COST-BENEFIT ANALYSIS FOR VARIOUS REHABILITATION STRATEGIES

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ABSTRACT

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Over the last decade, six major earthquakes that occurred in Turkey dramatically demonstrated the poor performance of the buildings that were designed and constructed far from Turkish seismic code's requirements. The Marmara region, where most of the population and industry is located, is in the active seismic zone. With the rising cost of damages due to earthquakes, the necessity of the cost-benefit analysis for various rehabilitation strategies used in existing buildings has become a major concern for the decision makers who are in the position of making decisions on the building rehabilitation strategies.

This study evaluates the performance of two different rehabilitation strategies applied to two five-story reinforced concrete buildings and assesses their cost-benefit analyses. These buildings were chosen to be representative of the typical residential buildings in Turkey.

To carry out the structural analysis of the buildings, three-dimensional models of the buildings were developed using SAP2000 [6]. Two alternative strengthening methods, insertion of reinforced concrete shear walls and application of Carbon Fiber Reinforced Polymers (CFRP) on hallow clay tile infill walls, were used for both of the buildings. While modeling infill walls strengthened with CFRP, two specific modeling attempts proposed by the researchers at Middle East Technical University were used. Pushover analyses were performed to evaluate seismic performance of the buildings. The Life Safety criterion was chosen as the rehabilitation objective. The global and component response acceptability limits were checked and the cost-benefit analysis was performed in order to determine the most attractive rehabilitation alternative.

The results and comparisons given here illustrated that strengthening with shear wall had the most significant improvement on the seismic performance and cost effectiveness of the case study buildings. Outcomes of this study are only applicable to the buildings employed here and are bound by the assumptions made, approximations used and parameters considered in this study. The findings cannot be generalized for the buildings rehabilitated with CFRP due to lack of the consistent models for CFRP application. More research needs to be conducted to provide solid guidelines and reliable models applicable to the CFRP rehabilitated infill walls.

Keywords: Rehabilitation, Performance Point, Pushover Analysis, Cost-Benefit Analysis, SAP2000, Shear Wall, Carbon Fiber Reinforced Polymer

ÖZ

ÇEŞİTLİ REHABİLİTASYON STRATEJİLERİ İÇİN MALİYET-FAYDA ANALİZİ

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Son on yılda Türkiye'de meydana gelen altı büyük deprem, Afet Yönetmeliği şartlarını sağlamadan tasarlanan ve inşa edilen binaların ne denli kötü performans sergilediğini ortaya koydu. Özellikle nüfusun ve endüstrinin yoğunlaştığı Marmara Bölgesi aktif sismik bölge üzerinde yer almaktadır. Deprem sonrası oluşan hasarların artan maliyetleri ile birlikte maliyet-fayda analizi ilgili karar mekanizmaları için başlıca önem arz eden unsur haline geldi.

Bu çalışma iki farklı rehabilitasyon stratejilerinin uygulandığı iki adet beş katlı binanın performansını değerlendirmekte ve maliyet-fayda analizlerini sunmaktadır. Seçilen binalar Türkiye'de inşa edilen çoğu konut tipi binanın özelliklerini içermektedir.

Binaların yapısal analizlerini yapmak amacıyla SAP2000 [6] kullanılmıştır. Her iki bina için güçlendirme metodu olarak perde duvarlar ile güçlendirme ve karbon lif takviyeli polimer ile tuğla duvarların güçlendirilmesi uygulanmıştır. Karbon lif takviyeli polimer ile güçlendirilmiş tuğla duvarlar modellenirken, ODTÜ-Yapı Mekaniği Laboratuarı'nda yürütülmüş olan araştırma programı sonucunda geliştirilmiş iki farklı model kullanılmıştır. Binaların davranışlarını değerlendirmek amacıyla statik itme analizleri yapıldı. Can Güvenliği performans kriteri rehabilitasyon amacı olarak seçildi. Bu analizler sonucunda, global kabul edilebilir limitler ve elemanlar için kabul edilebilir limitler kontrol edildi. En uygun alternatifi belirleyebilmek amacıyla maliyet-fayda analizleri yapıldı.

Ortaya çıkan sonuçlar ve karşılaştırmalar; perde duvar ile yapılan güçlendirme tekniğinin diğer yönteme kıyasla binanın sismik performansını daha çok artırdığını maliyet ve fayda açısından değerlendirildiğinde daha etkin olduğunu göstermiştir. Bu çalışmanın sonuçları yalnızca incelenen binalar için geçerlidir; yapılan kabuller, tahminler ve kullanılan parametrelerle sınırlıdır. Karbon lif takviyeli polimer uygulaması için önerilen iki model arasındaki farklılığa bağlı olarak, bu çalışmanın bulguları bu yöntem ile güçlendirilmiş binalar için genellenemez. Karbon lif takviyeli polimer uygulaması için daha güvenilir ve doğru modellerin oluşturulması için daha fazla araştırma yapılması gerekmektedir.

Anahtar Kelimeler: Rehabilitasyon, Performans Noktası, Statik İtme Analizleri, Fayda-Maliyet Analizi, SAP2000, Perde Duvar, Karbon Fiber Takviyeli Polimer

To Whom I Love,

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LIST OF SYMBOLS AND ABBREVIATIONS

3-D	Three dimensional
A_o	Effective Ground Acceleration Coefficient
A_{ch}	Gross section area of a solid wall, wall segment of a coupled wall, a floor or a floor segment of a perforated floor
ADRS	Acceleration-displacement response spectrum
a_{pi}	Spectral acceleration of trial performance point
ATC	Applied Technology Council
a_y	Yield spectral acceleration
B	Expected benefit attributed to the rehabilitation
BCR	Benefit/Cost Ratio
C	Cost of rehabilitation alternative
CFRP	Carbon Fiber Reinforced Polymer
C_i^M	Cost of retrofitting measures
C_m	Effective mass factor
CSM	Capacity spectrum method
d	Discount rate
d_p	Roof displacement
d_{pi}	Spectral displacement of trial performance point
d_y	Yield spectral displacement
Ec	Young's modulus of elasticity
f_{ck}	Characteristic compressive strength of concrete
f_{ctd}	Design tensile strength of concrete
FEMA	Federal Emergency Management Agency
f_{yd}	Design yield strength of longitudinal reinforcement
f_{yk}	Characteristic yield strength of longitudinal reinforcement

g	Acceleration of gravity
h	Height of column
h'	Height of infill
K_i	Initial stiffness
K_y	Yield stiffness
L	Length of member
l_p	Equivalent plastic hinge length
LS	Life Safety
I	Second moment of inertia
I	Building Importance Factor
IO	Immediate occupancy
M_{ux}	Component of biaxial flexural strength on the x-axis at the required inclination
M_{uy}	Component of biaxial flexural strength on the y-axis at the required inclination
M_{uxo}	Uniaxial flexural strength about the x-axis
M_{uyo}	Uniaxial flexural strength about the y-axis
NL	Expected number of fatalities
NPV	Net Present Value
PF	Modal participation factors
PGA	Peak ground acceleration
P_{yield}	Yield force
R	Seismic Load Reduction Factor
R_x	Percent of the ratio of shear wall area to the plan area in the x-direction
R_y	Percent of the ratio of shear wall area to the plan area in the y-direction
S_a	Spectral acceleration
S_d	Spectral displacement
SR_A	Spectral reduction factor
SR_V	Spectral reduction factor
SS	Structural Stability
t	Thickness of infill
T	Period

TN	Time horizon
V	Value of human life
V_p	Base shear force at performance point
V_r	Yield shear force capacity
V_u	Ultimate base shear force
V_y	Yield base shear force
W	Dead weight of the building plus live loads
w	Equivalent strut width of compression diagonal
α	Effective modal mass coefficients
β	Parameter dictating the shape of the inclination surface
β_0	Hysteretic damping
β_{eff}	Effective damping
θ	Slope of infill diagonal to horizontal
θ_p	Plastic rotation
θ_{yield}	Yield rotation
Δ / h	Global drift ratio
Δ_{roof}	Roof displacement
Δ_u	Ultimate displacement
Δ_y	Yield displacement
Ø	Diameter of the steel bar
κ	Modification factor for the simulation of probable imperfections in real building hysteresis loops
λh	Relative stiffness of the frame with respect to the infill
μ	Ductility ratio
ρ_{sh}	Ratio of horizontal web reinforcement of wall to the gross area of wall web
φ_{ult}	Ultimate curvature
φ_{yield}	Yield curvature

CHAPTER 1

INTRODUCTION & PREVIOUS STUDIES

1.1 INTRODUCTION

Turkey is one of the most seismically active countries in the world. The earthquakes that occurred in the last decade in Turkey led to huge economic loss and casualties. These consequences have demonstrated the lack of detailing, inadequate lateral resistance and important system deficiencies in the buildings. It is obvious that most of the buildings constructed in Turkey are far from overcoming the code demands.

According to the predictions much of the current building stock in Marmara Region is likely to experience strong ground shakings in a near future. Regarding the consequences of the recent devastating earthquakes, the concern over loss mitigation and cost-benefit analysis has significantly increased. The decision of strengthening a large number of building stock is very difficult to make for the administrative and public authorities, as the cost of such improvement requires great appreciation. Therefore it is urgent to conduct widespread researches consisting of cost-benefit analyses for different rehabilitation strategies of buildings throughout this region.

Cost-benefit analysis is a very functional tool in order to make decisions to understand whether it is worth or not and which rehabilitation strategy is the most beneficial and cost effective for the structure investigated. The most important matter in the cost-benefit analysis is the decision on the appropriate rehabilitation objective and choice of the most feasible rehabilitation strategy among a number of potential alternatives. For this reason, at the beginning of the analysis the performance objectives must be clearly identified by the decision makers.

Although it does not seem feasible in some ways, generally structures have been rehabilitated with conventional strengthening techniques like insertion of concrete shear walls and column jacketing after recent earthquakes in Turkey. Nowadays, new rehabilitation techniques are evolving gradually. One of them is strengthening infill walls with Carbon Fiber Reinforced Polymers (CFRP). One of the most important concerns is the cost-effectiveness of the new rehabilitation techniques as compared to the conventional procedures. In order to investigate the costs, benefits and feasibility of these two procedures and to compare their seismic performance, this research was undertaken. Two case study buildings selected for this purpose have been hypothetically upgraded using the two rehabilitation schemes and the results were compared.

1.2 EXISTING REHABILITATION METHODOLOGIES

Identifying the best rehabilitation strategy to reduce the risk to acceptable limits is one of the most important parameters influencing the cost-benefit analysis. The aim of rehabilitation strategies is to improve seismic performance of the building. For this reason, while conducting the cost-benefit analysis most suitable strategy must be chosen such that the desired performance objective is reached. For a residential building the scope of the project is effected primarily by the persons or establishments that pay the project cost.

In 1997, the Federal Emergency Management Agency (FEMA) published resource documents FEMA 273 [11] and FEMA 274 [12] which were aimed to be guidelines and commentary for the seismic rehabilitation of buildings. These two documents were later combined into a new document, "Prestandard for Seismic Rehabilitation" FEMA 356 [13] which advocates a displacement based method and non-linear pushover analysis. This prestandard was intended as an applicable tool for design professionals, code officials and building owners undertaking the seismic rehabilitation of buildings. Provisions that include technical requirements for seismic rehabilitation were set up. Moreover the study includes foundations and geologic site hazards, design, rehabilitation requirements for steel, concrete, masonry, wood and light metal framing, seismic isolation and energy dissipation, simplified rehabilitation, architectural, mechanical, and electrical components. It provides a general point of view before initiating rehabilitation strategies for the cost concept.

Available seismic rehabilitation procedures are summarized in the Seismic Evaluation and Retrofit of Concrete Buildings (ATC-40) [3] prepared by Applied Technology Council (ATC). Alternative retrofitting strategies were classified into two groups, technical strategies and management strategies. The following sections explain briefly each of these strategies that have different considerations in reducing seismic risk.

1.2.1 Technical strategies

Technical strategies provide reliable approaches for the seismic performance of the building by modifying demand and response elements of the building.

Basic factors affecting the lateral force resisting system's behavior are:

- Building mass
- Stiffness
- Damping
- Configuration
- Deformation capacity

There are four approaches used for technical strategies. They are system completion, system strengthening and stiffening, enhancing deformation capacity and reducing earthquake demands.

System completion

This approach should be applicable for the structures reaching an acceptable performance point with some local failure events and for the structures having walls, diaphragms and frames acting as a lateral force resisting system. Common causes of these local failures are listed below.

- Lack of inadequate chord and collector elements at diaphragms
- Inadequate bearing length at precast element supports
- Inadequate anchorage or bracing of structural or nonstructural components

General methods for system completion are using diaphragm chords, collectors and drags which are commonly used for timber diaphragms, using steel element connectors for buildings that consist of precast elements and bracing and anchoring the building.

System strengthening and stiffening

This approach is the most favorite and common seismic performance improvement. System stiffening and system strengthening are related to each other. They have to be introduced to the structure at the same time. Techniques used for stiffening strengthen the building and strengthening techniques stiffen the buildings. System strengthening increases total lateral force capacity of the building and system stiffening shifts performance point of the building to a better level.

i. Shear walls:

Introducing reinforced concrete shear walls into an existing building, one of the most favorable rehabilitation techniques, is very successful at increasing both building strength and stiffness.

Although this method has been used traditionally, placement of shear walls often poses problems for the architectural design. The necessity of evacuation of the rehabilitated building and being a time consuming methodology are other adverse effects of this strengthening method.

ii. Carbon Fiber Reinforced Polymer (CFRP) applied on the infill wall

Strengthening infill walls with carbon fiber reinforced polymer (CFRP) has become popular in the rehabilitation of reinforced concrete structures. However, limited number of studies exists on their use. These studies have revealed the significance of these techniques on the improvement of the seismic performance in terms of strength, stiffness and energy dissipation capacity. This technique is very simple and fast to apply in comparison with the other techniques. Furthermore it is a very efficient method because it does not require evacuation during rehabilitation.

iii. Braced frames

Although bracing frames with steel does not provide strength and stiffness as much as shear walls, it is another common method. As their mass is less than the mass of shear walls, they do not result in a significant increase in building mass and therefore increase seismic forces induced by the lateral load.

Besides its advantages, this technique has difficulties while attaching bracing steel members to the existing concrete structure.

iv. Buttresses

This system is appropriate when occupancy is essential during rehabilitation. It can be applied outside the building by adding an additional construction.

v. Moment resisting frames

Moment frames enhance improvement of strength of the building and have the advantage of occupying relatively a minimal floor space. However, their use is generally limited as they have relatively large lateral drift capacity than the building they are applied. This incompatibility is the main problem for the system.

vi. Diaphragm strengthening

The most commonly used methods for diaphragm strengthening are:

- Topping slabs, metal plates laminated onto the top of the surface of the slab
- Bracing diaphragms below the concrete slabs
- Increasing existing nailing in the covering and replacing the covering with stronger material or overlaying the existing covering with plywood. (For buildings with timber diaphragms)

Enhancing deformation capacity

Column jacketing, column strengthening and providing additional supports at places subjected to deformation are among the most typical applications of this technique.

i. Adding confinement

Another widespread method used in rehabilitation projects is confining the columns. Column jacketing improves deformation capacity of non-ductile columns. Jacketing can be made using two techniques, confining with continuous steel plates and with fiber-reinforced plastic fabrics. Effectiveness of the technique depends on attachment of confinement to resist pressure exerted on them.

ii. Column strengthening

Column strengthening becomes necessary for buildings in which strong beam-weak column configurations appear. It will permit formation of story mechanisms and much larger drifts.

iii. Local stress reductions

This technique is implemented for the elements that are not primary for the building's performance. Procedures for local stress reductions are:

- Demolition of local members which are quite stiff and respond lateral forces which they can not resist
- Introducing joints between face of the column and adjacent architectural elements.

iv. Supplemental support

Supplemental bearing supports should be effective for the gravity load bearing structure elements that are not effective in resisting lateral force induced by earthquake.

Reducing earthquake demands

Reducing earthquake demands includes new and very expensive special protective systems. Other techniques improve capacity of the building while these systems modify the demand spectrum rather than the capacity spectrum for the structure. Usage of these systems is appropriate for the important buildings like historical buildings or for the accommodation critical occupancies with valuable equipments and machinery.

i. Base isolation

It is applied by inserting bearings that have relatively low stiffness, extensive lateral deformation capacity and advanced energy dissipation capacities. These characteristics counter lateral deformation demands induced on the building.

Base isolation is applicable without performing significant modifications to the structure and suitable for important historic structures. This strategy may be cost effective when there are substantial performance objectives.

ii. Energy dissipation systems

Using energy dissipation units (EDUs) is another successful technique to reduce the damping of building response. Primary characteristics and use of these systems are:

- Directly reduce the displacement demands on the structure by dissipating energy.
- Most effective when introduced in structures having greater lateral deformation capacity. Most appropriate for frame structures.
- Should be considered for protection of critical systems and contents in a building.

iii. Mass reduction

Mass reduction is another method to reduce the demand imposed on the building. Mass reduction reduces natural period of the building. Some of the alternative ways of mass reduction are removing heavy nonstructural elements such as water tanks and storage, and removing one or more building stories.

1.2.2 Management strategies

Both design team and other participants who are controlling the project budget execute risk reduction projects. Management strategies are generally controlled by participants involved with the cost of rehabilitation projects. There are two primary types of management strategies:

1) Strategies directing building's performance after rehabilitation
2) Strategies controlling the way of employing technical strategy

Both of these strategies include such methods as:

- Occupancy chance
- Demolition
- Temporary retrofit
- Phased retrofit
- Retrofit while occupying building
- Retrofit while vacant
- Exterior retrofit
- Interior retrofit

1.3 COST BENEFIT ANALYSIS

Cost-benefit analysis is the fundamental necessity of all before initiating a rehabilitation strategy. Different strategies have widely varying costs. Rehabilitation programs include design costs, construction costs, transportation costs, tenant relocation costs, costs of engineering, and costs due to loss of floor space during and after construction etc. In most cases the value of life is the principal motivation. In addition to these criteria reduction of casualties, cost of repair, loss of building function and business interruption are the other motivating major criteria for the relevant authorities.

1.3.1 NATURE OF COST BENEFIT ANALYSIS

It is important to understand the basics of cost-benefit analysis. Cost-benefit analysis is a common measure for all hazard mitigation projects. A key criterion for this analysis is cost effectiveness of funding. A rehabilitation project should be considered as cost effective if project benefits after completion of the project are higher than the project cost.

Cost-benefit analysis can assist interested participants in determining whether the considered project is worth undertaking. However some of the parameters of this analysis are very complicated to assess their benefits. Estimating future losses is a very complex process in the nature of this analysis.

The outcomes of this analysis are the Net Present Values (NPV). There are two ways of calculating NPV:

1. Benefit / Cost ratios (BCR)
2. Present value of future benefits minus cost of the rehabilitation

The following conditions indicate the effectiveness of each of these alternative approaches.

$$\left.\begin{array}{r} \text{Benefit / Cost ratios } \geq 1.0 \text{ or,} \\ \text{Present value of future benefits - Cost of the rehabilitation} \geq 0 \end{array}\right\} \text{cost effective}$$

The Benefit / Cost Ratios (BCR) can be defined as given in Equation 1.1:

$$\boldsymbol{BCR} = \frac{\text{Potential Future Avoided Damages} + \text{Additional Benefits}}{\text{Total Project Cost}} \qquad \textbf{\textit{(1.1)}}$$

Figure 1.1 provides an example of the kind of comparative benefit and cost data after employing cost-benefit analysis.

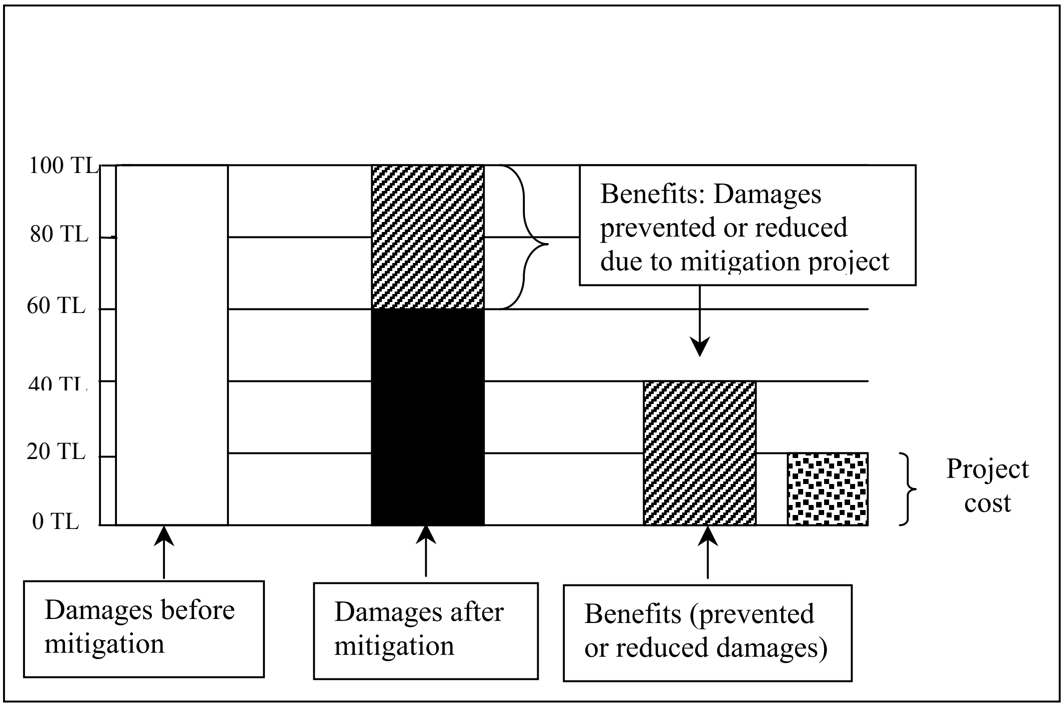

Figure 1.1 Benefit-cost model

1.3.2 PREVIOUS STUDIES

In many countries, after major earthquake disasters, several approaches are implemented to investigate the cost-effective way of reducing the life loss, injuries and property damage from future earthquakes.

Sortis, Pasquale and Romeo [25] analyzed three types of reinforced concrete buildings. A two-floor small housing, a nine-floor residential building and a thirteen-floor office building have been designed according to both Italian Code and Eurocode 8 (version 1994) considering three ductility levels; low, medium and high. Three different hazard studies were used for the economic loss estimation. Fragility curves were established for each building to estimate the economic damage using Peak Ground Acceleration (PGA) values. Life safety and economic losses were the primary parameters to evaluate an optimum combination of seismic zonation and design code that is cost effective. The authors concluded the following:

- The hazards were a dominating factor effecting the cost-benefit analysis.
- The design code and ductility level considered for the building were other important factors effecting economic loss estimation, consequently cost-benefit analysis. According to the results, changing the code from Italian code to EC8,

but leaving the zonation unchanged increased the expected loss with high ductility level.

- Increase in ductility level was not suitable if the objective was cost effectiveness.
- Changing the seismic zonation and keeping the code unchanged reduced the economic risk. Combined implementation of EC8 low ductility and a new classification developed performed best in reducing risk significantly.

In Turkey an extensive project was undertaken by METU Earthquake Engineering Research Center [METU-EERC] for the rehabilitation of moderately damaged buildings on site after Dinar earthquake in 1995 [30]. Cast in place reinforced concrete shear walls were used inside the buildings and the columns in which plastic rotations had occurred were strengthened with steel jacketing. Furthermore rehabilitation was carried out providing additional reinforcement and straightening of damaged steel reinforcement. The findings of the authors are as follows:

- Measures used for the rehabilitation programme in Dinar would be cost effective in time.
- The calculated lateral drift ratios and period of the buildings were reduced significantly.
- All buildings rehabilitated were conforming to the structural requirements stated in 1997 Turkish specifications.
- The performance of all buildings rehabilitated was adequate.

Another comprehensive project was held after the 27 June 1998 Adana-Ceyhan earthquake by METU-EERC [15]. The project consisted of rehabilitation of a building stock in Ceyhan consisting of 108 moderately damaged buildings that were 2-9 stories. The buildings that were found feasible were rehabilitated and the ones damaged severely were demolished. The project team used simple methodologies for the calculation of damage. Buildings that had damage scores indicating light damage were linearly modeled in 3 dimensional spaces and analyzed. Among them, buildings not conforming to the requirements of 1998 Turkish code were rehabilitated. Addition of shear walls and jacketing of deformed columns were used as rehabilitation measures. The cost analyses of each rehabilitated buildings were carried out for all buildings. Authors stated that performance of all rehabilitated buildings reached at least minimum acceptable levels.

1.3.2.1 FEMA approach for cost-benefit analysis

In 1990s after the occurrence of California earthquakes, Federal Emergency Management Agency (FEMA) undertook an effective program of seismic code development. In 1992 FEMA 227 [8] and its supporting documentations FEMA 228 [9] and FEMA 256 [10] provided a standard cost-benefit model for the rehabilitation of existing buildings. Two cost-benefit computer models, the single-class model and the multi-class model, written in a spreadsheet format were included in these reports. The single-class model analyzed groups of buildings that have single use and single group of economic assumptions. The multi-class model investigated groups of buildings having several structural types and uses. Specific cost-benefit models were developed for each classified building types. The concept of the computer programs provided two applications. First application was to make preliminary analysis quickly in order to understand whether rehabilitation strategy was feasible or not. Second application performed more detailed analyses. This application was initiated if the preliminary analyses suggest further consideration. In the documents, five examples illustrating single-class and two examples illustrating multi-class model were considered.

The four major elements considered in this program are:

1. An estimate of damages and losses before mitigation.
2. An estimate of damages and losses after mitigation.
3. An estimate of the frequency and severity of the hazard causing damages.
4. The economic factors of the analysis.

Primary variables used in the FEMA approach are:

- Geographic and Geologic Information
    - Earthquake probabilities for the site where building is located
    - Building site characteristics
- Structural and Engineering Information
    - Facility class
    - Building size
    - Fragility curves
    - Average effectiveness of rehabilitation technique
    - Rehabilitation cost
    - Salvage value

- Building use information
    - Social function classification
    - Occupancy
    - Death and Injury rates
- Building economic information
    - Replacement value
    - Rental income
    - Relocation cost
    - Income
    - Loss of function
    - Business inventory
    - Personal property
- General Economic factors
    - Discount rate
    - Planning horizon
    - Selected NPV coefficient
    - Present value of initial rehabilitation investment
    - Value of life

In the computer program the cost-benefit model considers risk as a combination of high hazard, high vulnerability and high value of investment for the hazard. The concept of risk is summarized in Figure 1.2.

HAZARD & RISK

EARTHQUAKE HAZARD (Frequency & Severity)	×	**PROPERTY EXPOSED TO EARTHQUAKES**	=	**EARTHQUAKE RISK** (Monetary)
Probability of damaging earthquakes		Value and vulnerability of property exposed to earthquake hazard		Severity of threat to the built environment

Figure 1.2 Hazard and risk concept

The computer program gives the Net Present Values (NPV) as an outcome to the user.

Determining cost-effectiveness of rehabilitation projects is of critical importance. In this respect, program maximizes its investment in damage reduction. Cost-benefit model proposed by FEMA requires data entry and each data entry for cost-benefit analysis affects the results. Many data inputs are specific for the project and must be documented by local data.

1.3.2.2 Kunruether's approach for cost benefit analysis

Kunreuther [16] suggests a simplified five-step procedure for estimating losses to structure and evaluating the benefits to the system. This procedure can be applied to lifeline systems like electric power systems, water distribution systems, transportation systems and also to residential buildings. Figure 1.3 illustrates the steps in Kunruether's approach for cost-benefit analysis.

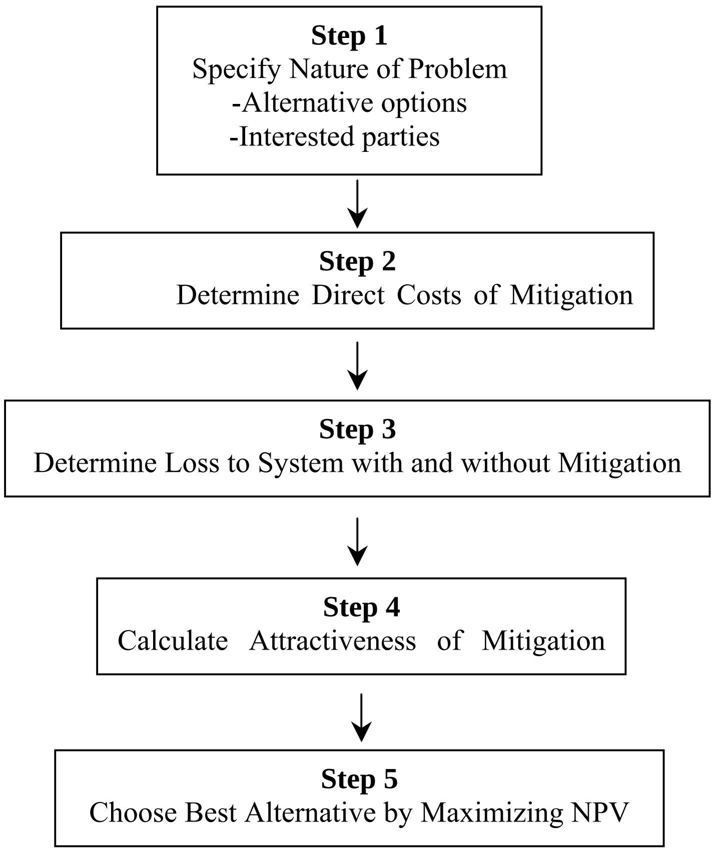

Figure 1.3 Simplified five-step procedures for cost-benefit analysis

Step1: Specify the Nature of the Problem

Rehabilitation options and participants of the project team must be verified at the beginning of cost-benefit analysis. Participants are engineers, public and private organization, building owners, tenants and others who will get benefit for a residential building.

Status quo, building without rehabilitation, is the first alternative and a reference point for the analysis. Other options have to be identified according to the rehabilitation objectives.

Step2: Determine Direct Cost of Mitigation Alternatives

Cost of rehabilitation alternatives except status quo must be specified in this step.

Step3: Determine Direct Benefits of Mitigation Alternatives

A scenario earthquake event or a set of scenario earthquakes, which have different magnitudes, location, duration and frequency, have to be considered to determine the benefits of rehabilitation alternatives. Potential benefits are reducing physical damages, reduced fatalities and casualties.

Step4: Calculate the Attractiveness of Rehabilitation Alternatives

To calculate the attractiveness of the rehabilitation options Net Present Values have to be calculated. To accept an alternative as cost effective either Net Present Value must be greater than zero, or benefit-cost ratio must be greater than 1.

In the calculation of NPV, identifying discount rate i have a major importance since the benefits and costs are expected to accrue over the life of the building. This rate is used in order to discount future benefits into a net present value.

Step5: Choose the Best Alternative by Maximizing Net Present Value

Finally in the last step, Net Present Values have to be maximized. Alternative giving the highest NPV or benefit-cost ratio at the end of the cost-benefit analysis is the most attractive one around alternatives considered.

In implementing this procedure fragility curves are used for the estimation of loss after a possible earthquake. Fragility curves can be developed either by using damage data from past earthquakes or conducting a numerical analysis. Hazard curves have to be obtained as a continuing part of analysis. The last process to obtain loss estimations is to combine hazard curves and fragility information. It is formulated as given below:

$$\sum_{T=1}^{T_N}\sum_{i=1}^{I}\int_{\min}^{\max} R(a,T)\times P(E_i only \perp a)\frac{C_i^D}{(1+d)^{T-1}}da \quad \textbf{(1.2)}$$

$$R(a,T)=R(a)\times e^{-R(a_{\min})(T-1)} \quad \textbf{(1.3)}$$

$R(a,T)$: The probability of exceeding the PGA value given that no earthquake has occurred in the previous years.

$P(E_i only \perp a)$: The probability of only event E_i occurring for a given PGA value a.

Altay, Deodatis, Franco, Gülkan, Kunreuther, Luş, Mete, Seeber, Smyth and Yüzügüllü [1] carried out an analytical study using Kunruther's approach for the cost-benefit analysis of a representative building in İstanbul. It was modeled 3 dimensionally using SAP2000 computer package. Pushover analyses were carried out for the original building and the three different strengthening alternatives of the building. These three retrofitting strategies were; strengthening with bracing, strengthening with partial shear walls and strengthening with full shear walls. The consequences of rehabilitation strategies on the expected damage were compared. Fragility curves for the four-damage states; namely slight, moderate, major and collapse were established and hazard curve for İstanbul region was provided in the form of annual probabilities of exceeding various values of PGA. Besides scientific and engineering data three factors; time horizon-expected period for the occurrence of earthquake (TN), social discount rate (d) and number of fatalities (NL) were taken into account in evaluating the benefits of mitigation alternatives. In this study TN was taken as 50 years, value of d=0.1 as a discount rate was utilized. The Net Present Values for each strategy were determined for considering 5 fatalities (NL=5) and 0 fatality (NL=0). The value of human life was estimated as $200.000 while employing analysis considering 5 fatalities. As a last step to find the best alternative, net present value was maximized varying one of the four variables constant among the variables of the time horizon (TN), the discount rate (d), the expected number of fatalities (NL) and the value of human life (V), and keeping the remaining variables constant. It was concluded in the paper that;

1. If the value of human life was not considered:

- Brace and full retrofit case became cost-effective for $TN \geq 4$, partial retrofit became cost-effective for $TN \geq 3$ (discounted benefits were greater than cost of the rehabilitation strategy)
- Partial retrofit strategy was the most feasible among the three alternative strategies.

2. If value of human life was considered:

- Brace and partial retrofit became cost-effective for $TN \geq 2$, full retrofit became cost-effective for $TN \geq 3$ (Net Present Value is greater than 0)
- Partial retrofit strategy was the most feasible among the three alternative strategies.

1.4 CONCLUDING REMARKS

Among the methods explained earlier, addition of shear walls, column jacketing, and strengthening of infill walls using CFRP are employed herein. For the cost-benefit analysis though, a different and simplified approach is used. The details of the modeling rules for implemented rehabilitation alternatives are explained in Chapter 2 and the details of cost-benefit analysis used in this study are explained in Chapter 3.

1.5 OBJECT AND SCOPE

The six major earthquakes that occurred in Turkey within the last decade caused substantial damage to buildings. As a result, many on-site rehabilitation programs were implemented. The Ministry of Public Works and Settlement provided funds required for this entire rehabilitation project. Assessments of damaged buildings were performed, severely damaged ones were demolished and moderately damaged ones were rehabilitated. Reinforced concrete shear walls were introduced for both rehabilitation and repair of reinforced concrete structures.

It was observed during the implementation that the cost of rehabilitation strategies was quite high and this led related participants to find new cost effective solutions. Recently there have been researches for easier and faster techniques like strengthening frame members with using CFRP. This technique does not require evacuation of the building during rehabilitation. Furthermore the experimental studies performed for reinforced concrete frames showed that this method increased the strength of the frames significantly.

Many studies were conducted by engineers and seismologists for the assessment of earthquake hazard in Marmara region. Strong shaking is expected from an expected earthquake in Marmara Sea. There is a large building stock in İstanbul consisting of multistory apartment buildings that are vulnerable to earthquake hazard. In view of this high risk in the area, buildings have been examined in an attempt to identify those that are highly vulnerable and rehabilitate them to reduce the loss. An indispensable task is to choose the most appropriate and beneficial method of rehabilitation. To achieve this goal, cost-benefit analysis is a highly desired economic tool. This way, responsible parties can decide on which rehabilitation strategy is feasible for the building stock.

The objective of this thesis is to carry out an analytical study to evaluate the performance of two reinforced concrete buildings before and after rehabilitation, and to perform their cost-benefit analysis. The buildings chosen have similar characteristics with the buildings that are vulnerable to earthquake hazard in Marmara region like poor concrete quality, poor lateral resistance, low ductility, insufficient detailing of reinforcement at the beam column connections. Two rehabilitation techniques, strengthening with concrete shear walls, and strengthening with CFRP are applied to the buildings. In both techniques, the column jacketing is also applied. Addition of concrete shear wall and jacketing is the simplest and most efficient method that has been widely used in Turkey. Strengthening with CFRP is a rather new method. These procedures are compared with regard to their contribution to lateral strength of the buildings and cost-benefit analysis is employed for determining the feasibility of these rehabilitation measures. In this respect twenty fragility curves were developed to estimate the damage corresponding to each rehabilitation alternatives in two principal directions. Then information gathered from these curves was used to calculate the Net Present Values.

The performances of the two buildings were checked according to the response limits proposed by ATC 40 [3] for both global building responses and component responses. In the analysis, Life Safety Performance Level was chosen as the performance objective. Both buildings were evaluated according to the design spectra of the 1998 Turkish Earthquake Code: Specifications for the Buildings to Be Constructed in Disaster Areas [27] and site specific spectra computed for a specific scenario earthquake.

Afterwards rehabilitation costs for each alternative were calculated. These calculations were essentially based on Unit Price List published by Ministry of Public Works and Settlement [17]. The unit prices for the CFRP technique were obtained from the private company, SİKA [23], which is one of the distributing firms of this material. The results are expected to provide an insight for the decision makers on various aspects of these two alternative techniques.

This thesis contains the following sections. Chapter 1 provides a general background on the cost benefit analysis and existing rehabilitation strategies. Chapter 2 deals with the overview of the modeling assumptions made for each rehabilitation strategy and the procedure employed for the analysis. In Chapter 3, work items for each rehabilitation strategy and methodology employed to find reduced damages and benefits are summarized. Evaluation of the analytical results in the form of capacity curves (pushover curves) and drift demands as well as the comparison of cost-benefit analyses are provided in Chapter 4. Finally, Chapter 5 introduces a summary and the conclusions of this thesis and some recommendations for future researches.

CHAPTER 2

STRUCTURAL ANALYSIS OF SELECTED BUILDINGS

2.1 INTRODUCTION

In this study two typical existing undamaged residential buildings were selected which are located in Marmara Region. Their finite element models were established using SAP2000 computer package [5]. Two rehabilitation alternatives, rehabilitation with inserting shear wall and applying Carbon Fiber Reinforced Polymer (CFRP) on the hollow clay tile, were introduced. Column jacketing was also applied to the columns that were found to have low axial load capacity in both rehabilitation alternatives.

2.2 GENERAL ASSUMPTIONS FOR STRUCTURAL MODELING

As mentioned before SAP2000 computer package [5] was used for three dimensional finite element models of the buildings. These models incorporate geometrical and structural details. Only column and beam components were modeled in the program. They were represented by massless line elements. Infill walls and slabs were not introduced in the models, the contribution of infill walls were ignored. The weights of slabs were transferred to beams as uniformly distributed loads and then to columns as point loads. Also self-weight of infill walls, columns, beams, shear walls and roof were considered in the weight calculations.

The rigid end offsets were assigned to signify the dimensions of beam-column joints. At each floor level diaphragm constraints were assigned to form a rigid diaphragm. Pushover analyses were carried out by assigning plastic hinges to all line elements representing beams, columns, shear walls and infill walls that were strengthened with CFRP. Contribution of infills were included only in the models of CFRP rehabilitated versions of the buildings. Flexural yielding was concentrated in plastic hinges that were located at the ends of beams, columns, and shear walls. The hinge properties for axial loads were concentrated at the middle of the compression and tension struts representing infill walls strengthened by CFRP and shear hinges were assigned at the mid point of shear walls.

Strain hardening of the reinforcement was neglected. While defining stiffness of reinforced concrete components, their flexural rigidity was reduced by the coefficients given in Table 2.1. The values in Table 2.1 are based on EcIg definitions which are given in ACI 318 [2].

Table 2.1 Component initial stiffness

Component	Flexural Rigidity
Beam, non prestressed	0.5 EcIg
Columns in compression	0.7 EcIg
Walls, uncracked	0.8 EcIg

Young's modulus of elasticity (Ec) for all components is calculated using Equation 2.1 given by TS-500, Requirements for the Design and Construction of Reinforced Concrete Structures [28].

$$\text{Ec}=3250\times(f_{ck})^{1/2}+14000 \quad \text{(MPa)} \qquad \textbf{\textit{(2.1)}}$$

Hinge properties for each member were calculated one by one. In this respect moment-curvature relationships of columns, shear walls, and beams were provided. The software RESPONSE-2000 [21] was used for computing moment-curvature relationships and interaction diagrams of the components. Once models were developed, user-defined hinges were assigned and models were analyzed under defined static pushover cases. Three pushover load cases, GRAV including dead load and live load, Push-x and Push-y including lateral forces in the two principal directions, were defined. No axial force in beams was assumed. The axial forces of the vertical components due to GRAV were used in determining the moment-curvature relationships. In order to convert moment-curvature relationships to the moment-rotation relationships, the procedure employed previously by

Saidii and Sozen (1979) [22] and Park and Paulay (1975) [19] was used. This procedure consists of some empirical equations given below.

$$\theta_{yield} = \frac{L \times \varphi_{yield}}{6} \qquad \textbf{\textit{(2.2)}}$$

where:

L: Length of member

θ_{yield} : Yield rotation

φ_{yield} : Yield curvature

$$\theta_p = (\varphi_{ult} - \varphi_{yield}) \times l_p \qquad \textbf{\textit{(2.3)}}$$

where:

l_p : Equivalent plastic hinge length

φ_{ult} : Ultimate curvature

θ_p : Plastic rotation

The program is able to modify ultimate moment and yield moment values by using interaction diagrams belonging to that component. For this reason five equally spaced axial force-moment capacity curves were input into SAP2000 [5]. Axial force-moment capacity curves corresponding to the two major axes, x-axis and y-axis, of each column and shear wall sections were obtained by solving defined sections in RESPONSE-2000 [21]. Three dimensional interaction surfaces for columns and shear walls were obtained using the following formula proposed by Parme et al. (1966) [20]:

$$\left[\frac{M_{ux}}{M_{uxo}}\right]^{\log 0.5/\log\beta} + \left[\frac{M_{uy}}{M_{uyo}}\right]^{\log 0.5/\log\beta} = 1 \qquad \textbf{\textit{(2.4)}}$$

where:

M_{uxo} : Uniaxial flexural strength about the x-axis

M_{uyo} : Uniaxial flexural strength about the y-axis

M_{ux}: Component of biaxial flexural strength on the x-axis at the required inclination

M_{uy}: Component of biaxial flexural strength on the y-axis at the required inclination

β : Parameter dictating the shape of the interaction surface

2.3 CHARACTERISTICS OF BUILDINGS AND SITES

2.3.1 BUILDING-1

The first case study building analyzed is located in Bursa. It is a residential building used as an employee housing and belongs to a state agency. It was designed and constructed in 1980's.

2.3.1.1 Building and Site Characteristics

Comprehensive field assessment studies were carried out by METU-team to obtain as-built properties of the building and the site characteristics. The results of tests on core samples revealed that the average in-site strength *(f_{ck})* was 9 MPa with yield strength of 220 MPa for the longitudinal reinforcement (f_{yk}).

Information about characteristics of site where Building-1 is located is listed in Table 2.2. This information was used in developing the code-based spectrum.

Table 2.2 Site characteristics according to 1998 code [27] (Building-1)

Seismic Zone	1.0
Effective Ground Acceleration Coefficient (Ao)	0.4
Building Importance Factor (I)	1.0
Soil Type	Z4
Spectrum Characteristics Period (TA) (sec)	0.2
Spectrum Characteristics Period (TB) (sec)	0.9
Seismic Load Reduction Factor (R)	1.0
Modal Damping Ratio	5 %

The site-specific spectrum was developed based on a magnitude 7.0 earthquake and using the attenuation relationship proposed by Gülkan and Kalkan [14]. The spectra used in the evaluation of this building are given in Figure 2.1.

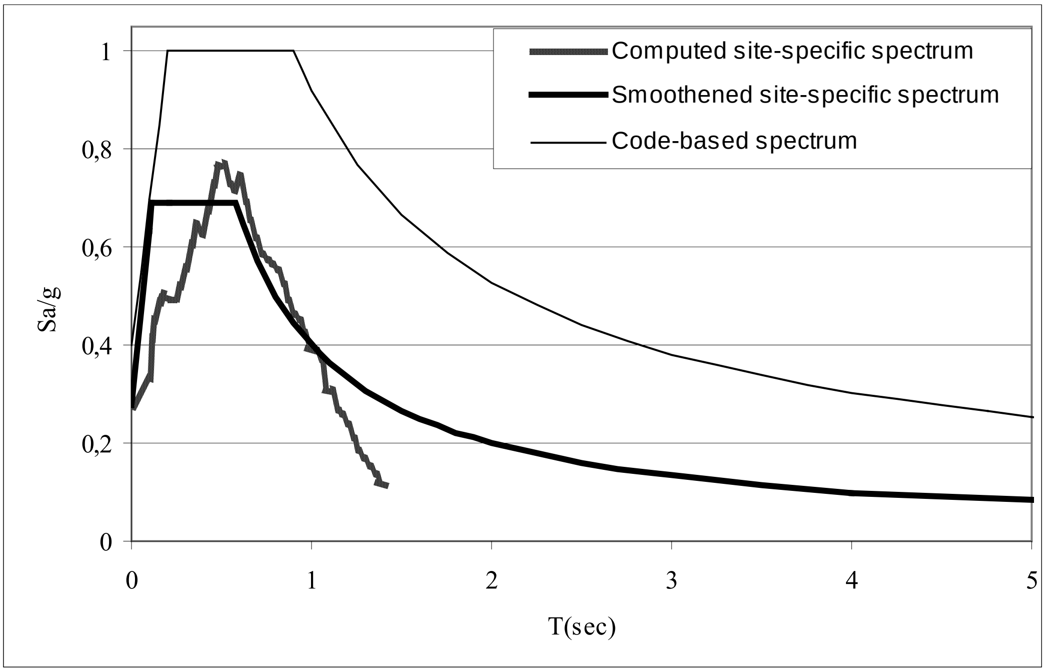

Figure 2.1 Code-based and Site-specific spectra – Building-1

2.3.1.2 Structural Features

Building-1 is a five-story reinforced concrete building with a plan area of approximately 1,310 m^2. It was constructed according to the 1975 Turkish Seismic Code, Specifications for the Buildings to Be Constructed in Disaster Areas. The first story height is 3 meters and other stories are 2.8 meters high. It is rectangular in shape, with seven bays in East-West (longitudinal) and three bays in North-South (transverse) directions. Plan views of the original and rehabilitated versions of Building-1 are shown in Figures 2.2-2.6. Both the dimensions and the amount of longitudinal reinforcement of six columns decrease beginning from the second floor. On the other hand, the dimensions and the reinforcement of beams do not change with height. All beams have a depth of 60 cm and a width of 20 cm. The reinforced concrete slab has a constant depth of 12 cm throughout the building.

