and horizontal connections. Finally, once the welds are in place, the entire connection region is grouted with mortar. It is important to keep in mind that the connections between adjacent floor and wall panels do not connect to one another via dowels. A typical horizontal and vertical connection detail is illustrated in Fig. 2-7.

Figure 2-8a: Typical wall splice connection between floor panel (Nenplan, 2012).
Figure 2.7b: Typical floor panel splice connection (Nenplan, 2012)
In order to construct an accurate model, the engineering and material properties of the building are required. The most important of these properties is the concrete used in the structural elements. A normal weight concrete of 2,400 kg/m$^3$ is used in all wall and floor elements. According to a cylinder test done in Dr. Nikolov’s lab, an elastic modulus of 25,500 MPa and a compressive strength of 20 MPa can be expected for panel buildings in Bulgaria (Nikolov, 2012). The reinforcement steel used at the time is specified to have a yield stress of 375 MPa and an elastic modulus of 200,000 MPa. A summary of these values is presented in Table 2-1 in both SI and English units for comparison.

<table>
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<tr>
<th>Material Properties</th>
<th>SI Units</th>
<th>English Units</th>
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<tr>
<td>Concrete Elastic Modulus, $E_C$</td>
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<td>3,700 ksi</td>
</tr>
<tr>
<td>Concrete Shear Modulus, $G$</td>
<td>11,090 MPa</td>
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<td>Conc. Compressive Strength, $f_{c}''$</td>
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<tr>
<td>Concrete Density</td>
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<td>150 lb/ft$^3$</td>
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<tr>
<td>Steel Elastic Modulus, $E_S$</td>
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<tr>
<td>Steel Tensile Strength, $f_y$</td>
<td>375 MPa</td>
<td>50 ksi</td>
</tr>
</tbody>
</table>
2.3.2 Assumptions

Large-panel buildings of this nature are structures with previously determined lines for cracking and points for non-linear behavior. Up to the formation of the first crack, the large-panels are working in the elastic range. The first cracks are formed in the weakest points of the building. These points are in the lintels over the doors and in the zones of connections between outside panels (façade panels are not parts of the main construction; this is why they are left out of the model).

If the earthquake excitation is strong enough the entire building starts to work in resisting it. Although there are joints between floor panels, it is assumed that the floor slab is stiff in its plane. In reality this is not true. In order to do a more accurate investigation a non-linear spring is necessary to be incorporated in the model (Nikolov, 2012). For the purpose of this paper the slab is considered to be stiff.

In the vertical direction every wall panel possess horizontal and vertical joints. Usually there are two on every side (four horizontal and four vertical). Every joint is characterized by its stiffness in the $x$, $y$ and $z$ directions. These characteristics are not independent of each other. It is possible for these characteristics to be substituted with constant values equal to the initial stiffness located in the elastic range. The joints can also be substituted with their hysteretic models and represented in time (time history).

For the purpose of this paper the materials and structural elements are assumed to behave in their elastic range according to Eurocode 8. It is also assumed the foundation is rigid. After the static analysis, a response spectrum analysis in both the $x$ and $y$ directions is used to study the building’s behavior. The results can be compared with the capacity of the R/C members and joints. To model the non-linear behavior another program is necessary.
2.4 Modeling

A linear elastic model was applied in performing the analysis on the building using finite element software (SAP 2000). First, a 3D AutoCAD model is constructed. This idealization is then imported into SAP 2000 where both a static and dynamic analysis is performed. The following sections describe in detail how the models were set up to model the building itself and the selected retrofit technique.

Following the information presented in the previous sections about type of buildings, interviews with professors & local engineers and an onsite survey conducted by Nikolov & Bonev, it was concluded that the external bracing system would be the most feasible to apply to such a building (Nikolov, 2012). The underlying reason behind this conclusion is that each apartment is privately owned. In order to approve any work inside the building a petition must be signed by every resident of a complex. Furthermore, it is extremely difficult to do rehabilitation work inside a fully occupied building. Therefore, the investigation will present only the external brace technique, leaving other options open for further research.