As stated before, Building-1 was strengthened using two rehabilitation alternatives, rehabilitation with inserting shear wall and applying Carbon Fiber Reinforced Polymer (CFRP) on the hollow clay tile infill walls. Four bays along the weak axis of the building were rehabilitated with shear walls. They were placed symmetrically with respect to the longitudinal axis on the first and eight axes. A total of five bays were rehabilitated with shear walls along the longitudinal axis. Dimensions and reinforcement of the shear walls were kept constant in the first and second floors. They were decreased in the upper stories. Additionally four columns were jacketed in the first and second floors with constant dimensions and reinforcement. The CFRP was applied to the infill walls where shear walls were placed in preceding rehabilitation alternative. In both rehabilitation schemes the same columns were jacketed.

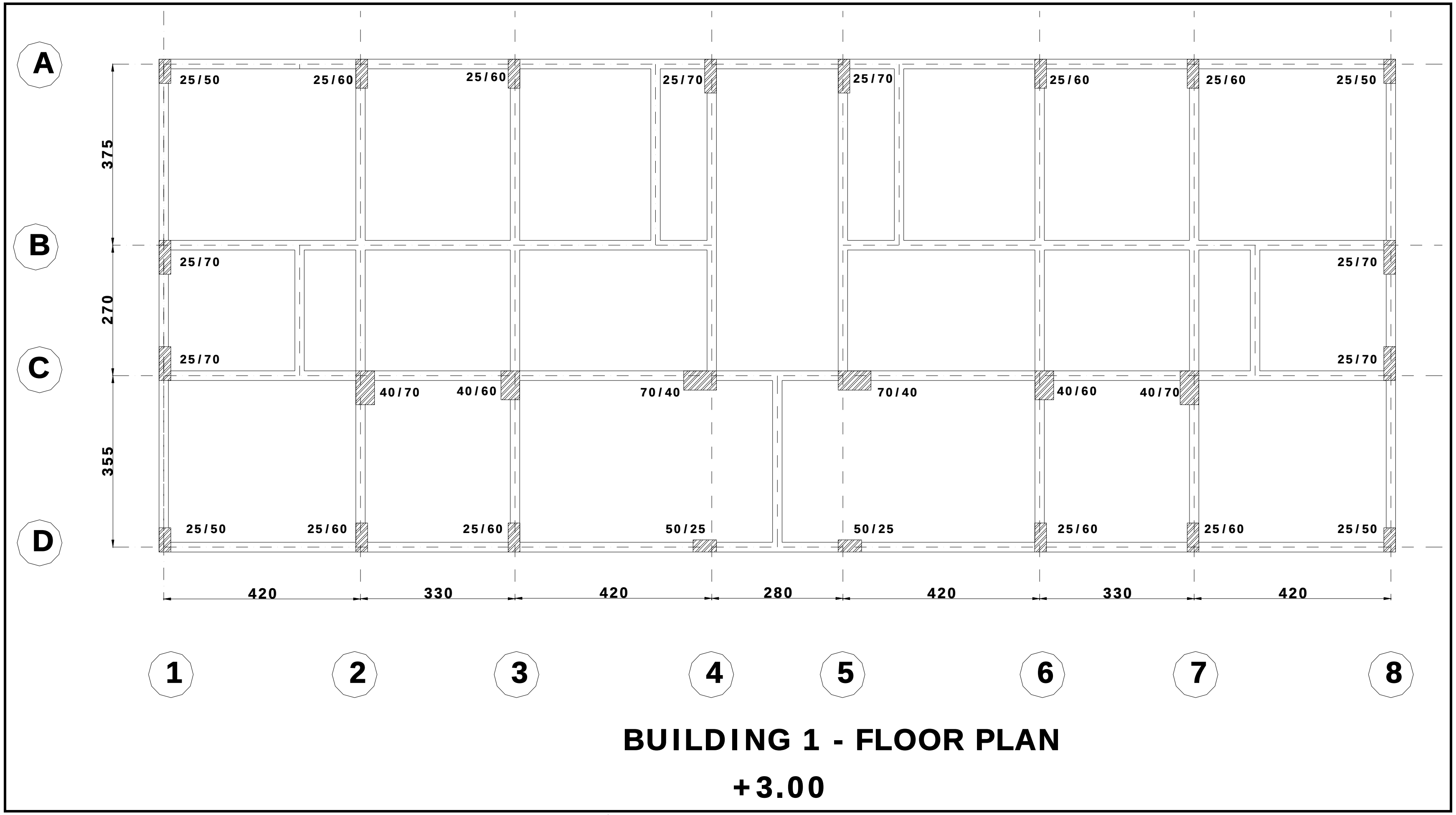

Figure 2.2 1st Floor plan of the Original Building-1

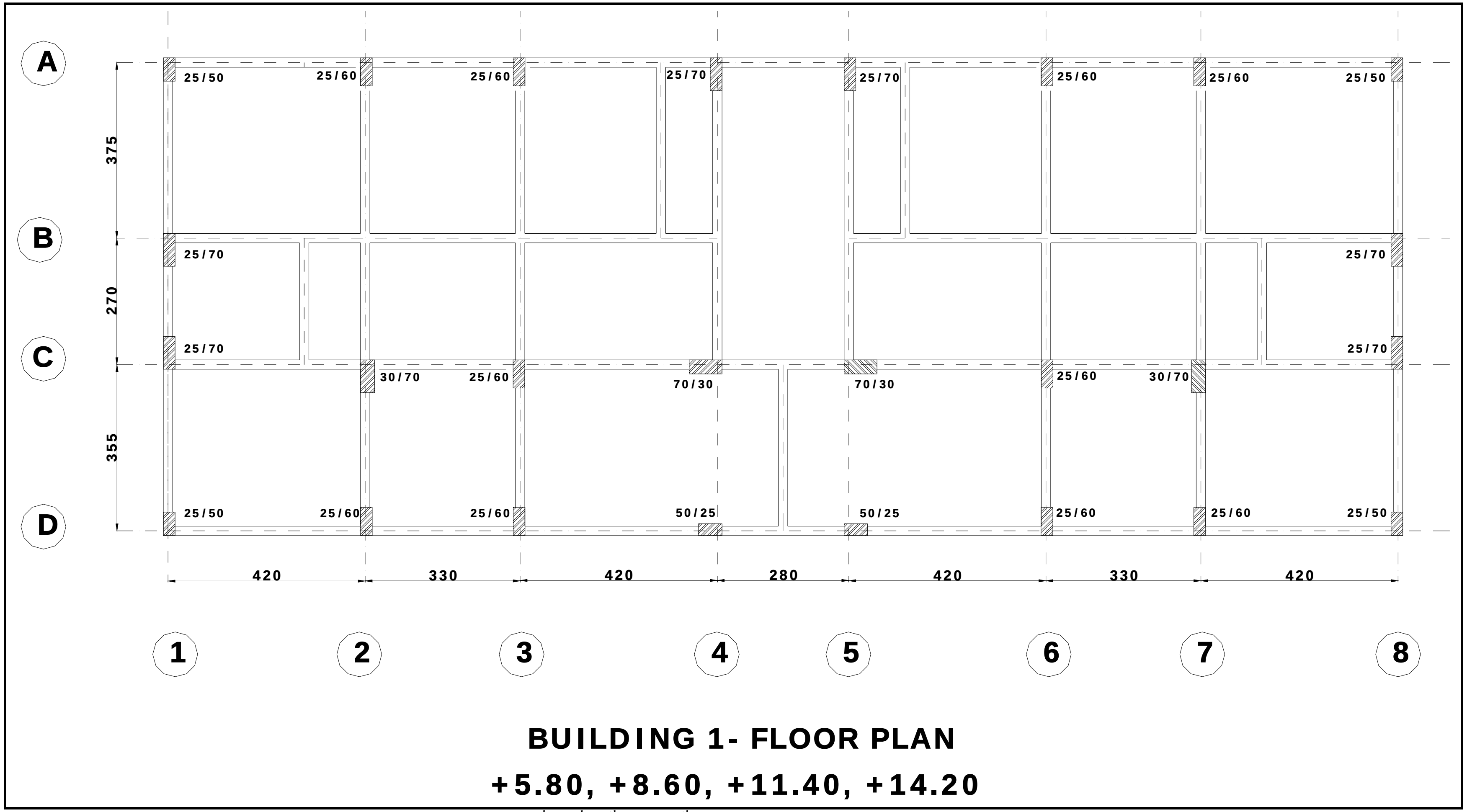

Figure 2.3 2nd, 3rd, 4th and 5th Floor plan of the Original Building-1

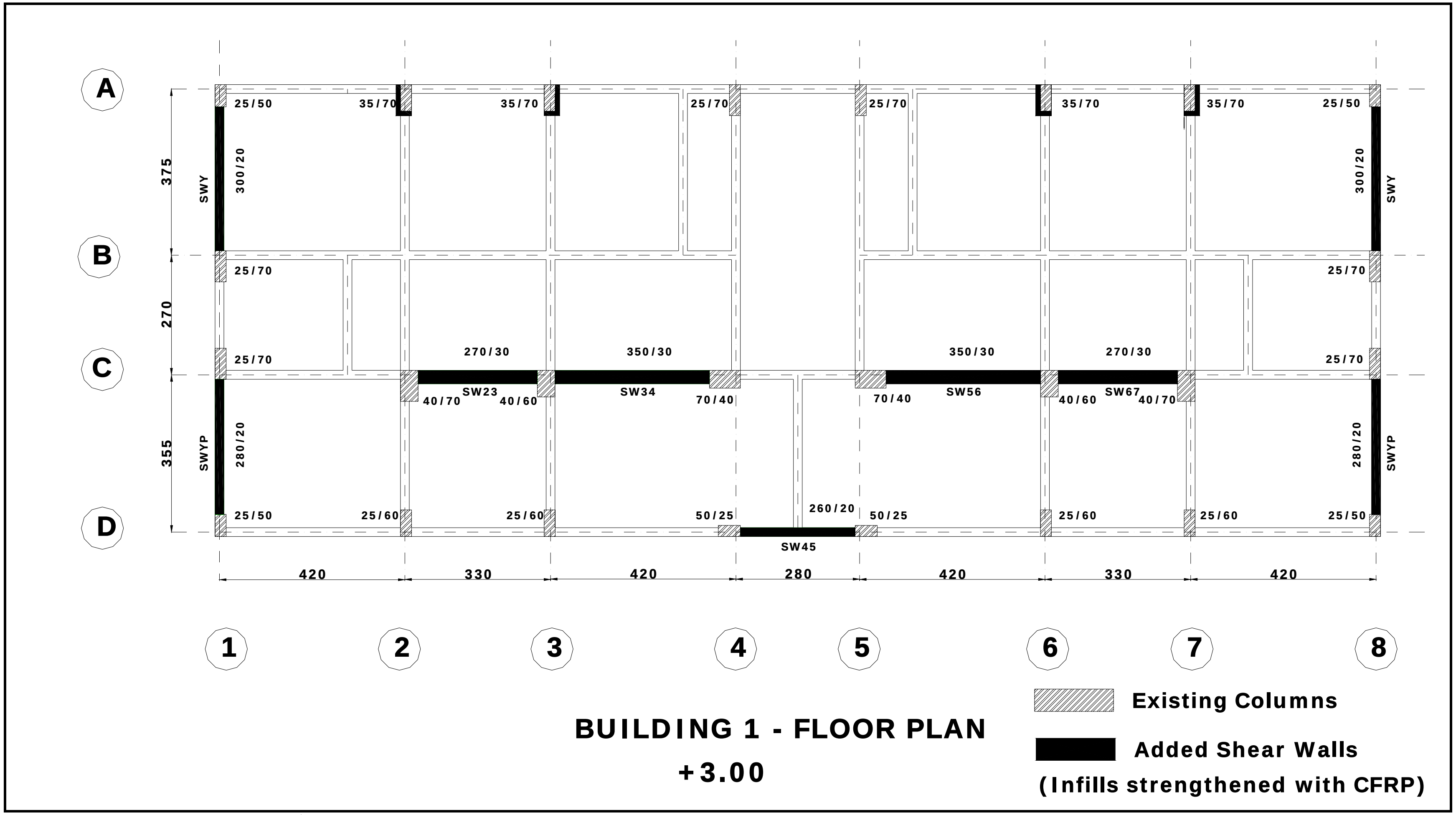

Figure 2.4 1st Floor plan of the shear wall (CFRP on infill walls) and jacketing rehabilitated version of Building-1

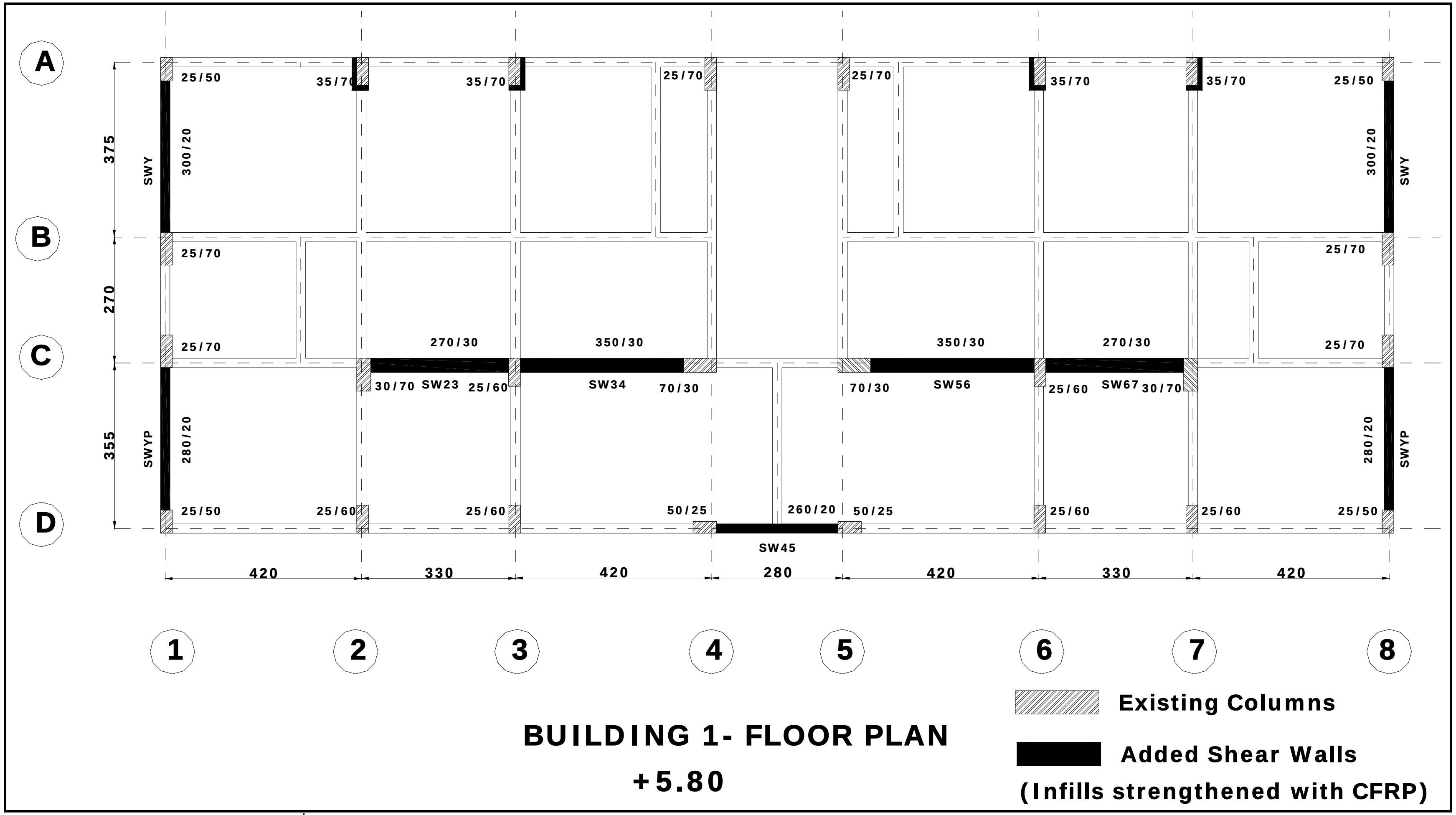

Figure 2.5 2nd Floor plan of the shear wall (CFRP on infill walls) and jacketing rehabilitated version of Building-1

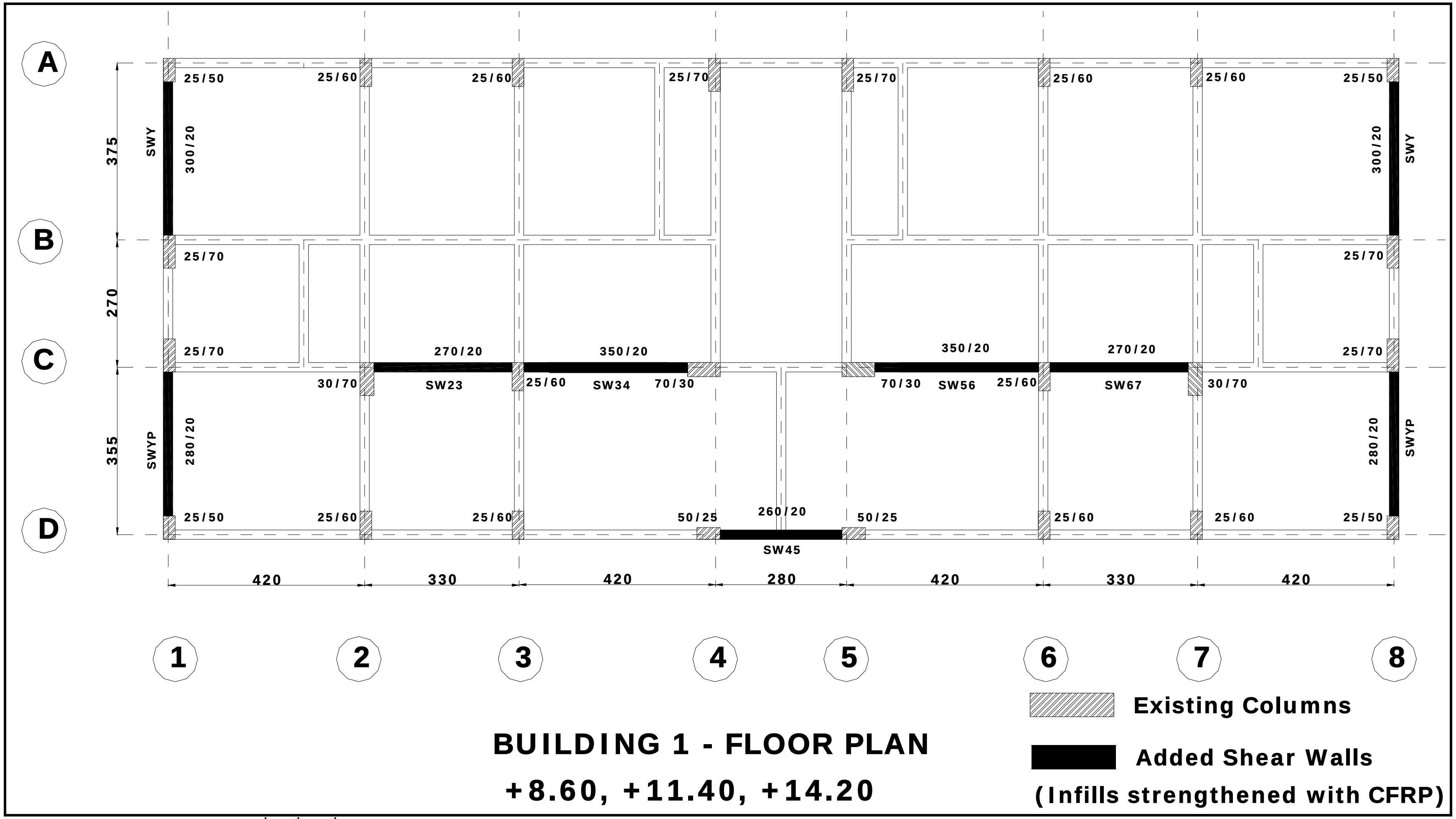

Figure 2.6 3rd, 4th, 5th Floor plan of the shear wall (CFRP on infill walls) and jacketing rehabilitated version of Building-1

2.3.2 BUILDING-2

The second case study building analyzed is also a residential building owned by the government and located in Levent, İstanbul. It was designed and constructed in 1980's like the first building.

2.3.2.1 Building and site characteristics

The characteristic yield strength of longitudinal reinforcement (f_{yk}) was 220 MPa. The test results indicated that the average in-site compressive strength (f_{ck}) of concrete was 10 MPa.

Information about characteristic of site where Building-2 is located is listed in Table 2.3.

Table 2.3 Site characteristics according to 1998 code [27] (Building-2)

Seismic Zone	1.0
Effective Ground Acceleration Coefficient (Ao)	0.4
Building Importance Factor (I)	1.0
Soil Type	Z2
Spectrum Characteristics Period (TA) (sec)	0.15
Spectrum Characteristics Period (TB) (sec)	0.4
Seismic Load Reduction Factor (R)	1.0
Modal Damping Ratio	5 %

The code-based spectrum and the site-specific spectrum developed based on a magnitude 7.2 earthquake and using the attenuation relationship proposed by Gülkan and Kalkan [14] are given in Figure 2.7.

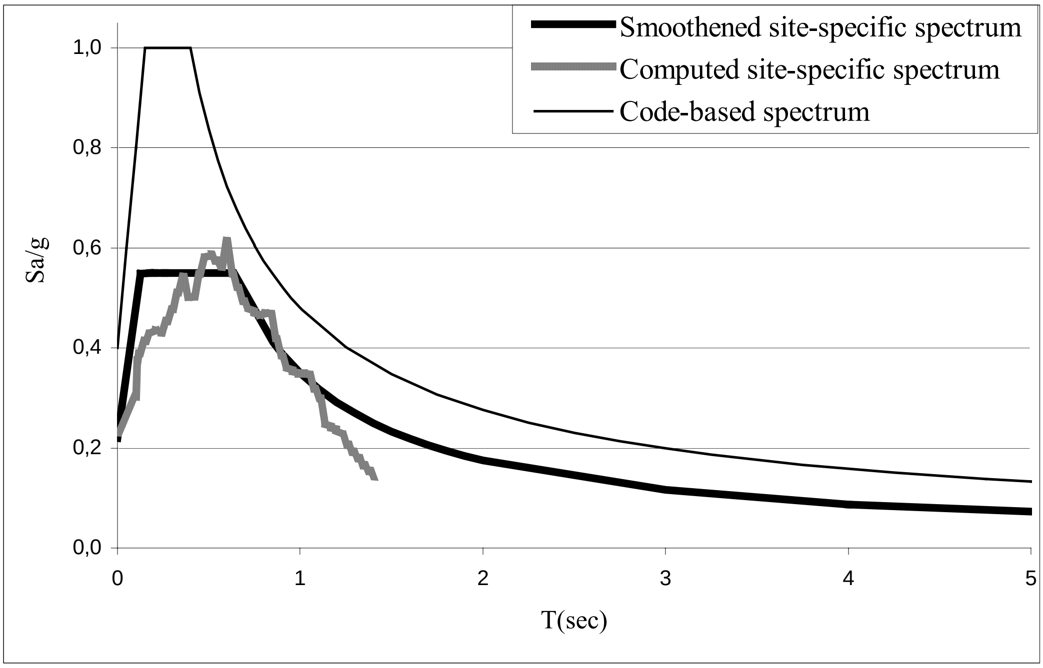

Figure 2.7 Code-based and Site-specific spectra – Building-2

2.3.2.2 Structural Features

Building-2 has similar structural characteristics as Building-1. It is a five-story reinforced concrete building with a plan area of approximately 1,330 m^2. Like Building-1 it was constructed according to the 1975 Turkish Seismic Code. All story heights of the building are 3 meters and total height is 15 meters. Plan views of the original and rehabilitated versions of Building-2 are shown in Figures 2.8-2.12.

Dimensions of 14 columns decrease at the fifth floor. Reinforcement of four of these columns decreases in the fifth floor while of the remaining columns does not vary. Dimensions of beams and their longitudinal reinforcement do not change with height. All beams have a depth of 60 cm and a width of 20 cm. The reinforced concrete slab has a constant depth of 12 cm throughout the building.

Similar to Building-1, Building-2 was strengthened using two rehabilitation alternatives, rehabilitation with inserting shear wall and applying Carbon Fiber Reinforced Polymer (CFRP) on the hollow clay tile infill walls. Six bays along the longitudinal axis of the building were rehabilitated with shear walls. They were placed symmetrically with respect to the short axis. Both dimensions and amount of the longitudinal reinforcement decrease from the third floor on. Additionally seven bays were rehabilitated with shear walls along the weak axis. Six of them were placed symmetrically with respect to the longitudinal axis of the building. Although dimensions of them do not chance with height, reinforcement of the shear walls decrease beginning from the third floor. Since columns were found to have enough axial load capacity, column jacketing was not applied to the columns of Building-2. CFRP was applied on the infill walls where shear walls were placed in the preceding rehabilitation alternative.

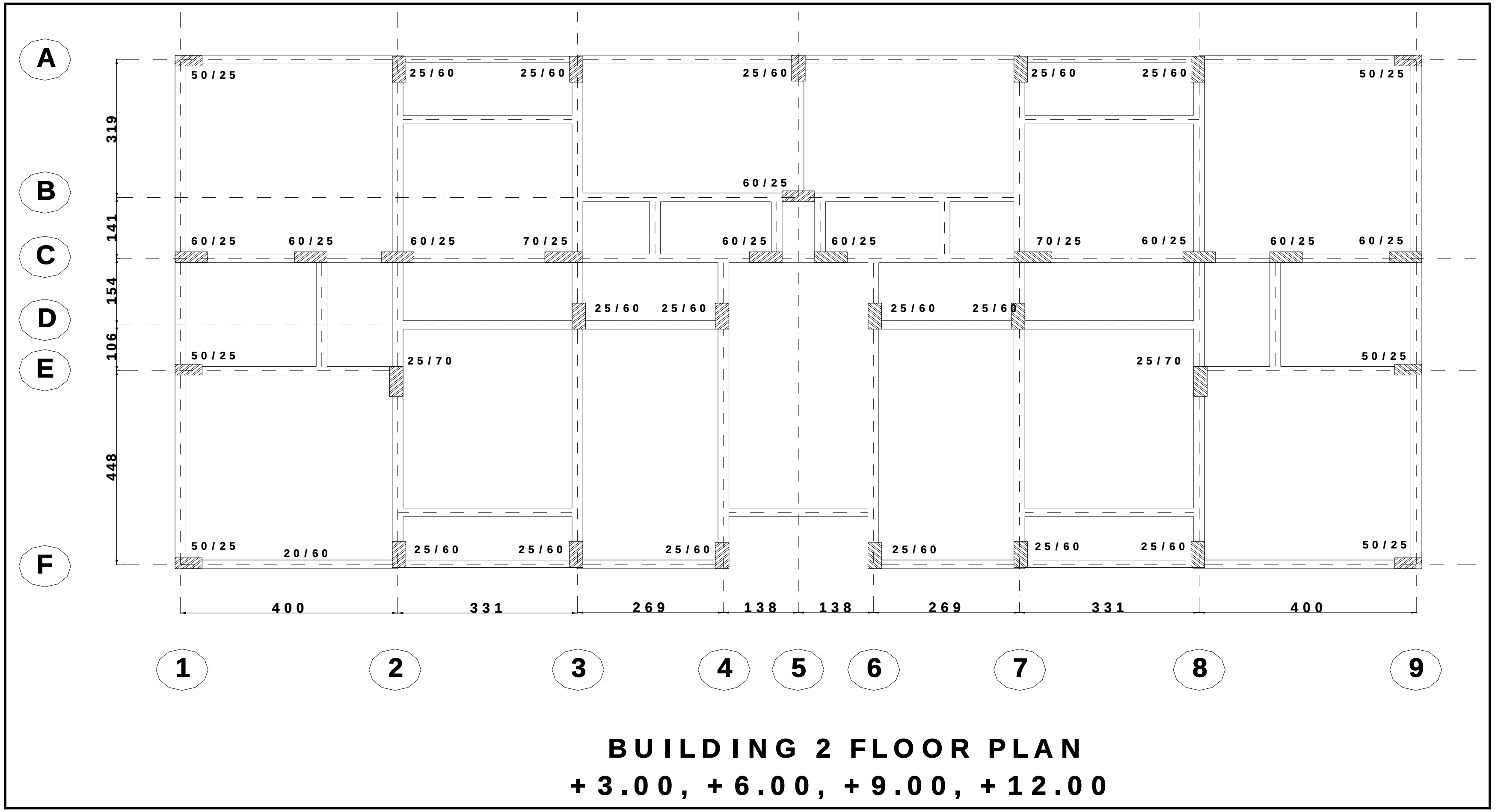

Figure 2.8 1st, 2nd, 3rd, 4th Floor plan of the original Building-2

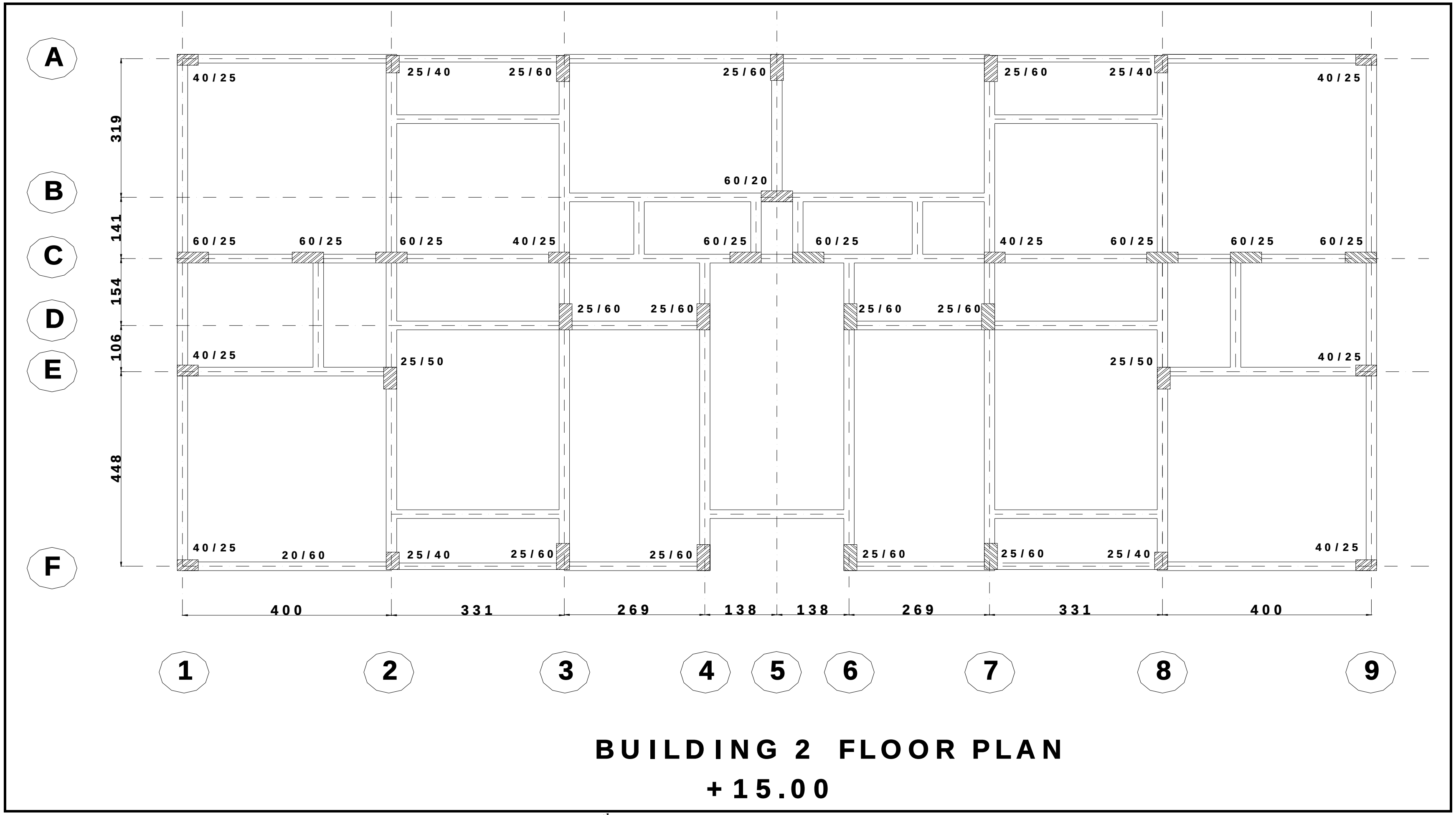

Figure 2.9 5th Floor plan of the original Building-2

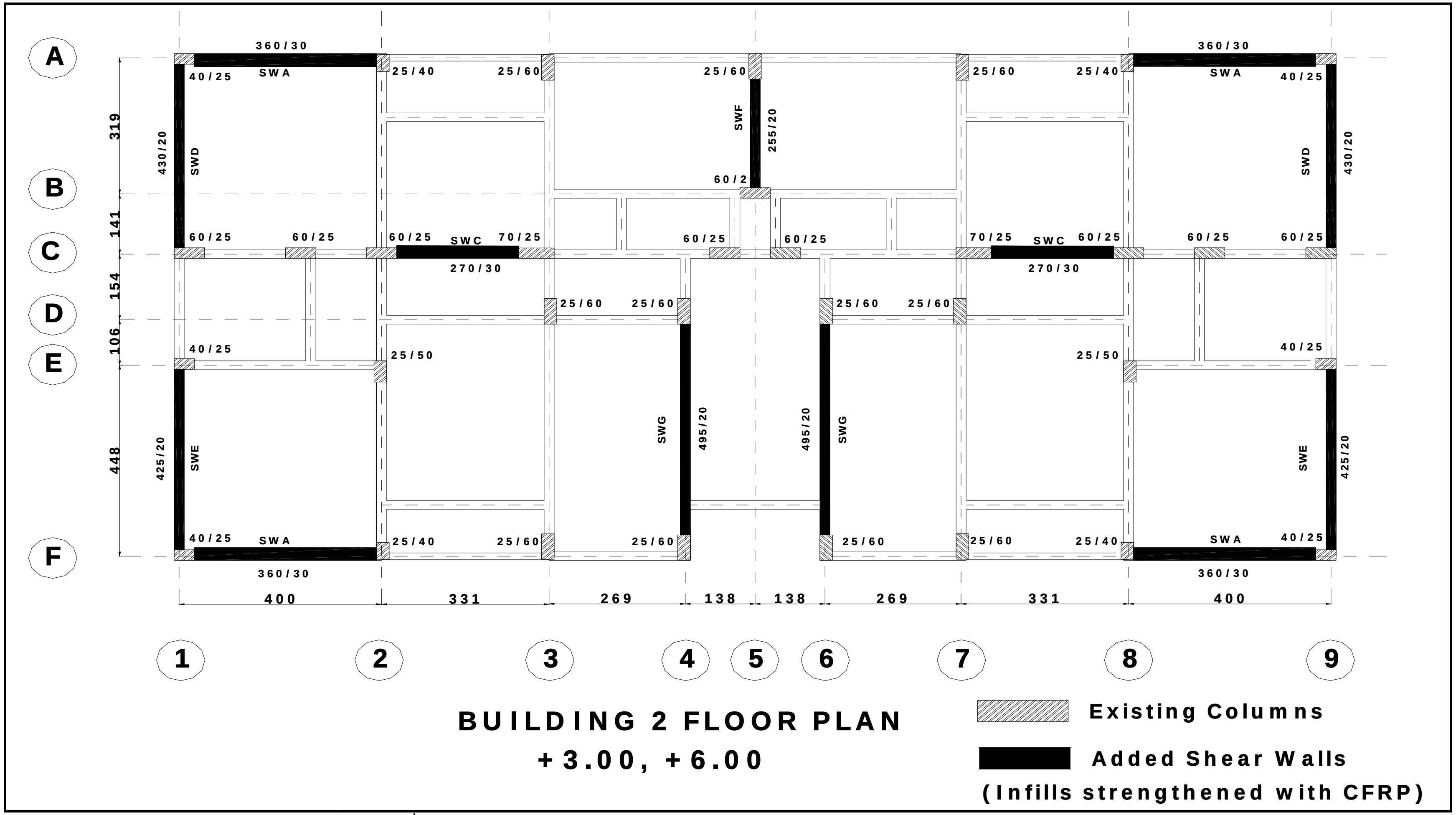

Figure 2.10 1st and 2nd Floor plan of the shear wall rehabilitated (CFRP on infill walls) version of Building-2

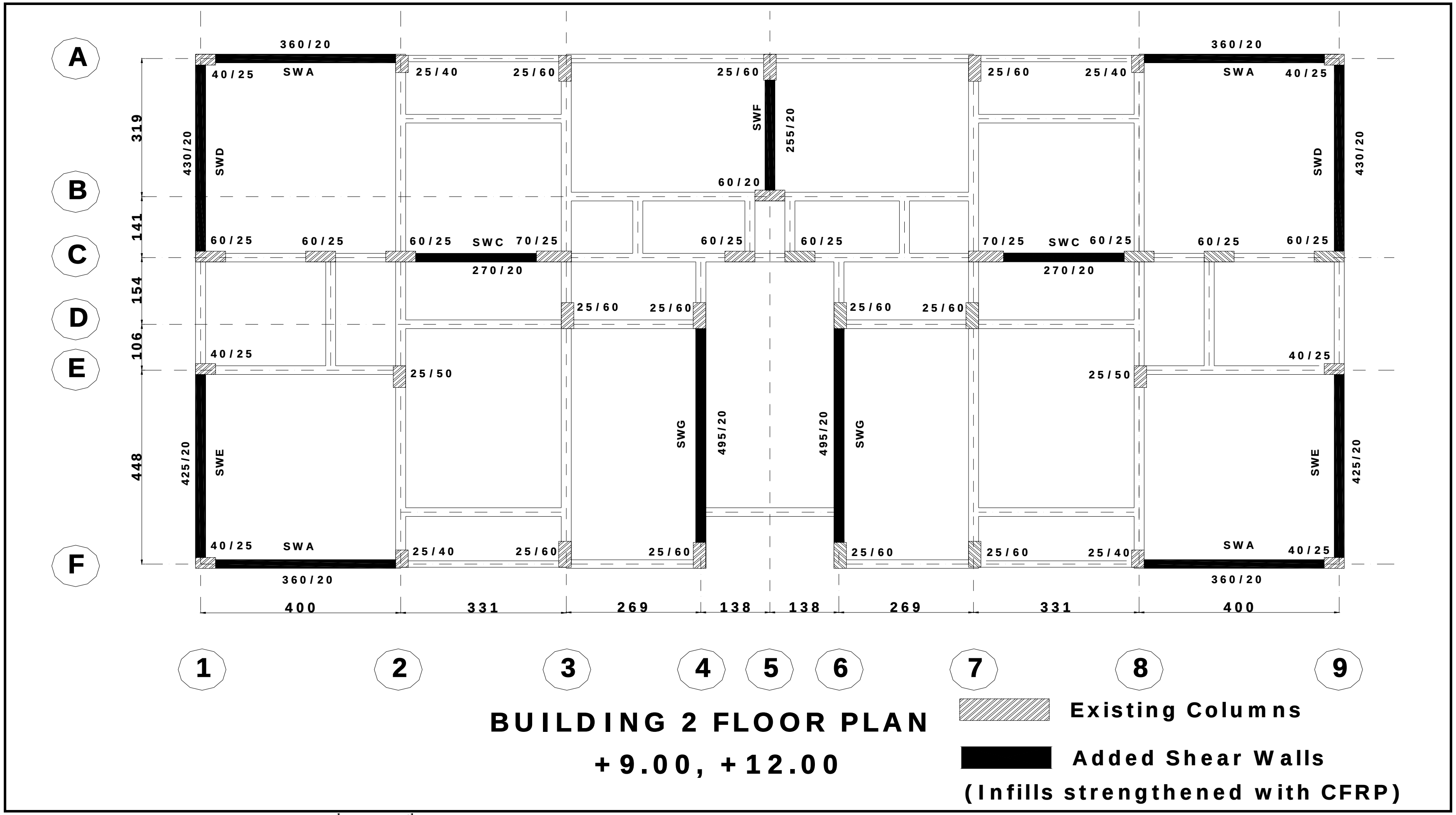

Figure 2.11 3rd and 4rth Floor plan of the shear wall rehabilitated (CFRP on infill walls) version of Building-2

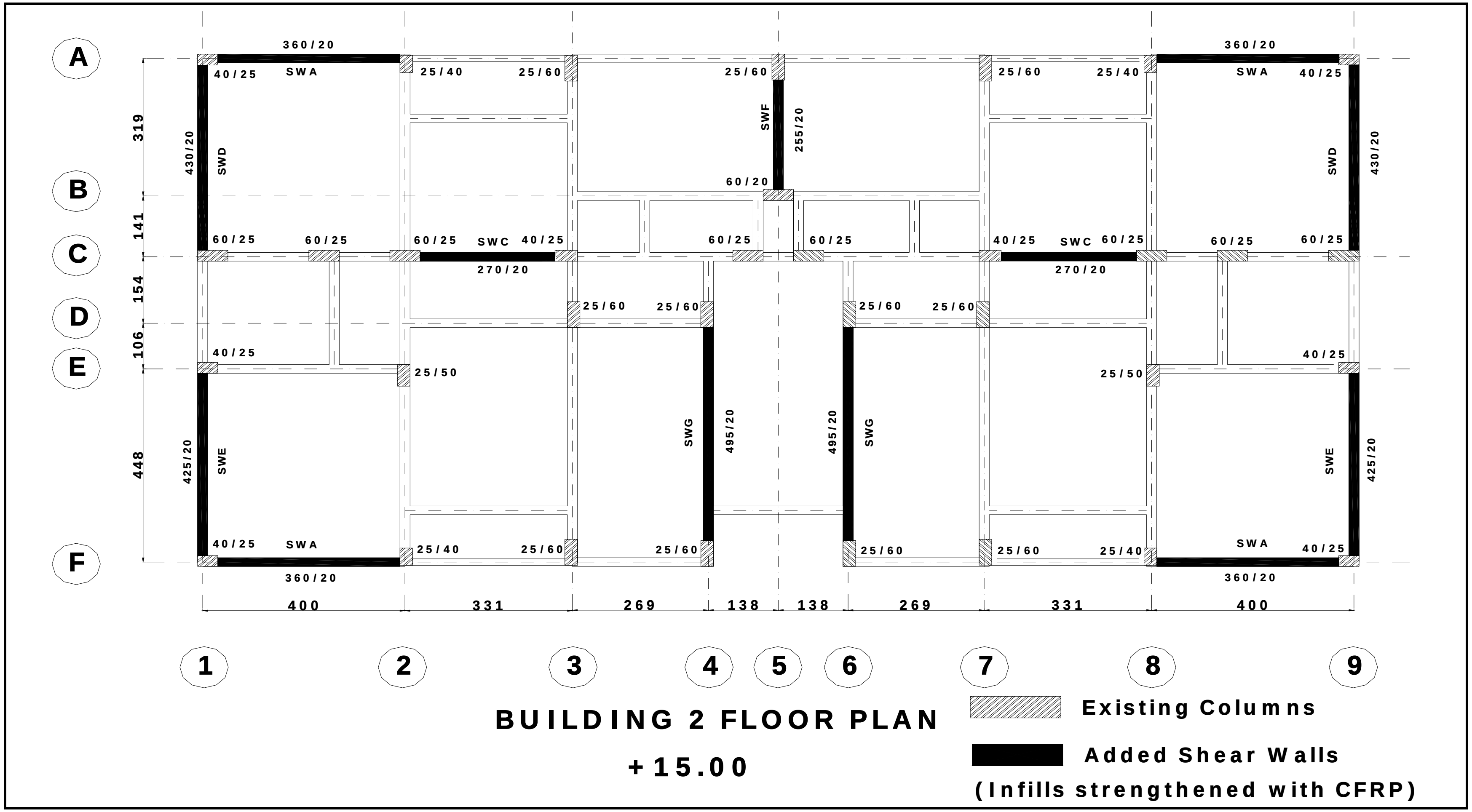

Figure 2.12 5th Floor plan of the shear wall rehabilitated (CFRP on infill walls) version of Building-2

2.4 STRUCTURAL MODELING OF REHABILITATION TECHNIQUES

For both of the buildings, the original (existing buildings without any rehabilitation measure) and rehabilitated structures were considered in the analyses. The details of component modeling and analyses are given next.

2.4.1 Shear wall

The shear walls were modeled as line elements at the center of their cross sections. They were connected to columns at their boundaries with rigid beams.

It is considered that some deficiencies might occur during the construction of the added shear walls due to insufficient anchorage to the surrounding frame members and inconvenience of pouring concrete especially near the boundaries. For this reason, the moment capacities of added shear walls were reduced by 25 % in order to account for these affects. In order to see the effect of this reduction, the rehabilitated building was analyzed with and without implementing this reduction.

The shear capacity of the reinforced concrete walls was calculated using Equation 2.5 as specified in 1998 Turkish Earthquake Code [27]. This formulation was used to define the yield shear force capacity of the shear hinge assigned to the shear walls.

$$V_r = A_{ch}(0.65 f_{ctd} + \rho_{sh} f_{yd}) \tag{2.5}$$

where:

A_{ch} : Gross section area of a solid wall, wall segment of a coupled wall, a floor or a floor segment of a perforated floor

f_{ctd} : Design tensile strength of concrete

ρ_{sh} : Ratio of horizontal web reinforcement of wall to the gross area of wall web

f_{yd} : Design yield strength of longitudinal reinforcement

Although shear walls were used both for the rehabilitation of Building-1 and Building-2, there is significant difference between the amount and densities of the walls added. The wall densities in the two orthogonal directions are compared in Figure 2.13 and Figure 2.14. In these figures, Rx and Ry indicate percent of the ratio of shear wall area to the plan area in the x-and y-directions, respectively. In general, the amount of walls in Building-2 is about twice as much as that of the Building-1. The respective wall densities in y-direction for Building-1 and Building-2 are 1.30 and 2.20 percent in all floors. In x-direction, on the other hand, the wall density reduces in upper floors.