2.4.1 Modeling Procedure

In order to properly model the building and its individual panel elements the following procedure is followed:

1. Starting with the first floor, a 3D AutoCAD model is constructed representing the wall elements and the floor above them (Fig.2-8).
2. The global wall and floor components are then discretized into smaller 0.5-meter x 0.5-meter mesh elements (Fig.2-9).
3. This single story is converted to a .dxf format and then imported into SAP 2000.
4. In SAP 2000, material properties are specified and most importantly area properties are created over the mesh: one for wall elements and one for floor elements. These area elements are assigned attributes of a thin-shell and two layers of reinforcement are specified (Fig.2-10).

5. It is important to note that all of the façade panels are not modeled because they are non-structural. Instead the joints above these elements are constrained from moving in the z direction.

6. This completes the story and now the assembly can be copied up into the final 8-story building (Fig.2-11).

7. Live area loads of 150 kg/m² and 300 kg/m² are applied to the living space and balconies, respectively. A response spectrum function per Eurocode 8 is created in both the x and y directions setting the ground conditions to level C and the behavior factor $q = 3$.

8. Finally, the analysis is run and the results gathered.

The elements used in this section are much like the elements of the actual building. Using finite elements the actual shape of the wall panels and floor slabs are modeled including door openings. The panels are given attributes of thickness and material properties, in this case the effective modulus of concrete including the actual reinforcement ratios.
Figure 2-9: 3D AutoCAD model of one story vertical elements.

Figure 2-10: Discretize panels into smaller mesh.
Figure 2-11: Create 3D SAP 2000 model with thin-shell elements.
Figure 2-12: Complete 8-story structure divided into finite elements.
2.4.2 Modeling Rehabilitation Technology

After the in-situ building is successfully modeled and analyzed it is ready for the retrofit technology to be applied to it. This procedure is quite similar to the one described in the previous section. Since a steel external brace will be used to stiffen the structure and reduce drift, the individual elements can be again drawn in AutoCAD (see Fig.2-12). These elements are then imported directly into the existing SAP 2000 model as frame elements. A frame section is established; in this study a general wide-flange section of W14x82 is used. However, for more detailed results, the required stiffness can be back calculated using the drift values from the original analysis. Members can be sized accordingly. The overall dimension of this scheme is a 4 m x 4 m chevron brace connected to the floor slab at every 2.8 m along the building height.

Figure 2-13: Apply external brace frame in SAP 2000.
3 RESULTS & INTERPRETATION

This chapter presents the results from the analysis performed on the in-situ building and compares them to the ones from the rehabilitated model. This will include mode shapes and periods associated with each for the overall building as well as drift and accelerations of nodes at specific locations on the building. Then, a simple cost analysis is performed to determine the feasibility of such a retrofit method. Finally, several methods of raising funds are discussed with the help of current initiatives around Europe.

3.1 Analysis Results

3.1.1 Dynamic Analysis without Bracing

Fig.3-1 represents the corresponding mode shapes from the modal analysis. The first, second and third mode has a period of 0.282 sec, 0.207 sec, and 0.141 sec, respectively. From this analysis we can fundamentally conclude that in fact the structure is stiff with all three modes falling in the peak region.

Figure 3-1: Modal periods plotted on response spectrum.
These results are quite undesirable and confirm that something needs to be done in order to lower the response. Intuitively, adding more mass or softening the overall structure will both increase the period and thereby lessen the response. However, both of these options are not viable. Instead, the structure will be stiffened to mitigate drift and damage to connections.

In the following figures and graphs the corresponding mode shapes are illustrated. Figures 3-2, 4 & 6 are taken directly from the SAP 2000 model. They show the roof (H = 22.4 meters) motion in plan relative to the static case. Each mode is accompanied with a graph (Figures 3-3, 5 & 7) of the nodal displacements in the x direction for each floor. The node selected is the right bottom most node when looking at the floor plan. From the graphs it is noticeable that the building behaves similar to a shear beam. Overall, all three mode shapes exhibit a torsional behavior as well.

Table 3-1 is an output of the absolute acceleration this specific floor node experiences when the response spectrum analysis is applied in both the global x and y directions.

### Table 3-1: Absolute floor accelerations in the local axis. RS is applied in both the x and y direction.

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<th>Height</th>
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<th>Y</th>
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<thead>
<tr>
<th>Height</th>
<th>X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m/sec²</td>
<td>m/sec²</td>
</tr>
<tr>
<td>2.8</td>
<td>0.194</td>
<td>0.384</td>
</tr>
<tr>
<td>5.6</td>
<td>0.486</td>
<td>0.980</td>
</tr>
<tr>
<td>8.4</td>
<td>0.841</td>
<td>1.688</td>
</tr>
<tr>
<td>11.2</td>
<td>1.231</td>
<td>2.429</td>
</tr>
<tr>
<td>14</td>
<td>1.633</td>
<td>3.145</td>
</tr>
<tr>
<td>16.8</td>
<td>2.024</td>
<td>3.796</td>
</tr>
<tr>
<td>19.6</td>
<td>2.386</td>
<td>4.357</td>
</tr>
<tr>
<td>22.4</td>
<td>2.694</td>
<td>4.813</td>
</tr>
</tbody>
</table>
Figure 3-2: Mode 1 deformed shape at the roof level.

Figure 3-3: Mode 1 shape in the global x direction.
Figure 3.4: Mode 2 deformed shape at the roof level.

Figure 3.5: Mode 2 shape in the global $x$ direction.
Figure 3.6: Mode 3 deformed shape at the roof level.

Figure 3.7: Mode 3 shape in global x direction.
Finally, Figures 3-8 & 9 illustrate the drift of the top floor (H = 22.4 meters) when the response spectrum is applied in the global $x$ direction and the $y$ direction, respectively. The drift in the local $(x, y)$ of the bottom most node on the right is (11.4 cm, 7.3 cm) when the RS is applied in the $x$ direction and (3.1 cm, 5.4 cm) with the RS applied in the $y$ direction.

Figure 3-9: Deformed shape after RS is applied in the global $x$ direction.

Figure 3-8: Deformed shape after RS is applied in the global $y$ direction.
3.1.2 Dynamic Analysis with Bracing

Following the in-situ dynamic analysis of the building the mentioned bracing scheme is applied. The same analysis as before is performed. Fig.3-10 maps the new modified periods on top of the same response spectrum. The new period for the first, second and third modes are 0.198 sec, 0.137 sec, and 0.112 sec, respectively. All three periods are less than the previous meaning that the structure is in fact stiffened by the external brace. Mode 2 and 3 are now in the linear region outside the peak ground acceleration that experiences less acceleration.

Figure 3-10: Plotted modal periods after applying external bracing.
Figures 3-11, 13 & 15 are taken directly from the SAP 2000 model. They show the roof (H = 22.4 meters) motion in plan relative to the static case. Each mode is accompanied with a graph (Figures 3-12, 14 & 16) of the nodal displacements in the $x$ direction for each floor. The node selected is the right bottom most node when looking at the floor in plan. In this braced case, the shear beam behavior is preserved although the displacements are all less than the unbraced scheme. Overall, all three mode shapes exhibit a torsional behavior as well.

Table 3-2 is the output of the absolute acceleration of the outer most node before and after bracing when the response spectrum analysis is applied in both the global $x$ and $y$ directions. The last column expresses the percent change in the acceleration values for this node. Compared to the unbraced case, the accelerations in both the local $x$ & $y$ direction experience a decrease in magnitude under the braced case.