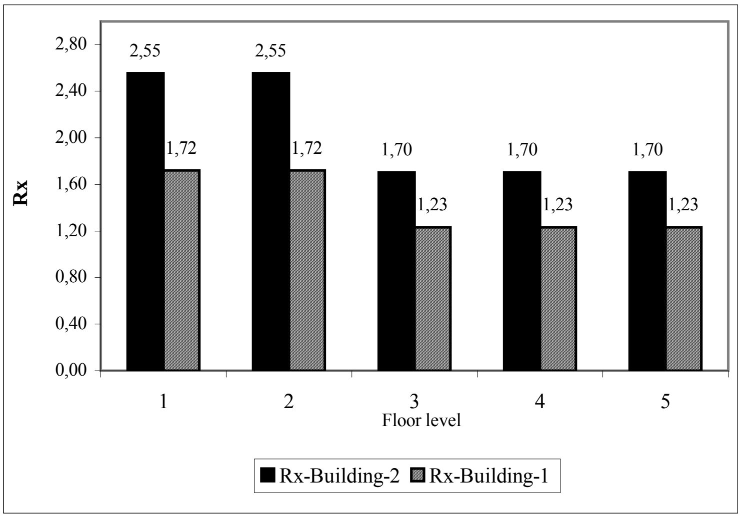

Figure 2.13 Shear wall density of buildings in the x-direction

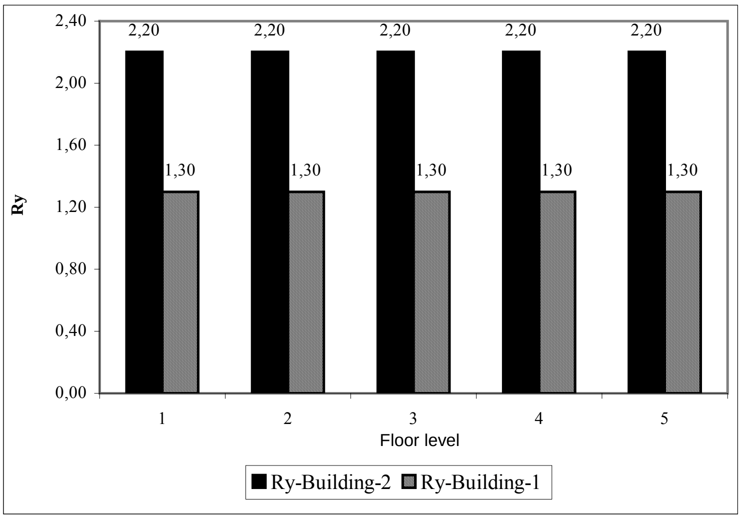

Figure 2.14 Shear wall density of buildings in the y-direction

2.4.2 Carbon Fiber Reinforced Polymer (CFRP)

The CFRP applied on hollow clay tile infills were modeled as diagonal struts based on the recommendations of studies that were conducted at Middle East Technical University [6, 7]. The contribution of masonry infill walls was incorporated through the compression strut and the composite behavior of CFRP on infill walls were reflected by the tension strut.

In this study two different attempts of modeling (will be called as models) proposed for CFRP application based on the experiments carried out at METU were considered. Erdem [6] studied the behavior of reinforced concrete frames strengthened with CFRP. He developed a bilinear elastic material model for the composite material (CFRP + plaster + clay tile). Another model was proposed as a design criterion for strengthening with CFRP by Özcebe, Ersoy, Tankut, Erduran, Keskin and Mertol in the TUBITAK Report 2003/1 [7]. Since the proposed models by these studies were different, two separate models for the stiffness and strength properties of the masonry infill walls and the composite material were used. For the sake of clarity and comparison, the individual test frames of each study were re-analyzed using both models with SAP2000 [5]. The comparison of the pushover curves obtained from these analyses revealed that there is significant difference between these models. The graphs illustrating the comparison of these two models and pushover curves obtained with SAP2000 [5] are given in Figures A1-A4 (Appendix A).

While defining hinge properties of infill walls strengthened with CFRP, a single force deformation relationship was employed. The force-deformation relation in compression was computed using infill wall properties, whereas this behavior in tension was obtained using the properties of the composite material formed with CFRP and the masonry infill. A sample force-deformation relation for a composite strut of CFRP and masonry infill wall is illustrated in Figure 2.15 where Δ_y and Δ_u indicate the yield and ultimate points, respectively.

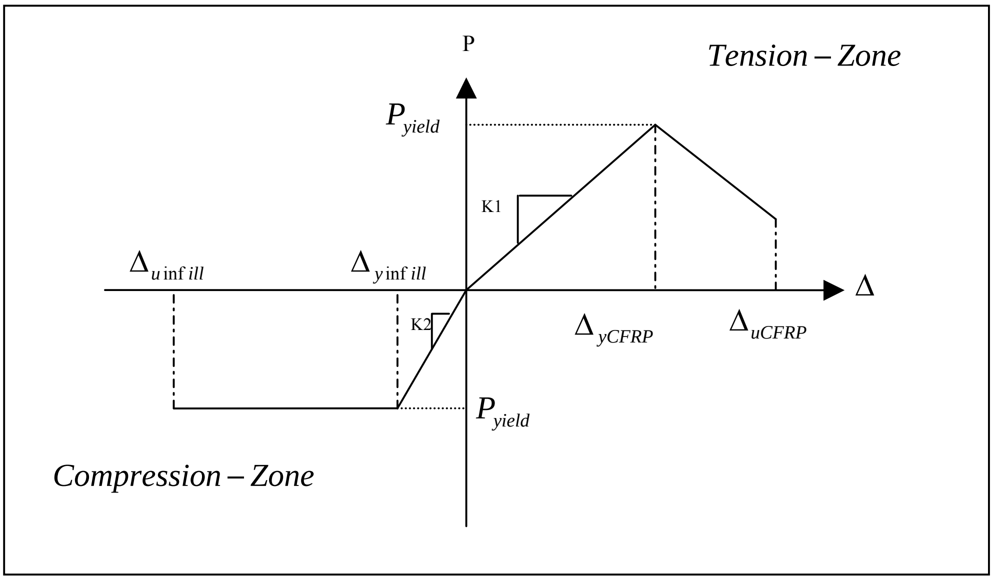

Figure 2.15 Hinge property defined for infill wall strengthened with CFRP

Although Erdem [6] used the graphs of λh versus equivalent strut width for various length/height proportions which were recommended by Smith (1969) [24], in this study equivalent strut widths of the compression struts were determined according to the formulation recommended by FEMA [11] (given in Appendix A) while implementing the models proposed by Erdem [6]. This empirical formulation was also used for determining the equivalent strut width while implementing the other proposed model (TUBITAK 2003/1 Report [7]). The calculated compression strut widths were modified based on the comparison of the results given in Appendix A.

2.4.2.1 Erdem's study

Erdem [6] tested two-story, three bay, 1/3 scale frames which were strengthened by CFRP applied on the hollow clay tile and by the reinforced concrete infill wall. An equivalent strut model to represent the CFRP strengthened wall is proposed based on the comparison of analytical and experimental pushover curves. The comparisons of these results are shown in Figure 2.16.

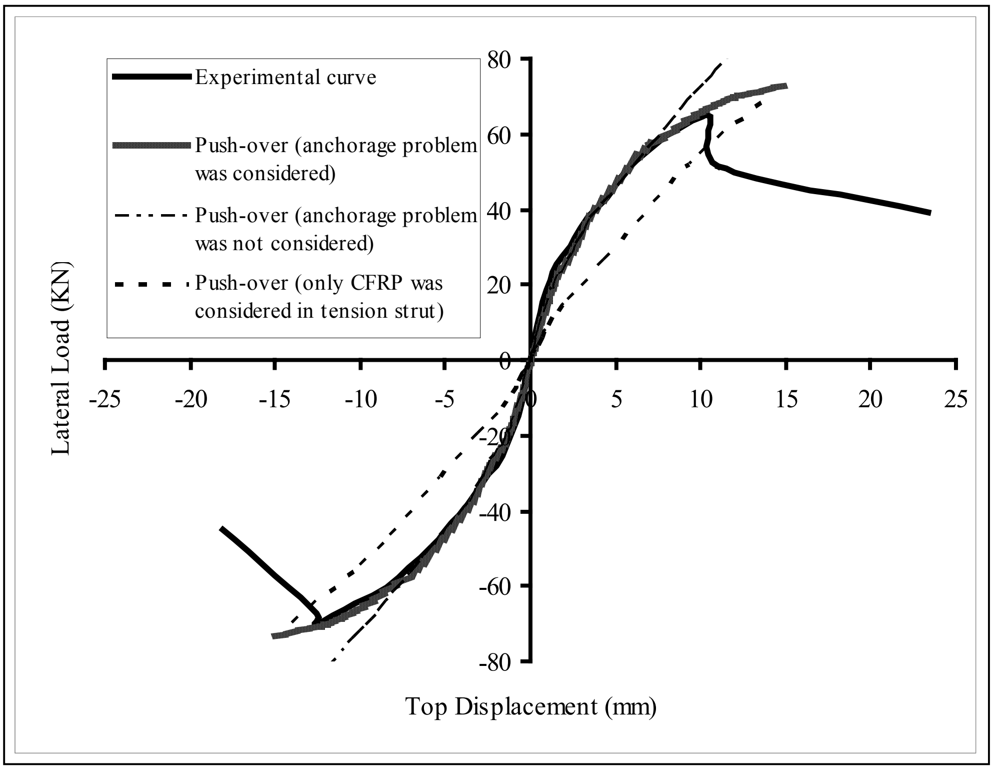

Figure 2.16 Response envelopes with analytical predictions for strengthening with CFRP, 2003 [6]

The CFRP was modeled as a tension strut and infill wall was represented by a compression strut in the numerical model. Equivalent strut widths for compression struts were determined according to the Smith method [24]. Properties of compression and tension struts used in his study are given in Table 2.4.

Table 2.4 Structural modeling data for diagonal struts

Width of compression strut in the first story	638 mm
Width of compression strut in the second story	480 mm
Width of tension strut in the first story	300 mm
Width of tension strut in the second story	200 mm
Thickness of plastered clay infill	90 mm
Thickness of composite material	92 mm
Elasticity of Modulus for compression strut	700 MPa
Elasticity of Modulus for tension strut	64,000 MPa

The material models defined for the tension and compression struts were bilinear (Figures 2.17 and 2.18).

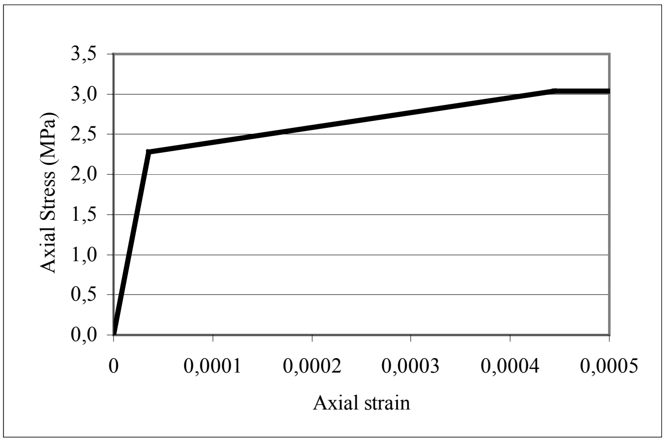

Figure 2.17 Material model for tension strut, 2003 [6]

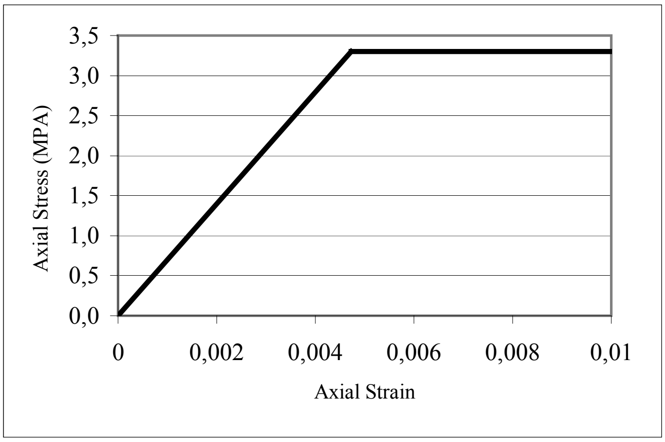

Figure 2.18 Material model for compression strut, 2003 [6]

2.4.2.2 Models proposed in TUBITAK Report 2003/1

Özcebe, Ersoy, Tankut, Erduran, Keskin and Mertol [7] studied the behavior of undamaged hollow brick infilled reinforced concrete frames strengthened with CFRP. In their experimental study they constructed seven two-story, one bay, 1/3 scale reinforced concrete frames that have representative characteristics of common deficiencies observed in buildings constructed in Turkey. They tested one reference specimen which was not strengthened with CFRP and six specimens strengthened with CFRP using different techniques. All specimens were tested under reversed cyclic load. At the end of the research they developed an analytical model for only one of the specimens. This specimen was the most efficient one considering the economy and performance among the other six specimens. The derived constitutive models for compression and tension struts are given in Figures 2.19 and 2.20.

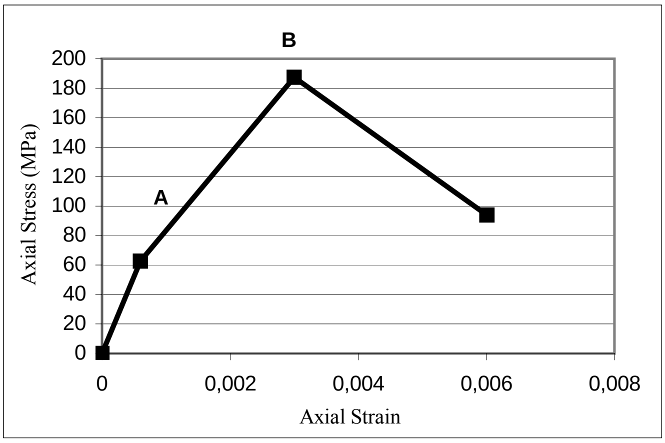

Figure 2.19 Strut model for the composite material, 2003 [7]

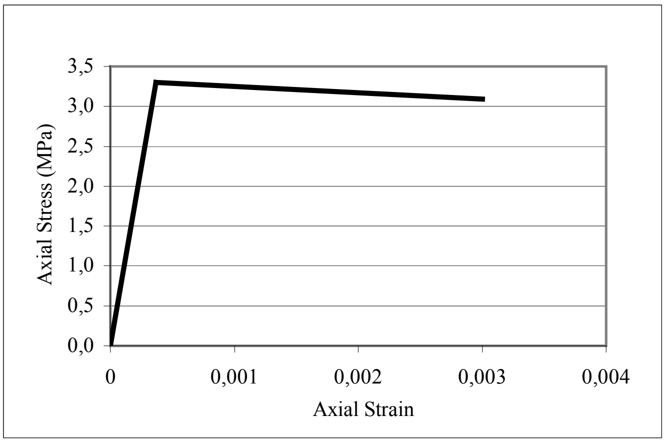

Figure 2.20 Strut model of the infill, 2003 [7]

Parameters of the initial stiffness, the post-yield stiffness, the post-peak stiffness, the first yield load, the first fracture load and equivalent strut width of the tension strut were adjusted until a sufficient agreement was observed between the experimentally observed and the analytically predicted responses. Therefore a series of pushover analyses were performed to predict the behavior of selected specimen (SP-5) and results of pushover analysis were compared with the test results (Figure 2.21).

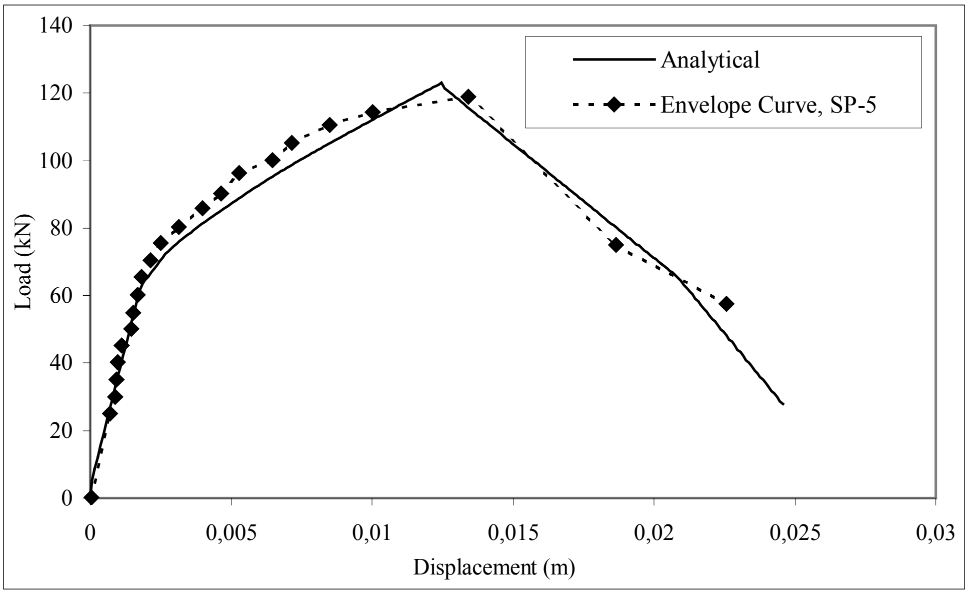

Figure 2.21 Comparison of experimental and analytical results of SP-5, 2003 [7]

2.4.3 COLUMN JACKETING

Both Building-1 and Building-2 were analyzed under the GRAV load combination that consists of dead loads and live loads. The axial loads applied on the columns were checked against their axial force capacities in both of the buildings. As a consequence of this checking, jacketing was applied only in Building-1 to the columns that were forced to carry axial loads greater than their axial load capacities.

A program was prepared using MATLAB for calculating interaction diagrams of the columns that were jacketed. The program considers different characteristic compressive strengths of both unjacketed column and confining concrete. The cross sectional view of the jacketed columns of Building-1 and the output of the program are given in Appendix A.

Suleiman, Ersoy and Tankut [26] conducted an experimental study to determine the differences of behavior of jacketed columns for strengthening and monolithic specimens that had identical cross-section and steel arrangement as the jacketed ones. These specimens were tested under reversed cyclic loading. They concluded that:

- When normalized moment curvature relationships of strengthened columns by jacketing and monolithically cast columns were compared; it was observed that the maximum moment carried by the strengthened specimen was almost the same as the one carried by the monolithic specimen.
- The stiffness of jacketed column was almost identical to that of the companion monolithic specimen up to the peak load. The stiffness degradations were also similar.
- According to the comparisons of experimental results with interaction diagrams, strengthened specimen behaved as good as the reference monolithic specimen which was also subjected to reversed cycling load.
- Strengthened and monolithic specimens had almost the same rigidity and the same capacity.
- Energy dissipation capacities of strengthened and monolithic specimen were different. Always total energy dissipated by the strengthened specimen was less than that of the companion monolithic specimen.

Although conclusions demonstrated that the behavior of strengthened specimen and the monolithic specimen which had identical cross-sections and steel arrangement as the jacketed ones was almost identical, it was also concluded that energy dissipation capacities of strengthened columns were less than that of monolithic specimens. As expected, the adherence of two different concrete cast at different times will not be perfect. Moreover jacketing of the columns inside an existing building has some construction difficulties.

Considering all the aforementioned concerns, the computed moment capacities of the jacketed columns were reduced by 40 % in the analyses.

2.5 SUMMARY OF THE REHABILITATION STRATEGIES EMPLOYED IN THIS STUDY

The five evaluation schemes based on the two rehabilitation alternatives for the selected buildings are summarized below.

Building-1 was evaluated for five states:

1. The original un-rehabilitated structure
2. Rehabilitation by shear wall and jacketing
3. Rehabilitation by modified shear wall and jacketing (For this alternative moment capacities of shear walls were reduced by 25 %)
4. Rehabilitation by CFRP on infill walls and jacketing (Erdem's proposal for the models of CFRP and infill wall were used in this version)
5. Rehabilitation by CFRP on infill walls and jacketing

(For the models of CFRP and infill walls, constitutive models proposed in TUBITAK Report 2003/1 were used in this version)

Building-2 was evaluated for same five states as Building-1. But as mentioned before column jacketing was not applied to Building-2.

Rehabilitation alternatives selected for Building-1 and Building-2 are shown in the 3 dimensional models given in Figures 2.22-2.27.

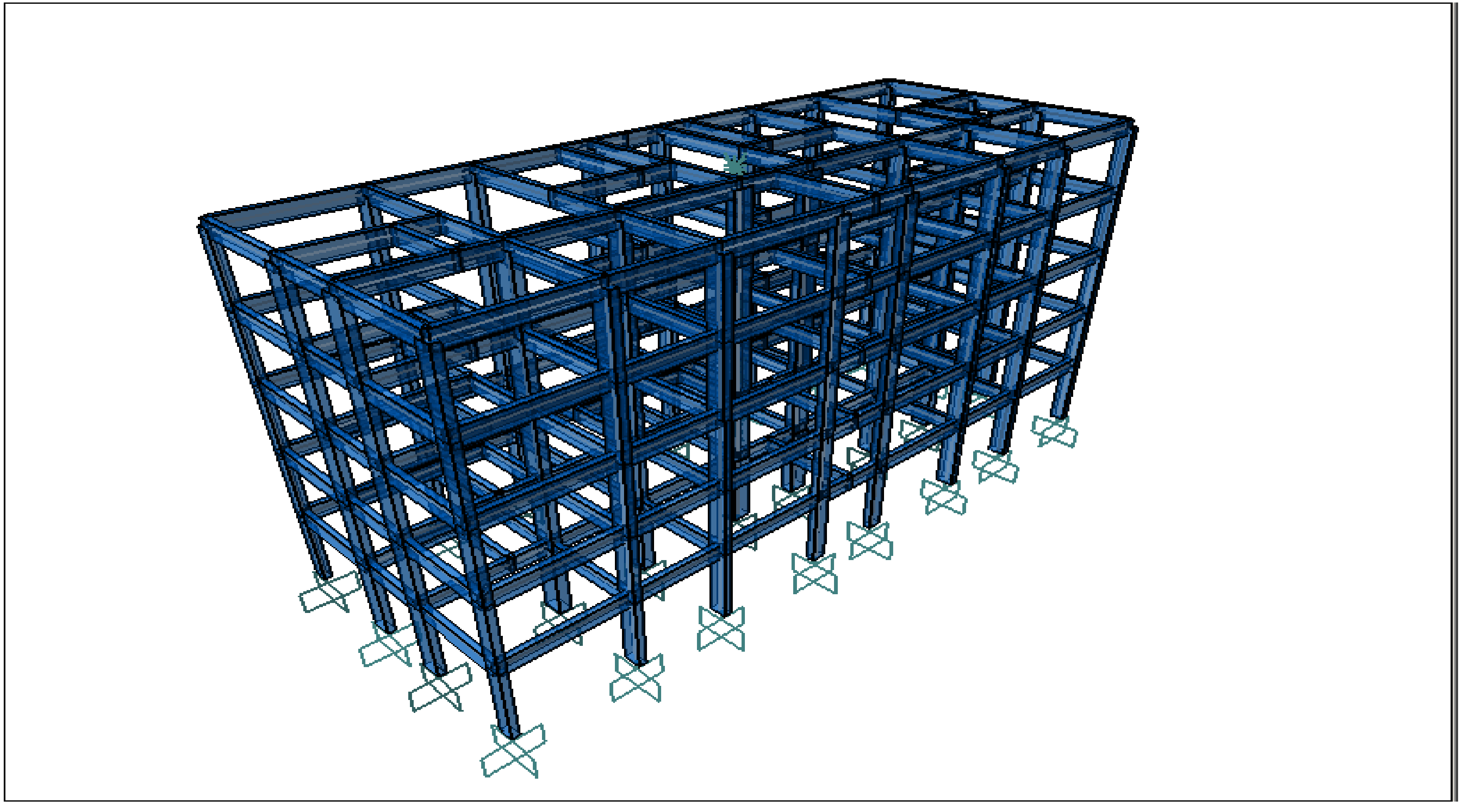

Figure 2.22 3-D Illustration of original Building-1

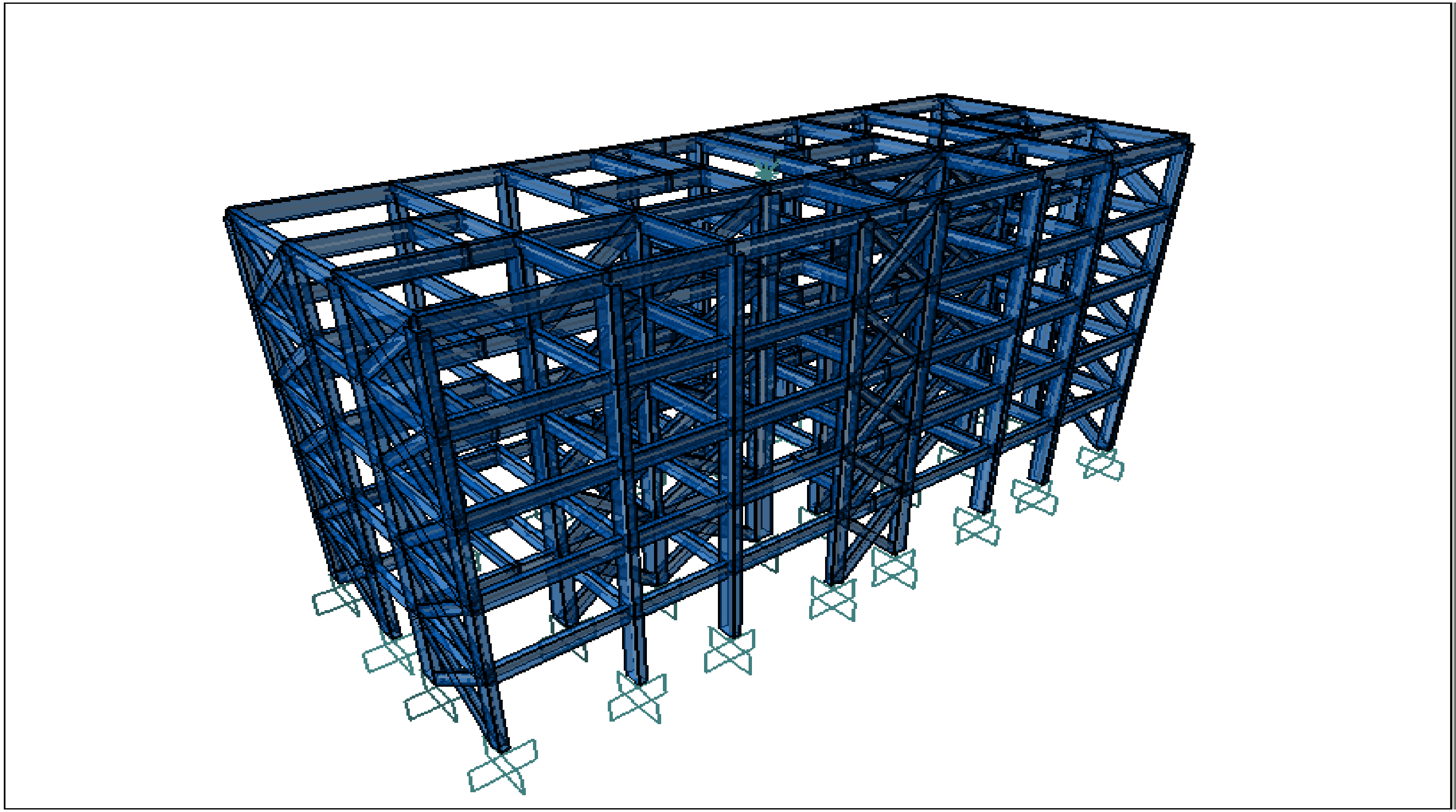

Figure 2.23 3-D Illustration of CFRP on infill walls and jacketing rehabilitated version of Building-1

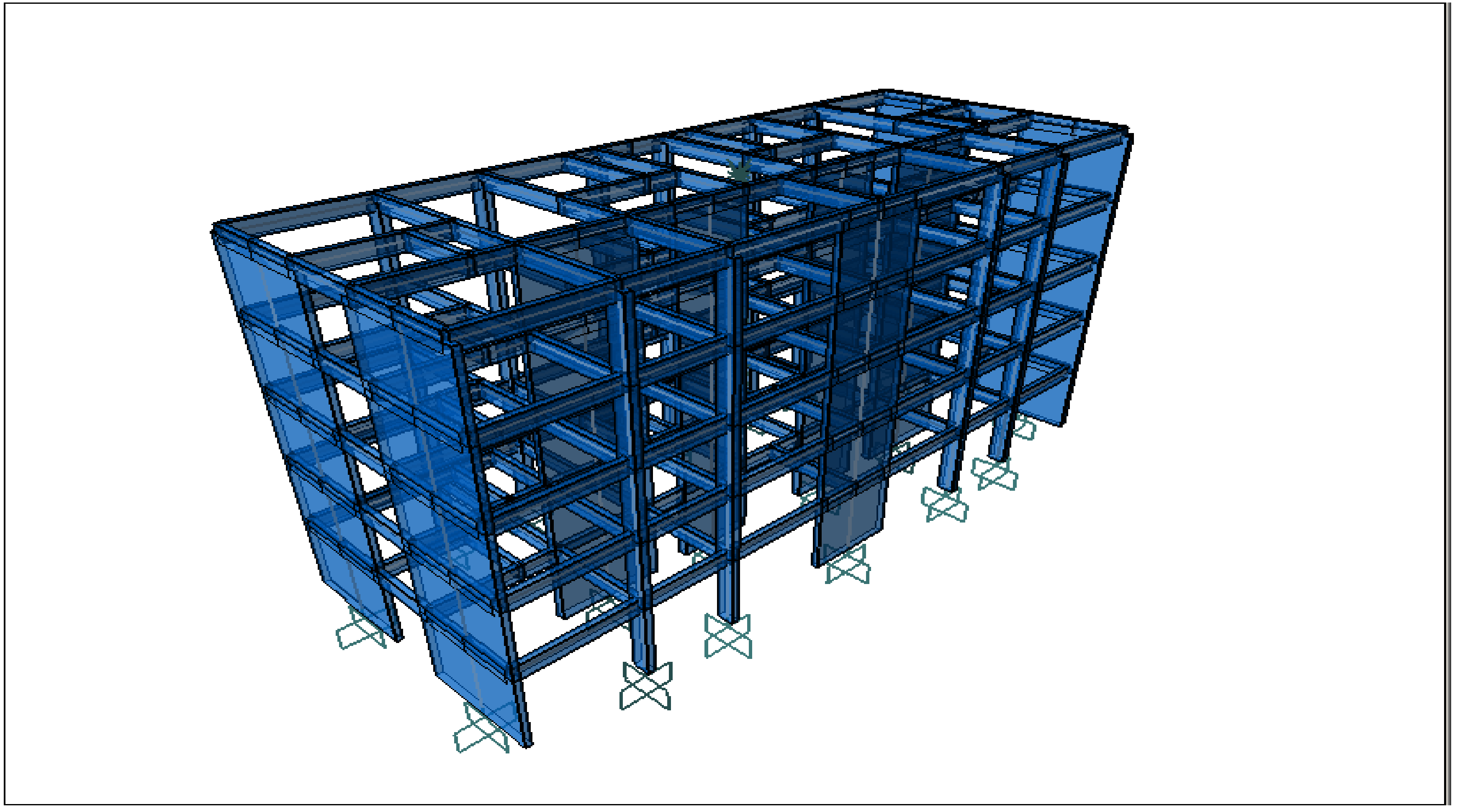

Figure 2.24 3-D Illustration of shear walls and jacketing rehabilitated version of Building-1

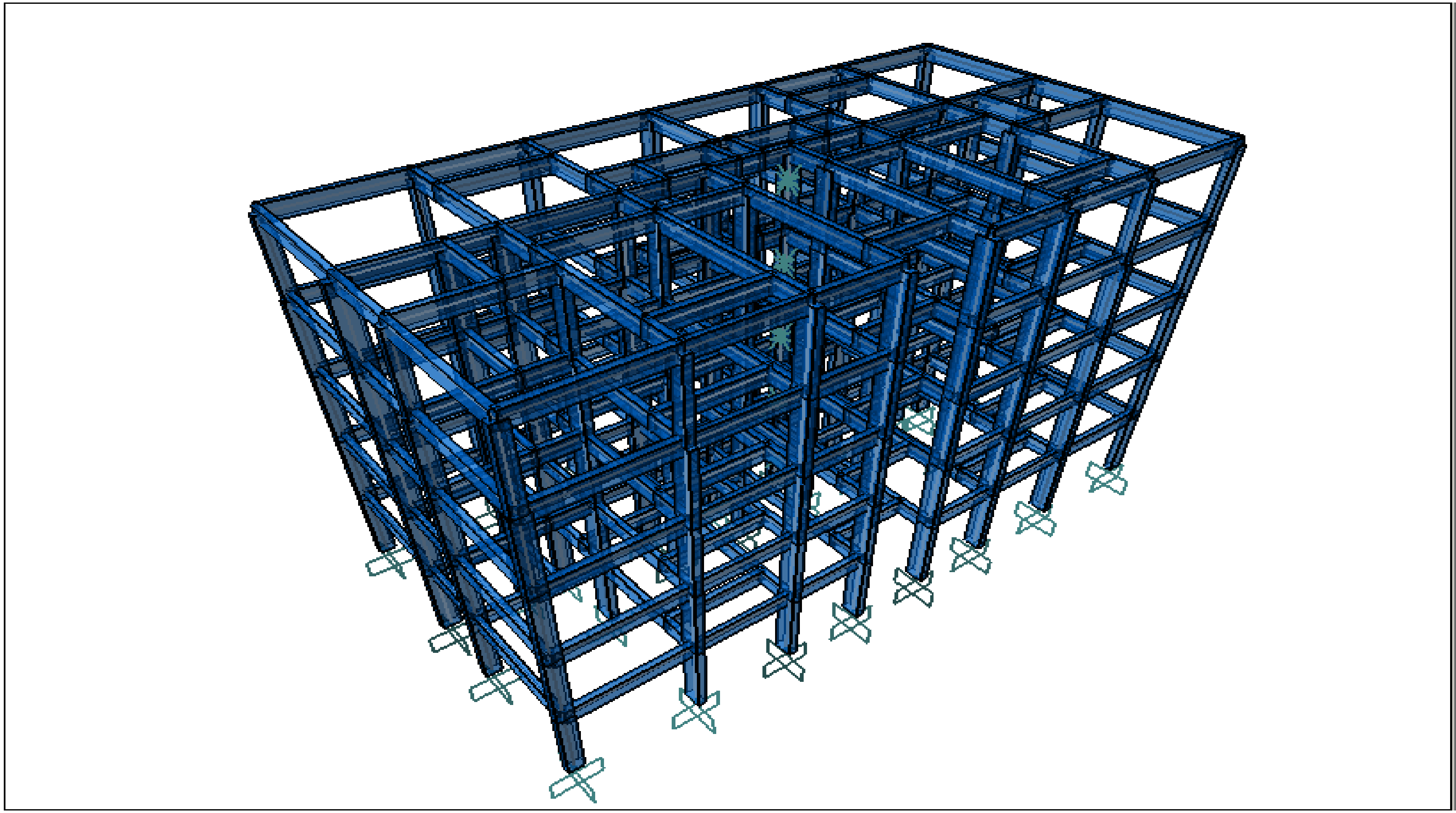

Figure 2.25 3-D Illustration of Original Building-2

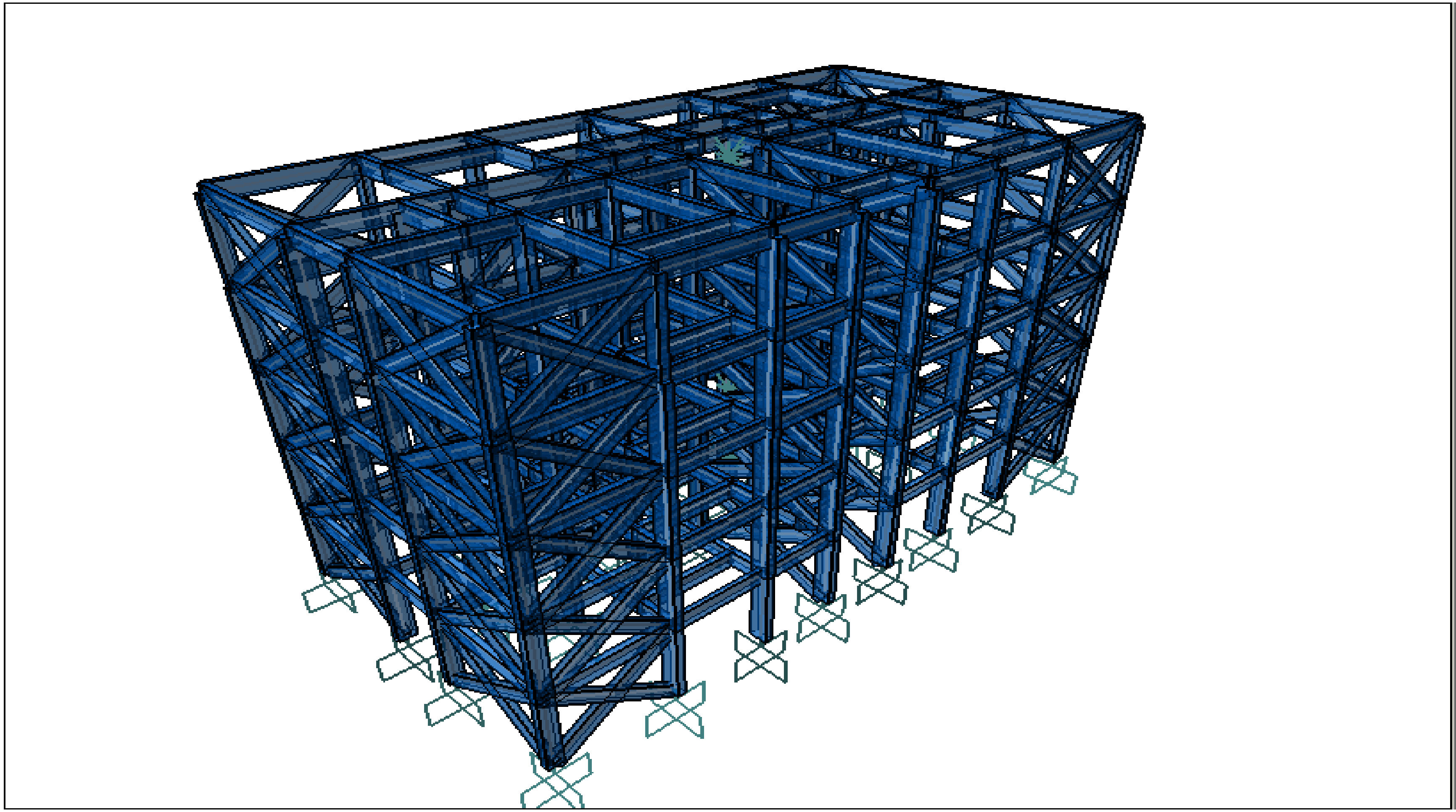

Figure 2.26 3-D Illustration of CFRP on infill walls rehabilitated version of Building-2

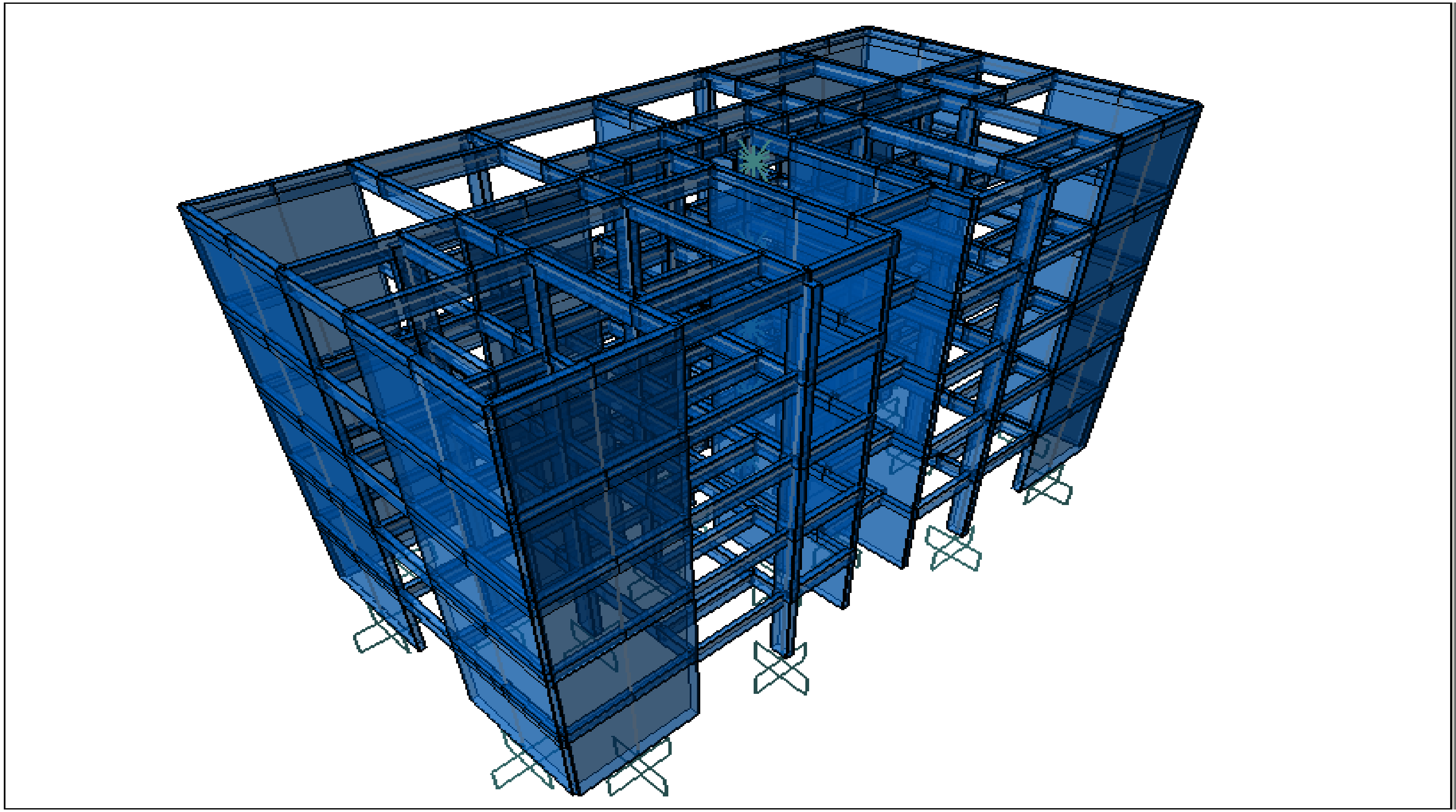

Figure 2.27 3-D Illustration of shear wall rehabilitated version of Building-2

2.6 PROCEDURE EMPLOYED FOR THE ASSESSMENT OF BUILDING PERFORMANCES

2.6.1 COMPUTATION OF PERFORMANCE POINT FOR THE BUILDINGS

In the analyses an approximate nonlinear procedure, Capacity Spectrum Method (CSM)-Procedure A, which is proposed by ATC40 [3] was used to find the performance points of buildings with and without rehabilitation measures using the code-based and site-specific spectra.

The CSM Procedure A is an iterative method. Performance point must lie both on the capacity spectrum and the spectral demand curve reduced from elastic 5 % damped spectrum (Figure 2.28).

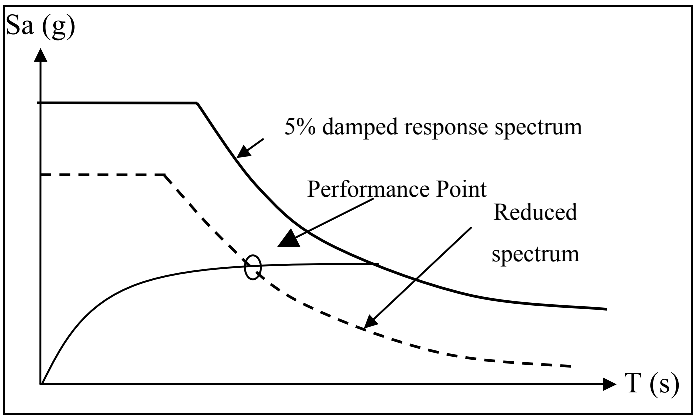

Figure 2.28 Performance point at the intersection point of reduced demand and capacity spectrum

In CSM structural behavior type must be specified. The selection of the structural behavior type depends on the quality of primary elements of the buildings and the duration of shaking. ATC40 [3] defines three different structural types (Table 2.5). In this study, structural behavior type was assumed to be Type B for both buildings.

Table 2.5 Structural behavior types defined by ATC-40 [3]

Shaking Duration	Essentially New Building	Average Existing Building	Poor Existing Building
Short	Type A	Type B	Type C
Long	Type B	Type C	Type C

A bilinear representation of capacity spectrum must be developed to estimate the effective damping ratio in order to reduce the demand spectrum. The initial line passes through the original and reflects the initial slope of the original curve. The slope of the second line (post-yield) is obtained such that it passes through the trial performance point (a_{pi},d_{pi}) and the yield point (a_y,d_y). The areas A1 and A2 shown in Figure 2.29 must be equal to each other.

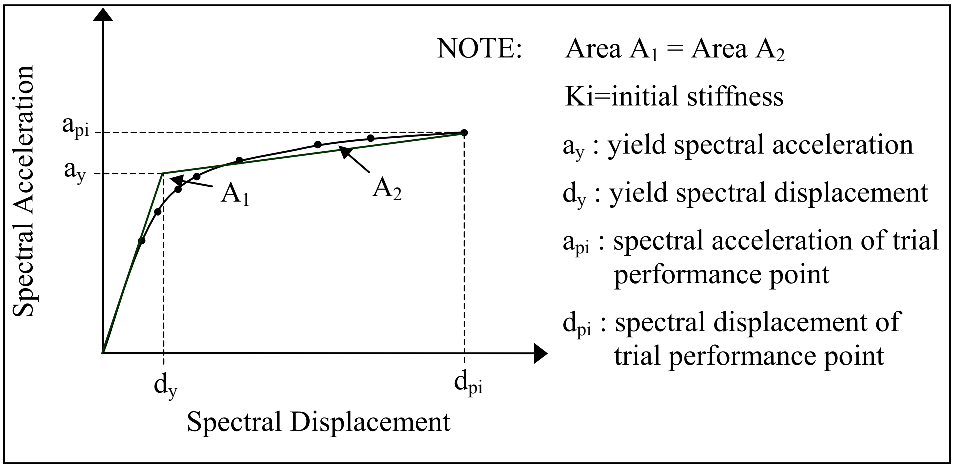

Figure 2.29 Bilinear representation of capacity spectrum

In order to reduce the 5 % damped response spectrum, effective damping is calculated using the equation given below:

$$\beta_{eff} = \kappa \cdot \beta_0 + 5 \qquad \textbf{\textit{(2.6)}}$$

Where:

β_{eff} : effective damping. This value should not be greater than 40 % for Type A, 29 % for Type B and 20 % for Type C structures.

5: viscous damping inherit in the structure (5 %)

β_0 : hysteretic damping represented as equivalent viscous damping calculated from equation 2.7.

$$\beta_0 = \frac{63.7(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} \qquad \textbf{\textit{(2.7)}}$$

κ : a modification factor for the simulation of probable imperfections in real building hysteretic loops, which may be pinching or degrading. Table 2.6 presents values for κ depending on structural behavior type and equivalent viscous damping.

Table 2.6 Values for damping modification factor, κ (ATC-40 [3])

Structural Behavior Type	β_0 (percent)	κ
Type A	≤ 16.25	1.0
	> 16.25	$1.13 - \dfrac{0.51(a_y d_{pi} - d_y a_{pi})}{(a_{pi} d_{pi})}$
Type B	≤ 25	0.67
	> 25	$0.845 - \dfrac{0.446(a_y d_{pi} - d_y a_{pi})}{(a_{pi} d_{pi})}$
Type C	Any value	0.33

Spectral reduction factors are calculated using the following equations.

$$SR_A \approx \frac{3.21 - 0.68 \cdot \ln(\beta_{eff})}{2.12} \tag{2.8}$$

$$SR_V \approx \frac{2.31 - 0.41 \cdot \ln(\beta_{eff})}{1.65} \tag{2.9}$$

Minimum values of SR_A and SR_V found from equations 2.8 and 2.9 should not be less than the values provided in Table 2.7.