<table>
<thead>
<tr>
<th>Floor Level</th>
<th>Height (m)</th>
<th>$x$</th>
<th>$y$</th>
<th>$x$</th>
<th>$y$</th>
<th>Maximum Change (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.8</td>
<td>0.49</td>
<td>0.38</td>
<td>0.39</td>
<td>0.35</td>
<td>-19.4%</td>
</tr>
<tr>
<td>2</td>
<td>5.6</td>
<td>1.25</td>
<td>0.98</td>
<td>1.05</td>
<td>0.90</td>
<td>-16.0%</td>
</tr>
<tr>
<td>3</td>
<td>8.4</td>
<td>2.12</td>
<td>1.69</td>
<td>1.84</td>
<td>1.54</td>
<td>-13.1%</td>
</tr>
<tr>
<td>4</td>
<td>11.2</td>
<td>3.01</td>
<td>2.43</td>
<td>2.70</td>
<td>2.20</td>
<td>-10.4%</td>
</tr>
<tr>
<td>5</td>
<td>14</td>
<td>3.84</td>
<td>3.15</td>
<td>3.54</td>
<td>2.82</td>
<td>-10.5%</td>
</tr>
<tr>
<td>6</td>
<td>16.8</td>
<td>4.58</td>
<td>3.80</td>
<td>4.33</td>
<td>3.36</td>
<td>-11.4%</td>
</tr>
<tr>
<td>7</td>
<td>19.6</td>
<td>5.19</td>
<td>4.36</td>
<td>5.02</td>
<td>3.82</td>
<td>-12.3%</td>
</tr>
<tr>
<td>8</td>
<td>22.4</td>
<td>5.68</td>
<td>4.81</td>
<td>5.59</td>
<td>4.19</td>
<td>-12.9%</td>
</tr>
</tbody>
</table>
After addressing the floor accelerations interstory drift is examined. Table 3-3 shows the drift for each floor before and after the rehabilitation is applied. In order to better understand the effects of the external bracing on drift, the maximum percent change in drift is presented in the final column. From the initial analysis results it can be directly concluded that this precast panel building, modeled in this particular fashion, does not meet the prescribed drift requirement of $\delta = 0.005$. However, after the external bracing is applied the drift levels see a significant change in magnitude. Over half of the floor levels experience a change of damage category from significant structural damage to only likely nonstructural damage. It does not fully solve the problem in the upper floors, with the building still seeing drift levels above 0.007, but certainly the external bracing system mitigates the seismic effects.

<table>
<thead>
<tr>
<th>Floor Level</th>
<th>Height (m)</th>
<th>Rel. Displ. (m)</th>
<th>Drift</th>
<th>Rel. Displ. (m)</th>
<th>Drift</th>
<th>Maximum Change (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.8</td>
<td>0.017</td>
<td>0.0061</td>
<td>0.0023</td>
<td>0.0008</td>
<td>-86.5%</td>
</tr>
<tr>
<td>2</td>
<td>5.6</td>
<td>0.028</td>
<td>0.0100</td>
<td>0.0072</td>
<td>0.0026</td>
<td>-74.3%</td>
</tr>
<tr>
<td>3</td>
<td>8.4</td>
<td>0.049</td>
<td>0.0175</td>
<td>0.0104</td>
<td>0.0037</td>
<td>-78.8%</td>
</tr>
<tr>
<td>4</td>
<td>11.2</td>
<td>0.06</td>
<td>0.0214</td>
<td>0.0269</td>
<td>0.0096</td>
<td>-55.2%</td>
</tr>
<tr>
<td>5</td>
<td>14</td>
<td>0.08</td>
<td>0.0286</td>
<td>0.0221</td>
<td>0.0079</td>
<td>-72.4%</td>
</tr>
<tr>
<td>6</td>
<td>16.8</td>
<td>0.087</td>
<td>0.0311</td>
<td>0.0419</td>
<td>0.0150</td>
<td>-51.8%</td>
</tr>
<tr>
<td>7</td>
<td>19.6</td>
<td>0.102</td>
<td>0.0364</td>
<td>0.0313</td>
<td>0.0112</td>
<td>-69.3%</td>
</tr>
<tr>
<td>8</td>
<td>22.4</td>
<td>0.105</td>
<td>0.0375</td>
<td>0.0548</td>
<td>0.0196</td>
<td>-47.8%</td>
</tr>
</tbody>
</table>

Table 3-3: Percent change in the interstory drift due to external bracing.
Figure 3-11: Modified mode 1 deformed shape with external brace.

Figure 3-12: Modified mode 1 shape in global x direction.
Figure 3-13: Modified mode 2 deformed shape with external brace.

Figure 3-14: Modified mode 2 shape in global x direction.
Figure 3-15: Modified mode 3 deformed shape with external brace.

Figure 3-16: Modified mode 3 shape in global x direction.
Figures 3-17 & 18 illustrate the drift of the top floor (H = 22.4 meters) when the response spectrum is applied in the global $x$ direction and the $y$ direction, respectively. The drift in the local $(x, y)$ of the bottom most node on the right is (8.1 cm, 5.9 cm) when the RS is applied in the $x$ direction and (2.4 cm, 4.1 cm) with the RS applied in the $y$ direction. Again, these drifts are less than before.

**Figure 3-17**: Modified deformed shape after applying RS in global $x$ direction.

**Figure 3-18**: Modified deformed shape after applying RS in global $y$ direction.
3.2 Cost Analysis

This section gives a brief cost analysis of how much a rehabilitation system like the one analyzed would cost. The cost calculation includes the direct costs of construction materials and labor. According to FEMA, typical costs for seismic rehabilitation should also include: A/E design fees, permit fees, demolition, site clearing and financing (FEMA, 1994). Some indirect costs to consider include: social impacts, occupant relocation, construction delays and so on. In a detailed cost evaluation, all of these factors need to be considered.

3.2.1 Construction Calculations

In this study, construction costs include the foundations, structural steel by ton and labor. The following procedure is used to determine a cost for using a private contractor:

Structural Steel:

\[
465.6 \text{ m} = 1527.2 \text{ ft} \times \frac{82 \text{ lb}}{\text{ft}} = 125,230 \text{ lbs} = 56.8 \text{ tons of steel}
\]

\[
56.8 \text{ tons} \times \frac{\$800}{\text{ton}} = \$45,440
\]

Foundations:

\[
5 \text{ m} \times 5 \text{ m} \times 3 \text{ m} = 75 \text{ m}^3 \text{ reinforced concrete}
\]

\[
75 \text{ m}^3 \times \frac{\$250}{\text{m}^3} = \$18,750
\]

Labor:

Based on average labor rates of $18/hr and 5 workers over a 1.5-month construction schedule the labor cost is estimated around $21,600 (Nenplan Engineering Ltd.).
Therefore, after the simple calculations the total construction cost is estimated to be close to $85,790. With consideration for a 5% contingency and projected consultant fees the overall cost should be roughly $100,000 that is equivalent to 150,000 BGN (local currency). This price is relatively high if the private residents of the building solely funded the rehabilitation. Considering a rather low standard of living of the occupants this construction is not feasible. However, if a private investor or the federal government were to get involved the cost is not out of their budget.

3.2.2 Raising Funds

From the conclusion of the previous section, it is evident that different funding sources need to be considered. Tenants of these buildings, especially ones in Bulgaria, one of the EU’s poorest countries, simply cannot afford to fund these projects out of their own pockets. Several alternative funding methods are presented in this section.

Panel housing communities still existing in Western European countries like Austria, France and Germany have already began to tackle this problem. It should be mentioned that all three nations have enhanced social welfare programs compared to that of Eastern European countries. In Germany for example, the state subsidizes rehabilitation efforts across the country by constructing additional floors to existing panel buildings. It then sells the new living space at a reduced cost to low income families and uses the money to rehabilitate the entire building. The retrofit efforts primarily benefit the thermal and architectural condition of the complexes.

Until the early 1990s energy sources for household needs (district heating, electricity, etc.) were heavily subsidized by the state and thus were not an expensive item for residents (Csagoly, 2003). In sense, “low-cost” heating compensated for the poor energy characteristics of the buildings. Since then
modernization initiatives have spread across several European countries. To help offset the rise in energy prices and the thermal inefficiency of panel buildings, the EU launched the Solanova pilot project in Dunaújváros, Hungary in 2005. A 42-story panel apartment building was renovated and converted to a low energy consumption building (Zsebik, 2005).

The aim of the reconstruction was to restrict the annual space heat demand by 15 to 45 kWh/m², which is characteristic of ultra-low energy consumption of buildings (Zsebik, 2005). Typical average annual space heat consumption of panel buildings is ca. 200 kWh/m². The renovation efforts mainly focused on a complete restructuring of the insulation as well as changing or modifying all external doors and windows. After the renovation the percent achieved in energy savings in the following four seasons was 78.7%, 88.4%, 84.6% and 84.3% (Zsebik, 2005). Fig.3-19 illustrates how the building looked before and after renovation.