Table 2.7 Minimum allowable values for SR_A and SR_V proposed by ATC-40 [3]

Structural Behavior Type	SR_A	SR_V
Type A	0.33	0.50
Type B	0.44	0.56
Type C	0.56	0.67

Figure 2.30 summarizes the steps involved in the Capacity Spectrum Method Procedure A.

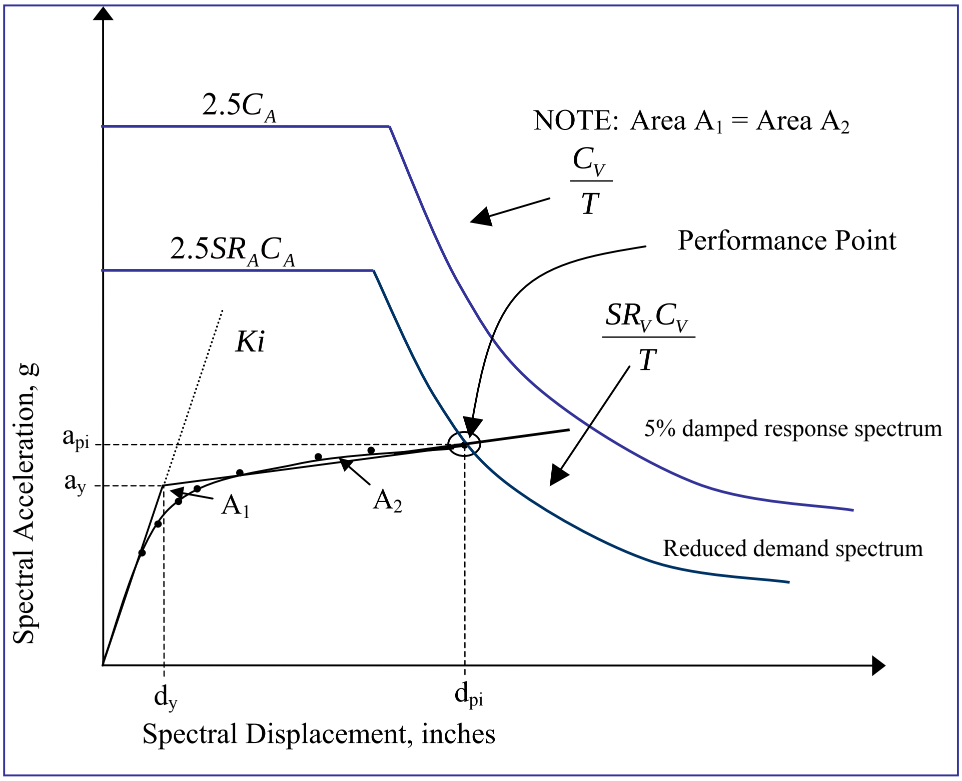

Figure 2.30 Determining performance point using CSM Procedure A

2.6.2 EVALUATION OF BUILDING RESPONSES

In ATC-40, the deformation capacity at Structural Stability Performance Level is defined as the deformation at which significant lateral load strength degradation initiates [3]. The deformation capacity for primary components at the Life Safety Performance Level is specified as 75 % of the Structural Stability Performance Level.

In the performance evaluation of the selected buildings, both before and after rehabilitation, the life safety performance objective was employed. The Life Safety performance level criteria are described in FEMA 273 [11] as:

- There might be extensive damage to structural and nonstructural components of the buildings.
- Repair might be needed prior to reoccupancy.
- Repair cost may be too much and economically impractical
- Probability of loss of life in the building is low.

Figure 2.31 shows the three widely used performance levels, namely Immediate occupancy (IO), Life Safety (LS) and Structural Stability (SS), on a capacity curve.

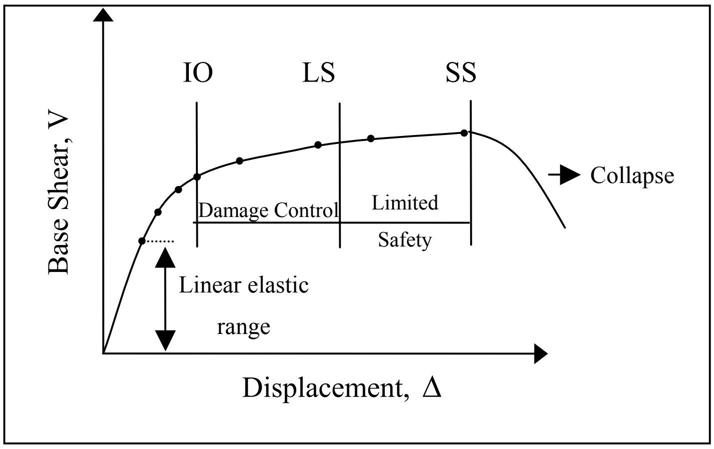

Figure 2.31 Performance levels on capacity curve

In this study, the buildings were evaluated for life safety performance objective using two separate acceptance criteria; global building acceptability limits and component acceptability limits. The response quantities calculated at performance point from nonlinear static analyses were compared with the corresponding component response limits to identify the critical members. The global response limits were used to determine the expected performance of the building as a whole.

2.6.2.1 EVALUATION OF GLOBAL BUILDING RESPONSE

The lateral deformations in the direction of loading corresponding to the performance point were obtained at each floor level of the building. The global drift ratio defined as the roof drift divided by the building height and interstory drift ratios were calculated and checked with the corresponding limits proposed by ATC40 [3].

The limits proposed by ATC40 [3] are shown in Table 2.8. In this table, the maximum total drift is defined as interstory drift ratio at the performance point of the structure. In order to account for the likely differences between the Turkish buildings and those in United States, the ATC limits given in Table 2.8 were reduced by 30 %.

Table 2.8 Deformation limits for performance levels, (ATC-40 [3])

	Performance Level			
Interstory Drift Limit	Immediate Occupancy	Damage Control	***Life Safety***	Structural Stability
Maximum total drift	0.01	0.01-0.02	***0.02***	0.33 V/P
Maximum inelastic drift	0.005	0.005-0.015	***no limit***	no limit

2.6.2.2 EVALUATION OF COMPONENT RESPONSE

The buildings with and without rehabilitation were checked using the numerical acceptance criteria that were proposed by ATC 40 [3] using the component level plastic hinge rotations limits. Only columns and shear walls, the primary lateral load carrying members, were checked against strength and deformation limits. It is recommended in the document, unless specific data are available to indicate deformability with lightweight aggregate concrete should be assumed to be about 25 % lower than those of equivalent components made with normal-weight aggregate concrete. For both of the buildings 70 percent of tabulated values were used in the analysis.

The document [3] gives numerical acceptance values for the structures whose boundary reinforcement effectively satisfies and does not satisfy the requirements of ACI 318 [2]. Both cases were checked and compared for the buildings.

The building response quantities obtained from nonlinear static analysis were used to evaluate their performance under the response spectra given previously. The global level and component-based evaluations were made separately to determine the efficiency of the recommended rehabilitation strategies. The results of these evaluations are presented in Chapter 4.

CHAPTER 3

COST-BENEFIT ANALYSIS

3.1 INTRODUCTION

Cost-benefit analysis is essential for determining the economic feasibility of the alternative rehabilitation strategies. Decision makers save time and resources by identifying unsuitable or unfeasible projects through evaluation of the consequences of cost-benefit analysis.

The following potential economic criteria are considered in this study for performing a systematic cost-benefit analysis:

- Estimating the cost required for implementation of the rehabilitation strategies,
- Estimating the benefits after implementation of the rehabilitation alternatives,
- Determining the cost effective alternative by computing the Net Present Value (NPV) or Benefit/Cost Ratio (BCR).

Detailed cost estimates must be performed for reasonably accurate estimates. Detailed estimation requires determination of the quantities and all associated costs. The costs may include the project cost, material cost, labor cost, cost of equipment and overhead expenses.

Estimating benefits is the most difficult step of cost-benefit analysis, since expected future benefits of rehabilitation strategies depend on the avoided (mitigated) damages.

The last step of the cost-benefit analysis is the decision of the most appropriate alternative. In this phase, despite its importance in cost-benefit analysis the economic value of life was excluded in the analyses.

3.2 COST OF REHABILITATION ALTERNATIVES

Using the construction drawings and specifications available, detailed cost estimates were made for each rehabilitation alternative considered in this study. In this respect detailed quantity takeoffs were performed.

Detailed estimates for the cost of the conventional rehabilitation method, insertion of shear walls and column jacketing, were performed according to the current unit prices list published by the Ministry of Public Works and Settlement in March, 2004 [17]. The prices given in this list are valid, through 01.01.2004, for the year 2004. Although many of the unit prices were gathered from this list, some of the unit prices were calculated in accordance with the special poses specifically provided for the strengthening techniques [29]. These special poses are consistent with the construction details of the rehabilitation strategies employed.

There are no available unit prices for determining the cost of the new rehabilitation technique, strengthening with CFRP, in the unit price list published by the Ministry of Public Works and Settlement [17]. In order to provide special poses for estimating the cost of rehabilitation with CFRP, unit prices of the materials used in this technique were acquired from a private firm which is one of the distributing agents of CFRP [23]. In calculation of these poses the overhead expenses and profit were taken as 25 % of cost of material and labor to be consistent with the methodology used in the calculation of unit prices provided by Ministry of Public Works [17]. While providing these special poses, CFRP application procedure of Specimen 5 which was studied in TUBITAK 2003/1 [7] Report was considered and implementation conditions of this new technique were analyzed carefully in METU-Structural Laboratory. Since prices of the materials used for this technique were given in Euro in the product data sheets, they were converted into Turkish Liras in order to make comparison with the cost of the conventional method. To be consistent with the cost of the conventional method, value of one Euro was taken as 1,630,000 TL based on rates posted on 01.01.2004.

Analyses of the unit price estimates derived for various special work items of rehabilitation alternatives which are not included in the unit price list of Ministry of Public Works and Settlement [17] are given in Appendix B.

Results of the detailed cost estimation for each rehabilitation alternative are presented in Table 3.1. As can be seen from the table, for both of the buildings cost of rehabilitation by CFRP application is greater than that of the strengthening with shear wall. This picture is not unexpected because the cost of CFRP itself is quite high and the amount of this material used in the techniques employed here is substantial. The information provided in the following section gives details of how the rehabilitation costs are calculated along with the descriptions of the work items and the assumptions used. Detailed quantity takeoffs were provided in the CD attached to the thesis.

Table 3.1 Rehabilitation Costs for Building-1 and Building-2

Building	Rehabilitation Alternative	Cost
Building-1	Shear Wall +Jacketing	121,536,379,318 TL
	CFRP + Jacketing	137,113,028,757 TL
Building-2	Shear Wall	204,157,763,493 TL
	CFRP	240,600,240,484 TL

3.2.1 ASSUMPTIONS FOR ESTIMATING THE COST OF REHABILITATION STRATEGIES

The cost variables that were considered in estimating the cost of rehabilitation alternatives are given in Table 3.2. Some assumptions and basic descriptions of each work item are explained in the following sections.

Table 3.2 Work items considered for the detailed cost estimation

Rehabilitation with insertion of shear walls and column jacketing	**Rehabilitation with CFRP**
Excavation and filling	Wet application of CFRP (for diagonal struts)
Crushing rubble concrete at the foundation level	CFRP for lap splices
Making 300 dose rubble concrete for new foundation of shear walls and jacketed columns	CFRP used as anchor dowels
Dismantling of floorboard	Scaffolding
Dismantling of vertical rain water pipes	Plastering
Demolishing infill walls	Exterior plastering
Crushing concrete cover of columns and beams	Whitewashing old whitewashed surfaces with 3 layers of plastic wall painting
Using C20 ready mixed concrete for the construction of shear walls and jacketed columns	Whitewashing new plaster with 3 layers of plastic wall painting
Providing non-shrink concrete	Acrylic painting for exterior sides of building
Construction steel used for longitudinal reinforcement, transverse reinforcement, and anchorage	Design project cost
Welding for jacketing	Cost of delivery
Wooden formwork	
Scaffolding for formwork	
Scaffolding (painting external faces of the buildings)	
Dismantling of door, window and etc	
Covering slab with mosaic having all types of color	
Plastering	
Exterior plastering	
Whitewashing old whitewashed surfaces with 3 layers of plastic wall painting	
Whitewashing new plaster with 3 layers of plastic wall painting	
Acrylic painting for exterior sides of building	
Fitting waste water pipe	
Rental cost	
Design project cost	
Dismantling and installation of electricity, plumbing, and gas	
Cost of delivery	

3.2.1.1 Rehabilitation with introducing Shear Walls and Column Jacketing

The variables considered in estimating the cost of rehabilitation by introducing shear walls and column jacketing are listed below along with the associated assumptions made:

- Excavation and filling: In order to make foundations of shear walls, earth has to be excavated and afterwards has to be filled back. It was assumed that 1 m length from both sides of the wall area is going to be excavated (Figure 3.1).

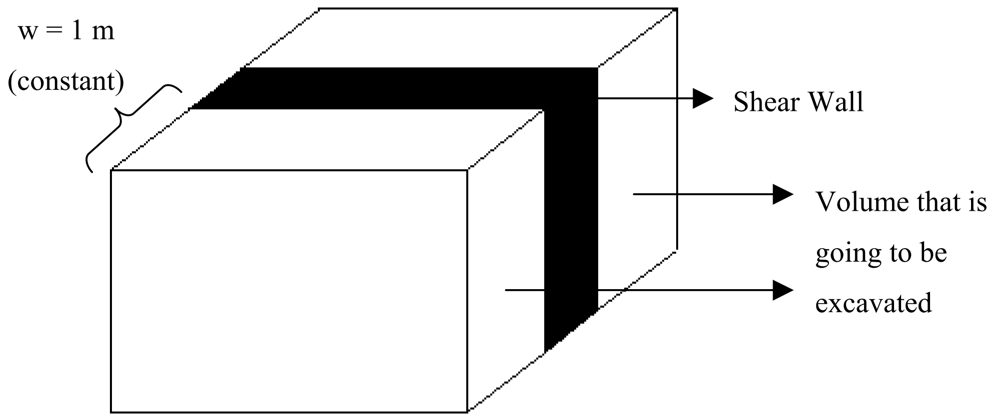

Figure 3.1 Area that is going to be excavated and filled back

- Crushing rubble concrete at the foundation level: The thickness and the width of the rubble concrete to be crushed from both sides of the wall area were assumed as 0.1 m and 1m, respectively.
- Making 300 dose rubble concrete for new foundation of shear walls and jacketed columns: 300 dose mixed rubble concrete is placed at the base level on which foundations of shear walls and jacketed columns are constructed. Same assumptions made for crushing rubble concrete were considered for the dimensions of thickness and width of the new rubble concrete.

- Dismantling of floorboard: 1 m length of board from both sides of the wall area was assumed to be dismantled at the 1st story level and 0.25 m length of board was assumed for the upper floors.
- Dismantling of vertical rain water pipes: Vertical rain water pipe was assumed to be dismantled for this rehabilitation alternative.
- Demolishing infill walls: Masonry infill walls are demolished for the insertion of shear walls and column jacketing.
- Crushing concrete cover of columns and beams: The cover of columns and beams has to be crushed till transverse reinforcement appears. It is implemented to ensure development of effective bond strength between the old and the new concrete. Unit price (Appendix B, Table B.6) calculated for this item also includes cleaning of the crushed surface with pressurized air and water.
- Using C20 ready mixed concrete for the construction of shear walls and jacketed columns: It was assumed that reinforced concrete grade C20 was used for the construction of shear walls and column jacketing. The unit price determined for this work item was different from the price given in the unit price list of Ministry of Public Works and Settlement [17]. To reflect this difference additional materials used were included and difficult working conditions were considered in terms of labor prices in the analysis of the unit price (Appendix B, Table B.1).
- Providing non-shrink concrete: Considering the construction inconveniences it was assumed that a gap forms between beams and added shear walls. The height of this gap was assumed to be 0.15 meter. This gap is filled with non-shrink concrete.
- Application of adhesive material to the interface of old and new concrete: To increase the bond between old and new concrete adhesive substance is applied.
- Construction steel used for longitudinal reinforcement, transverse reinforcement, and anchorage: This work item includes the construction steel used for longitudinal reinforcement, transverse reinforcement and anchoring. Two special unit price analyses, one for longitudinal and transverse reinforcement and one for anchorage were prepared (Appendix B, Table B.7-B.14). Their analyses are slightly different from the analyses of ordinary construction steel. Especially considering the difficult working conditions quantities estimated for labor were greater than the estimated labor hours in the analyses performed for the unit prices of Ministry of Public Works and Settlement. For anchoring unit price analyses were performed for different depths. In these analyses

anchoring consists of opening hole, cleaning inside the hole with pressurized air, application of resin and placing of steel reinforcement inside the hole.

- Welding for jacketing: Reinforcement bars shaped like "Z" would be welded to the 8 cm length of the existing reinforcement of the columns that are jacketed.
- Wooden formwork: The quantity and cost of wooden formwork that has to be constructed for foundations, sides of shear walls and jacketed columns were calculated.
- Scaffolding for formwork: The base length of the triangle was taken 1.2 meter for the quantity analysis of scaffolding for formwork. Quantity calculated for scaffolding for formwork per m^3 is illustrated in Figure 3.2.

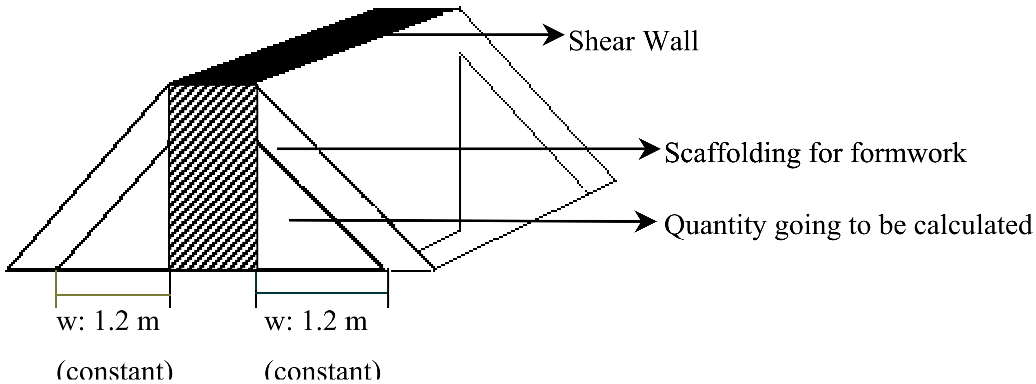

Figure 3.2 Scaffolding for formwork

- Scaffolding: After the completion of rehabilitation, scaffolding has to be erected outside the building to paint the external face of the building.
- Dismantling of door, window and etc.: Doors and windows were assumed to be dismantled before demolishing infill walls.
- Covering slab with mosaic having all types of color: After constructions of shear walls and column jacketing, dismantled floorboards are covered with new mosaic having same dimensions as the dismantled floorboards.
- Plastering: Interior surfaces of the buildings were assumed to be plastered with washable plastic paint entirely.

- Exterior plastering: Exterior face of the building was assumed to be plastered completely.
- Whitewashing old whitewashed surfaces with 3 layers of plastic wall painting: All old whitewashed surfaces are rewhitewashed completely for a unique view of interior side of the buildings.
- Whitewashing new plaster with 3 layers of plastic wall painting: This work item is a common application after plastering.
- Acrylic painting for exterior sides of building: Exterior faces of the buildings were assumed to be painted with acrylic painting.
- Fitting waste water pipe - ø100 MM PVC: 100 mm diameter of plastic wastewater pipe would be installed outside the buildings for water drainage.
- Rental cost: Evacuation of the buildings is required while introducing shear walls and column jacketing Average local rents for buildings vary widely. Local rents for both of the residential buildings were estimated as 250,000,000 TL per month for one apartment house in the cost analysis.
- Design project cost: This includes initial project development cost and the cost of maintaining projects over time. Project cost was estimated 3,000,000 TL per m^2 for both of the buildings.
- Dismantling and installation of electricity, plumbing, and gas: Five percent of the total rehabilitation cost was assumed for the cost of dismantling the electrical, gas, and plumbing systems.
- Cost of delivery: The prices and rates used in the calculation of overall price include all costs and expenses required to complete the corresponding work item as described in the preceding section. However some of the unit prices used in the analysis were not quoted for cost of delivery to the site. To include this an adjustment was made such that ten percent of the total rehabilitation cost was estimated for the cost of delivery.

The rates assumed for dismantling and installation of electricity, plumbing, and gas and cost of delivery were analyzed carefully. They are consistent with the rates taken in implemented and confirmed rehabilitation projects by Ministry of Public Works and Settlement.

Bills of quantities estimates for shear wall and jacketing rehabilitated versions of Building-1 and Building-2 are summarized in Tables 3.3 and 3.4.

Table 3.3 Bill of quantities estimates

(Shear wall and jacketing rehabilitated version of Building-1)

WORK ITEM	PRICE
Excavation and filling	916,155,628 TL
Crushing rubble concrete	6,253,875 TL
Making 300 dose rubble concrete	199,880,009 TL
C20 Ready mixed concrete	6,865,254,521 TL
Covering between old and new concrete with adhesive material	3,523,425,125 TL
Wooden formwork	10,155,328,046 TL
Scaffolding for formwork	773,381,510 TL
Scaffolding	3,181,191,824 TL
Demolishing infill walls	933,634,050 TL
Dismantling of door, window and etc.	25,832,807 TL
Providing non shrink concrete	104,741,804 TL
Crushing concrete cover of columns and beams	897,337,764 TL
Welding for jacketing	361,336,125 TL
Construction Steel Ø 8 - Ø 12 Ribbed bars	7,169,567,844 TL
Construction Steel Ø 14 - Ø 24 Ribbed bars	8,228,572,843 TL
Ø20-40cm-110cm (Anchorage)	1,647,296,651 TL
Ø20-30cm-100cm (Anchorage)	18,383,539,275 TL
Ø20-25cm-95cm (Anchorage)	490,343,035 TL
Ø22-45cm-120cm (Anchorage)	1,604,499,596 TL
Ø22-35cm-100cm (Anchorage)	3,558,878,862 TL
Ø22-12cm-85cm (Anchorage)	57,487,986 TL
Ø22-35cm-100cm (For foundation) (Anchorage)	873,542,993 TL
Acrylic painting for exterior sides of building	7,306,483,620 TL
Plastering	2,262,371,976 TL
Exterior plastering	1,448,841,472 TL
Whitewashing new plaster with 3 layers of plastic wall painting	280,821,722 TL
Dismantling of vertical rain water pipes	79,832,400 TL
Fitting waste water pipe - ø100 mm PVC	254,677,085 TL
Dismantling floor board, concrete plate, paving stone and rubble stone	325,910,273 TL
Covering slab with mosaic having all types of color	2,257,007,538 TL
Whitewashing old whitewashed surfaces with 3 layers of plastic wall painting	4,971,179,844 TL
Rental cost (250,000,000 TL/month/apartment house)	12,500,000,000 TL
Design project cost	4,039,200,000 TL
Sub-total	105,683,808,102 TL
Dismantling and installation of electricity, plumbing, gas (5 %)	5,284,190,405 TL
Cost of delivery (10 %)	10,568,380,810 TL
TOTAL	***121,536,379,318 TL***

Table 3.4 Bill of quantities estimates

(Shear wall rehabilitated version of Building-2)

WORK ITEM	**PRICE**
Excavation and filling	1,610,137,945 TL
Crushing rubble concrete	8,843,443 TL
Making 300 dose rubble concrete	282,645,139 TL
C20 ready mixed concrete	15,158,529,506 TL
Covering between old and new concrete with adhesive material	5,437,440,580 TL
Wooden formwork	21,017,749,627 TL
Scaffolding for formwork	1,595,897,368 TL
Scaffolding	1,079,149,621 TL
Demolishing infill walls	1,854,482,400 TL
Dismantling of door, window and etc.	102,861,541 TL
Providing non shrink concrete	232,831,376 TL
Crushing concrete cover of shear walls, columns and beams	1,182,181,619 TL
Construction Steel Ø 8 - Ø 12 ribbed bars	15,291,098,851 TL
Construction Steel Ø 14 - Ø 24 ribbed bars	16,566,656,056 TL
Ø20-40cm-110cm Anchorage	3,548,023,556 TL
Ø20-30cm-100cm Anchorage	26,160,643,670 TL
Ø22-45cm-120cm Anchorage	722,024,818 TL
Ø22-35cm-100cm Anchorage	21,547,393,837 TL
Ø22-40cm-110cm	1,764,949,556 TL
Ø22-35cm-100cm (For foundation)	1,261,784,324 TL
Acrylic painting for exterior sides of building	7,306,483,620 TL
Plastering	4,400,495,722 TL
Exterior plastering	3,356,248,025 TL
Whitewashing new plaster with 3 layers of plastic wall painting	525,844,260 TL
Dismantling of vertical rain water pipes	84,330,000 TL
Fitting waste water pipe - ø100 mm PVC	269,025,090 TL
Dismantling floor board, concrete plate, paving stone and rubble stone	417,096,403 TL
Covering slab with mosaic having all types of color	2,888,493,564 TL
Whitewashing old whitewashed surfaces with 3 layers of plastic wall painting	5,315,948,480 TL
Rental cost (250,000,000 TL/month/apartment house)	12,500,000,000 TL
Design project cost	4,039,200,000 TL
Sub-total	177,528,489,994 TL
Dismantling and installation of electricity, plumbing, gas (5 %)	8,876,424,500 TL
Cost of delivery (10 %)	17,752,848,999 TL
TOTAL	***204,157,763,493 TL***

•Detailed quantity takeoffs was appended to the CD attached to the thesis.

3.2.1.2 Rehabilitation by applying Carbon Fiber Reinforced Polymer on infill walls

The procedures used for the implementation of the CFRP strips to strengthen the RC frames with infill walls were described in Chapter 2. As stated before, in this study the procedure used and recommended for Specimen-5 of METU experimental research program [7] was used. The purpose of this selection was to remain consistent with the analytical model used in the analytical phase of this research that was based on the experimental results of Specimen-5.

This technique can be applied in different configurations. There isn't yet a generalized method of applying the technique. To be consistent with the modeling of the buildings rehabilitated by CFRP on infill walls, special poses were provided for computing cost of this technique. The poses derived according to the construction details of Specimen-5 are given in Appendix B (Tables B.16-22).

The rent cost and the cost of dismantling and installation of electricity, plumbing and gas were not included in cost estimates for this technique. The work items, which are special to this technique, are listed below:

- <u>Wet application of CFRP</u>: Primarily there are two techniques of applying CFRP:

1. Dry application: It is the simplest way of applying CFRP. Firstly resin based mixture is applied on the application surface and then fiber sheets prepared are applied onto the resin coating using a roller.

2. **Wet application:** In this application epoxy based matrix of CFRP is composed of concressive 1305, putty and saturant [6]. Firstly, Carbon Fiber Sheets are cut to the desired sizes. For the next step surfaces where CFRP will be applied are coated with concressive 1305 to fill the micro cracks on the wall to ensure an appropriate bond between application surface and resin. Putty is used to make the surface smooth. After that saturant (adhesive base) is applied on the surface where CFRP sheets will be placed on. The prepared sheets are put onto the resin coating in the required direction and finally additional resin layer is applied on the fiber sheets.

The wet and dry applications differ in cost and in the way CFRP is applied. Since wet application was preferred for Specimen-5, to be compatible with the experimental results, the cost was calculated according to this method.

The data acquired from the manufacturer was used in deriving the special poses. Further technical data was extracted from the product data sheets. The basic information used for the cost analysis of CFRP application is given in Table 3.5.

Amount of the CFRP sheets needed for strengthening infills of buildings were calculated proportional to the amount of CFRP sheets used for Specimen-5. The amount of the CFRP required was calculated based on the ratio of the areas of the wall surface in hand to that of Specimen-5.

Amount of resin consumption for wet application was calculated according to the available data acquired from the manufacturer. It depends on the amount of the CFRP sheets. While performing unit price analysis, surface conditions of the wall for both inside and outside of the buildings were considered. For wet application method resin consumption depends on the roughness of the substrate.

Furthermore to adjust the calculated quantities for including the wasted material during application, 5 percent of CFRP and resin amount were assumed as waste material in the calculation of unit price analysis.

Special work items for which unit price analyses were prepared are briefly explained next:

- <u>CFRP for lap splices:</u> Lap splice regions of Specimen 5 were confined using two layers of CFRP fibers in orthogonal directions.
- <u>CFRP used as anchor dowels:</u> Four different types of anchor dowels, Type A, B, C and D [7], were used for Specimen-5. However two types of anchor dowels, Type A and Type C, were assumed to be used for the anchorage system in this study. Type A was considered to be applied inside the infills and Type C inside the concrete. The properties of the anchor dowels are summarized in Table 3.6

Table 3.5 Parameters used in deriving special poses

<table>
<tr><th>Material</th><th>Price (Euro)</th><th>Price (TL)</th><th>Description</th><th colspan="3">Resin Consumptions for Wet Application</th></tr>
<tr><td>Carbon Fiber Sheets</td><td>40 (Euro/m^2)</td><td>65,200,000 (TL/m^2)</td><td>Unidirectional, woven carbon fiber fabric</td><td colspan="3"></td></tr>
<tr><td rowspan="4">Resin A</td><td rowspan="4">14.7 (Euro/kg)</td><td rowspan="4">23,961,000 (TL/kg)</td><td rowspan="4">Impregnation Epoxy Resin for Carbon Fabric Reinforcement with the Wet and Dry Application, A: B=100:34.5 by weight</td><td colspan="3">Wet Application</td></tr>
<tr><td colspan="2">Primer Consumption</td><td>Resin Consumption</td></tr>
<tr><td>Smooth Surface</td><td>Rough Surface</td><td rowspan="2">0.7 kg/m^2</td></tr>
<tr><td>0.5 kg/m^2</td><td>0.8 kg/m^2</td></tr>
<tr><td rowspan="3">Resin B</td><td rowspan="3">16.1 (Euro/kg)</td><td rowspan="3">26,243,000 (TL/kg)</td><td rowspan="3">Impregnation Epoxy Resin for Carbon Fabric Reinforcement with the Dry Application, A: B=4:1 by weight</td><td colspan="3">Wet Application</td></tr>
<tr><td colspan="3">Resin Consumption</td></tr>
<tr><td colspan="3">0.8 kg/m^2</td></tr>
</table>

Table 3.6 Properties of anchor dowels considered in cost analysis

Anchor Type	Applied to	Depth in reinforcement (mm)	Width of Strip (mm)	Length of strip (mm)	Diameter of Hole (mm)
A	Infill	--	50	320	10
C	Concrete	60	25	240	10

- Design project Cost: Expected cost of project blueprints and design calculations for this rehabilitation technique was assumed to be less than that for the conventional methods as this rehabilitation technique does not entail time as much as conventional methods do. Besides in the project development phase a few construction plans and drawings are required. Therefore, the project cost was estimated as 1,500,000 TL per m^2.
- Cost of delivery: The cost of transport depends on the distance that the materials of construction will be transported to and the amount of the material. Since CFRP is a lightweight material, amount of material to be transported was expected to be less. For this reason 7 percent of rehabilitation cost was estimated for the cost of delivery.

Some of the work items implemented in the rehabilitation procedure are same with the work items considered in other rehabilitation alternatives. Therefore same assumptions explained before were made for the work items listed below:

- Plastering
- Exterior plastering
- Whitewashing new plaster with 3 layers of plastic wall painting
- Whitewashing old whitewashed surfaces with 3 layers of plastic wall painting
- Scaffolding
- Acrylic painting for exterior sides of building

Bills of quantities estimates for cost of Carbon Fiber Reinforced Polymer (CFRP) on infill walls for both buildings are summarized in Tables 3.7 and 3.8.

Table 3.7 Bill of quantities estimates

CFRP on infill walls and column jacketing in Building-1

WORK ITEM	PRICE
Excavation and filling (Jacketed columns)	134,907,665 TL
Crushing rubble concrete (Jacketed columns)	2,038,300 TL
Making 300 dose rubble concrete (Jacketed columns)	65,146,077 TL
C20 concrete ready mixed concrete (Jacketed columns)	446,057,707 TL
Covering between old and new concrete with adhesive material (Jacketed columns)	700,138,670 TL
Wooden formwork (Jacketed columns)	119,335,517 TL
Scaffolding for formwork (Jacketed columns)	34,413,644 TL
Scaffolding	3,181,191,824 TL
Crushing concrete cover (Jacketed columns)	304,532,833 TL
Welding for jacketing (Jacketed columns)	361,336,125 TL
Steel Ø 14 - Ø 24 (Jacketed columns)	763,695,007 TL
Anchorage (for jacketing)	6,323,218,702 TL
Acrylic painting for exterior sides of building	7,306,483,620 TL
Plastering	2,262,371,976 TL
Exterior plastering	1,448,841,472 TL
Whitewashing new plaster with 3 layers of plastic wall painting	280,821,722 TL
Dismantling floor board, concrete plate, paving stone and rubble stone	18,993,250 TL
Covering slab with mosaic having all types of color	131,532,854 TL
Whitewashing old whitewashed surfaces with 3 layers of plastic wall painting	4,971,179,844 TL
Wet application - CFRP	75,871,173,649 TL
CFRP used for anchor dowels	14,380,958,985TL
CFRP for lap splices	7,015,048,086 TL
Design project cost	2,019,600,000 TL
Sub Total	128,143,017,530 TL
Cost of delivery (7 %)	8,970,011,227 TL
TOTAL	***137,113,028,757 TL***

Table 3.8 Bill of quantities estimates

CFRP on infill walls version of Building-2

WORK ITEM	PRICE
Scaffolding	2,642,438,284 TL
Acrylic painting for exterior sides of building	7,262,621,100 TL
Plastering	4,400,495,722 TL
Exterior plastering	3,390,737,407 TL
Whitewashing new plaster with 3 layers of plastic wall painting	525,844,260 TL
Whitewashing old whitewashed surfaces with 3 layers of plastic wall painting	5,315,948,480 TL
Wet application CFRP	171,352,258,173 TL
CFRP used for anchor dowels	21,840,295,143 TL
CFRP for lap splices	6,109,798,466 TL
Design project cost	2,019,600,000 TL
Sub Total	224,860,037,836 TL
Cost of delivery (7 %)	15,740,202,649 TL
TOTAL	***240,600,240,484 TL***

3.3 ESTIMATING BENEFITS

Predicting benefits arising from a seismic rehabilitation strategy in a probable future earthquake is the most difficult part of the cost-benefit analysis since there is lack of available experimental and empirical data on the performance of the rehabilitated buildings. Developing fragility curves, which represent the probability of exceeding a damage limit state for a given structure type subjected to ground excitations, are useful means of estimating the probable loss. Since the modifications to the building change its structural properties these curves need to be developed for each rehabilitation alternative and that requires thorough analyses and information. In this respect twenty fragility curves were established for the buildings to assess the expected earthquake damage for each rehabilitation alternative in the x- and y-directions. ATC –13 [4] provides quantitative damage ratios (on a scale of 100) corresponding to certain prescribed physical damage states. In other words, damage factors are assigned to seven damage states in percentages as shown in Table 3.9.

Table 3.9 Damage factors corresponding to damage states, ATC-13 [4]

Damage State	Damage Factor Range (%)	Central Damage Factor (%)
None	0	0
Slight	0-1	0.5
Light	1-10	5
Moderate	10-30	20
Heavy	30-60	45
Major	60-100	80
Destroyed	100	100

In this study, the upper limits corresponding to four damage states, none, light, moderate and heavy were considered. In other words, four damage states with corresponding quantitative values (upper limits) taken from Table 3.9 were used in conjunction with the performance limits at yield and ultimate global drift ratios obtained from the capacity (pushover) curves (Table 3.10) to develop the fragility curves.

Table 3.10 Damage factors corresponding to limit global drift ratios

Damage State	Damage Factor (%)	Limit Global Drift Ratios (%)
None	0	0
Light	10	$\Delta y / H$
Moderate	30	$0.5(\Delta y + \Delta u)/H$
Heavy	60	$\Delta u / H$

Because of the convergence problems complete pushover curves could not be obtained in some cases. For this reason ultimate global drift limits of buildings rehabilitated with shear walls were assumed to be equal to that of original building (bare frame) and ultimate global drift limits of buildings rehabilitated with the two different CFRP models [6, 7] were taken equal to each other. These assumptions are consistent with the experimental results presented in Erdem [6]. Global drift ratios for each rehabilitation alternative used in establishing the fragility curves of Building-1 and Building-2 are summarized in Tables 3.11 and 3.12, respectively.

Table 3.11 Limit global drift ratios for Building-1

Rehabilitation Strategy	BUILDING-1					
	x-direction			y-direction		
	Δy /H	**0.5(Δy+Δu)/H**	**Δu/H**	**Δy /H**	**0.5(Δy+Δu)/H**	**Δu/H**
Original Building	0.18	0.71	1.23	0.10	0.71	1.32
Shear Wall	0.13	0.68	1.23	0.10	0.71	1.32
Reduced-Shear Wall	0.09	0.66	1.23	0.07	0.70	1.32
Erdem’s CFRP Model	0.08	0.44	0.80	0.10	0.48	0.87
TUBITAK CFRP Model	0.11	0.45	0.80	0.20	0.54	0.87

Table 3.12 Limit global drift ratios for Building-2

Rehabilitation Strategy	BUILDING-2					
	x-direction			y-direction		
	Δy /H	**0.5(Δy+Δu)/H**	**Δu/H**	**Δy /H**	**0.5(Δy+Δu)/H**	**Δu/H**
Original Building	0.17	0.91	1.66	0.20	0.82	1.44
Shear Wall	0.12	0.89	1.66	0.20	0.82	1.44
Reduced-Shear Wall	0.11	0.88	1.66	0.19	0.81	1.44
Erdem’s CFRP Model	0.07	0.45	0.82	0.09	0.46	0.84
TUBITAK CFRP Model	0.14	0.48	0.82	0.17	0.50	0.84

Fragility curves were fitted using the least squares fitting technique. Equation giving the damage function in percentage is given below:

$$Damage = 1 - \exp^{-(\frac{\Delta/h}{a})^{b}} \tag{3.1}$$

where:

Δ / h : Global drift ratio

a and b are the unknown equation variables

The fragility curves were established combining the information explained above with the purpose of assessing the earthquake damage before and after rehabilitation. By using these fragility curves, probable damage induced on the buildings was obtained in percent of replacement value of the buildings.

The fragility curves developed for cost-benefit analysis are shown in Figures 3.3-3.6. Examination of these curves reveals that fragility curves for the RC systems are almost identical since they have the same ultimate global drift ratios. In case of the CFRP rehabilitated buildings the difference between the curves is more significant due to differences in their yield drift ratios. Examination of the properties of pushover curves given in Chapter 4 provides better insight for these observations.

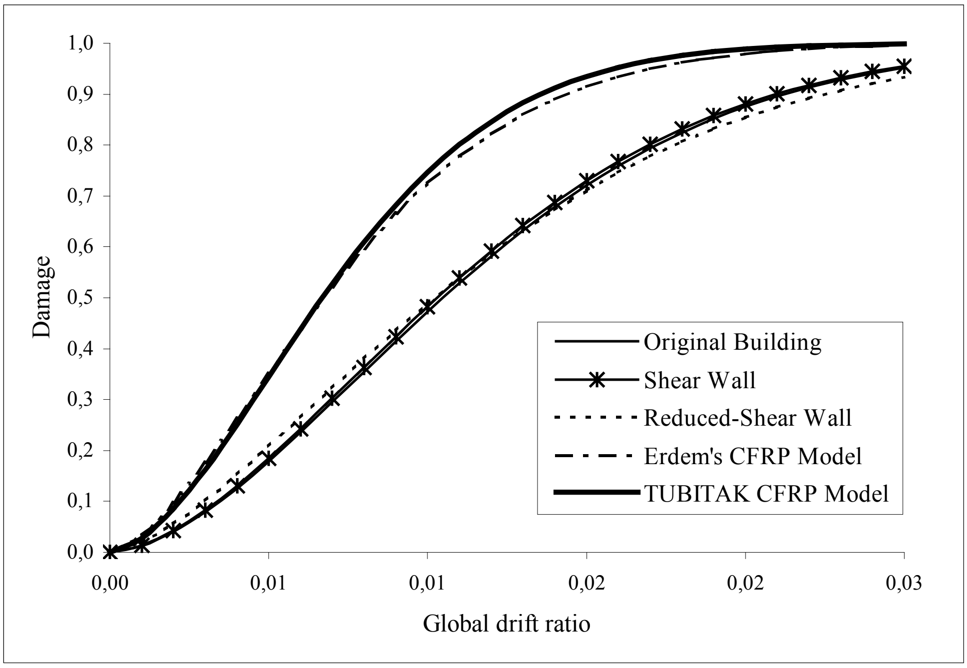

Figure 3.3 Fragility curves established for Building-1 (x-direction)

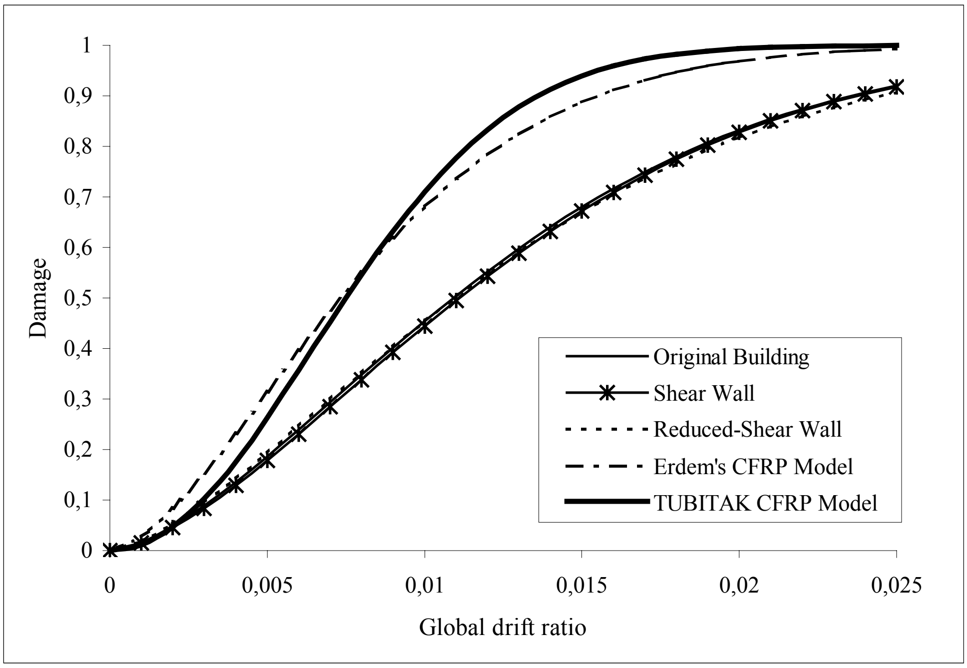

Figure 3.4 Fragility curves established for Building-1 (y-direction)

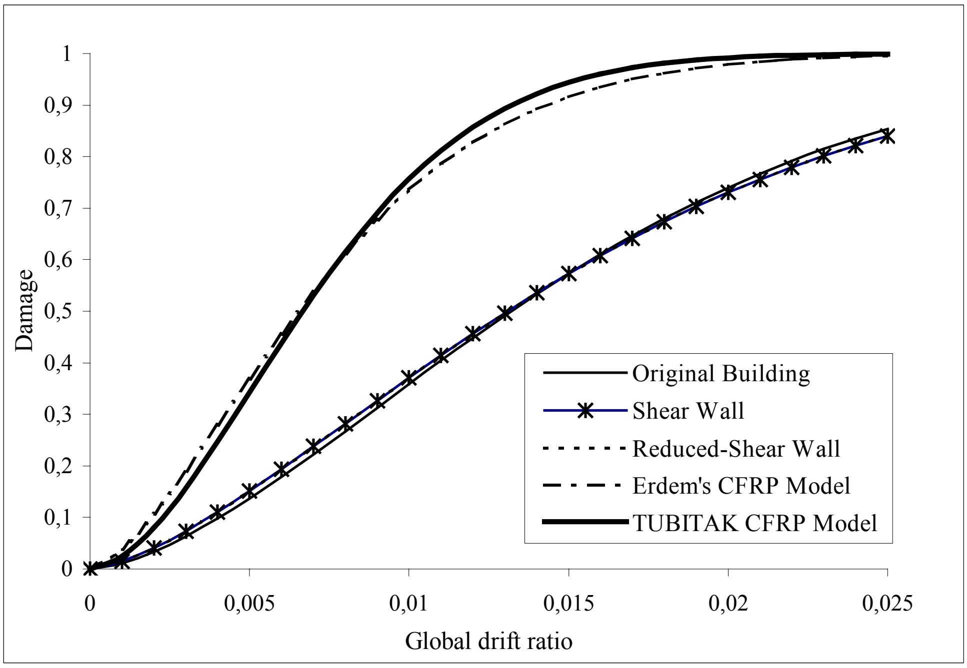

Figure 3.5 Fragility curves established for Building-2 (x-direction)

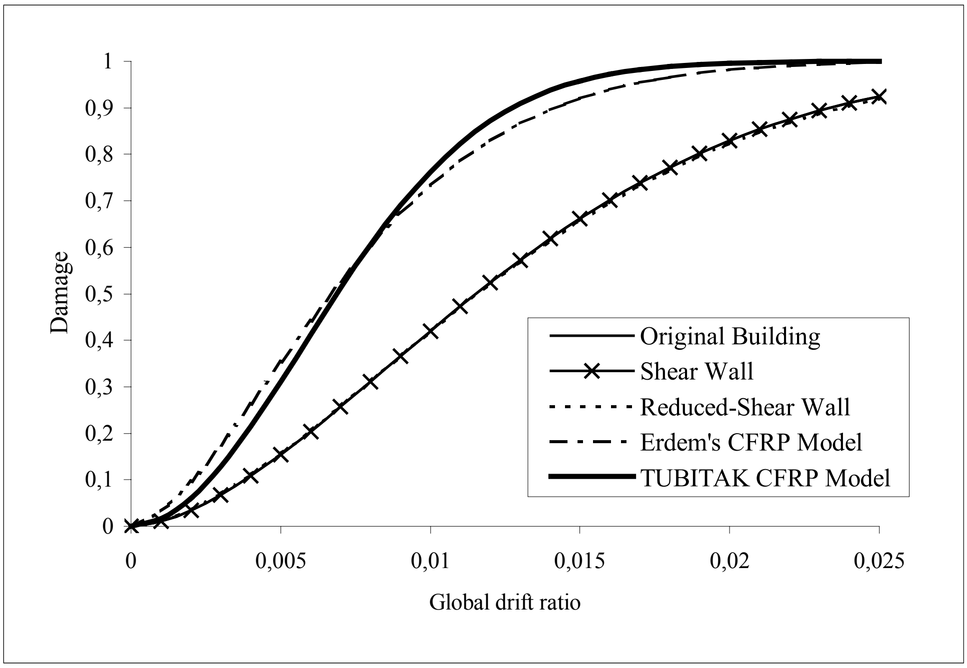

Figure 3.6 Fragility curves established for Building-2 (y-direction)

A recent official notification including the approximate unit cost values for separately categorized structures for the year 2004 was published by Ministry of Public Works and Settlement [18]. The replacement values of the buildings investigated were calculated using these approximate unit costs given in the notification. Approximate unit cost values corresponding to the buildings investigated was same. Both of the buildings are classified in the third category as B Class Structures and approximate unit cost assigned for this class of structures is 322,000,000 TL/m^2. Estimated replacement values for each building are given in Table 3.13.

Table 3.13 Replacement values of buildings

Building	Replacement value (TL)
Building-1	421,820,000,000 TL
Building-2	427,997,248,000 TL

The cost of damage calculated for the first alternative, i.e. buildings without any rehabilitation strategy, is the reference value in identifying the benefits of rehabilitation alternatives. Benefits were estimated as the reduced cost of damage compared with the cost of damage computed for the first alternative. The difference of the estimated cost of damage corresponding to each rehabilitation alternative and the original building was calculated as the benefit of each rehabilitation strategy.

Benefits were estimated only in monetary terms. Besides the improvement in the capacity curves, lateral drift capacities and component response of the buildings with and without rehabilitation are compared in Chapter 4. Although the number of fatalities, and injuries, and other properties are among the most important motivating factors in making decisions, these concerns were not included herein.

3.4 DETERMINATION OF COST EFFECTIVE ALTERNATIVE

Once costs of rehabilitation alternatives were estimated and benefits were determined, as the last step attractiveness of each rehabilitation alternative was evaluated in terms of both Net Present Values (NPV) and Benefit/Cost ratios (BCR). Relevant formulas used in this phase are given below:

$$NPV = B - C \tag{3.2}$$

$$BCR = B / C \tag{3.3}$$

where,

NPV: Net Present Value

B: Expected benefit attributed to the rehabilitation

C: Cost of rehabilitation alternative

The Net Present Values were calculated in monetary terms and BCR in terms of a nondimensional ratio. For the rehabilitation alternatives NPV greater than zero and BCR greater than one are considered to be cost effective alternatives. In the following chapter NPV and BCR are determined for each rehabilitation alternative applied to the selected buildings and comparisons are made through discussion of the results.

CHAPTER 4

EVALUATION OF RESULTS

In this chapter seismic performance of the case study buildings with and without implementation of the rehabilitation strategies are compared in terms of natural vibration periods, base shear versus roof displacement relationships (pushover curve) and interstory drift ratios. The computed responses were checked both for global response limits and component response limits given in ATC40 [3] to determine whether the buildings meet the life safety performance criteria or not.

The results of the cost-benefit analysis are also presented in this chapter. Consequences of both Net Present Values (NPV) and Benefit/Cost Ratios (BCR) corresponding to the rehabilitation alternatives were comparatively evaluated to determine the most reliable rehabilitation alternative.

The rehabilitation strategies employed in this study are referred by the following names; “Original Building” indicates the reference building without any rehabilitation, “Erdem’s-CFRP Model” indicates the building that was modeled using the guidelines proposed by Erdem [6], “TUBITAK-CFRP Model” represents the building rehabilitated based on the principles given in TUBITAK Report 2003/1 [7], ”Shear Wall” indicates the option for which shear walls have been introduced in the building and “Reduced-Shear Wall” designates the case in which the building was rehabilitated with insertion of shear walls whose capacity was reduced by 25 %.

4.1 EVALUATION OF SEISMIC PERFORMANCE OF THE BUILDINGS

Seismic performances of the case study buildings were obtained using nonlinear static analysis procedure as described in Chapter 2. Computed pushover curves were compared for each case. In some cases, a complete pushover curve could not be obtained due to convergence problems encountered by the computer software [5] used for the analysis. In these cases, the results were extrapolated and indicated by dotted lines in the pushover curves (Figures 4.1, 4.2, 4.11, 4.12). Furthermore, pushover curves for buildings rehabilitated with Carbon Reinforced Polymers consist of some sudden drops which were observed just after yielding. This is due to high axial load capacity of the strut models employed.

For global response evaluation, global drift ratios and interstory drift ratios were computed at the performance points from nonlinear analysis as well as for the case of the equivalent linear analysis described in the Turkish Seismic Code [27] for each rehabilitation strategy.

Plastic hinge rotations at columns and shear walls were checked at the performance points against strength and deformation limits proposed by ATC 40 [3]. Members exceeding the numerical acceptance criteria are tabulated and presented in the CD-ROM attached to the thesis.

4.1.1 BUILDING-1

4.1.1.1 COMPARISON OF PUSHOVER CURVES

The capacity curves obtained for Building-1 in the x-direction and in the y-direction are given in Figures 4.1 and 4.2, respectively. As can be seen from these figures all the strengthening techniques employed resulted in a significant increase in the strength and stiffness of Building-1. The numerical results describing the general features of these curves are tabulated in Appendix C (Tables C.1, C.2). Of the rehabilitation strategies employed, insertion of the shear walls (Shear Wall) increased the yield and ultimate base shear capacity of the building the most. This alternative achieved approximately 300 % increase in the yield base shear capacity and 200 % increase in the ultimate shear capacity in each direction.

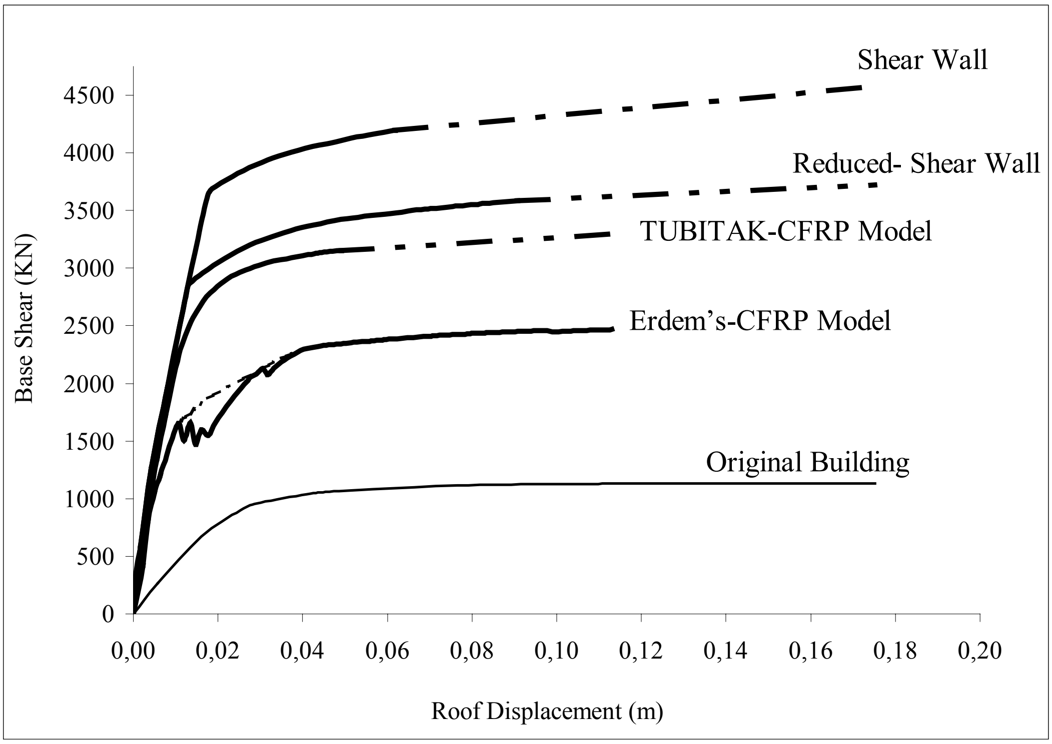

Figure 4.1 Comparison of capacity curves in the x-direction for Building 1

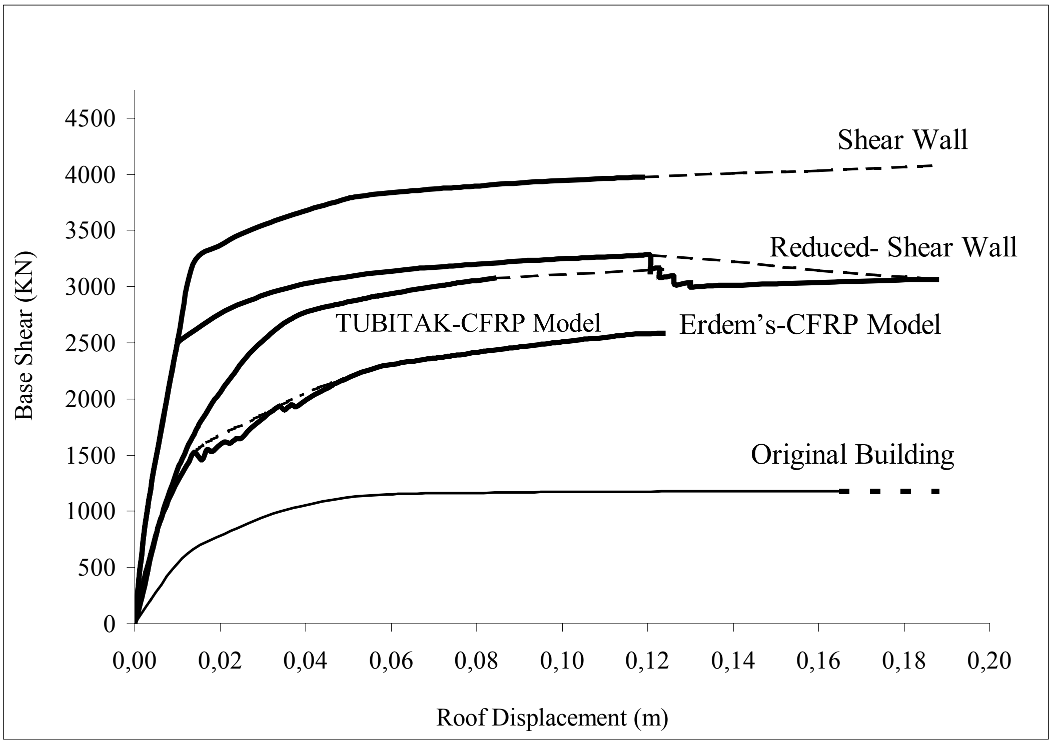

Figure 4.2 Comparison of capacity curves in the y-direction for Building 1

The initial stiffness was calculated considering the effective periods obtained in the pushover analyses. The initial stiffness of the pushover curves in the x- and y-directions corresponding to Building-1 rehabilitated with "Shear Wall" is the largest (C.3, C.4) when compared with other rehabilitation techniques and in the y-direction it is approximately twice as much as that of "TUBITAK CFRP Model" and "Erdem's CFRP Model " (Table C4).

Another conclusion that can be drawn from the numerical values shown in Tables C.3 and C.4 is that; the yield stiffness of the pushover curves corresponding to Building-1 rehabilitated with "Shear Wall" are higher than those of the other rehabilitation alternatives for both x- and y-directions.

4.1.1.2 BUILDING PERFORMANCE

Seismic performances of original and rehabilitated versions of Building-1 in both directions for code-based spectrum and site-specific spectrum are tabulated in Table 4.1. The performance point is presented in terms of the base shear force (V_p), and the roof displacement (d_p). It is worth nothing that in some cases (all cases for code spectrum) the CSM method did not yield a performance point because an intersection between the capacity curve and the demand curve could not be obtained due to extremely large demands. In such cases, evaluations were made at the last point on the capacity curve that is the performance point was taken as the last point obtained in the last step of the pushover analysis.

When performance points of each rehabilitation alternative are compared with the performance point of "Original Building", it can be seen that all rehabilitation alternatives reduced the performance point significantly. "TUBITAK CFRP Model" reduced the performance point the most under the code-based spectrum in both directions and under site-specific spectrum in the x-direction. Strengthening with "Shear Wall" reduced the performance point under site-specific spectrum in the y-direction, the weak direction.

Table 4.1 Seismic performance of Building-1

Rehabilitation Strategy	Code-based spectrum				Site-specific spectrum			
	x-direction		y-direction		x-direction		y-direction	
	V_p (KN)	d_p (m)	V_p (KN)	d_p (m)	V_p (KN)	d_p (m)	V_p (KN)	d_p (m)
Shear Wall	4,196.15*	0.062*	3,976.32*	0.119*	4,196.15*	0.062*	3,879.47	0.073
Reduced-Shear Wall	3,586.31*	0.091*	3,066.05*	0.188*	3,540.24	0.077	3,213.56	0.085
TUBITAK CFRP Model	3,154.26*	0.049*	3,073.04*	0.084*	3,154.26*	0.049*	3,030.39	0.075
Erdem's CFRP Model	2,467.69*	0.113*	2,585.73*	0.124*	2,445.50	0.088	2,470.91	0.092
Original Building	1,132.56*	0.175*	1,177.62*	0.165*	1,132.56*	0.175*	1,177.62*	0.165*

* An intersection point could not be found; performance point was taken as the last point.

Table 4.2 shows the periods obtained for the first mode shapes at the performance point; modal participation factors (PF), effective modal mass coefficients (α) and the total weight for each rehabilitation strategy in the x- and y-directions. The periods of Original Building is about twice as much as that of all rehabilitation strategies in two directions. The minimum period was obtained for Building-1 strengthened with shear wall in the y-direction due to significant increase in stiffness. Besides strengthening the building with CFRP (Erdem's CFRP Model) produced the minimum period in the x-direction.

Table 4.2 Modal properties for Building-1

Rehabilitation Strategy	**T_x (sec)**	**T_y (sec)**	**(PF_x)**	**(PF_y)**	**(α_x)**	**(α_y)**	**Weight (KN)**
Original Building	0.95	0.80	1.229	1.342	0.886	0.803	16,007.29
TUBITAK CFRP Model	0.44	0.51	1.317	1.341	0.819	0.779	16,007.29
Erdem's CFRP Model	0.34	0.45	1.196	1.282	0.891	0.853	16,007.29
Reduced-Shear Wall	0.36	0.39	1.377	1.391	0.742	0.812	19,769.99
Shear Wall	0.36	0.39	1.381	1.384	0.782	0.795	19,769.99

All rehabilitation measures effected the ductility of the building significantly. The ductility ratios were obtained from the capacity curves using the projected ultimate points.

The building rehabilitated with “Reduced-Shear Wall” has the largest ductility in the x- and y-directions when compared with the other rehabilitation strategies (Table 4.3). In the x-direction ductility of “Reduced-Shear Wall” is almost twice as much as that of the original building. Of the rehabilitation strategies “TUBITAK CFRP Model” has the lowest ductility in the y-direction.

Table 4.3 Ductility of Building-1

Rehabilitation Strategy	μ_x **(Δ u/Δ y)**	μ_y **(Δ u/Δ y)**
Original Building	6.96	13.21
Shear Wall	9.63	13.79
Reduced-Shear Wall	13.18	18.57
Erdem’s CFRP Model	10.56	8.83
TUBITAK CFRP Model	7.20	4.29

4.1.1.3 GLOBAL BUILDING RESPONSE

Story drifts for Building-1 under code-based and site-specific spectra in both x- and y-directions are illustrated through Figures 4.3-4.6. In general strengthening with “Shear Wall” reduced story drifts in first three floor levels and strengthening with “TUBITAK CFRP Model” reduced the story drifts in the upper floor levels except for code-based spectrum in y-direction. For this case “TUBITAK CFRP Model” reduced story drifts at all floor levels. The displacement profile of Building-1 in the y-direction is unusual because the roof displacement in the case with shear wall (Reduced Shear Wall) appears to be more than that in the Original Building. This consequence is essentially due to the code-based spectrum with large demands that cannot be accommodated by any of the systems. It is worth mentioning that the strength reduction factor (R) was taken as 1 for establishing code-based spectrum and this code-spectrum was used here for evaluation purposes not design. In such cases, the performance point was taken as the ultimate point leading to a smaller ultimate roof deformation of the original building than that of the “Reduced-Shear Wall” alternative indicating that the original building is less ductile (Figure 4.2). This condition was only observed for Building-1 in the case of the code-based spectrum.

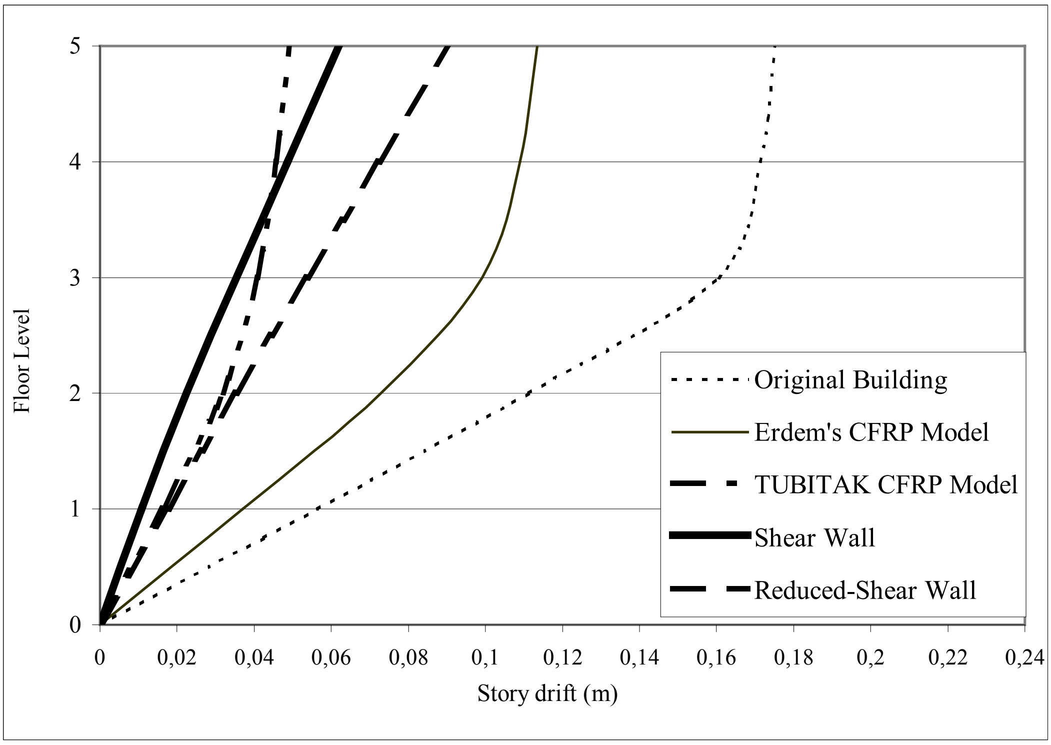

Figure 4.3 Story drifts for Building-1 under code-based spectrum in the x-direction

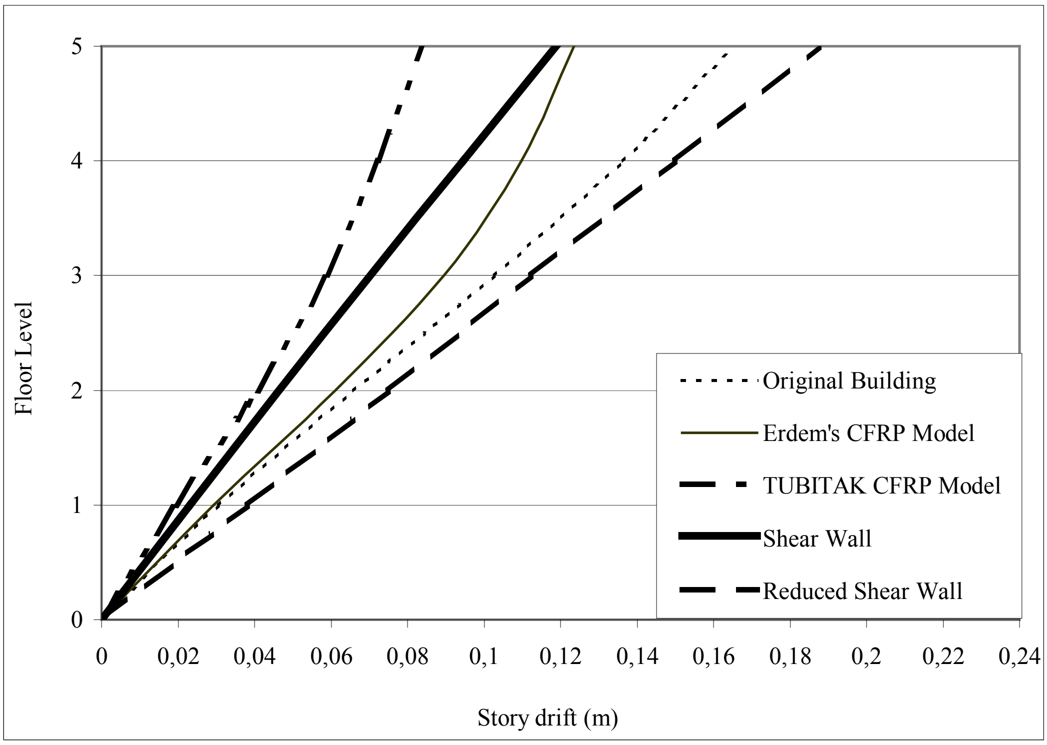

Figure 4.4 Story drifts for Building-1 under code-based spectrum in the y-direction

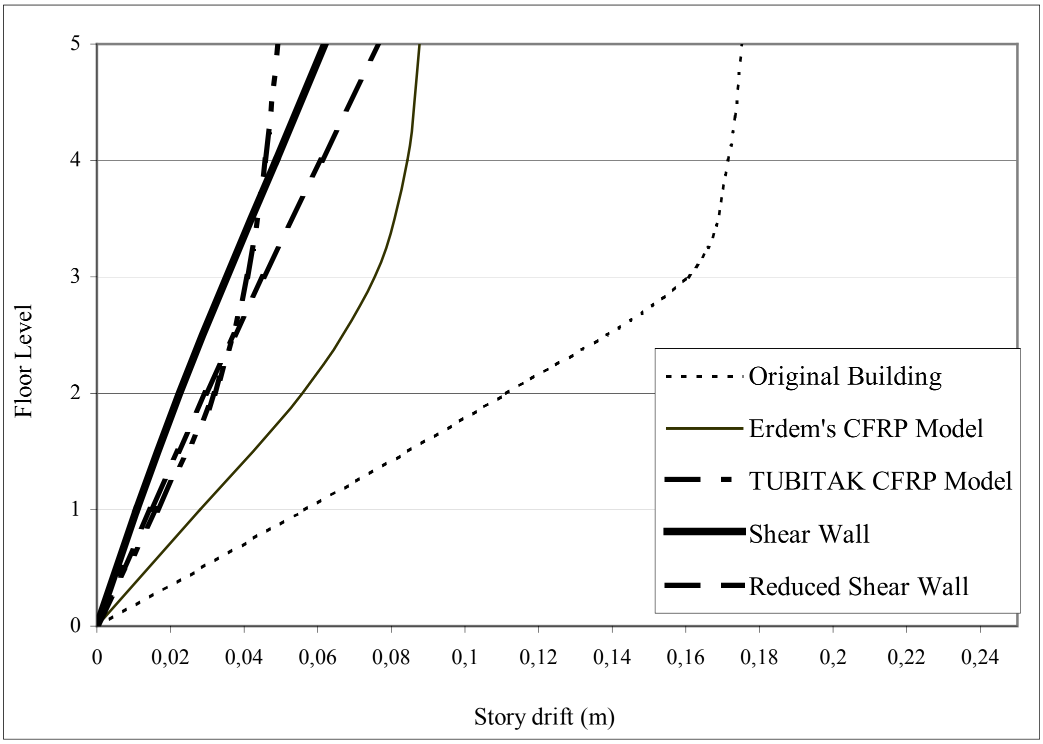

Figure 4.5 Story drifts for Building-1 under site-specific spectrum in the x-direction

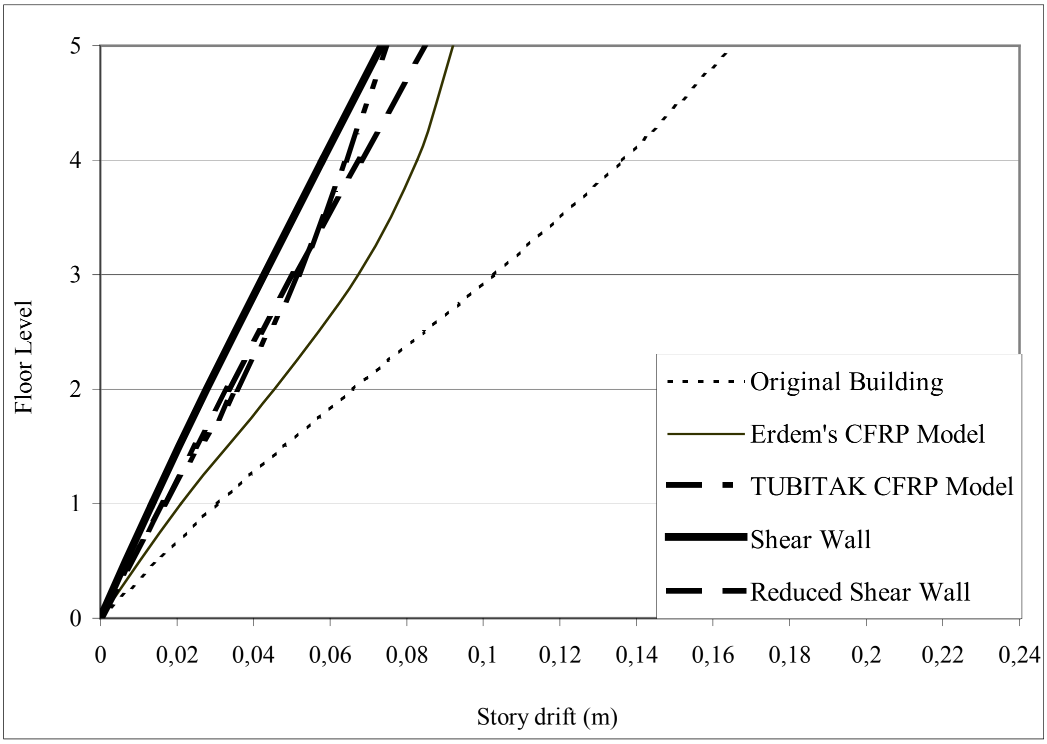

Figure 4.6 Story drifts for Building-1 under site-specific spectrum in the y-direction

Tables 4.4 and 4.5 give the global drift ratios in percentages. In these tables "C" designates conforming cases and "NC" designates nonconforming cases with respect to the modified ATC-40 [3] criteria described in Chapter 2. All the global drift ratios determined meet acceptability limit of 1.4 % given for life safety in Chapter 2. Significant reductions in global drift ratio were sustained by all rehabilitation alternatives.

Table 4.4 Global drift ratios of Building-1 under code-based spectrum

Rehabilitation Strategy	Roof Drift (m)		Global Drift Ratio (%)		Acceptability	
	x-direction	y-direction	x-direction	y-direction	x-direction	y-direction
Original Building	0.175	0.165	1.23	1.16	C	C
Erdem's CFRP Model	0.113	0.124	0.80	0.87	C	C
Reduced-Shear Wall	0.091	0.188	0.64	1.32	C	C
Shear Wall	0.062	0.119	0.44	0.84	C	C
TUBITAK CFRP Model	0.049	0.084	0.35	0.59	C	C

Table 4.5 Global drift ratios of Building-1 under site-specific spectrum

Rehabilitation Strategy	Roof Drift (m)		Global Drift Ratio (%)		Acceptability	
	x-direction	y-direction	x-direction	y-direction	x-direction	y-direction
Original Building	0.175	0.165	1.23	1.16	C	C
Erdem's CFRP Model	0.088	0.092	0.62	0.65	C	C
Reduced-Shear Wall	0.077	0.085	0.54	0.60	C	C
Shear Wall	0.062	0.073	0.44	0.52	C	C
TUBITAK CFRP Model	0.049	0.075	0.35	0.53	C	C

As can be seen from the tables, the building rehabilitated with "TUBITAK CFRP Model" reduced the global drift ratio the most under code-based spectrum in both directions and under site-specific spectrum in the x-direction. On the other hand drift ratio corresponding to the building rehabilitated with "Shear Wall" is smaller than that of the other alternatives under site-specific spectrum in the y-direction. An important point that needs to be highlighted is that even the ATC-40 [3] modified limits employed here are quite large for Turkish buildings that have limited deformation capacity as illustrated by conformance of all cases in Tables 4.4 and 4.5. Therefore, this evaluation alone does not seem to be appropriate especially for the buildings which are far from resisting the anticipated demands (original building in this case) but given here for the sake of completeness. Moreover, the rehabilitation based on CFRP is a nonductile application and might be better suited for force-based evaluations.

Figures 4.7 and 4.8 illustrate interstory drift ratios of Building-1 under code-based spectrum and Figures 4.9 and 4.10 illustrate interstory drift ratios under site-specific spectrum for x- and y-directions, respectively. As can be observed from the figures, generally insertion of shear walls reduced the interstory drift ratios in first two floors the most. However strengthening with CFRP reduced the Interstory drift ratios in the upper floors. Also results reveal that only under code-based spectrum in the y-direction strengthening with CFRP reduced the interstory drift ratios in all floor levels.

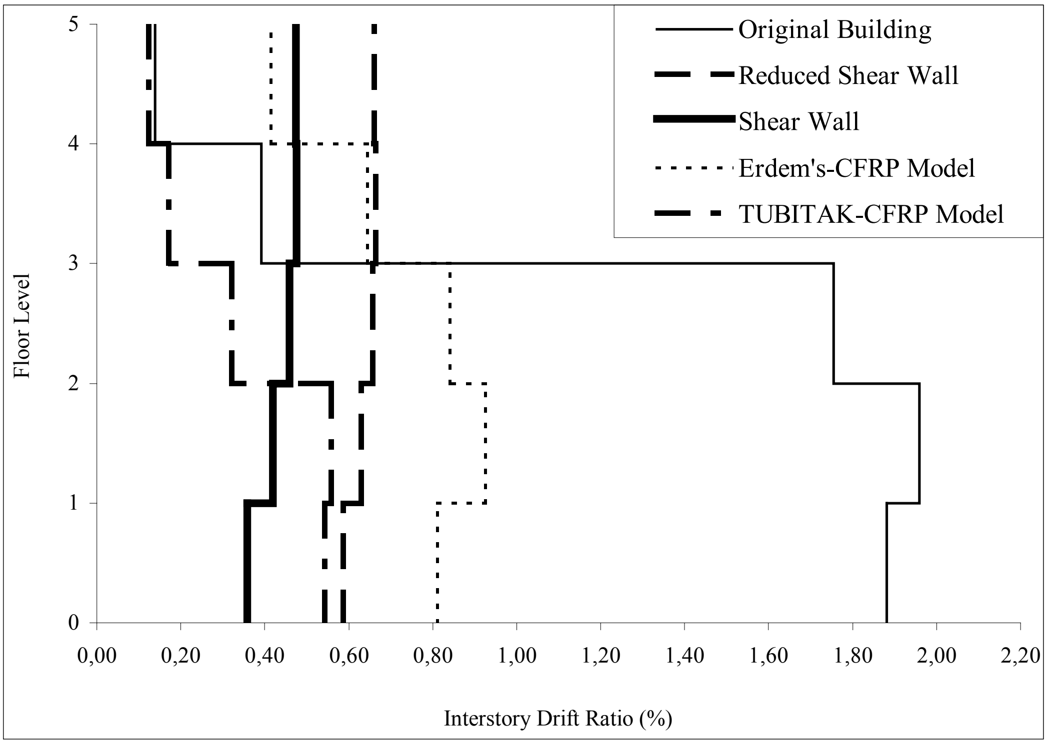

Figure 4.7 Interstory drift ratios of Building-1 under code-based spectrum in the x-direction

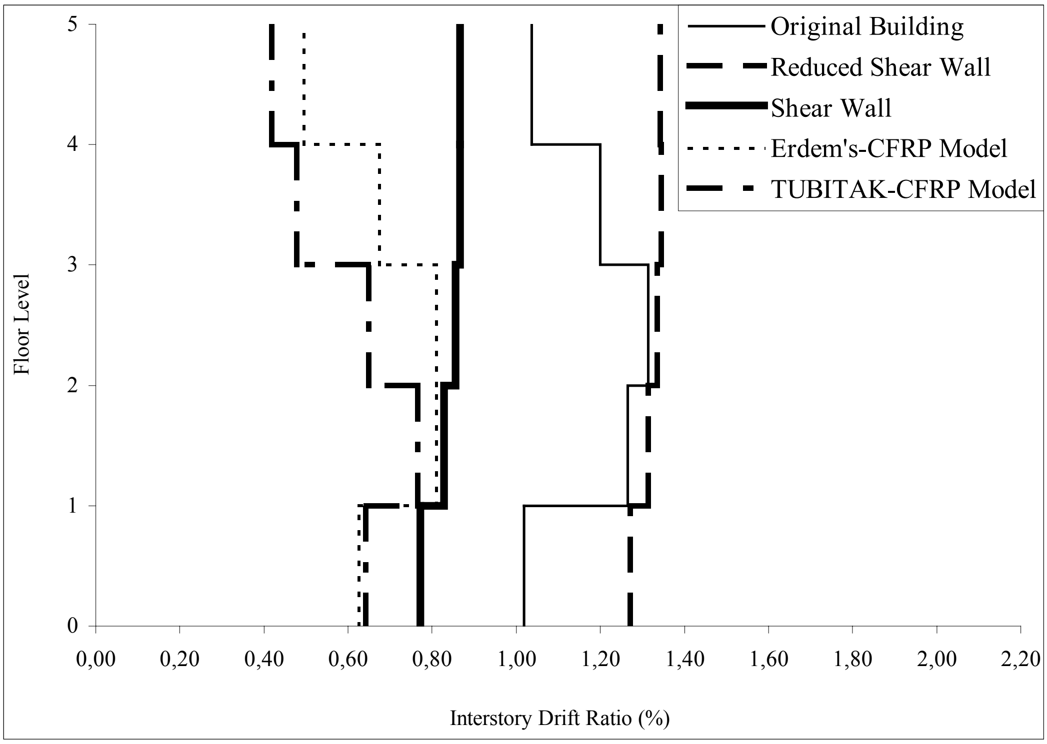

Figure 4.8 Interstory drift ratios of Building-1 under code-based spectrum in the y-direction

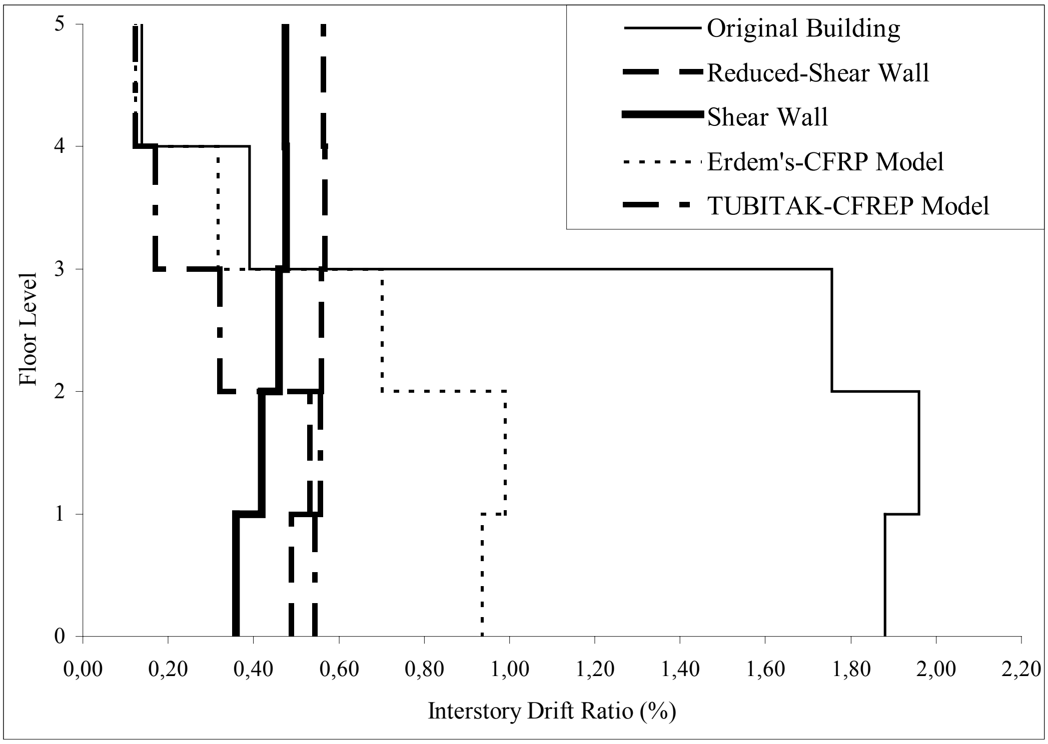

Figure 4.9 Interstory drift ratios of Building-1 under site-specific spectrum in the x-direction

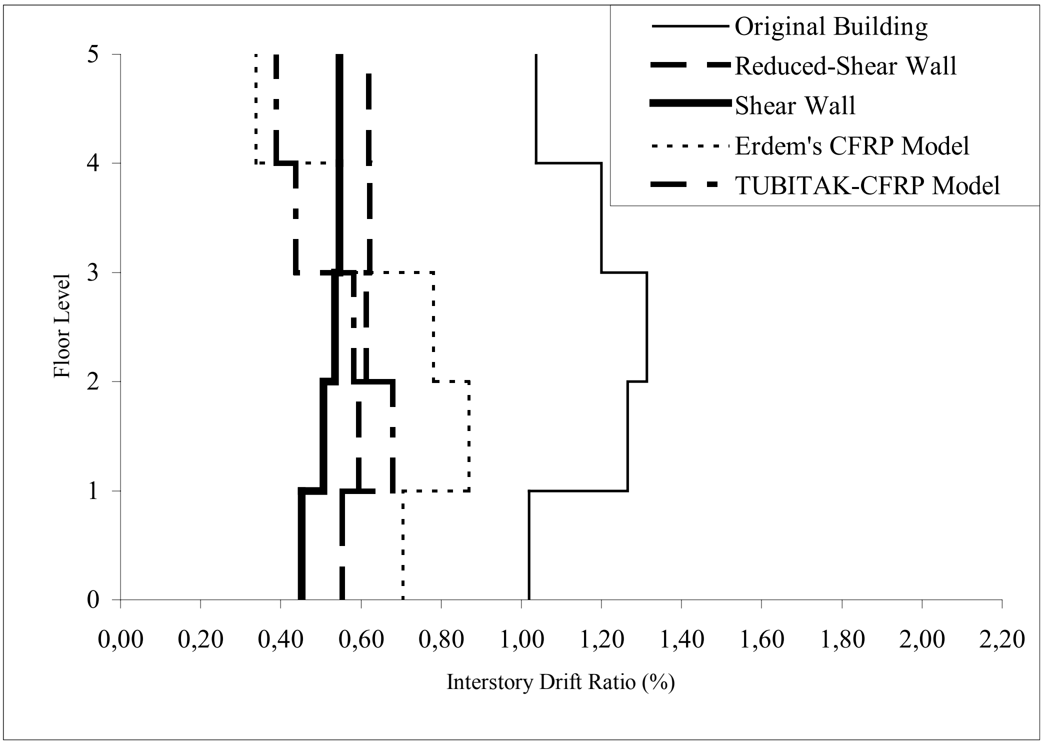

Figure 4.10 Interstory drift ratios of Building-1 under site-specific spectrum in the y-direction

Comparison of the maximum interstory drift ratios is given in Appendix C (Table C.5 and Table C.6). These tables demonstrate that in the x-direction the original building does not meet the acceptability limit under either of the spectra. Maximum drifts were generally obtained at the 2nd floor level for the original building and the building strengthened with CFRP. However they are the largest at the 4th floor level for the building rehabilitated with shear walls. All strengthening techniques reduced maximum interstory drift ratios to reasonable values and all meet the acceptability limits for life safety performance level. In general strengthening with "Shear Wall" reduced maximum interstory drift ratios the most, except strengthening with "TUBITAK CFRP Model" under code-based spectrum in the y-direction.

Interstory drift ratios calculated with implementing linear static analysis are presented in Appendix C (Figures C.1 and C.2). In the x-direction strengthening with "Shear Wall" reduced the elastic interstory drift ratios in first two floors whereas strengthening with "TUBITAK CFRP Model" reduced the interstory drift ratios in upper floors. The elastic interstory drift ratios in the y-direction are the lowest for strengthening with "Shear Wall".

4.1.1.4 COMPONENT BUILDING RESPONSE

The component check results for Building-1 and Building-2 under the two considered spectra in both x- and y-directions were evaluated at their performance points. The evaluations were made for the two cases specified in ACI 318 [2], namely conforming reinforcement and nonconforming reinforcement cases. Due to high volume of the results, tables that contain member level evaluations are made available in the disk attached to the thesis. The tables illustrate which elements satisfy the specified acceptability limits and which do not. The symbol "X" indicates the components that do not meet the acceptability limits and "√" indicates the components that are acceptable. Additionally, the total number of components that failed is identified through Tables C.7-C.14 for Building-1 and Tables C.21-C.28 for Building-2 in Appendix C. When these tables are investigated it is obvious that in most cases the total number of failing members are reduced the most when strengthening with shear wall technique is employed.

Another conclusion observed from the tables is that all rehabilitation alternatives adopted here provide a significant improvement over the response of the building. The most efficient alternatives seem to be insertion of the shear wall and use of TUBITAK models for the CFRP rehabilitated buildings.

The overall evaluation of results and discussions for Building-1 indicate that there are substantial differences in the response of the building between the selected rehabilitation and analysis methods. Inspection of the displacement profiles reveal that the behavior mode of the buildings rehabilitated with shear wall is quite different from that when CFRP is employed. The application of CFRP results in smaller drifts and drift ratios at upper floors whereas insertion of shear walls imposes a rather uniform drift throughout the building. In general, the profiles obtained from CFRP rehabilitated cases are similar and closer to the behavior of the original building.

4.1.2 BUILDING-2

4.1.2.1 COMPARISON OF PUSHOVER CURVES

Figures 4.11 and 4.12 reveal that the performance of Building-2 strengthened with CFRP is not as favorable as that of Building-2 strengthened with shear walls. Both in the x- and y-directions capacity curves show that the strength increase attained for Building-2 rehabilitated with shear walls is relatively high compared to the case with CFRP as far as change in V_y (yield base shear force) and V_u (ultimate base shear force) are concerned

(Tables C.15, C.16). The ultimate and the yield strength of “Shear Wall” is approximately twice as much as that of “TUBITAK CFRP Model”.

Among the rehabilitation strategies, insertion of the shear walls (Shear Wall) resulted a significant increase in yield and ultimate base force capacity of the building. Moreover strengthening with shear walls increased both the initial and yield lateral stiffness of the building. These observations are similar to those of Building-1 except that strengthening Building-2 with “Shear Wall” resulted in relatively more significant increase in both lateral strength and stiffness as compared with Building-1. The major factor leading to this is the differences in the amounts of shear walls inserted. As mentioned before the density of the shear walls in Building-2 is relatively higher than that of Building-1 in both directions.

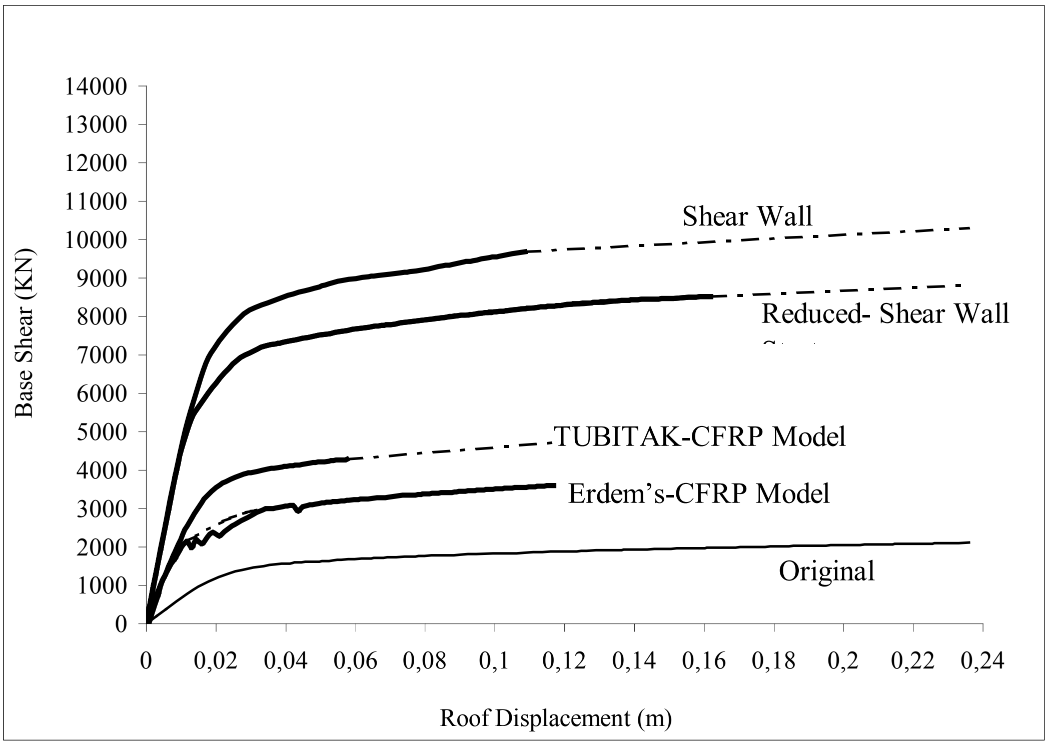

Figure 4.11 Comparison of capacity curves in the x-direction for Building 2

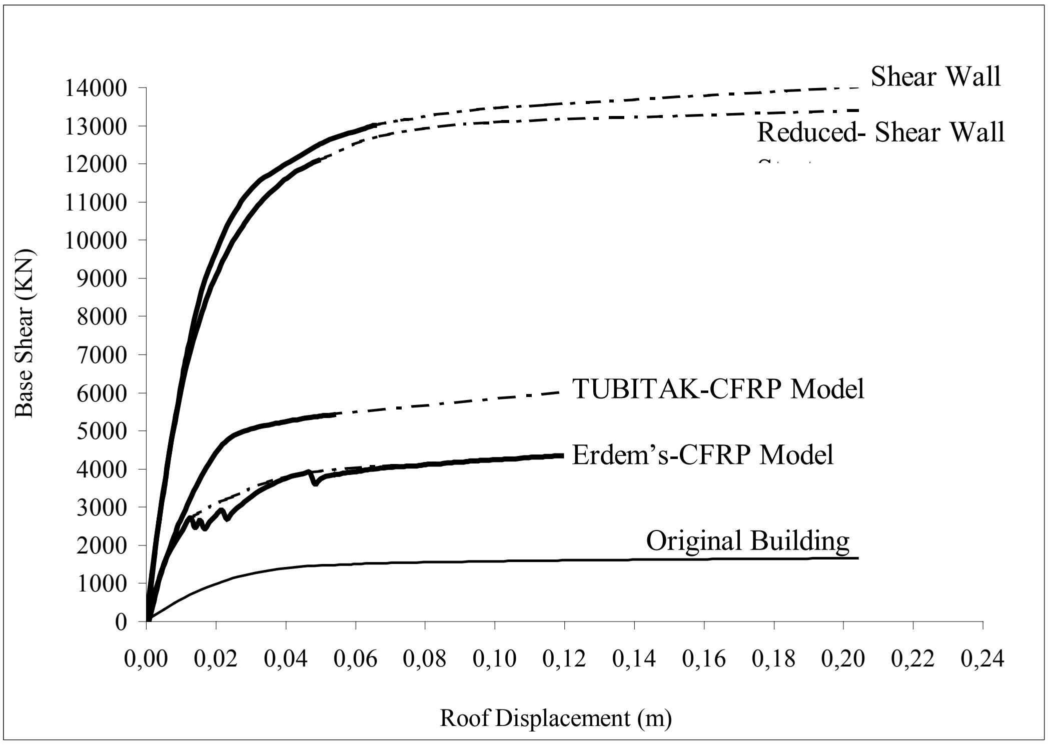

Figure 4.12 Comparison of capacity curves in the y-direction for Building 2

4.1.2.2 BUILDING PERFORMANCE

Performance points of Building-2 with and without rehabilitations for the considered spectra are given for comparison in Tables 4.6. Strengthening with shear wall improved seismic behavior of Building-2 significantly in all cases. As can be seen from the tables among all strengthening techniques the most effective technique to reduce the performance point of the building is strengthening the building using shear walls.