*Figure 3-19: Before and after photos of the first Solanova pilot building (www.solanova.eu).*
This pilot study in Hungary is a world novelty when it comes to renovation of panel structures. Never before has a building of this size and type been refurbished to a standard matching the present and future demands for new buildings (Zsebik, 2005). It is clear that this case study can serve as a reference for sustainable renovation for all the EU countries, and especially for those have large number of panel buildings. Most importantly, these results prove to lawmakers the necessity to modify the policy of governmental subsidies for large-scale building renovations to include the requirements of sustainable renovation.
4 CONCLUSION

4.1 Summary

This thesis first presented the industrial construction method that revolutionized the building industry in countries across Eastern Europe in the 1960s, 70s and 80s. This period was marked by a massive influx of people from rural areas into an urban working environment. In order to meet the growing housing demands, the State developed a “cookie-cutter” approach in order to build large scale apartments for a low cost and time. These industrial methods of construction primarily focused on the use of the frameless large-panel structural system.

Following the history of this building type, the focus shifted to the problem. Several Eastern European countries, Bulgaria in particular, are located in a high seismic zone. In fact, the area has been very seismically active in recent decades with several 7+ magnitude earthquakes. This leaves any structure with poor structural performance extremely vulnerable to damage or failure. Following the 2012 Pernik earthquake, a pair of Bulgarian structural engineering professors performed a series of field investigations. The first of its kind, this study gave new insight on the structural health, the behavior of these buildings, and the need for their strengthening.

In the next chapter the case study building is presented. All relevant floor plans, drawing details and material properties used are summarized. Then these parameters are used as the basis for creating a finite element model. Once the in-situ building is analyzed using a response spectrum, the braced frame retrofit is added and the analysis is performed once more. The corresponding drifts and accelerations are then summarized and compared. To close the chapter a simple cost analysis is performed and several examples of current EU renovation initiatives are discussed.
4.2 Key Conclusions

Finally, after considering the history of these structures, the information presented in this thesis and the results of the dynamic analysis on the existing panel building with the applied rehabilitation scheme, the following key conclusions can be made:

1. From section 1.2 it can be concluded that construction by industrial methods was effective at meeting the rising housing demand in cities across Eastern Europe. These construction efforts were successful in meeting a low cost and completion time but neglected the quality of the design. Unfortunately, as building codes progressed and got more advanced these buildings did not. They remain essentially untouched since their assembly nearly 30-40 years ago.

2. From the on-site inspections and lab tests done by Profs. Bonev & Nikolov it can be concluded that large-panel buildings in Bulgaria are not designed according to EU standards (capacity design rules and performance requirements are not satisfied). Thus they are in need strengthening. Their conclusions are backed up by the analysis results in section 3.1. The case study building exceeds prescribed interstory drifts for comfort and structural damage. Floor accelerations are also greater than desirable.

3. In the latter portion of section 3.1, results suggest the external bracing system effectively stiffens the structure. As a final outcome, the bracing scheme used does in fact decrease both the floor accelerations and the interstory drift by at least 10% and in some cases as much as 85%.

4. The modeling techniques described in chapter 2 provide a foundation for more detailed analytical models. This also supports the need for further research.

5. The cost analysis in section 3.2 shows the external bracing scheme is an economical solution. Although not affordable for individual residents, it is feasible through government subsidies or private investments.
4.3 Suggestions for Further Research

The precast large-panel structural system is one that will go down in history with great significance. Unfortunately, as quickly as it became the most popular building style across Eastern Europe, it took even less time for it to become virtually extinct by the 21st century. For decades there has been plenty of literature and research on dynamic analysis as well as that of individual shear wall/panel systems. However, after the fall of the USSR, many of the designers, planners and constructors of these buildings are no longer available as resources, leaving an empty gap in research, literature and the understanding of these historical structures. This sends a clear message: there is a need for further research to better understand the behavior of this specific style of building.

For the simplicity of this thesis only an elastic response spectrum was implemented. In further research it is essential to construct a more detailed model with specific emphasis being placed on the connections between individual elements. A non-linear time history analysis must also be performed to better understand the structure’s behavior. It is also important that this analysis be performed on several different building models with a variety of other retrofit schemes in order to detect any inconsistencies or unforeseen behavior. Finally, since there are few of people that know about these problems or have an interest in helping to solve them, it is crucial to disseminate this information and look for ways to increase public awareness and sources of funding.
References

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