Table 4.6 Performance of Building-2

Rehabilitation Strategy	Code-based spectrum				Site-specific spectrum			
	x-direction		y-direction		x-direction		y-direction	
	V_p (KN)	d_p (m)	V_p (KN)	d_p (m)	V_p (KN)	d_p (m)	V_p (KN)	d_p (m)
Shear Wall	8,298.13	0.033	10,144.4	0.022	6,790.18	0.017	6,724.62	0.011
Reduced-Shear Wall	7,456.77	0.046	9,968.33	0.025	6,008.70	0.018	6,600.93	0.011
TUBITAK CFRP Model	4,281.93*	0.058*	5,397.32	0.052	4,003.68	0.033	4,817.51	0.024
Erdem's CFRP Model	3,601.47	0.117	4,205.63	0.094	3,258.27	0.064	3,866.99	0.055
Original Building	2,110.14*	0.236*	1,660.17*	0.204*	1,852.06	0.109	1,609.69	0.125

* An intersection point could not be found; performance point was taken as the last point.

Table 4.7 presents dynamic characteristics and the total weights of each rehabilitation strategy applied to Building-2. The minimum period was obtained for the building strengthened with shear wall in both directions due to significant increase in stiffness. Additionally the periods of the building strengthened with "Erdem's CFRP Model" in both x- and y-directions are smaller than that of building rehabilitated with "TUBITAK CFRP Model".

Table 4.7 Modal properties for Building-2

Rehabilitation Strategy	**T_x (sec)**	**T_y (sec)**	**(PF_x)**	**(PF_y)**	**(α_x)**	**(α_y)**	**Weight (KN)**
Original Building	0.84	0.92	1.256	1.301	0.775	0.82	18,913.76
TUBITAK CFRP Model	0.48	0.43	1.322	1.326	0.768	0.77	18,913.76
Erdem's CFRP Model	0.39	0.33	1.205	1.229	0.873	0.866	18,913.76
Reduced-Shear Wall	0.32	0.26	1.354	1.398	0.774	0.723	21,065.52
Shear Wall	0.32	0.26	1.395	1.400	0.705	0.724	21,065.52

Insertion of shear wall enhanced the ductility behavior of the building in the x-direction considerably (Table 4.8). The building strengthened with CFRP (Erdem's CFRP Model) showed the largest computed displacement ductile factor in the y-direction among other rehabilitation alternatives.

Table 4.8 Ductility of Building-2

Rehabilitation Strategy	**μ_x (Δ u/Δ y)**	**μ_y (Δ u/Δ y)**
Original Building	9.87	7.29
Shear Wall	13.80	7.23
Reduced-Shear Wall	15.52	7.67
Erdem's CFRP Model	11.13	9.44
TUBITAK CFRP Model	5.85	4.96

4.1.2.3 GLOBAL BUILDING RESPONSE

Figures 4.13-4.16 clearly show that among all strengthening techniques response of Building-2 rehabilitated with "Shear Wall" is the most effective one. It significantly reduced the story drifts and improved the behavior of lateral load resistance of the building under code-based and site-specific spectrums in both directions. There is no significant difference in the response between the cases with shear walls and with reduced shear wall capacities in the y-direction. This is probably due to the large number of shear walls that go through progressive yielding and benefit from redistribution.

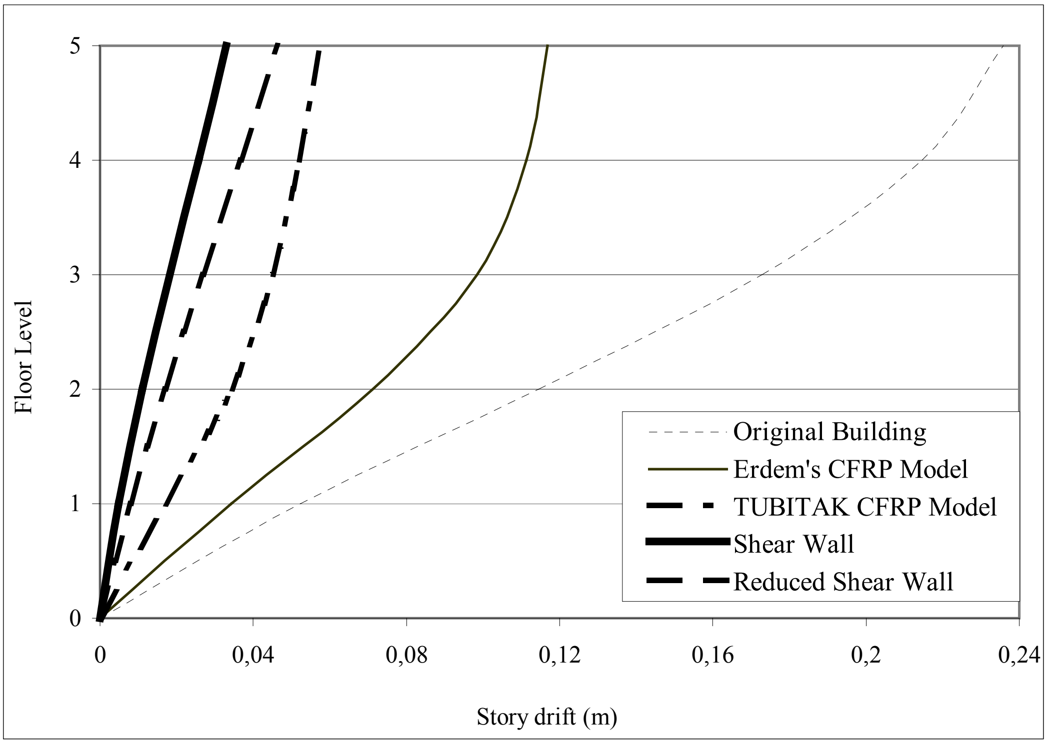

Figure 4.13 Story drifts for Building-2 under code-based spectrum in the x-direction

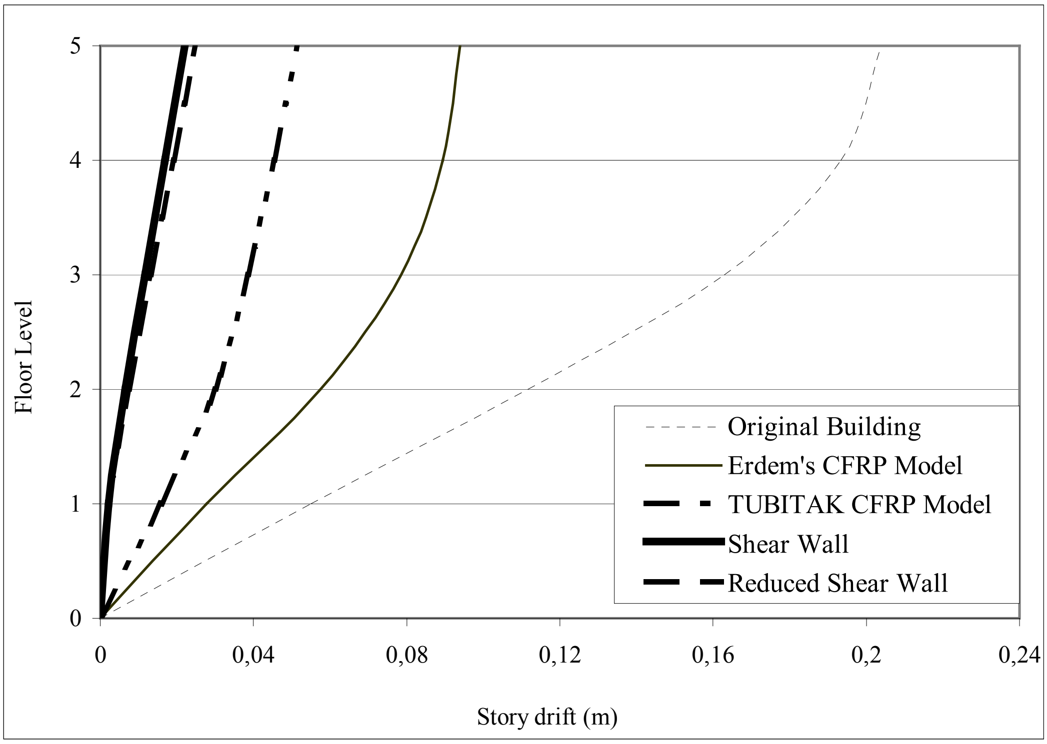

Figure 4.14 Story drifts for Building-2 under code-based spectrum in the y-direction

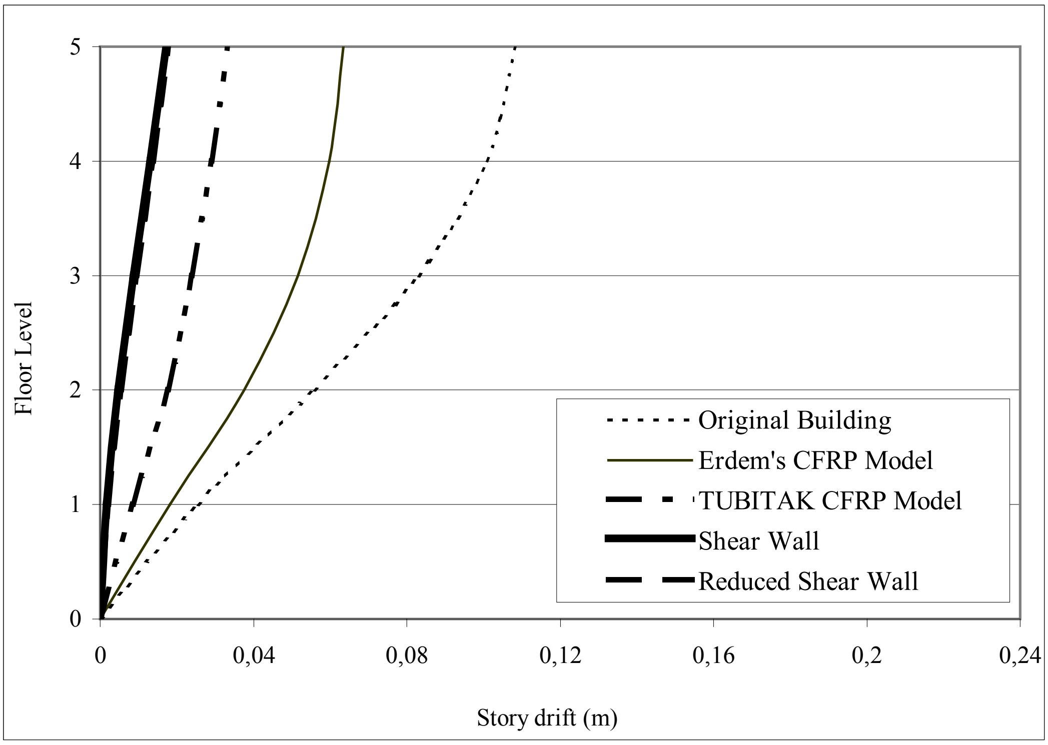

Figure 4.15 Story drifts of Building-2 under site-specific spectrum in the x-direction

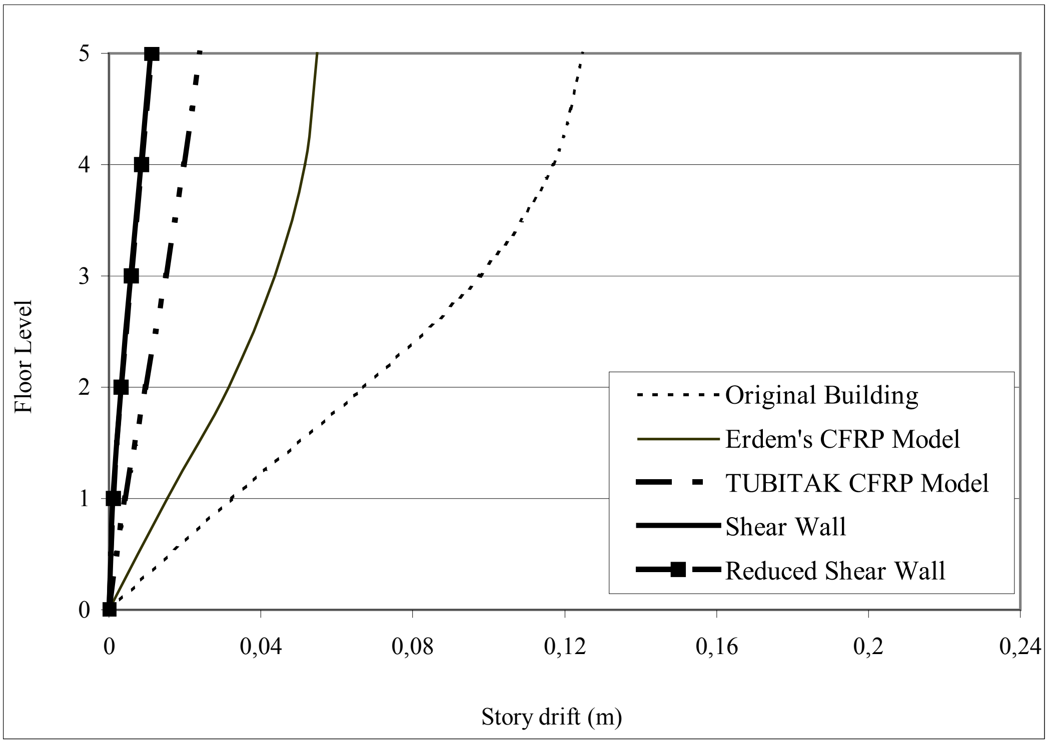

Figure 4.16 Story drifts for Building-2 under site-specific spectrum in the y-direction

Tables 4.9 and 4.10 present the roof drifts and the evaluation of the building considering these drifts. When the drift ratios given in these tables are compared, due to large allowable deformation limits, even the "Original Building" seems to satisfy the acceptability criterion except under the code-based spectrum in the x-direction. All strengthening methods meet the acceptability criterion. Among all techniques "Shear Wall" reduced the global drift ratio to the smallest value in all cases. It can be also noted that, the global drift ratios of the building under code-based spectrum is about twice as much as that under site-specific spectrum for all alternatives in the x- and y-directions. This difference can be observed clearly in Figures 4.13-4.16.

Table 4.9 Global drift ratios of Building-2 under code-based spectrum

Rehabilitation Strategy	Roof Drift (m)		Global Drift Ratio (%)		Acceptability	
	x-direction	y-direction	x-direction	y-direction	x-direction	y-direction
Original Building	0.236	0.204	1.57	1.36	NC	C
Erdem's CFRP Model	0.117	0.094	0.78	0.63	C	C
TUBITAK CFRP Model	0.058	0.052	0.38	0.34	C	C
Reduced-Shear Wall	0.046	0.025	0.31	0.17	C	C
Shear Wall	0.033	0.022	0.22	0.15	C	C

Table 4.10 Global drift ratios of Building-2 under site-specific spectrum

Rehabilitation Strategy	Roof Drift (m)		Global Drift Ratio (%)		Acceptability	
	x-direction	y-direction	x-direction	y-direction	x-direction	y-direction
Original Building	0.108	0.125	0.72	0.83	C	C
Erdem's CFRP Model	0.063	0.055	0.42	0.37	C	C
TUBITAK CFRP Model	0.033	0.024	0.22	0.16	C	C
Reduced-Shear Wall	0.018	0.011	0.12	0.08	C	C
Shear Wall	0.017	0.011	0.11	0.07	C	C

Figures 4.17 through 4.20 present interstory drift ratios calculated for all strategies at the performance points. As is evident from the figures, "Shear Wall" and "Reduced-Shear Wall" give similar results. Similar to Building-1, it is confirmed that the insertion of shear wall reduced interstory drifts in the first three floor levels whereas strengthening with CFRP reduced interstory drifts in upper floors.

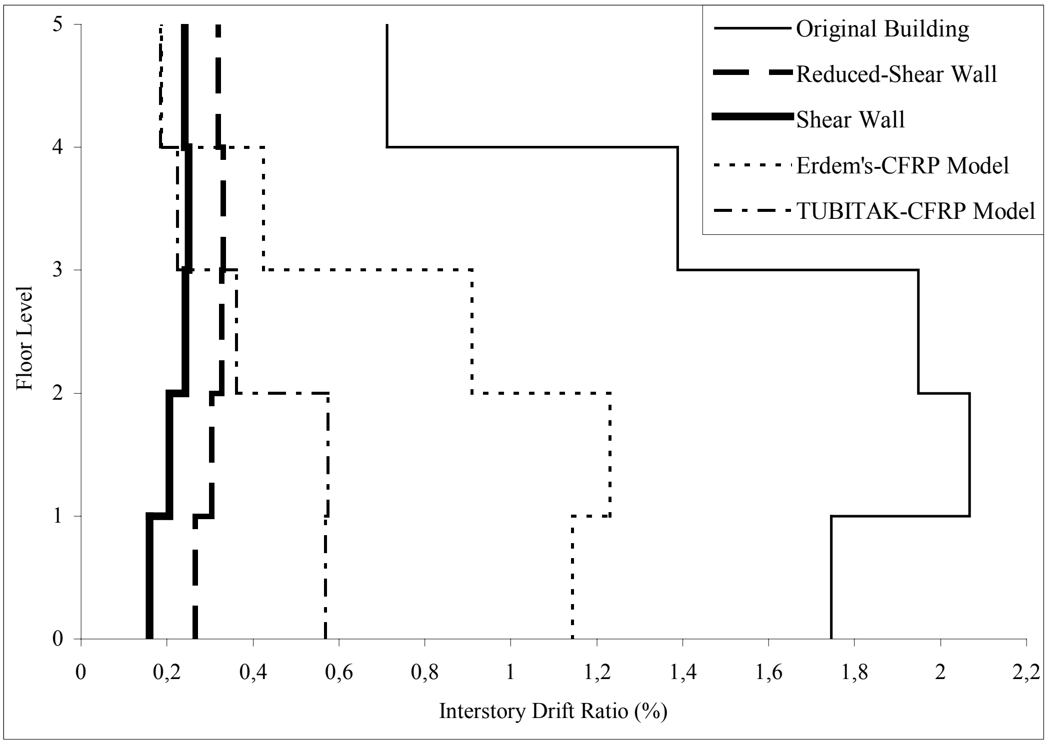

Figure 4.17 Interstory drift ratios of Building-2 under code-based spectrum in the x-direction

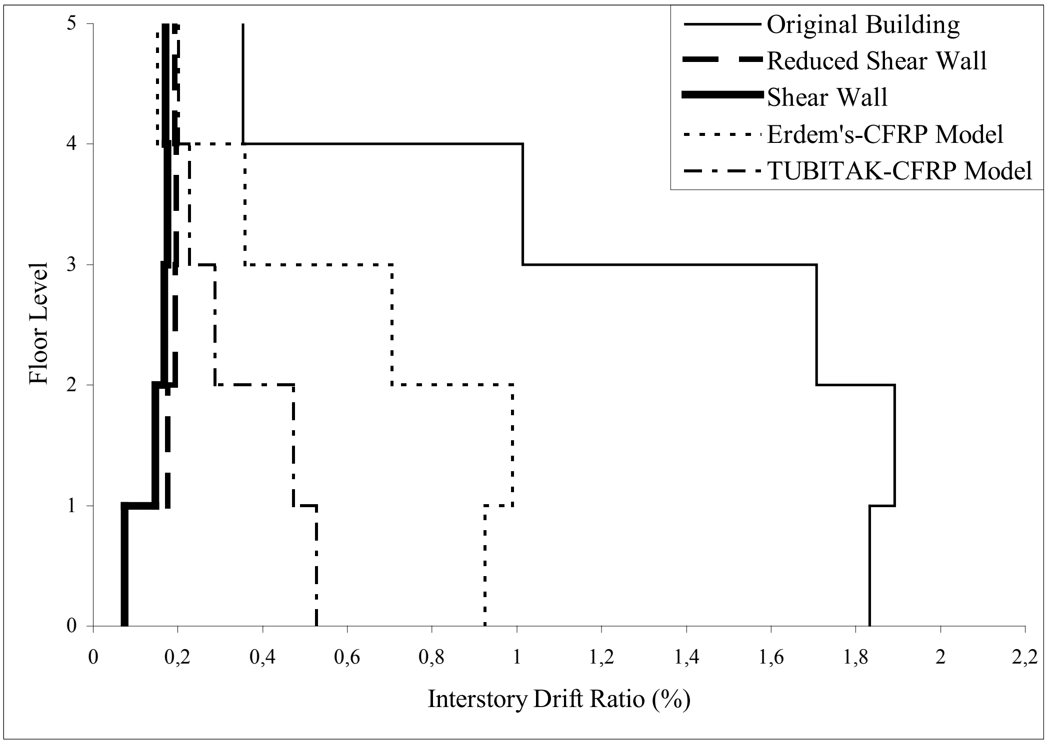

Figure 4.18 Interstory drift ratios of Building-2 under code-based spectrum in the y-direction

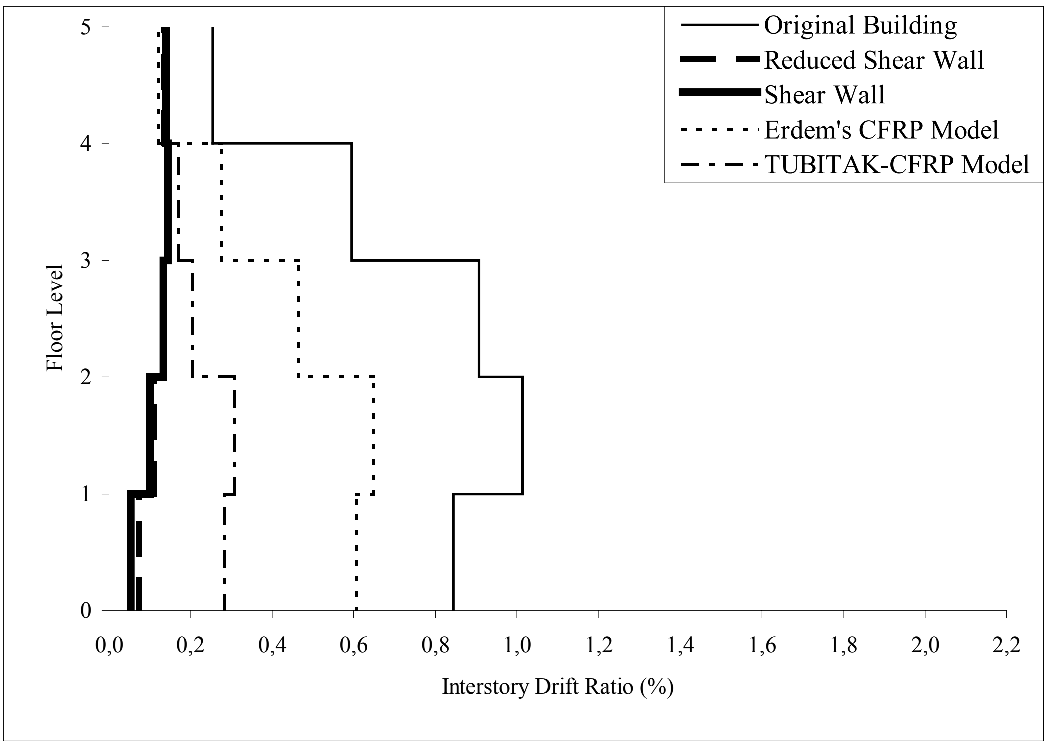

Figure 4.19 Interstory drift ratios of Building-2 under site-specific spectrum in the x-direction

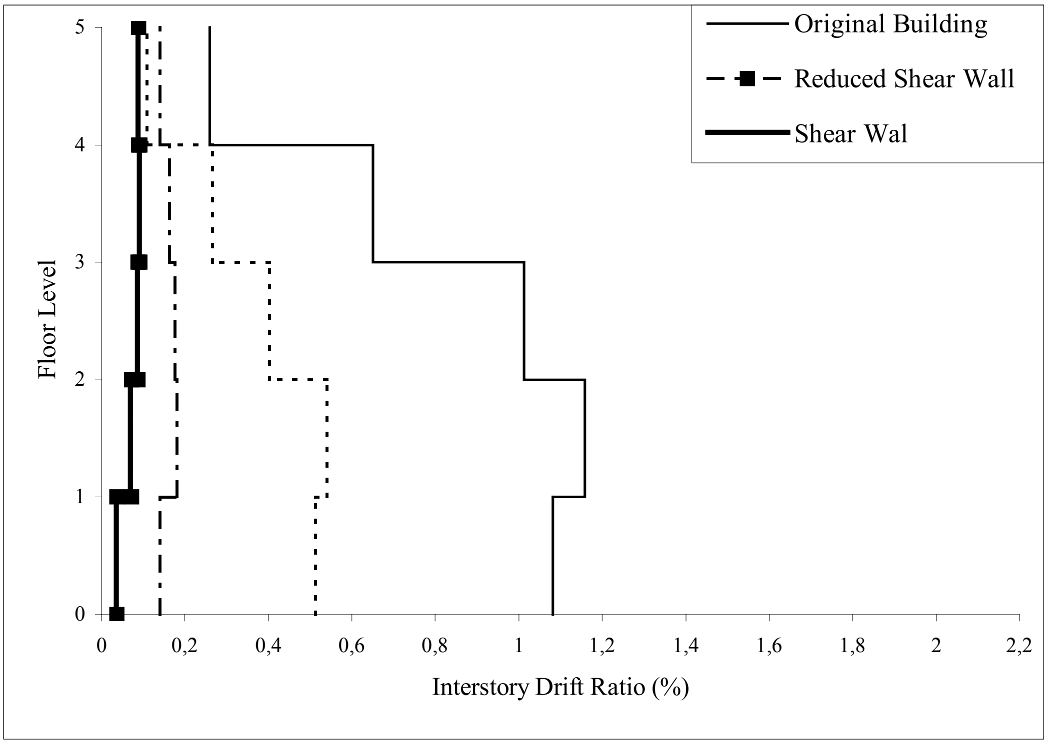

Figure 4.20 Interstory drift ratios of Building-2 under site-specific spectrum in the y-direction

The results of the linear static analyses performed for Building-2 according to the Turkish Seismic Code are given in Appendix C (Figures C. 3 and C. 4). These plots demonstrate that strengthening with "Shear Wall" is the most efficient alternative in reducing the elastic interstory drift ratios.

The maximum Interstory drift ratios that were calculated for Building-2 are given in Tables C.19, C.20 in Appendix C. Under the code-based spectrum the calculated maximum interstory drift ratios for "Original Building" do not conform to the acceptability limit. Similar to Building-1 the maximum drift ratios are obtained at 2^{nd} floor level for strengthening with CFRP and at 4^{th} floor level for strengthening with shear walls. All rehabilitated cases satisfy the acceptability limit for drift. Moreover strengthening with "Shear Wall" reduced the maximum drift ratios the most compared to the other strengthening techniques.

The comments made for Building-1 are also observed to be valid for Building-2. However, the performance of the building with shear walls is clearly the best as all response parameters change favorable when evaluated for the two alternative ground motion effects. The evaluations made at this stage consider only the change in the performance of the building not considering the costs that are involved for upgrading the buildings and that result when a likely earthquake occurs. Therefore, the final selection of the most feasible rehabilitation alternative is made after the cost-benefit analysis that is provided next.

4.2 COST-BENEFIT ANALYSIS

In this section the feasibility of the rehabilitation alternatives is evaluated using cost-benefit analysis. Analysis was performed only for site-specific spectra because in most of cases a performance point could not be obtained for the buildings investigated under the code-based spectrum. The results of this evaluation are given in terms of both Net Present Values (NPV) and Benefit/Cost ratios (BCR). Cost effectiveness was determined by comparing cost of rehabilitation and value of damage prevented after rehabilitation. As mentioned previously in Chapter 3, NPV greater than zero or BCR greater than one are considered to be cost effective. Using the numerical values obtained comparisons were made to find the most advantageous alternative.

In this study the analysis was employed both for global drift ratios and maximum drift ratios that were computed at the performance points of each alternative. The consequences of the analysis employed using maximum drift ratios are given in Appendix C, here only the results based on the global drift ratios are provided.

The details of estimated rehabilitation costs calculated for both buildings were presented in Chapter 3. Table 4.11 depicts the summary of cost of retrofit for each rehabilitation alternative. As can be inferred from the table, the cost of rehabilitation for strengthening with CFRP is 12.82 % and 17.85 % more than the cost estimated for strengthening with shear wall for Building-1 and Building-2, respectively. It can be concluded that strengthening CFRP is more expensive than strengthening with shear walls.

Table 4.11 Estimated cost of rehabilitation strategies

BUILDING	Rehabilitation Strategy	Cost of retrofit (TL)
BUILDING-1	Shear Wall	121,536,379,318 TL
	CFRP on Infill Walls	137,113,028,757 TL
BUILDING-2	Shear Wall	204,157,763,493 TL
	CFRP on Infill Walls	240,600,240,484 TL

4.2.1 COST-BENEFIT ANALYSIS EMPLOYED USING GLOBAL DRIFT RATIOS

In this section expected cost of damage, expected benefits and calculated NPV and BCR for Building-1 and Building-2 are investigated, respectively.

4.2.1.1 BUILDING-1

4.2.1.1.1 EXPECTED COST OF DAMAGE

The expected cost of damages from the selected ground motions was computed for both the global drift ratios and the maximum drift ratios. As explained in Chapter 3, this is calculated by multiplying the percent damage obtained from the fragility curve with the replacement value of the building. Tables 4.12 and 4.13 show the expected cost of damages calculated using global drift ratios. The damage is expressed in terms of the ratio of the building's replacement value obtained from the fragility curves described in Chapter 3.

All strengthening techniques reduced the expected damage significantly when compared with the Original Building. Minimum expected cost of damage was obtained for the case of strengthening with "Shear Wall" when global drift ratios were considered. This is consistent with the deformation profiles presented earlier. Expected cost of damage calculated for "Shear Wall" is almost half of the cost of damage computed for "TUBITAK CFRP Model" in the x-direction (Table 4.12). In the y-direction "Shear Wall" reduced the expected cost of damage relatively more than "TUBITAK CFRP Model" (Table 4.13).

Table 4.12 Expected damages for Building-1 in the x-direction

Rehabilitation Strategy	Global drift ratio (Δ / h %)	Damage (%)	Cost of damage
Original Building	1.23	60.00	253,092,000,000 TL
Erdem's CFRP Model	0.62	45.32	191,181,877,090 TL
TUBITAK CFRP Model	0.44	28.79	121,443,390,471 TL
Reduced-Shear Wall	0.54	23.17	97,735,838,454 TL
Shear Wall	0.44	14.97	63,135,638,598 TL

Table 4.13 Expected damages for Building-1 in the y-direction

Rehabilitation Strategy	Global drift ratio (Δ / h %)	Damage (%)	Cost of damage
Original Building	1.25	57.43	242,257,377,320 TL
Erdem's CFRP Model	0.65	43.57	183,770,087,537 TL
TUBITAK CFRP Model	0.53	28.86	121,723,079,448 TL
Reduced-Shear Wall	0.60	24.64	103,939,674,218 TL
Shear Wall	0.52	18.68	78,811,598,806 TL

4.2.1.1.2 EXPECTED BENEFITS

The expected benefit is calculated as the difference between the expected damage of the original building and the rehabilitated ones. Strengthening with "Shear Wall" maximized the benefit for Building-1. The strengthening with shear wall maximized the expected benefit under site-specific spectrum in both directions. Tables 4.14 and 4.15 present the expected benefits computed for Building-1. When the expected benefits corresponding to "Shear Wall" and "Reduced Shear Wall" are compared, it can be seen that the expected benefits calculated for "Shear Wall" are about 15 % greater than that of "Reduced Shear Wall".

Table 4.14 Expected benefits calculated considering global drift ratios -Building-1 (in the x-direction)

Rehabilitation Strategy	**EXPECTED BENEFITS**
Original Building	--
Shear Wall	189,956,361,402 TL
Reduced-Shear Wall	155,356,161,546 TL
TUBITAK CFRP Model	131,648,609,529 TL
Erdem's CFRP Model	61,910,122,910 TL

Table 4.15 Expected benefits calculated considering global drift ratios Building-1 (in the y-direction)

Rehabilitation Strategy	**EXPECTED BENEFITS**
Original Building	--
Shear Wall	163,445,778,513 TL
Reduced-Shear Wall	138,317,703,102 TL
TUBITAK CFRP Model	120,534,297,871 TL
Erdem's CFRP Model	58,487,289,782 TL

It can also be noted that expected benefits computed for "TUBITAK CFRP Model" is approximately twice of that calculated for Erdem's CFRP Model in both directions.

4.2.1.1.3 CHOOSING THE BEST REHABILITATION ALTERNATIVE

Determination of cost-effectiveness of rehabilitation alternatives is of critical importance for the determination of minimized economic loss.

Tables 4.16 and 4.17 illustrate that strengthening of Building-1 with shear wall seemed to be the most cost effective rehabilitation strategy among other rehabilitation alternatives. According to the results given in the tables below, "Shear Wall" and "Reduced Shear Wall" are the only alternatives that ensures criteria for NPV and BCR in the x- and y-directions and strengthening with CFRP is not feasible for Building-1.

Table 4.16 Expected NPV and BCR for Building-1 in the x-direction

Rehabilitation Strategy	**NPV**	**BCR**
Shear Wall	68,419,982,085 TL	1.56
Reduced-Shear Wall	33,819,782,228 TL	1.28
TUBITAK CFRP Model	-5,464,419,228 TL	0.96
Erdem's CFRP Model	-75,202,905,846 TL	0.45

Table 4.17 Expected NPV and BCR for Building-1 in the y-direction

Rehabilitation Strategy	**NPV**	**BCR**
Shear Wall	41,909,399,196 TL	1.34
Reduced-Shear Wall	16,781,323,784 TL	1.14
TUBITAK CFRP Model	-16,578,730,885 TL	0.88
Erdem's CFRP Model	-78,625,738,974 TL	0.43

4.2.1.2 BUILDING-2

4.2.1.2.1 EXPECTED COST OF DAMAGE

Like Building-1 the lowest estimated damage was calculated for strengthening with “Shear Wall” for Building-2 in the x- and y-directions. Tables 4.18 and 4.19 give the comparison of expected cost of damage calculated considering global drift ratios.

Table 4.18 Expected damages for Building-2 in the x-direction

Rehabilitation Strategy	Global drift ratio (Δ / h %)	Damage (%)	Cost of damage
Erdem’s CFRP Model	0.42	29.28	125,321,935,981 TL
Original Building	0.72	23.25	99,488,344,902 TL
TUBITAK CFRP Model	0.22	9.55	40,869,850,352 TL
Reduced-Shear Wall	0.12	1.88	8,064,227,733 TL
Shear Wall	0.11	1.77	7,585,568,081 TL

Table 4.19 Expected damages for Building-2 in the y-direction

Rehabilitation Strategy	Global drift ratio (Δ / h %)	Damage (%)	Cost of damage
Original Building	0.83	32.88	140,719,748,778 TL
Erdem’s CFRP Model	0.37	23.23	99,403,794,232 TL
TUBITAK CFRP Model	0.16	3.94	16,871,593,287 TL
Reduced-Shear Wall	0.08	0.73	3,103,756,707 TL
Shear Wall	0.07	0.64	2,752,085,694 TL

These results reveal that the most feasible rehabilitation strategy is the insertion of shear walls, which leads to the lowest expected cost of damage. There is a significant difference in the expected cost of damage of strengthening with shear walls and strengthening with CFRP. The significant reduction in the global drift ratio is due to the efficiency of the substantial amount of shear walls introduced.

4.2.1.2.2 EXPECTED BENEFITS

Strengthening with shear wall maximized the expected benefits for Building-2 for all cases. The expected benefits obtained for "Shear Wall" are almost same with the expected benefits calculated for "Reduced Shear Wall" due to similar response of the two rehabilitation alternatives. Tables 4.20 and 4.21 illustrate the expected benefits in monetary terms for each rehabilitation alternative.

Table 4.20 Expected benefits calculated considering global drift ratios-Building-2 (in the x-direction)

Rehabilitation Strategy	EXPECTED BENEFITS
Original Building	--
Shear Wall	91,902,776,820 TL
Reduced-Shear Wall	91,424,117,169 TL
TUBITAK CFRP Model	58,618,494,549 TL
Erdem's CFRP Model	-25,833,591,080 TL

Table 4.21 Expected benefits calculated considering global drift ratios-Building-2 (in the y-direction)

Rehabilitation Strategy	EXPECTED BENEFITS
Original Building	--
Shear Wall	137,967,663,083 TL
Reduced-Shear Wall	137,615,992,070 TL
TUBITAK CFRP Model	123,848,155,490 TL
Erdem's CFRP Model	41,315,954,546 TL

4.2.1.2.3 CHOOSING THE BEST REHABILITATION ALTERNATIVE

Tables 4.22 and 4.23 show that, insertion of shear walls should be selected as the most suitable and beneficial rehabilitation alternative for Building-2 among the alternatives employed. Although Building-2 rehabilitated with "Shear Wall" does not satisfy NPV and BCR criteria in the x- and y-directions, it is the most preferable alternative among other alternatives. NPV obtained for strengthening with shear wall is more than twice of NPV determined for strengthening with CFRP. NPV and BCR calculated for "Shear Wall" and "Reduced Shear Wall" are almost same. NPV and BCR of "Reduced Shear Wall " are slightly less than that of "Shear Wall".

Table 4.22 Expected NPV and BCR for Building-2 in the x-direction

Rehabilitation Strategy	NPV	BCR
Shear Wall	-112,254,986,673 TL	0.45
Reduced-Shear Wall	-112,733,646,325 TL	0.45
TUBITAK CFRP Model	-181,981,745,935 TL	0.24
Erdem's CFRP Model	-266,433,831,564 TL	0.11

Table 4.23 Expected NPV and BCR for Building-2 in the y-direction

Rehabilitation Strategy	NPV	BCR
Shear Wall	-66,190,100,410 TL	0.68
Reduced-Shear Wall	-66,541,771,423 TL	0.67
TUBITAK CFRP Model	-116,752,084,994 TL	0.51
Erdem's CFRP Model	-199,284,285,939 TL	0.17

The substantial amount of shear wall added to Building-2 led to the increased cost of rehabilitation that effected the economic feasibility adversely. However, when the difference between the NPV and the BCR values corresponding to the Shear Wall and TUBITAK CFRP alternatives of the two buildings are compared, the influence of shear wall density on the cost effectiveness becomes more obvious.

4.2.2 COST-BENEFIT ANALYSIS EMPLOYED USING MAXIMUM DRIFT RATIOS

The expected cost of damages, expected benefits and comparisons of NPV and BCR calculated using maximum drift ratios for both Building-1 and Building-2 are given in Appendix C (Tables C.29-C.34 and C.35-C.40).

Tables C.29, C.30 and C.35, C.36 present the expected cost of damage considering maximum drift ratios for Building-1 and Building-2, respectively. It should be pointed out that, for all rehabilitation strategies the minimum expected cost of damage was obtained for strengthening with shear wall for both of the buildings when maximum drift ratios were considered in the calculation of expected damage. The expected cost of damage computed for Building-1 strengthened with "Shear Wall" is twice of that computed for Building-1 strengthened with CFRP when TUBITAK model is used in the x- and y-directions. The difference between the expected cost of damage calculated for CFRP and shear wall is evident for Building-2 when the values given in Tables C.35 and C.36 are evaluated. These results are similar to the ones obtained when global drift ratios were used.

Tables C.31, C.32 and C.37, C.38 present the expected benefits computed considering maximum drift ratios for Building-1 and Building-2, respectively. Strengthening with shear wall produced the largest expected benefits for both buildings in the x and y-directions.

Tables C.33, C.34 and C.39, C.40 give the attractiveness of each rehabilitation strategy that was calculated considering maximum drift ratios for Building-1 and Building-2, respectively. It is obvious that for both of the buildings, strengthening with shear wall is the most attractive rehabilitation strategy for all cases.

The findings highlight a strong correlation between the cost and benefit when making decisions on the best alternative. It is also important that the most attractive alternative might be case dependent and might also be strongly influenced by the selection of the response parameter.

Overall evaluation made for the results of cost-benefit analyses implemented considering both global drift ratios and the maximum drift ratios reveals that the best alternative for Building-1 and Building-2 seem to be the rehabilitation with shear wall.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 SUMMARY

In this thesis, an analytical study was undertaken to evaluate the seismic performance and to carry out cost benefit analysis of two existing undamaged buildings having similar characteristics. The buildings reflect features of typical buildings in Turkey and thus are considered vulnerable to earthquake hazards in the Marmara region. Two rehabilitation techniques, strengthening with reinforced concrete shear walls and application of Carbon Fiber Reinforced Polymers (CFRP) on hollow clay tile infill walls, were investigated under two ground motion effects represented by the code-based and site-specific spectra. The performance evaluations were based on the life safety performance level and the modified criteria given in ATC-40. Two separate models [6,7], developed for CFRP application based on the experimental test results and analytical studies conducted at Middle East Technical University, were considered. The selected buildings were analyzed by SAP 2000 [5] using three-dimensional models. All response quantities deemed necessary for the performance evaluation were computed. The cost-benefit analysis was performed in order to determine the most appropriate rehabilitation alternative. The probable earthquake damage induced on the buildings under the selected ground motion effects was assessed using the simple fragility curves developed for this study.

The results were compared in terms of seismic performance and cost effectiveness of the buildings. The seismic performance of the case study buildings was evaluated considering strength, stiffness and lateral drifts both at the building level and at the level of the components. The comparisons were also made considering the results of cost-benefit analysis used to determine the most beneficial rehabilitation alternative. The primary goal in this study was to introduce all the steps involved in a complete cost-benefit analysis for the purpose of comparing the efficiency of the two alternative rehabilitation schemes that are applicable to the buildings in Turkey. It is important to note that the outcomes of this study are only applicable to the buildings employed here and are bound by the assumptions made, approximations used and approach taken in this study. The behavior of the building rehabilitated with CFRP is not known with an adequate level of confidence due to lack of data available. The inconsistency present between the two proposals for modeling the same application is a clear indication of this lack. More research needs to be conducted to provide solid guidelines and reliable models applicable to the CFRP rehabilitated infill walls. Therefore, findings presented here can not be generalized for the buildings rehabilitated by CFRP applied on infill walls but are valid for the other cases. The consideration of other important parameters like speed of application, political decisions and the size of the rehabilitation project (number of buildings to be rehabilitated) would strongly affect the decision.

5.2 CONCLUSIONS

The observations made through the comparison of performances and examination of the results the following general conclusions can be drawn:

- Analysis of individual test frames that were tested at METU revealed that there is a significant difference between two specific modeling attempts proposed by Erdem [6] and TUBITAK Report 2003/1 [7]. Strengthening buildings with CFRP using the model given in TUBITAK Report 2003/1 [7] enhanced the behavior more than that using the model proposed by Erdem [6].
- Reliability of the results obtained for seismic performance and cost-benefit analysis using CFRP strongly depends on the accuracy and validity of the CFRP model proposed based on the experimental studies carried out at METU.
- All rehabilitation techniques employed improved the seismic performances of the buildings as expected. The computed global responses of the rehabilitated buildings satisfied the acceptability limits given in ATC 40 [3]. Strengthening with shear wall increased the yield and ultimate base force capacity of the buildings more than strengthening with CFRP. In general strengthening with shear wall increased the strength and stiffness of the buildings the most. It was observed that this change is directly related with the shear wall density.

- Global response evaluation showed that, for the first building strengthening with shear wall reduced the story drifts in first three floors while strengthening with CFRP reduced the interstory drift ratio in upper floors. For the second building strengthening with shear wall resulted in the most significant reduction in the story drifts at all floors compared to the other rehabilitation alternatives. This is attributed to the change in the behavior mode of the building imposed by different techniques.
- The maximum interstory drift ratios were observed at 4^{th} floor level for the buildings strengthened with shear walls and at 2^{nd} floor levels for the buildings strengthened with CFRP.
- Considering the results of component response evaluation under selected spectra, generally strengthening with shear walls improved the performance of the elements significantly. This shows the influence of the assessment procedure and points out that the component-based evaluations would favor the rehabilitation with shear walls. Furthermore component response evaluation revealed that the degree of improvement is strongly correlated with the wall density.
- Detailed cost estimations performed for rehabilitation alternatives demonstrated that, strengthening with CFRP is more expensive than strengthening with shear walls. For the application considered here, i.e. the CFRP applied on the infill walls where shear walls were added, the difference in cost is about 15 %, which may increase if more CFRP is used to achieve the same level of capacity as the shear wall insertion.
- The major part of the cost in CFRP application comes from the cost of the CFRP material. This resulted from the assumption that the amount of CFRP is proportional to the area of the infill walls. If this proportionality changes then the cost figures computed here will change and in case of less material required to achieve the same level of performance, the cost of rehabilitation with CFRP might be less than that of the shear wall. Further changes in the cost are likely because different sources were used for the two techniques here due to lack of availability of the unit price lists for CFRP application.
- According to the outcomes of the cost-benefit analysis carried out for global and maximum drift responses, for both of the case study buildings strengthening with shear walls was found to be the most reliable alternative. As inferred from the deformation profiles, the selection of the damage inducing parameter is very

important. In certain cases, the global drift ratio is misleading and does not reflect the actual deformation profile.

- Overall results of the thesis showed that seismic performance and cost effectiveness of the rehabilitation alternatives investigated here are directly related to the shear wall density, the selection of the response quantity and the level of evaluation (component or building), the level of the earthquake (response spectrum in this case) and also, although not investigated here, to the approximate performance analysis procedure.
- The analyses presented here focused on the individual residential building performances considering only relevant parameters. In other cases, the final decision on the selection of the appropriate rehabilitation technique may depend on other important criteria such as speed of rehabilitation project, problems with accommodation for a large population when many buildings are needed to be rehabilitated in which case the CFRP technique becomes more appropriate.

5.3 RECOMMENDATIONS FOR FUTURE STUDY

- Further experimental and analytical studies should be performed in order to determine a valid and reliable CFRP model and generalize the conclusions.
- Analytical analysis including seismic performance evaluation and cost-benefit analysis should be carried out for many buildings and should be extended to different types of buildings with other features.
- Existence of different distributing firms, which could quote unit prices of materials used for CFRP application, should be searched.
- More detailed cost-benefit analysis concerning probabilistic seismic loss estimation methodology should be conducted. In this respect fragility curves should be established and a seismic hazard curve should be provided for the Marmara Region. A discount rate that can be determined by Turkish economists should be included in the analysis.
- Cost-benefit analysis should be done for other types of rehabilitation measures.

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APPENDIX A

COMPARISON OF PROPOSED MODELS [6, 7] & MODELING MASONRY INFILL WALL AND JACKETED COLUMN

A.1 COMPARISON OF CONSTITUTIVE MODELS PROPOSED BY ERDEM [6] AND TUBITAK REPORT 2003/1 [7]

The individual test frames of Erdem [6] and TUBITAK Report 2003/1 [7] were re-analyzed using both models proposed by each study using SAP2000. Comparisons of pushover curves are given in Figures A.1-A.4.

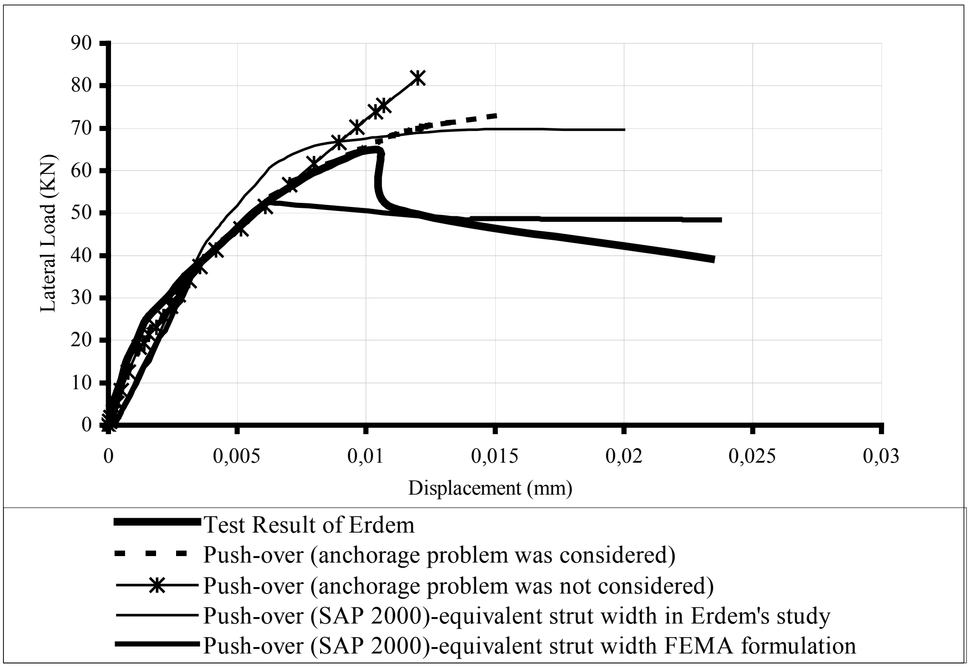

Figure A.1 Analytical predictions in Erdem's study [6] with pushover curves obtained using SAP2000

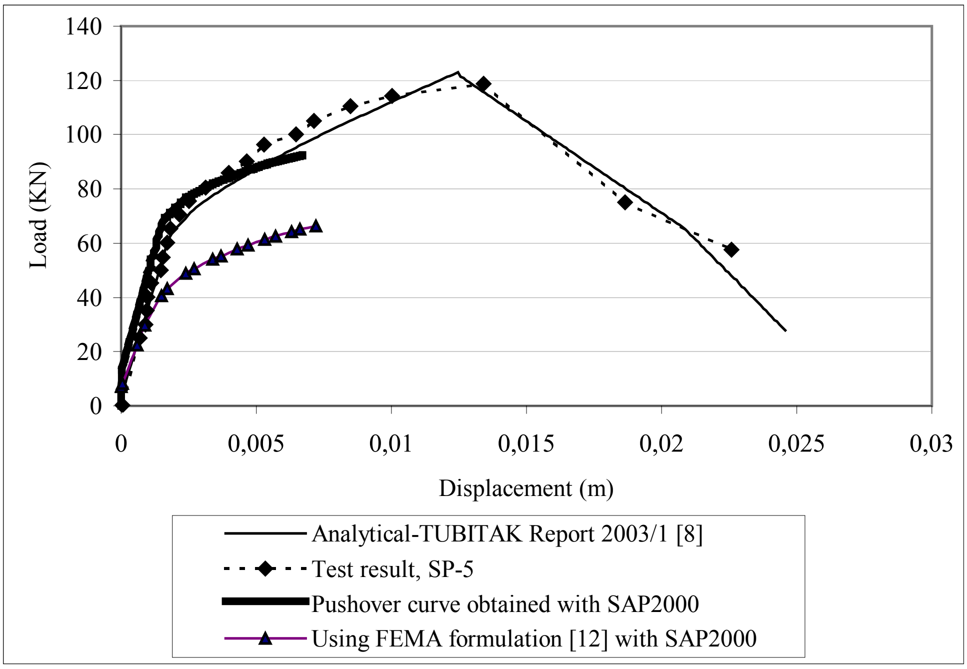

Figure A.2 Analytical prediction in TUBITAK 2003/1 Report [7] and pushover curves obtained with SAP2000

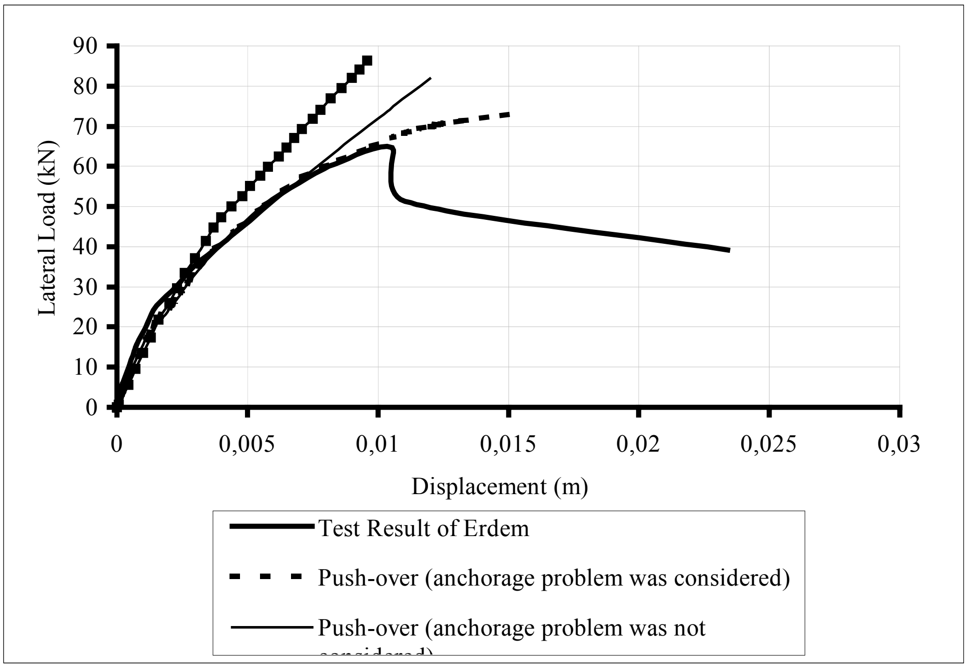

Figure A.3 Pushover curve using TUBITAK CFRP Model for the specimen in Erdem's study [6]

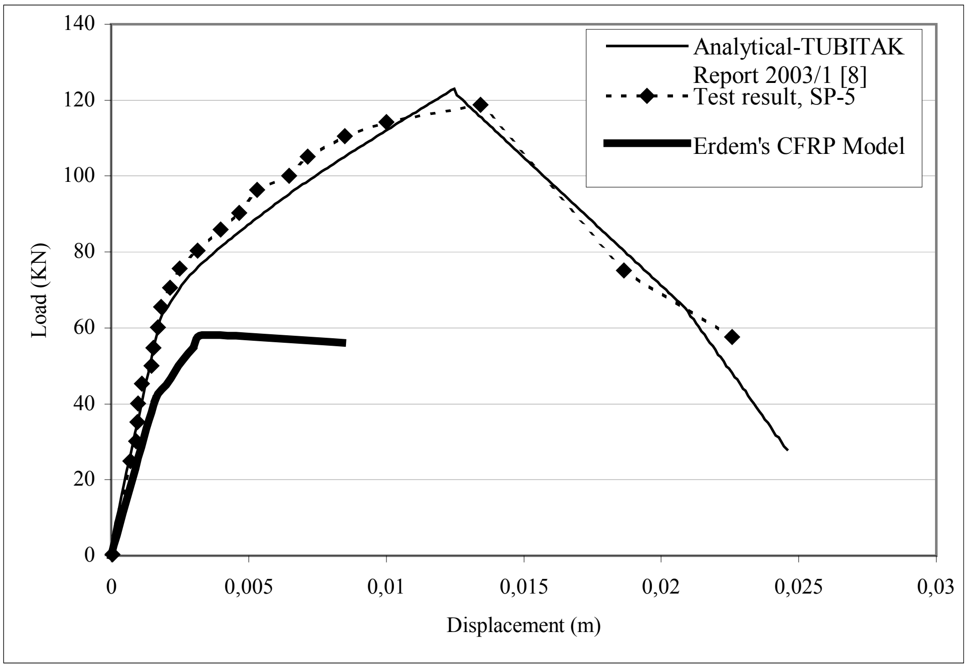

Figure A.4 Pushover curve using Erdem's CFRP Model [6] for the test frame of TUBITAK Report 2003/1 [7]

A.2 MODELLING OF MASONRY INFILL WALL

In Figure A.5 diagonal compression strut is illustrated.

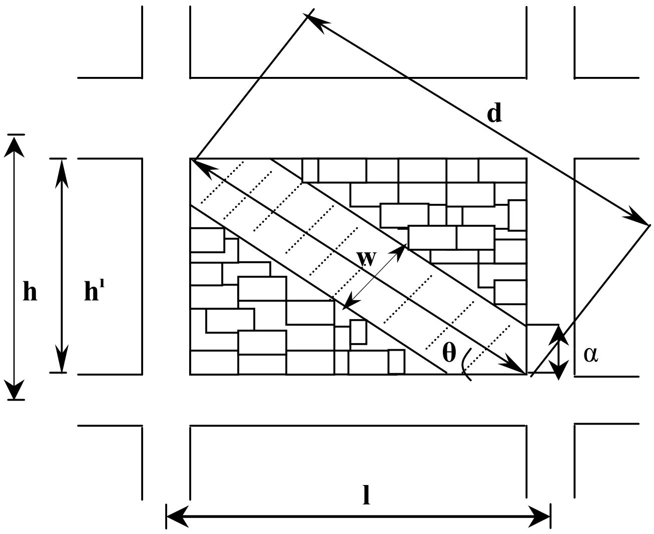

Figure A.5 Compression diagonal

Empirical formulation used in this study is given below.

$$w = 0.175(\lambda h)^{-0.4} d \qquad \lambda = \sqrt[4]{\frac{E_I t \sin 2\theta}{4EIh'}}$$

In the formulas given above; w is the equivalent strut width of compression diagonal; E_I, t and h' are Young's modulus, thickness and height of infill respectively. E, I and h are Young's modulus, second moment of inertia and height of column and θ is the slope of infill diagonal to horizontal. Here λh represents a non-dimensional parameter that represents the relative stiffness of the frame with respect to the infill.

A.3 COLUMN JACKETING

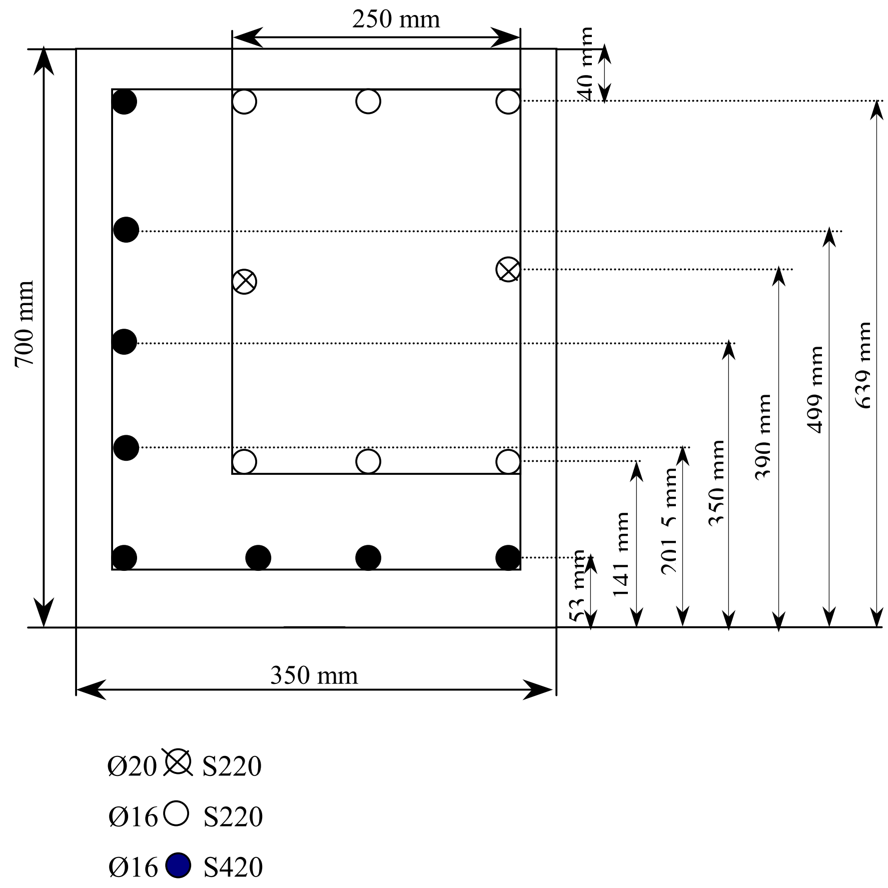

Figure A.6 Cross sectional view of jacketed column of Building-1

Output of the program, which provides the interaction diagram of the jacketed columns for Building-1, is given below:

```
Clear
Format long
% INPUT DATA %
%Input units are in N,mm and MPa; Output units are in KN, KNm and MPA%
hor=530;
h1=35;
h2=135;
bor=180;
b1=135;
b2=35;
h=h1+hor+h2
n=8; % Number of lines of steel %
x=[297,296,148.5,50,0,-148.5,-197,-297]; % Position of steel lines %
A=[603.186,201.062,201.062,628.32,201.062,201.062,603.186,804.25]; % Area of steel
lines %

k1=0.85;
Cj=20;
Cor=9;

fyk(1)=220;fyk(4)=220;fyk(7)=220;
fyk(2)=420;fyk(3)=420;fyk(5)=420;fyk(6)=420;fyk(8)=420;esu=0.1;
for i=1:8
   esy(i)=fyk(i)/200000;
end

etop=0.003;

say=1;
for c=1:h
   % STRESS FORCES AND MOMENTS IN STEEL %
```

```
for i=1:n
   if c<=(h-35)*etop/(esu+etop)
      Fs(i)=[A(i)*fyk(i)*(-1)/1000]
      Ms(i)=[0];
   else
      es(i)=[(c-(h/2)+x(i))/c*etop];
      if abs(es(i))>0 & abs(es(i))<=esy(i)
         Ss(i)=fyk(i)/esy(i)*es(i);
      elseif abs(es(i))>esy(i) & abs(es(i))<=0.1
         Ss(i)=fyk(i)*sign(es(i));
      else
         Ss(i)=0;
      end
      Fs(i)=[A(i)*Ss(i)/1000];
      Ms(i)=[Fs(i)*x(i)/1000];
   end
end
%
% CALCULATION OF FORCE AND MOMENT IN JACKETED CONCRETE %
%
% 1
if k1*c<=h1
   FC1=0.85*Cj*k1*(b1+b2+bor)*c/1000;
   MC1=FC1*((h1+hor+h2)/2-k1*c/2)/1000;
else
   FC1=0.85*Cj*(b1+b2+bor)*h1/1000;
   MC1=FC1*((h1+hor+h2)/2-h1/2)/1000;
end
% 2
if k1*c <=h1
   FC2=0;
   MC2=0;
elseif k1*c<=h1+hor & k1*c>h1
   FC2=0.85*Cj*b1*(k1*c-h1)/1000;
```

```
   MC2=FC2*((h1+hor+h2)/2-h1-(k1*c-h1)/2)/1000;
elseif k1*c>h1+hor
   FC2=0.85*Cj*b1*hor/1000;
   MC2=FC2*((h1+hor+h2)/2-h1-(hor/2))/1000;
end
% 3
if k1*c<=h1
   FC3=0;
   MC3=0;
elseif k1*c<=h1+hor & k1*c>h1
   FC3=0.85*Cj*b2*(k1*c-h1)/1000;
   MC3=FC3*((h1+hor+h2)/2-h1-(k1*c-h1)/2)/1000;
elseif k1*c>h1+hor
   FC3=0.85*Cj*b2*hor/1000;
   MC3=FC3*((h1+hor+h2)/2-h1-(hor/2))/1000;
end
% 4
if k1*c<=h1+hor
   FC4=0;
   MC4=0;
elseif k1*c<=h1+hor+h2
   FC4=0.85*Cj*(b1+b2+bor)*(k1*c-h1-hor)/1000;
   MC4=FC4*((h1+hor+h2)/2-(k1*c-h1-hor)/2-h1-hor)/1000;
elseif k1*c>h1+hor+h2
   FC4=0.85*Cj*(b1+b2+bor)*h2/1000;
   MC4=FC4*((h1+hor+h2)/2-h2/2-h1-hor)/1000;
end
%
% CALCULATION OF FORCE AND MOMENT IN UNJACKETED CONCRETE %
%
% 5
if k1*c<=h1
  FC5=0;
  MC5=0;
```

```
    elseif k1*c<=h1+hor & k1*c>h1
      FC5=0.85*Cor*bor*(k1*c-h1)/1000;
      MC5=FC5*((h1+h2+hor)/2-h1-(k1*c-h1)/2)/1000;
    elseif k1*c>h1+hor
      FC5=0.85*Cor*bor*hor/1000;
      MC5=FC5*((h1+h2+hor)/2-h1-hor/2)/1000;
    end
    % Axial Force in Section %
    N(say)=FC1+FC2+FC3+FC4+FC5+sum(Fs);
    % Moment Section %
    M(say)=sum(Ms)+MC1+MC2+MC3+MC4+MC5;
    say=say+1;
end
for c=h:200:1000*h
    etop=-0.001/999/h*c+0.003+0.001/999
    if k1*c<=h
        K1C=k1*c
    else
        K1C=h
    end
    % STRESS FORCES AND MOMENTS IN STEEL %
    for i=1:n
        es(i)=[(c-(h/2)+x(i))/c*etop];
        if abs(es(i))>0 & abs(es(i))<=esy(i)
            Ss(i)=fyk(i)/esy(i)*es(i);
        elseif abs(es(i))>esy(i) & abs(es(i))<=0.1
            Ss(i)=fyk(i)*sign(es(i));
        else
            Ss(i)=0;
        end
        Fs(i)=[A(i)*Ss(i)/1000];
        Ms(i)=[Fs(i)*x(i)/1000];
    end
    %
```

```
% CALCULATION OF FORCE AND MOMENT IN JACKETED CONCRETE %
%
% 1
if K1C<=h1
   FC1=0.85*Cj*K1C*(b1+b2+bor)/1000;
   MC1=FC1*((h1+hor+h2)/2-k1*c/2)/1000;
else
   FC1=0.85*Cj*(b1+b2+bor)*h1/1000;
   MC1=FC1*((h1+hor+h2)/2-h1/2)/1000;
end
% 2
if K1C<=h1
   FC2=0;
   MC2=0;
elseif K1C<=h1+hor & K1C>h1
   FC2=0.85*Cj*b1*(K1C-h1)/1000;
   MC2=FC2*((h1+hor+h2)/2-h1-(K1C-h1)/2)/1000;
elseif K1C>h1+hor
   FC2=0.85*Cj*b1*hor/1000;
   MC2=FC2*((h1+hor+h2)/2-h1-(hor/2))/1000;
end
% 3
if K1C<=h1
   FC3=0;
   MC3=0;
elseif K1C<=h1+hor & K1C>h1
   FC3=0.85*Cj*b2*(K1C-h1)/1000;
   MC3=FC3*((h1+hor+h2)/2-h1-(K1C-h1)/2)/1000;
elseif K1C>h1+hor
   FC3=0.85*Cj*b2*hor/1000;
   MC3=FC3*((h1+hor+h2)/2-h1-(hor/2))/1000;
end
% 4
if K1C<=h1+hor
```

```
    FC4=0;
    MC4=0;
  elseif K1C<=h1+hor+h2
    FC4=0.85*Cj*(b1+b2+bor)*(K1C-h1-hor)/1000;
    MC4=FC4*((h1+hor+h2)/2-(K1C-h1-hor)/2-h1-hor)/1000;
  elseif K1C>h1+hor+h2
    FC4=0.85*Cj*(b1+b2+bor)*h2/1000;
    MC4=FC4*((h1+hor+h2)/2-h2/2-h1-hor)/1000;
  end
  %
  % CALCULATION OF FORCE AND MOMENT IN UNJACKETED CONCRETE %
  %
  % 5
  if K1C<=h1
    FC5=0;
    MC5=0;
  elseif K1C<=h1+hor & K1C>h1
    FC5=0.85*Cor*bor*(K1C-h1)/1000;
    MC5=FC5*((h1+h2+hor)/2-h1-(K1C-h1)/2)/1000;
  elseif K1C>h1+hor
    FC5=0.85*Cor*bor*hor/1000;
    MC5=FC5*((h1+h2+hor)/2-h1-hor/2)/1000;
  end
  % Axial Force in Section %
  N(say)=FC1+FC2+FC3+FC4+FC5+sum(Fs);
  % Moment Section %
  M(say)=sum(Ms)+MC1+MC2+MC3+MC4+MC5;
  say=say+1;
end
AxialForce=transpose(N);
Moment=transpose(M);
plot(M,N)
```

APPENDIX B

SPECIAL ANALYSIS OF THE UNIT PRICES ESTIMATES

Pose numbers and the corresponding unit prices given in the tables were taken from the unit price list of Ministry of Public Works and Settlement [30].

B.1 UNIT PRICE ANALYSIS PROVIDED FOR REHABILITATION WITH SHEAR WALL AND JACKETING COLUMNS

Table B.1 Unit price analysis for C20 ready mixed concrete

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
16.058 / 1A	**Material**				
04.031	Irrigation of concrete	m^3	0.40	2,500,000 TL	1,000,000 TL
04.043/1A	Ready concrete mortar (C20)	m^3	1	62,462,000 TL	62,462,000 TL
04.613/1A3	Concrete plasticiser	kg	3	2,200,000 TL	6,600,000 TL
	Labor				
01.015	Concrete master worker	hour	0.45	2,831,000 TL	1,273,950 TL
01.501	Unqualified worker	hour	0.90	1,853,000 TL	1,667,700 TL
03.527	Spud vibrator	hour	0.05	2,881,350 TL	144,068 TL
	Material + Labor				73,147,718 TL
	25 % Contractor profit				18,286,929 TL
(For 1 m^3)	TOTAL				***91,434,647 TL***

Table B.2 Unit price analysis for covering between old and new concrete with adhesive material

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
	Material				
04.613/8A	Adhesive resin	kg	0.8	32,400,000 TL	25,920,000 TL
	Labor				
01.012	Plastering master worker	hour	2	2,831,000,00 TL	5,662,000 TL
01.501	Unqualified Worker	hour	0.8	1,853,000,00 TL	1,482,400 TL
	Material + Labor				33,064,400 TL
	25 % Contractor profit				8,266,100 TL
(For 1 m^2)	**TOTAL**				***41,330,500 TL***

Table B.3 Unit price analysis for wooden formwork

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
	Material				
04.151	Deal	m^3	0.012	555,000,000 TL	6,660,000 TL
04.270	Nail	kg	0.1	655,000 TL	65,500 TL
	Labor				
01.017	Carpenter master worker	hour	1.5	2,831,000 TL	4,246,500 TL
01.501	Unqualified worker (Carrying and picking up waste are included)	hour	1.5	1,853,000 TL	2,779,500 TL
	Material + labor				13,751,500 TL
	25 % Contractor profit				3,437,875 TL
(For 1 m^2)	TOTAL				***17,189,375 TL***

Table B.4 Unit price analysis for wooden scaffolding for formwork

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
Supplementary Analysis					
	Material				
04.255	Deal	kg	1.10	439,000 TL	482,900 TL
04.255	Lama bars (as connector)	kg	0.11	439,000 TL	48,290 TL
01.018	Ironsmith worker	hour	0.40	2,831,000 TL	1,132,400 TL
01.503	Apprentice	hour	0.40	1,555,000 TL	622,000 TL
				Subtotal	2,285,590 TL
Price Analysis	**Material**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
04.152	Deal	m^3	0.0024	416,250,000 TL	999,000 TL
04.275/1	Anchor bolt	kg	0.018	1,073,000 TL	19,314 TL
08.010	Lama hook	kg	0.009	2,150,000 TL	19,350 TL
04.270	Nail	kg	0.03	655,000 TL	19,650 TL
	Labor				
01.017	Carpenter master worker	hour	0.252	2,359,000 TL	594,468 TL
01.501	Unqualified worker (Carrying and picking up waste are included)	hour	0.108	1,544,000 TL	166,752 TL
	Dismantling Workmanship				
01.017	Carpenter master worker	hour	0.036	2,831,000 TL	101,916 TL
01.501	Unqualified worker (Carrying and picking up waste are included)	hour	0.024	1,853,000 TL	44,472 TL
	Material + Labor				1,964,922 TL
	25 % Contractor profit				491,231 TL
(For 1 m^3)	TOTAL				***2,456,153 TL***

Table B.5 Unit price analysis for non-shrink concrete

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
	Material				
04.613 / 3C	Ready mortar that does not make shrinkage. (Possible waste is included)	kg	1.70	1,750,000 TL	2,975,000 TL
04.031	Water (for cleaning beneath of surfaces)	m^3	0.283	2,500,000 TL	707,500 TL
	Labor				
01.015	Plastering Master Worker	hour	5	2,831,000 TL	14,155,000 TL
01.501	Unqualified Worker	hour	2	1,853,000 TL	3,706,000 TL
	Material + Labor				21,543,500 TL
	25 % Contractor profit				5,385,875 TL
(For 1 m^2)	TOTAL				***26,929,375 TL***

Table B.6 Unit price analysis for crushing concrete cover of beams and columns

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
	Material				
04.031	Water	m^3	0.025	2,500,000 TL	62,500 TL
Additional analysis	**Pose No: 03,517**	**Price of compressor for 1 hour**			
POSE NUMBER	**Material**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
03,017	Machinery	A	0.000274	16,140,000,000 TL	4,422,360 TL
04,109	Diesel Oil	kg	2.85	1,594,000 TL	4,542,900 TL
04,110	Diesel Oil (mach.	kg	0.57	3,380,000 TL	1,926,600 TL
01,011	Firing Foreman	hour	1	2,831,000 TL	2,831,000 TL
01.403	Engine Driver	month	0.01	679,440,000 TL	6,794,400 TL
01.502	Skilled Worker	hour	4	2,004,000 TL	8,016,000 TL
03,517	**Compressor**	**hour**	**0.10**	**28,533,260 TL**	**2,853,326 TL**
	Workmanship				
01.501	Unqualified worker	hour	2	1,853,000 TL	3,706,000 TL
01.501	Unqualified worker (Carrying and picking up waste are included)	hour	0.2	1,853,000 TL	370,600 TL
01.502	Qualified Worker	hour	0.25	2,004,000 TL	501,000 TL
	Material + Labor				7,493,426 TL
	25 % Contractor profit				1,873,357 TL
(For 1 m^2)	TOTAL				***9,366,783 TL***

Table B.7 Unit price analysis for construction steel Ø8-Ø12

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
	Material				
04.253	Ø8-Ø12	ton	1.1	493,000,000 TL	542,300,000 TL
	Labor				
01.501	Unqualified worker	hour	50	1,853,000 TL	92,650,000 TL
01.501	Unqualified worker (Carrying and picking up waste are included)	hour	30	1,853,000 TL	55,590,000 TL
01.019	Bar bender master worker	hour	60	2,831,000 TL	169,860,000 TL
01.219	Assistant bar bender master worker	hour	90	2,004,000 TL	180,360,000 TL
	Material + Labor				1,040,760,000 TL
	25 % Contractor profit				260,190,000 TL
(For 1 m^2)	TOTAL				***1,300,950,000 TL***

Table B.8 Unit price analysis for construction steel Ø14-Ø22

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
	Material				
04.254	Ø14-Ø22	ton	1.1	493,000,000 TL	542,300,000 TL
	Labor				
01.501	Unqualified worker	hour	40	1,853,000 TL	74,120,000 TL
01.501	Unqualified worker (Carrying and picking up waste are included)	hour	30	1,853,000 TL	55,590,000 TL
01.019	Bar bender master worker	hour	50	2,831,000 TL	141,550,000 TL
01.219	Assistant bar bender master worker	hour	70	2,004,000 TL	140,280,000 TL
	Material + Labor				953,840,000 TL
	25 % Contractor profit				238,460,000 TL
(For 1 m^2)	TOTAL				***1,192,300,000 TL***

Table B.9 Unit price analysis for 1 piece of anchorage (Ø22 – 45 cm of depth)

POSE NUMBER	**DESCRIPTION**	Anchorage (Ø22 – 45 cm of depth)			
Ø (mm)	22				
Ø-HOLE (mm)	27				
Depth of anchorage (cm)	45				
Supplementary Analysis	Pose No: 03,517	**Price of compressor (For 1 hour)**			
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
03,017	Machinery	--	0.000274	16,140,000,000 TL	4,422,360 TL
04,109	Diesel oil	kg	2.85	1,594,000 TL	4,542,900 TL
04,110	Diesel oil (mach. oiling)	kg	0.57	3,380,000 TL	1,926,600 TL
01,011	Firing foreman	hour	1	2,831,000 TL	2,831,000 TL
01.403	Engine driver	month	0.01	679,440,000 TL	6,794,400 TL
01.502	Qualified worker	hour	4	2,004,000 TL	8,016,000 TL
				Total	28,533,260 TL
Price Analysis					
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
04.613/8E	Epoxy resin	kg	0.1523	8,400,000 TL	1,279,500 TL
03,517	Compressor	hour	0.07	28,533,260 TL	1,997,328 TL
01.507	First class worker	hour	0.48	2,712,850 TL	1,302,168 TL
236.301	Opening hole	cm	9	1,274,000 TL	11,466,000 TL
					14,765,496 TL
	25 % Contractor profit				3,691,374 TL
(For 1 piece)	TOTAL				***20,056,245 TL***

Table B.10 Unit price analysis for 1 piece of anchorage (Ø22 – 35 cm of depth)

POSE NUMBER	**DESCRIPTION**				
Ø (mm)	22	Anchorage (Ø22 – 35 cm of depth)			
Ø-HOLE (mm)	27				
Depth of anchorage (cm)	35				
Supplementary Analysis	Pose No: 03,517	Price of compressor (For 1 hour)			
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
03,017	Machinery	--	0.000274	16,140,000,000 TL	4,422,360 TL
04,109	Diesel oil	kg	2.85	1,594,000 TL	4,542,900 TL
04,110	Diesel oil (mach. oiling)	kg	0.57	3,380,000 TL	1,926,600 TL
01,011	Firing foreman	hour	1	2,831,000 TL	2,831,000 TL
01.403	Engine driver	month	0.01	679,440,000 TL	6,794,400 TL
01.502	Qualified worker	hour	4	2,004,000 TL	8,016,000 TL
				Total	28,533,260 TL
Price Analysis					
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
04.613/8E	Epoxy resin	kg	0.1185	8,400,000 TL	995,165 TL
03,517	Compressor	hour	0.07	28,533,260 TL	1,997,328 TL
01.507	First class worker	hour	0.38	2,712,850 TL	1,030,883 TL
236.301	Opening hole	cm	7	1,274,000 TL	8,918,000 TL
					12,941,376 TL
	25 % Contractor profit				3,235,344 TL
(For 1 piece)	TOTAL				***16,176,720 TL***

Table B.11 Unit price analysis for 1 piece of anchorage (Ø22 – 12 cm of depth)

POSE NUMBER	**DESCRIPTION**	Anchorage (Ø22 – 12 cm of depth)			
Ø (mm)	22				
Ø-HOLE (mm)	27				
Depth of anchorage (cm)	12				
Supplementary Analysis	Pose No: 03,517	Price of compressor (For 1 hour)			
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
03,017	Machinery	--	0.0003	16,140,000,000 TL	4,422,360 TL
04,109	Diesel oil	kg	2.85	1,594,000 TL	4,542,900 TL
04,110	Diesel oil (mach. oiling)	kg	0.57	3,380,000 TL	1,926,600 TL
01,011	Firing foreman	hour	1	2,831,000 TL	2,831,000 TL
01.403	Engine driver	month	0.01	679,440,000 TL	6,794,400 TL
01.502	Qualified worker	hour	4	2,004,000 TL	8,016,000 TL
				Total	28,533,260 TL
Price Analysis					
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
04.613/8E	Epoxy resin	kg	0.0406	8,400,000 TL	341,200 TL
03,517	Compressor	hour	0.07	28,533,260 TL	1,997,328 TL
01.507	First class worker	hour	0.13	2,712,850 TL	352,671 TL
236.301	Opening hole	cm	2.40	1,274,000 TL	3,057,600 TL
					5,748,798 TL
	25 % Contractor profit				1,437,200 TL
(For 1 piece)	TOTAL				***7,185,998 TL***

Table B.12 Unit price analysis for 1 piece of anchorage (Ø20 – 40 cm of depth)

POSE NUMBER	**DESCRIPTION**	Anchorage (Ø20 – 40 cm of depth)			
Ø (mm)	20				
Ø-HOLE (mm)	25				
Depth of anchorage (cm)	40				
Supplementary Analysis	Pose No: 03,517	Price of compressor (For 1 hour)			
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
03,017	Machinery	--	0.0003	16,140,000,000 TL	4,422,360 TL
04,109	Diesel oil	kg	2.85	1,594,000 TL	4,542,900 TL
04,110	Diesel oil (mach. oiling)	kg	0.57	3,380,000 TL	1,926,600 TL
01,011	Firing foreman	hour	1	2,831,000 TL	2,831,000 TL
01.403	Engine driver	month	0.01	679,440,000 TL	6,794,400 TL
01.502	Qualified worker	hour	4	2,004,000 TL	8,016,000 TL
				Total	28,533,260 TL
Price Analysis					
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
04.613/8E	Epoxy resin	kg	0.124344	8,400,000 TL	1,044,490 TL
03,517	Compressor	hour	0.07	28,533,260 TL	1,997,328 TL
01.507	First class worker	hour	0.46	2,712,850 TL	1,247,911 TL
236.301	Opening hole	cm	8	1,274,000 TL	10,192,000 TL
					14,481,729 TL
	25 % Contractor profit				3,620,432 TL
(For 1 piece)	TOTAL				***18,102,161 TL***

Table B.13 Unit price analysis for 1 piece of anchorage (Ø20 – 30 cm of depth)

POSE NUMBER	**DESCRIPTION**				
Ø (mm)	20	Anchorage (Ø20 – 30 cm of depth)			
Ø-HOLE (mm)	25				
Depth of anchorage (cm)	30				
Supplementary Analysis	Pose No: 03,517	Price of compressor (For 1 hour)			
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
03,017	Machinery	--	0.0003	16,140,000,000 TL	4,422,360 TL
04,109	Diesel oil	kg	2.85	1,594,000 TL	4,542,900 TL
04,110	Diesel oil (mach. oiling)	kg	0.57	3,380,000 TL	1,926,600 TL
01,011	Firing foreman	hour	1	2,831,000 TL	2,831,000 TL
01.403	Engine driver	month	0.01	679,440,000 TL	6,794,400 TL
01.502	Qualified worker	hour	4	2,004,000 TL	8,016,000 TL
				Total	28,533,260 TL
Price Analysis					
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
04.613/8E	Epoxy resin	kg	0.0933	8,400,000 TL	783,367 TL
03,517	Compressor	hour	0.07	28,533,260 TL	1,997,328 TL
01.507	First class worker	hour	0.35	2,712,850 TL	949,498 TL
236.301	Opening hole	cm	6	1,274,000 TL	7,644,000 TL
					11,374,193 TL
	25 % Contractor profit				2,843,548 TL
(For 1 piece)	TOTAL				***14,217,741 TL***

Table B.14 Unit price analysis for 1 piece of anchorage (Ø20 – 25 cm of depth)

POSE NUMBER	**DESCRIPTION**				
Ø (mm)	20	Anchorage (Ø20 – 25 cm of depth)			
Ø-HOLE (mm)	25				
Depth of anchorage (cm)	25				
Supplementary Analysis	Pose No: 03,517	Price of compressor (For 1 hour)			
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
03,017	Machinery	--	0.0003	16,140,000,000 TL	4,422,360 TL
04,109	Diesel oil	kg	2.85	1,594,000 TL	4,542,900 TL
04,110	Diesel oil (mach. oiling)	kg	0.57	3,380,000 TL	1,926,600 TL
01,011	Firing foreman	hour	1	2,831,000 TL	2,831,000 TL
01.403	Engine driver	month	0.01	679,440,000 TL	6,794,400 TL
01.502	Qualified worker	hour	4	2,004,000 TL	8,016,000 TL
				Total	28,533,260 TL
Price Analysis					
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
04.613/8E	Epoxy resin	kg	0.07772	8,400,000 TL	652,806 TL
03,517	Compressor	hour	0.07	28,533,260 TL	1,997,328 TL
01.507	First class worker	hour	0.29	2,712,850 TL	786,727 TL
236.301	Opening hole	cm	5	1,274,000 TL	6,370,000 TL
					9,806,861 TL
	25 % Contractor profit				2,451,715 TL
(For 1 piece)	TOTAL				***12,258,576 TL***

Table B.15 Unit price analysis for welding for jacketing

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
	Material				
04.112	Electricity Energy	kwh	0.7	155,000 TL	108,500 TL
04.122	Welding Electrode	Pieces	2	75,000 TL	150,000 TL
	Labor				
01.021	Welding Master Worker	hour	0.15	2,831,000 TL	424,650 TL
01.501	Ordinary Worker (Carrying and Picking up Waste are included)	hour	0.125	1,853,000 TL	231,625 TL
	Material + Labor				914,775 TL
	25 % Contractor profit				228,694 TL
(For 1 m^2)	TOTAL				***1,143,469 TL***

B.2 UNIT PRICE ANALYSIS PROVIDED FOR REHABILITATION WITH CARBON REINFORCED POLYMERS

Table B.16 Unit price analysis for the application of 1 layer CFRP on the interior face of the wall inside building.

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
	Material				
	Carbon fiber	m^2	0.6462	65.200.000 TL	42.132.240 TL
	Conressive 1305 + Putty (for smooth surface)	kg	0.3393	23,961,000 TL	8,128,889 TL
	Saturant	kg	0.4750	23,961,000 TL	11,380,445 TL
03.139	Mixer		0.002139	456,000,000 TL	975,384 TL
03.517	Compressor	hour	0.05	28,533,260 TL	1,426,663 TL
04.112	Electricity Energy (For mixing resin)	Kwh	0.85	155,000 TL	131,750 TL
	Labor				
01.010	Insulation Foreman	hour	0.75	2,831,000 TL	2,123,250 TL
01.213	Assistant Bricklayer (Holding up and picking up waste are included)	hour	0.67	2,004,000 TL	1,336,668 TL
	Material + Labor				67,635,289 TL
	25 % Contractor profit				16,908,822 TL
(For 1 m^2)	TOTAL				***84,544,111 TL***

Table B.17 Unit price analysis for the application of 1 layer CFRP on the exterior face of the wall inside the building.

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
	Material				
	Carbon fiber	m^2	0.7887	65.200.000 TL	51.425.280 TL
	Conressive 1305 + Putty (for smooth surface)	kg	0.4141	23,961,000 TL	9,921,865TL
	Saturant	kg	0.5797	23,961,000 TL	13,890,611 TL
03.139	Mixer		0.002611	456,000,000 TL	1,190,624 TL
03.517	Compressor	hour	0.05	28,533,260 TL	1,426,663 TL
04.112	Electricity Energy (For mixing resin)	Kwh	1.04	155,000 TL	160,810 TL
	Labor				
01.010	Insulation Foreman	hour	0.92	2,831,000 TL	2,591,572 TL
01.213	Assistant Bricklayer (Holding up and picking up waste are included)	hour	0.81	2,004,000 TL	1,631,495 TL
	Material + Labor				82,238,919 TL
	25 % Contractor profit				20,559,730 TL
(For 1 m^2)	TOTAL				***102,798,648 TL***

Table B.18 Unit price analysis for the application of 1 layer CFRP on the exterior face of the wall outside the building.

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
	Material				
	Carbon fiber	m^2	0.7887	65.200.000 TL	51.425.280 TL
	Conressive 1305 + Putty (for smooth surface)	kg	0.7256	23,961,000 TL	17,386,887 TL
	Saturant	kg	0.5797	23,961,000 TL	13,890,611 TL
03.139	Mixer		0.003429	456,000,000 TL	1,563,875 TL
03.517	Compressor	hour	0.10	28,533,260 TL	2,853,260 TL
04.112	Electricity Energy (For mixing resin)	Kwh	1.30	155,000 TL	201,012 TL
	Labor				
01.010	Insulation Foreman	hour	0.92	2,831,000 TL	2,591,572 TL
01.213	Assistant Bricklayer (Holding up and picking up waste are included)	hour	0.81	2,004,000 TL	1,631,495 TL
	Material + Labor				91,544,058 TL
	25 % Contractor profit				22,886,014 TL
(For 1 m^2)	TOTAL				***114,430,072 TL***

Table B.19 Unit price analysis for the application of 2 layers CFRP (orthogonal) for lap splice regions inside the building

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
	Material				
	Carbon fiber	m^2	2.1	65,200,000 TL	136,920,000 TL
	Conressive 1305 + Putty (for smooth surface)	kg	0.5513	23,961,000 TL	13,208,501 TL
	Saturant (2 layers)	kg	0.7718 + 0.5513	23,961,000 TL	31,700,403 TL
03.139	Mixer		0.00492422	456,000,000 TL	2,245,445 TL
03.517	Compressor	hour	0.05	28,533,260 TL	1,426,663 TL
04.112	Electricity Energy (For mixing resin)	Kwh	1.96	155,000 TL	303,278 TL
	Labor				
01.010	Insulation	hour	2.44	2,831,000 TL	6,900,070 TL
01.213	Assistant Bricklayer (Holding up and picking up waste	hour	2.17	2,004,000 TL	4,343,861 TL
	Material + Labor				197,048,221 TL
	25 % Contractor profit				49,262,055 TL
(For 1 m^2)	TOTAL				***246,310,276 TL***

Table B.20 Unit price analysis for the application of 2 layers CFRP (orthogonal) for lap splice regions outside the building

POSE NUMBER	DESCRIPTION	UNIT	AMOUNT	UNIT PRICE (TL)	TOTAL
	Material				
	Carbon fiber	m^2	2.1	65,200,000 TL	136,920,000 TL
	Conressive 1305 + Putty (for smooth surface)	kg	0.8820	23,961,000 TL	34,342,103 TL
	Saturant (2 layers)	kg	0.7718 + 0.5513	23,961,000 TL	13,208,501 TL
03.139	Mixer		0.0057932	456,000,000 TL	2,641,700 TL
03.517	Compressor	hour	0.10	28,533,260 TL	2,853,326 TL
04.112	Electricity Energy (For mixing resin)	Kwh	2.30	155,000 TL	356,797 TL
	Labor				
01.010	Insulation Foreman	hour	2.44	2,831,000 TL	6,900,070 TL
01.213	Assistant Bricklayer (Holding up and picking up waste are included)	hour	2.17	2,004,000 TL	4,343,861 TL
	Material + Labor				206,849,759 TL
	25 % Contractor profit				51,712,440 TL
(For 1 m^2)	TOTAL				***258,562,199 TL***

Table B.21 Unit price analysis for making anchorage with CFRP Ø 8.5 - 20 cm depth

POSE NUMBER	DESCRIPTION				
Width of strip (cm)	5	**Anchorage** Ø 8.5 depth : 20 cm			
Ø (mm)	8,5				
Ø-HOLE (mm)	10				
Depth of anchorage (cm)	20				
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
Carbon fiber	Carbon fiber sheet	m^2	0.0160	65,200,000 TL	1,043,200 TL
Saturant	Saturant	kg	0.0061	23,961,000 TL	145,387 TL
04.278	Galvanized wire	kg	0.0700	1,250,000 TL	87,500 TL
03,517	Compressor	hour	0.07	28,533,260 TL	1,997,328 TL
01.507	First class worker	hour	0.23	2,712,850 TL	623,956 TL
236.102	Opening hole	cm	1	1,840,000 TL	1,840,000 TL
					5,737,371 TL
	25 % Contractor profit				1,434,343 TL
(For 1 piece)	TOTAL				***7,171,714 TL***

Table B.22 Unit price analysis for making anchorage with CFRP Ø 8.5 - 6 cm depth

POSE NUMBER	DESCRIPTION				
Width of strip (cm)	5	**Anchorage** Ø 8.5 depth : 6 cm			
Ø (mm)	8.5				
Ø-HOLE (mm)	10				
Depth of anchorage (cm)	6				
POSE NUMBER	**Material + Labor**	**UNIT**	**AMOUNT**	**UNIT PRICE (TL)**	**TOTAL**
Carbon fiber	Carbon fiber sheet	m^2	0.006	65,200,000 TL	391,200 TL
Saturant	Saturant	kg	0.0017	23,961,000 TL	39,982 TL
04.278	Galvanized wire	kg	0.07	1,250,000 TL	87,500 TL
03,517	Compressor	hour	0.07	28,533,260 TL	1,997,328 TL
01.507	First class worker	hour	0.06	2,712,850 TL	157,345 TL
236.102	Opening hole	cm	1	1,319,500 TL	1,319,500 TL
					3,992,855 TL
	25 % Contractor profit				998,214 TL
(For 1 piece)	TOTAL				***4,991,069 TL***

APPENDIX C

EVALUATION OF RESULTS

C.1 EVALUATION OF SEISMIC PERFORMANCES

C.1.1 BUILDING-1

Table C.1 Comparison of capacity curve parameters in the x-direction for Building 1

Rehabilitation Strategy	V_y (KN)	Δ_y (m)	Percent Change (%) - A	V_u (KN)	Δ_u (m)	Percent Change (%) – B
Shear Wall	3,675.95	0.018	309	4,196.15	0.062	271
Reduced-Shear Wall	2,869.19	0.013	219	3,586.31	0.091	217
TUBITAK CFRP Model	2,671.33	0.016	197	3,154.26	0.049	179
Erdem's CFRP Model	1,656.27	0.011	84	2,467.70	0.113	118
Original Building	898.46	0.025	0	1,132.02	0.175	0

Table C.2 Comparison of capacity curve parameters in the y-direction for Building 1

Rehabilitation Strategy	V_y (KN)	Δ_y (m)	Percent Change (%) - A	V_u (KN)	Δ_u (m)	Percent Change (%) – B
Shear Wall	3,200.14	0.014	361	3,832.32	0.119	225
Reduced-Shear Wall	2,516.04	0.010	262	3,066.05	0.188	160
TUBITAK CFRP Model	2,479.01	0.029	257	3,073.04	0.084	161
Erdem's CFRP Model	1,524.46	0.014	119	2,585.73	0.124	120
Original Building	694.73	0.014	0	1,177.62	0.165	0

where:

Δ_y **:** Yield displacement

Δ_u **:** Ultimate displacement

V_y: Yield base shear force

V_u: Ultimate base shear force

Percent Change (%) – A $= \dfrac{(Vyi - Vyo)}{Vyo}$

Percent Change (%) – B $= \dfrac{(Vui - Vuo)}{Vuo}$

Vyi : Yield base shear force corresponding to the rehabilitation strategy

Vyo : Yield base shear force corresponding to the original building

Vui : Ultimate base shear force corresponding to the rehabilitation strategy

Vuo : Ultimate base shear force corresponding to the original building

Table C.3 Initial and yield stiffness of Building-1 in the x-direction

Rehabilitation Strategy	K_i	**Percent Change (%) - C**	K_y	**Percent Change (%) - D**
Shear Wall	293,761	544	170,148	377
Reduced-Shear Wall	293,761	544	215,728	505
Erdem's CFRP Model	231,785	408	201,975	467
TUBITAK CFRP Model	218,072	378	154,792	334
Original Building	45,600	0	35,653	0

where:

Percent Change (%) – C $= \dfrac{(Ki - Kio)}{Kio}$

Percent Change (%) – D $= \dfrac{(Ky - Kyo)}{Kyo}$

Ki: Initial stiffness

Kio: Initial stiffness corresponding to the original building

Ky: Yield stiffness

Kyo: Yield stiffness corresponding to the original building

Table C.4 Initial and yield stiffness of Building-1 in the y-direction

Rehabilitation Strategy	K_i	Percent Change (%) - C	K_y	Percent Change (%) - D
Shear Wall	249,113	346	249,113	409
Reduced-Shear Wall	249,113	346	235,304	381
Erdem's CFRP Model	156,695	181	86,077	76
TUBITAK CFRP Model	154,011	176	108,890	123
Original Building	55,850	0	48,925	0

Table C.5 Maximum interstory drift ratios of Building-1under code-based spectrum

Rehabilitation Strategy	Maximum Interstory Drift Ratio (%)		Floor level	
	x-direction	y-direction	x-direction	y-direction
Original Building	1.96	1.31	2	3
Erdem's CFRP Model	1.28	1.13	2	2
Reduced-Shear Wall	0.66	1.34	4	4
TUBITAK CFRP Model	0.56	0.77	2	2
Shear Wall	0.48	0.87	4	4

Table C.6 Maximum interstory drift ratios of Building-1under site-specific spectrum

Rehabilitation Strategy	Maximum Interstory Drift Ratio (%)		Floor level	
	x-direction	y-direction	x-direction	y-direction
Original Building	1.96	1.31	2	3
Erdem's CFRP Model	0.99	0.87	2	2
Reduced-Shear Wall	0.57	0.62	4	4
TUBITAK CFRP Model	0.56	0.68	2	2
Shear Wall	0.48	0.55	4	4

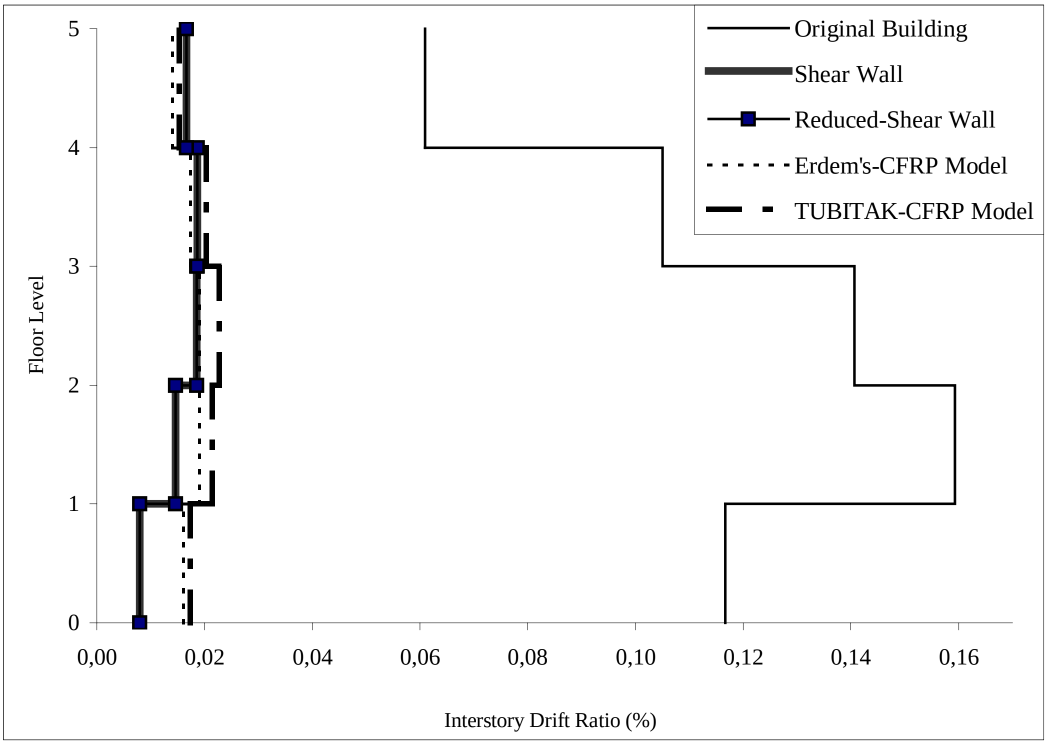

Figure C.1 Interstory Drift Ratios of Building-1 in the x-direction (Linear Static Analysis)

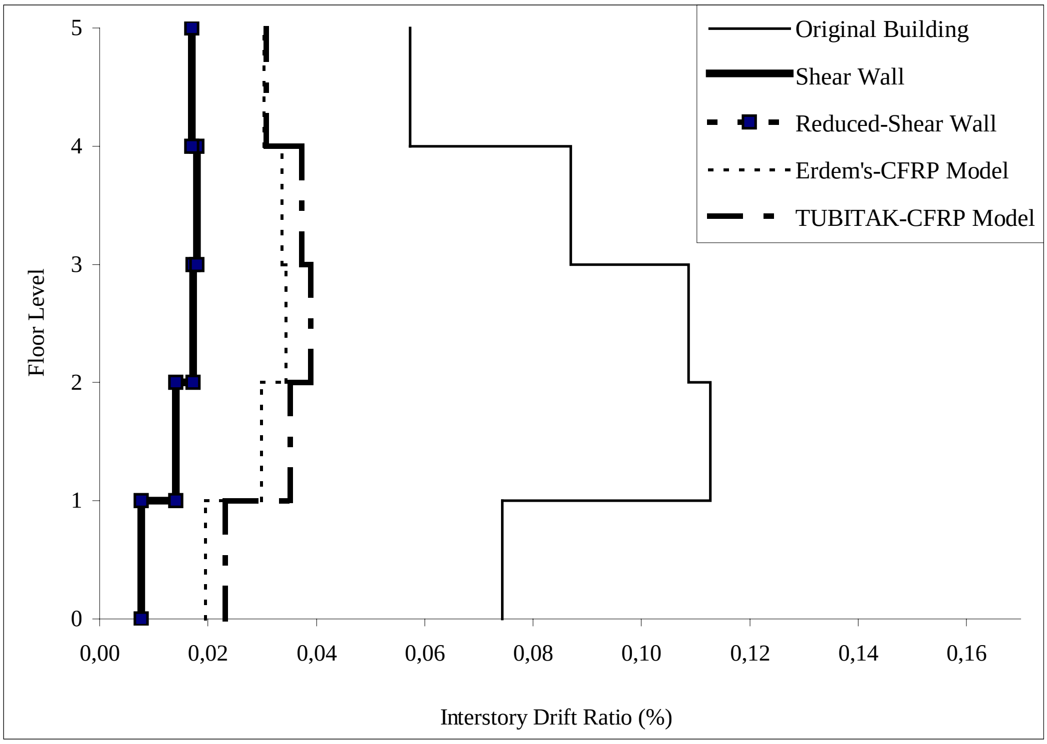

Figure C.2 Interstory Drift Ratios of Building-1 in the y-direction (Linear Static Analysis)

Table C.7 Number of confirming and nonconforming members of Building-1

(Under code-based spectrum in the x-direction, boundary element: C)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	26	0	-	-
	2	0	26	-	-
	3	26	0	-	-
	4	0	26	-	-
	5	0	26	-	-
Shear Wall	1	0	10	0	4
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Reduced Shear Wall	1	0	10	0	4
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Erdem's CFRP Model	1	26	0	-	-
	2	2	24	-	-
	3	0	26	-	-
	4	0	26	-	-
	5	0	26	-	-
TUBITAK CFRP Model	1	3	23	-	-
	2	0	26	-	-
	3	0	26	-	-
	4	0	26	-	-
	5	0	26	-	-

Table C.8 Number of confirming and nonconforming members of Building-1

(Under code-based spectrum in the y-direction, boundary element: C)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	26	0	-	-
	2	0	26	-	-
	3	0	26	-	-
	4	0	26	-	-
	5	0	26	-	-
Shear Wall	1	10	0	4	0
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Reduced Shear Wall	1	10	0	4	0
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Erdem's CFRP Model	1	25	1	-	-
	2	4	22	-	-
	3	4	22	-	-
	4	0	26	-	-
	5	0	26	-	-
TUBITAK CFRP Model	1	18	8	-	-
	2	4	22	-	-
	3	4	22	-	-
	4	0	26	-	-
	5	0	26	-	-

Table C.9 Number of confirming and nonconforming members of Building-1 (Under code-based spectrum in the x-direction, boundary element: NC)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	26	0	-	-
	2	6	20	-	-
	3	26	0	-	-
	4	0	26	-	-
	5	0	26	-	-
Shear Wall	1	0	10	4	0
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Reduced Shear Wall	1	10	0	4	0
	2	0	10	0	4
	3	0	10	0	4
	4	2	8	0	4
	5	9	1	0	4
Erdem's CFRP Model	1	26	0	-	-
	2	9	17	-	-
	3	17	9	-	-
	4	0	26	-	-
	5	0	26	-	-
TUBITAK CFRP Model	1	22	4	-	-
	2	5	21	-	-
	3	1	25	-	-
	4	0	26	-	-
	5	0	26	-	-

Table C.10 Number of confirming and nonconforming members of Building-1 (Under code-based spectrum in the y-direction, boundary element: NC)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	26	0	-	-
	2	2	24	-	-
	3	0	26	-	-
	4	0	26	-	-
	5	0	26	-	-
Shear Wall	1	10	0	4	0
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Reduced Shear Wall	1	10	0	4	0
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Erdem's CFRP Model	1	26	0	-	-
	2	6	20	-	-
	3	6	20	-	-
	4	4	22	-	-
	5	0	26	-	-
TUBITAK CFRP Model	1	24	2	-	-
	2	8	18	-	-
	3	6	20	-	-
	4	2	24	-	-
	5	0	26	-	-

Table C.11 Number of confirming and nonconforming members of Building-1 (Under site-specific spectrum in the x-direction, boundary element: C)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	26	0	-	-
	2	0	26	-	-
	3	0	26	-	-
	4	0	26	-	-
	5	0	26	-	-
Shear Wall	1	10	0	4	0
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Reduced Shear Wall	1	10	0	4	0
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Erdem's CFRP Model	1	25	1	-	-
	2	4	22	-	-
	3	4	22	-	-
	4	0	26	-	-
	5	0	26	-	-
TUBITAK CFRP Model	1	18	8	-	-
	2	4	22	-	-
	3	4	22	-	-
	4	0	26	-	-
	5	0	26	-	-

Table C.12 Number of confirming and nonconforming members of Building-1
(Under site-specific spectrum in the y-direction, boundary element: C)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	26	0	-	-
	2	0	26	-	-
	3	0	26	-	-
	4	0	26	-	-
	5	0	26	-	-
Shear Wall	1	0	10	0	4
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Reduced Shear Wall	1	2	8	0	4
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Erdem's CFRP Model	1	21	5	-	-
	2	2	24	-	-
	3	0	26	-	-
	4	0	26	-	-
	5	0	26	-	-
TUBITAK CFRP Model	1	9	17	-	-
	2	4	22	-	-
	3	4	22	-	-
	4	0	26	-	-
	5	0	26	-	-

Table C.13 Number of confirming and nonconforming members of Building-1

(Under site-specific spectrum in the x-direction, boundary element: NC)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	26	0	-	-
	2	6	20	-	-
	3	26	0	-	-
	4	0	26	-	-
	5	0	26	-	-
Shear Wall	1	0	10	4	0
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Reduced Shear Wall	1	8	2	4	0
	2	0	10	0	4
	3	0	10	0	4
	4	1	9	0	4
	5	9	1	0	4
Erdem's CFRP Model	1	26	0	-	-
	2	5	21	-	-
	3	6	20	-	-
	4	0	26	-	-
	5	0	26	-	-
TUBITAK CFRP Model	1	21	5	-	-
	2	3	23	-	-
	3	0	26	-	-
	4	0	26	-	-
	5	0	26	-	-

Table C.14 Number of confirming and nonconforming members of Building-1 (Under site-specific spectrum in the y-direction, boundary element: NC)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	26	0	-	-
	2	2	24	-	-
	3	0	26	-	-
	4	0	26	-	-
	5	0	26	-	-
Shear Wall	1	10	0	4	0
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Reduced Shear Wall	1	10	0	4	0
	2	0	10	0	4
	3	0	10	0	4
	4	0	10	0	4
	5	0	10	0	4
Erdem's CFRP Model	1	26	0	-	-
	2	4	22	-	-
	3	4	22	-	-
	4	0	26	-	-
	5	0	26	-	-
TUBITAK CFRP Model	1	24	2	-	-
	2	4	22	-	-
	3	4	22	-	-
	4	0	26	-	-
	5	0	26	-	-

C.1.2 BUILDING-2

Table C.15 Comparison of Capacity Curve Parameters in the x-direction for Building 2

Rehabilitation Strategy	V_y (KN)	Δ_y (m)	Percent Change (%) - A	V_u (KN)	Δ_u (m)	Percent Change (%) – B
Shear Wall	6,790.18	0.017	418	9,685.25	0.109	359
Reduced-Shear Wall	5,682.97	0.015	333	8,517.74	0.162	304
TUBITAK CFRP Model	3,542.00	0.020	170	4,281.93	0.058	103
Erdem's CFRP Model	2,063.67	0.011	57	3,601.47	0.117	71
Original Building	1,311.03	0.024	0	2,110.14	0.236	0

Table C.16 Comparison of Capacity Curve Parameters in the y-direction for Building 2

Rehabilitation Strategy	V_y (KN)	Δ_y (m)	Percent Change (%) - A	V_u (KN)	Δ_u (m)	Percent Change (%) – B
Shear Wall	11,127.74	0.028	815	13,012.6	0.065	684
Reduced-Shear Wall	10,225.15	0.027	740	12,091.1	0.049	628
TUBITAK CFRP Model	4,817.00	0.024	296	5,417.13	0.054	226
Erdem's CFRP Model	2,713.98	0.013	123	4,350.68	0.119	162
Original Building	1,216.72	0.028	0	1,660.18	0.204	0

Table C.17 Initial and yield stiffness of Building-2 in the x-direction

Rehabilitation Strategy	K_i	Percent Change (%) - C	K_y	Percent Change (%) - D
Shear Wall	423,214	540	397,087	624
Reduced-Shear Wall	423,214	540	373,880	582
Erdem's CFRP Model	238,921	261	177,100	223
TUBITAK CFRP Model	184,000	178	196,540	258
Original Building	66,176	0	54,855	0

Table C.18 Initial and yield stiffness of Building-2 in the y-direction

Rehabilitation Strategy	K_i	Percent Change (%) - C	K_y	Percent Change (%) - D
Shear Wall	642,898	1,040	394,601	808
Reduced-Shear Wall	642,898	1,040	384,404	785
Erdem's CFRP Model	309,100	448	200,708	362
TUBITAK CFRP Model	221,568	293	215,395	396
Original Building	56,400	0	43,454	0

Table C.19 Maximum Interstory Drift Ratios of Building-2 under code-based spectrum

Rehabilitation Strategy	**Maximum Interstory Drift Ratio (%)**		**Floor level**	
	x-direction	**y-direction**	**x-direction**	**y-direction**
Original Building	2.07	1.71	2	3
Erdem's CFRP Model	1.23	0.99	2	2
TUBITAK CFRP Model	0.58	0.47	2	2
Reduced-Shear Wall	0.33	0.20	4	4
Shear Wall	0.25	0.18	4	4

Table C.20 Maximum Interstory Drift Ratios of Building-2 under site-specific spectrum

Rehabilitation Strategy	**Maximum Interstory Drift Ratio (%)**		**Floor level**	
	x-direction	**y-direction**	**x-direction**	**y-direction**
Original Building	1.01	1.01	2	3
Erdem's CFRP Model	0.65	0.54	2	2
TUBITAK CFRP Model	0.31	0.18	2	2
Reduced-Shear Wall	0.14	0.09	4	4
Shear Wall	0.14	0.09	4	4

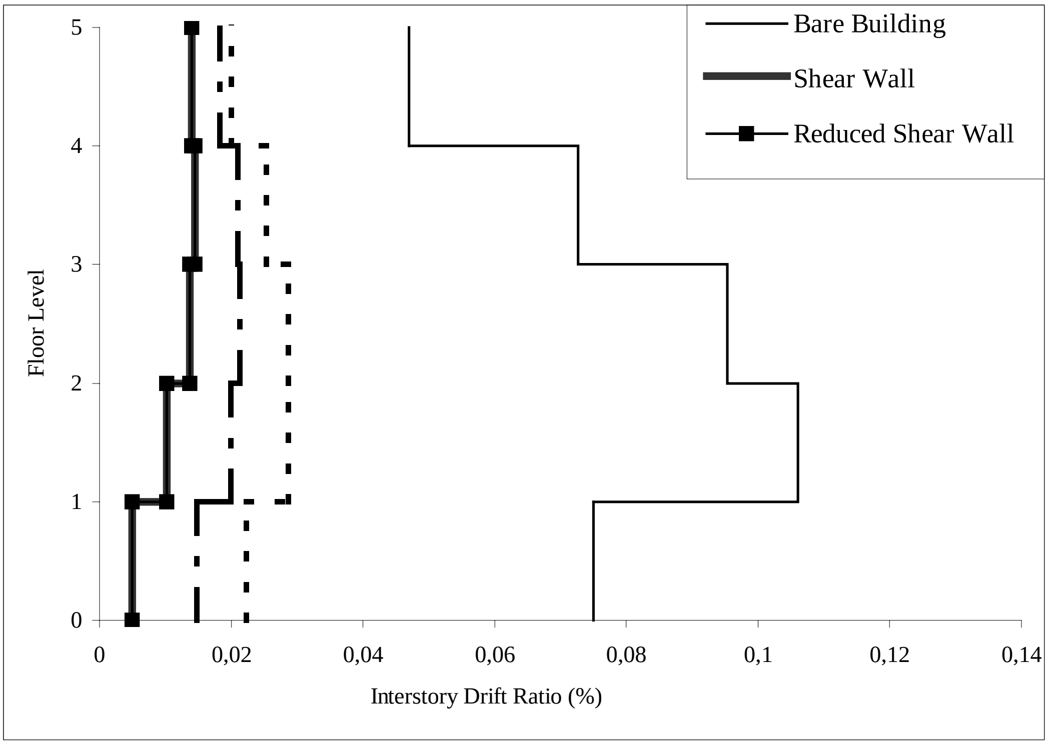

Figure C.3 Interstory Drift Ratios-Linear Static Analysis- Building-2 in the x-direction

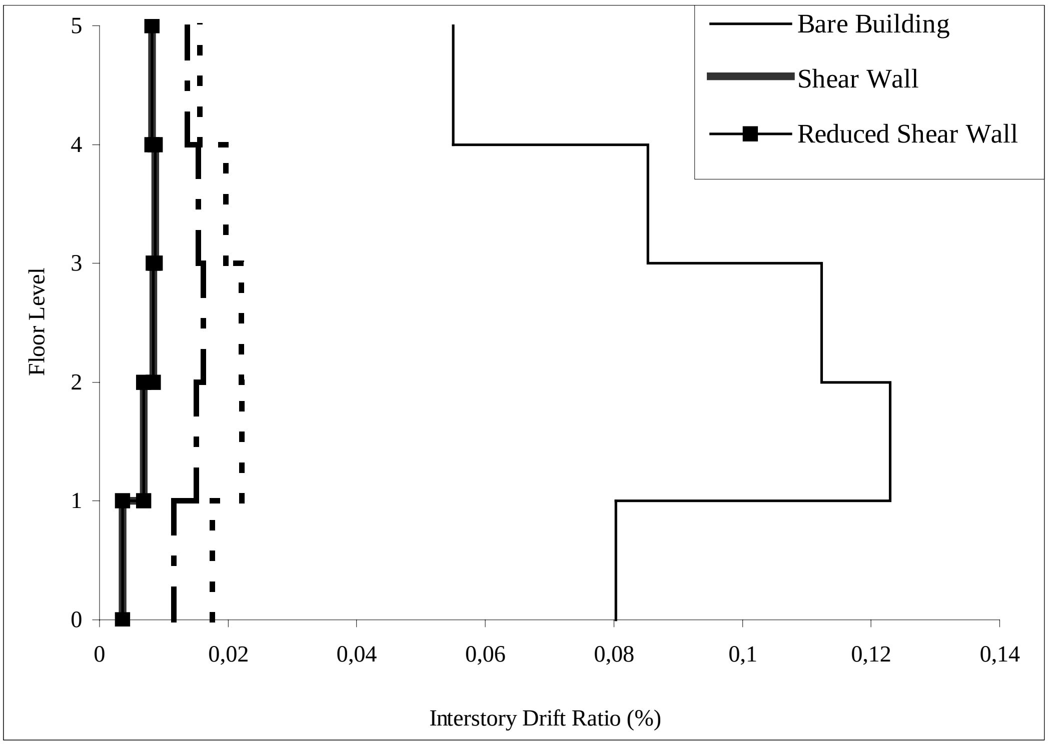

Figure C.4 Interstory Drift Ratios-Linear Static Analysis- Building-2 in the y-direction

Table C.21 Number of confirming and nonconforming members of Building-2

(Under code-based spectrum in the x-direction, boundary element: C)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	34	0	-	-
	2	14	20	-	-
	3	15	19	-	-
	4	16	18	-	-
	5	2	32	-	-
Shear Wall	1	0	12	0	7
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Reduced Shear Wall	1	0	12	0	7
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Erdem's CFRP Model	1	34	0	-	-
	2	11	23	-	-
	3	9	25	-	-
	4	0	34	-	-
	5	0	34	-	-
TUBITAK CFRP Model	1	13	21	-	-
	2	2	32	-	-
	3	0	34	-	-
	4	0	34	-	-
	5	0	34	-	-

Table C.22 Number of confirming and nonconforming members of Building-2

(Under code-based spectrum in the y-direction, boundary element: C)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	34	0	-	-
	2	9	25	-	-
	3	13	21	-	-
	4	7	27	-	-
	5	0	34	-	-
Shear Wall	1	0	12	0	7
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Reduced-Shear Wall	1	0	12	2	5
	2	0	12	2	5
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Erdem's CFRP Model	1	34	0	-	-
	2	6	28	-	-
	3	0	34	-	-
	4	0	34	-	-
	5	0	34	-	-
TUBITAK CFRP Model	1	13	21	-	-
	2	5	29	-	-
	3	0	34	-	-
	4	0	34	-	-
	5	0	34	-	-

Table C.23 Number of confirming and nonconforming members of Building-2 (Under code-based spectrum in the x-direction, boundary element: NC)

Rehabilitation Strategy	Floor Level	# of nonconfirming columns	# of confirming columns	# of nonconfirming shear walls	# of confirming shear walls
Original Building	1	34	0	-	-
	2	22	12	-	-
	3	30	4	-	-
	4	32	2	-	-
	5	24	10	-	-
Shear Wall	1	0	12	0	7
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Reduced Shear Wall	1	3	9	2	5
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Erdem's CFRP Model	1	34	0	-	-
	2	20	14	-	-
	3	18	16	-	-
	4	2	32	-	-
	5	0	34	-	-
TUBITAK CFRP Model	1	34	0	-	-
	2	13	21	-	-
	3	2	32	-	-
	4	0	34	-	-
	5	0	34	-	-

Table C.24 Number of confirming and nonconforming members of Building-2
(Under code-based spectrum in the y-direction, boundary element: NC)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	34	0	-	-
	2	11	23	-	-
	3	34	0	-	-
	4	32	2	-	-
	5	0	34	-	-
Shear Wall	1	34	22	4	3
	2	0	12	4	3
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Reduced-Shear Wall	1	0	12	4	3
	2	0	12	4	3
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Erdem's CFRP Model	1	34	0	-	-
	2	19	15	-	-
	3	10	24	-	-
	4	0	34	-	-
	5	0	34	-	-
TUBITAK CFRP Model	1	34	0	-	-
	2	13	21	-	-
	3	3	31	-	-
	4	0	34	-	-
	5	0	34	-	-

Table C.25 Number of confirming and nonconforming members of Building-2

(Under site-specific spectrum in the x-direction, boundary element: C)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	32	2	-	-
	2	10	24	-	-
	3	11	23	-	-
	4	0	34	-	-
	5	0	34	-	-
Shear Wall	1	0	12	0	7
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Reduced-Shear Wall	1	0	12	0	7
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Erdem's CFRP Model	1	10	24	-	-
	2	1	33	-	-
	3	0	34	-	-
	4	0	34	-	-
	5	0	34	-	-
TUBITAK CFRP Model	1	13	21	-	-
	2	2	32	-	-
	3	0	34	-	-
	4	0	34	-	-
	5	0	34	-	-

Table C.26 Number of confirming and nonconforming members of Building-2

(Under site-specific spectrum in the y-direction, boundary element: C)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	0	34	-	-
	2	5	29	-	-
	3	5	29	-	-
	4	0	34	-	-
	5	0	34	-	-
Shear Wall	1	0	12	0	7
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Reduced-Shear Wall	1	0	12	0	7
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Erdem's CFRP Model	1	6	28	-	-
	2	0	34	-	-
	3	0	34	-	-
	4	0	34	-	-
	5	0	34	-	-
TUBITAK CFRP Model	1	13	21	-	-
	2	5	29	-	-
	3	0	34	-	-
	4	0	34	-	-
	5	0	34	-	-

Table C.27 Number of confirming and nonconforming members of Building-2 (Under site-specific spectrum in the x-direction, boundary element: NC)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	34	0	-	-
	2	14	20	-	-
	3	14	20	-	-
	4	12	22	-	-
	5	0	34	-	-
Shear Wall	1	0	12	0	7
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Reduced-Shear Wall	1	0	12	0	7
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Erdem's CFRP Model	1	34	0	-	-
	2	14	20	-	-
	3	5	29	-	-
	4	0	34	-	-
	5	0	34	-	-
TUBITAK CFRP Model	1	34	0	-	-
	2	13	21	-	-
	3	2	32	-	-
	4	0	34	-	-
	5	0	34	-	-

Table C.28 Number of confirming and nonconforming members of Building-2 (Under site-specific spectrum in the y-direction, boundary element: NC)

Rehabilitation Strategy	Floor Level	# of nonconforming columns	# of confirming columns	# of nonconforming shear walls	# of confirming shear walls
Original Building	1	34	0	-	-
	2	11	23	-	-
	3	13	21	-	-
	4	7	27	-	-
	5	0	34	-	-
Shear Wall	1	0	12	0	7
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Reduced-Shear Wall	1	0	12	0	7
	2	0	12	0	7
	3	0	12	0	7
	4	0	12	0	7
	5	0	12	0	7
Erdem's CFRP Model	1	34	0	-	-
	2	9	25	-	-
	3	2	32	-	-
	4	0	34	-	-
	5	0	34	-	-
TUBITAK CFRP Model	1	34	0	-	-
	2	13	21	-	-
	3	3	31	-	-
	4	0	34	-	-
	5	0	34	-	-

C.2 COST-BENEFIT ANALYSIS - MAXIMUM DRIFT RATIOS

C.2.1 BUILDING-1

Table C.29 Expected damages for Building-1 in the x-direction

Rehabilitation Strategy	Maximum drift ratio (Δ / h %)	Damage (%)	COST OF DAMAGE
Original Building	1.96	86.58	365,213,614,467 TL
Erdem's CFRP Model	0.99	71.92	303,385,570,671 TL
TUBITAK CFRP Model	0.56	39.84	168,036,574,891 TL
Reduced-Shear Wall	0.57	24.69	104,144,392,463 TL
Shear Wall	0.48	17.10	72,115,573,650 TL

Table C.30 Expected damages for Building-1 in the y-direction

Rehabilitation Strategy	Maximum drift ratio (Δ / h %)	Damage (%)	COST OF DAMAGE
Original Building	1.31	60.38	254,709,970,844 TL
Erdem's CFRP Model	0.87	59.83	252,378,399,111 TL
TUBITAK CFRP Model	0.68	43.14	181,955,827,631 TL
Reduced-Shear Wall	0.62	25.82	108,906,551,580 TL
Shear Wall	0.55	20.26	85,459,786,956 TL

Table C.31 Expected benefits calculated considering max. drift ratios-Building-1 (in the x-direction)

Rehabilitation Strategy	EXPECTED BENEFITS
Original Building	--
Shear Wall	293,098,040,817 TL
Reduced-Shear Wall	261,069,222,004 TL
TUBITAK CFRP Model	197,177,039,576 TL
Erdem's CFRP Model	61,828,043,797 TL

Table C.32 Expected benefits calculated considering max. drift ratios-Building-1 (in the y-direction)

Rehabilitation Strategy	EXPECTED BENEFITS
Original Building	--
Shear Wall	169,250,183,888 TL
Reduced-Shear Wall	145,803,419,264 TL
TUBITAK CFRP Model	72,754,143,212 TL
Erdem's CFRP Model	2,331,571,733 TL

Table C.33 Expected NPV and BCR for Building-1 in the x-direction

Rehabilitation Strategy	NPV	BCR
Shear Wall	171,561,661,500 TL	2.41
Reduced-Shear Wall	139,532,842,686 TL	2.15
TUBITAK CFRP Model	60,064,010,819 TL	1.44
Erdem's CFRP Model	-75,284,984,960 TL	0.45

Table C.34 Expected NPV and BCR for Building-1 in the y-direction

Rehabilitation Strategy	NPV	BCR
Shear Wall	47,713,804,571 TL	1.39
Reduced-Shear Wall	24,267,039,946 TL	1.20
TUBITAK CFRP Model	-64,358,885,544 TL	0.53
Erdem's CFRP Model	-134,781,457,024 TL	0.02

C.2.2 BUILDING-2

Table C.35 Expected damages for Building-2 in the x-direction

Rehabilitation Strategy	Maximum drift ratio (Δ / h %)	Damage (%)	COST OF DAMAGE
Original Building	1.01	36.52	156,306,351,996 TL
Erdem's CFRP Model	0.65	49.58	212,190,070,441 TL
TUBITAK CFRP Model	0.31	16.33	69,885,552,899 TL
Reduced-Shear Wall	0.14	2.43	10,408,734,362 TL
Shear Wall	0.14	2.51	10,737,682,954 TL

Table C.36 Expected damages for Building-2 in the y-direction

Rehabilitation Strategy	Maximum drift ratio (Δ / h %)	Damage (%)	COST OF DAMAGE
Original Building	1.01	42.68	182,664,057,835 TL
Erdem's CFRP Model	0.54	38.96	166,747,998,612 TL
TUBITAK CFRP Model	0.18	5.02	21,484,955,334 TL
Reduced-Shear Wall	0.09	1.01	4,308,302,775 TL
Shear Wall	0.09	0.91	3,893,395,352 TL

Table C.37 Expected benefits calculated considering max. drift ratios-Building-2 (in the x-direction)

Rehabilitation Strategy	EXPECTED BENEFITS
Original Building	--
Shear Wall	145,897,617,634 TL
Reduced-Shear Wall	145,568,669,043 TL
TUBITAK CFRP Model	86,420,799,098 TL
Erdem's CFRP Model	-55,883,718,444 TL

Table C.38 Expected benefits calculated considering max. drift ratios-Building-2 (in the y-direction)

Rehabilitation Strategy	EXPECTED BENEFITS
Original Building	--
Shear Wall	178,770,662,483 TL
Reduced-Shear Wall	178,355,755,060 TL
TUBITAK CFRP Model	161,179,102,501 TL
Erdem's CFRP Model	15,916,059,223 TL

Table C.39 Expected NPV and BCR for Building-2 in the x-direction

Rehabilitation Strategy	NPV	BCR
Shear Wall	-58,260,145,859 TL	0.71
Reduced-Shear Wall	-58,589,094,451 TL	0.71
TUBITAK CFRP Model	-154,179,441,387 TL	0.36
Erdem's CFRP Model	-296,483,958,929 TL	0.23

Table C.40 Expected NPV and BCR for Building-2 in the y-direction

Rehabilitation Strategy	NPV	BCR
Shear Wall	-25,387,101,010 TL	0.88
Reduced-Shear Wall	-25,802,008,434 TL	0.87
TUBITAK CFRP Model	-79,421,137,983 TL	0.67
Erdem's CFRP Model	-224,684,181,262 TL	0.07

Component check of Building-1 rehabilitated with shear wall (rec
(site-specific spectrum in the x-direction, boundary element: (

Column	Acceptability	Column	Acceptability	Column	Acceptability	Column
A102	√	A202	√	A302	√	A402
A103	√	A203	√	A303	√	A403
A104	√	A204	√	A304	√	A404
A105	√	A205	√	A305	√	A405
A106	√	A206	√	A306	√	A406
A107	√	A207	√	A307	√	A407
D102	√	D202	√	D302	√	D402
D103	√	D203	√	D303	√	D403
D106	√	D206	√	D306	√	D406
D107	√	D207	√	D307	√	D407
TOTAL # OF NONCONFIRMING COLUMNS	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**
Wall	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**	**Wall**
SW23-1	√	SW23-2	√	SW23-3	√	SW23-4
SW34-1	√	SW34-2	√	SW34-3	√	SW34-4
SW56-1	√	SW56-2	√	SW56-3	√	SW56-4
SW67-1	√	SW67-2	√	SW67-3	√	SW67-4
TOTAL # OF NONCONFIRMING SHEAR WALLS	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**

Component check of Building-1 rehabilitated with shear wall (rec
(site-specific spectrum in the x-direction, boundary element: N

Column	Acceptability	Column	Acceptability	Column	Acceptability	Column
A102	X	A202	√	A302	√	A402
A103	X	A203	√	A303	√	A403
A104	X	A204	√	A304	√	A404
A105	X	A205	√	A305	√	A405
A106	X	A206	√	A306	√	A406
A107	X	A207	√	A307	√	A407
D102	√	D202	√	D302	√	D402
D103	X	D203	√	D303	√	D403
D106	√	D206	√	D306	√	D406
D107	X	D207	√	D307	√	D407
TOTAL # OF NONCONFIRMING COLUMNS	***8***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**
Wall	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**	**Wall**
SW23-1	X	SW23-2	√	SW23-3	√	SW23-4
SW34-1	X	SW34-2	√	SW34-3	√	SW34-4
SW56-1	X	SW56-2	√	SW56-3	√	SW56-4
SW67-1	X	SW67-2	√	SW67-3	√	SW67-4
TOTAL # OF NONCONFIRMING SHEAR WALLS	***4***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**

Component check of Building-1 rehabilitated with shear wall (rec
(site-specific spectrum in the y-direction, boundary element: (

Column	Acceptability	Column	Acceptability	Column	Acceptability	Column
A102	√	A202	√	A302	√	A402
A103	√	A203	√	A303	√	A403
A104	X	A204	√	A304	√	A404
A105	X	A205	√	A305	√	A405
A106	√	A206	√	A306	√	A406

A107	√	A207	√	A307	√	A407
D102	√	D202	√	D302	√	D402
D103	√	D203	√	D303	√	D403
D106	√	D206	√	D306	√	D406
D107	√	D207	√	D307	√	D407
TOTAL # OF NONCONFIRMING COLUMNS	***2***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**
Wall	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**	**Wall**
SWYA-1	√	SWYA-2	√	SWYA-3	√	SWYA-4
SWYA-1	√	SWYA-2	√	SWYB-3	√	SWYB-4
SWYPA-1	√	SWYPA-2	√	SWYPA-3	√	SWYPA-4
SWYPB-1	√	SWYPB-2	√	SWYPB-3	√	SWYPB-4
TOTAL # OF NONCONFIRMING SHEAR WALLS	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**

Component check of Building-1 rehabilitated with shear wall (rec
(site-specific spectrum in the y-direction, boundary element: N

Column	Acceptability	Column	Acceptability	Column	Acceptability	Column
A102	X	A202	√	A302	√	A402
A103	X	A203	√	A303	√	A403
A104	X	A204	√	A304	√	A404
A105	X	A205	√	A305	√	A405
A106	X	A206	√	A306	√	A406
A107	X	A207	√	A307	√	A407
D102	X	D202	√	D302	√	D402
D103	X	D203	√	D303	√	D403
D106	X	D206	√	D306	√	D406
D107	X	D207	√	D307	√	D407
TOTAL # OF NONCONFIRMING COLUMNS	***10***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**
SWYA-1	X	SWYA-2	√	SWYA-3	√	SWYA-4
SWYA-1	X	SWYA-2	√	SWYB-3	√	SWYB-4
SWYPA-1	X	SWYPA-2	√	SWYPA-3	√	SWYPA-4
SWYPB-1	X	SWYPB-2	√	SWYPB-3	√	SWYPB-4
TOTAL # OF NONCONFIRMING SHEAR WALLS	***4***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**

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Acceptability	Column	Acceptability
√	A502	√
√	A503	√
√	A504	√
√	A505	√
√	A506	√
√	A507	√
√	D502	√
√	D503	√
√	D506	√
√	D507	√
0	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***
Acceptability	**Wall**	**Acceptability**
√	SW23-5	√
√	SW34-5	√
√	SW56-5	√
√	SW67-5	√
0	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***

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Acceptability	Column	Acceptability
√	A502	X
√	A503	X
√	A504	X
√	A505	X
√	A506	X
X	A507	X
√	D502	X
√	D503	X
√	D506	√
√	D507	X
1	**TOTAL # OF NONCONFIRMING COLUMNS**	***9***
Acceptability	**Wall**	**Acceptability**
√	SW23-5	√
√	SW34-5	√
√	SW56-5	√
√	SW67-5	√
0	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***

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Acceptability	Column	Acceptability
√	A502	√
√	A503	√
√	A504	√
√	A505	√
√	A506	√

√	A507	√
√	D502	√
√	D503	√
√	D506	√
√	D507	√
0	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***
Acceptability	**Wall**	**Acceptability**
√	SWYA-5	√
√	SWYB-5	√
√	SWYPA-5	√
√	SWYPB-5	√
0	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***

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NC

Acceptability	**Column**	**Acceptability**
√	A502	√
√	A503	√
√	A504	√
√	A505	√
√	A506	√
√	A507	√
√	D502	√
√	D503	√
√	D506	√
√	D507	√
0	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***
√	SWYA-5	√
√	SWYB-5	√
√	SWYPA-5	√
√	SWYPB-5	√
0	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***

Component check of Building-2 rehabilitated with shear wall (reduced)
(site-specific spectrum in the x-direction, boundary element: C)

Column	Acceptability	Column	Acceptability	Column	Acceptability	Column	Acceptability	Column	Acceptability
A104	√	A204	√	A304	√	A404	√	A504	√
A110	√	A210	√	A310	√	A410	√	A510	√
C102	√	C202	√	C302	√	C402	√	C502	√
C106	√	C206	√	C306	√	C406	√	C506	√
C108	√	C208	√	C308	√	C408	√	C508	√
C112	√	C212	√	C312	√	C412	√	C512	√
D104	√	D204	√	D304	√	D404	√	D504	√
D110	√	D210	√	D310	√	D410	√	D510	√
E103	√	E203	√	E303	√	E403	√	E503	√
E111	√	E211	√	E311	√	E411	√	E511	√
F104	√	F204	√	F304	√	F404	√	F504	√
F110	√	F210	√	F310	√	F410	√	F510	√
TOTAL # OF NONCONFIRMING COLUMNS	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***
Wall	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**
SWAA-1	√	SWAA-2	√	SWAA-3	√	SWAA-4	√	SWAA-5	√
SWAB-1	√	SWAB-2	√	SWAB-3	√	SWAB-4	√	SWAB-5	√
SWAC-1	√	SWAC-2	√	SWAC-3	√	SWAC-4	√	SWAC-5	√
SWAD-1	√	SWAD-2	√	SWAD-3	√	SWAD-4	√	SWAD-5	√
SWCA-1	√	SWCA-2	√	SWCA-3	√	SWCA-4	√	SWCA-5	√
SWCB-1	√	SWCB-2	√	SWCB-3	√	SWCB-4	√	SWCB-5	√
TOTAL # OF NONCONFIRMING SHEAR WALLS	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***

Component check of Building-2 rehabilitated with shear wall (reduced)
(site-specific spectrum in the x-direction, boundary element: NC)

Column	**Acceptability**	**Column**	**Acceptability**	**Column**	**Acceptability**	**Column**	**Acceptability**	**Column**	**Acceptability**
A104	√	A204	√	A304	√	A404	√	A504	√
A110	√	A210	√	A310	√	A410	√	A510	√
C102	√	C202	√	C302	√	C402	√	C502	√
C106	√	C206	√	C306	√	C406	√	C506	√
C108	√	C208	√	C308	√	C408	√	C508	√
C112	√	C212	√	C312	√	C412	√	C512	√
D104	√	D204	√	D304	√	D404	√	D504	√
D110	√	D210	√	D310	√	D410	√	D510	√
E103	√	E203	√	E303	√	E403	√	E503	√
E111	√	E211	√	E311	√	E411	√	E511	√
F104	√	F204	√	F304	√	F404	√	F504	√
F110	√	F210	√	F310	√	F410	√	F510	√
TOTAL # OF NONCONFIRMING COLUMNS	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***
Wall	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**
SWAA-1	√	SWAA-2	√	SWAA-3	√	SWAA-4	√	SWAA-5	√
SWAB-1	√	SWAB-2	√	SWAB-3	√	SWAB-4	√	SWAB-5	√
SWAC-1	√	SWAC-2	√	SWAC-3	√	SWAC-4	√	SWAC-5	√
SWAD-1	√	SWAD-2	√	SWAD-3	√	SWAD-4	√	SWAD-5	√
SWCA-1	√	SWCA-2	√	SWCA-3	√	SWCA-4	√	SWCA-5	√
SWCB-1	√	SWCB-2	√	SWCB-3	√	SWCB-4	√	SWCB-5	√
TOTAL # OF NONCONFIRMING SHEAR WALLS	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***

Component check of Building-2 rehabilitated with shear wall (reduced)
(site-specific spectrum in the y-direction, boundary element: C)

Column	Acceptability	Column	Acceptability	Column	Acceptability	Column	Acceptability	Column	Acceptability
A104	√	A204	√	A304	√	A404	√	A504	√
A110	√	A210	√	A310	√	A410	√	A510	√
C102	√	C202	√	C302	√	C402	√	C502	√
C106	√	C206	√	C306	√	C406	√	C506	√
C108	√	C208	√	C308	√	C408	√	C508	√
C112	√	C212	√	C312	√	C412	√	C512	√
D104	√	D204	√	D304	√	D404	√	D504	√
D110	√	D210	√	D310	√	D410	√	D510	√
E103	√	E203	√	E303	√	E403	√	E503	√
E111	√	E211	√	E311	√	E411	√	E511	√
F104	√	F204	√	F304	√	F404	√	F504	√
F110	√	F210	√	F310	√	F410	√	F510	√
TOTAL # OF NONCONFIRMING COLUMNS	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***
Wall	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**
SWDA-1	√	SWDA-2	√	SWDA-3	√	SWDA-4	√	SWDA-5	√
SWDB-1	√	SWDB-2	√	SWDB-3	√	SWDB-4	√	SWDB-5	√
SWEC-1	√	SWEC-2	√	SWEC-3	√	SWEC-4	√	SWEC-5	√
SWED-1	√	SWED-2	√	SWED-3	√	SWED-4	√	SWED-5	√
SWF-1	√	SWF-2	√	SWF-3	√	SWF-4	√	SWF-5	√
SWGA-1	√	SWGA-2	√	SWGA-3	√	SWGA-4	√	SWGA-5	√
SWGB-1	√	SWGB-2	√	SWGB-3	√	SWGB-4	√	SWGB-5	√
TOTAL # OF NONCONFIRMING SHEAR WALLS	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***

Component check of Building-2 rehabilitated with shear wall (reduced)
(site-specific spectrum in the y-direction, boundary element: NC)

Column	Acceptability	Column	Acceptability	Column	Acceptability	Column	Acceptability	Column	Acceptability
A104	√	A204	√	A304	√	A404	√	A504	√
A110	√	A210	√	A310	√	A410	√	A510	√
C102	√	C202	√	C302	√	C402	√	C502	√
C106	√	C206	√	C306	√	C406	√	C506	√
C108	√	C208	√	C308	√	C408	√	C508	√
C112	√	C212	√	C312	√	C412	√	C512	√
D104	√	D204	√	D304	√	D404	√	D504	√
D110	√	D210	√	D310	√	D410	√	D510	√
E103	√	E203	√	E303	√	E403	√	E503	√
E111	√	E211	√	E311	√	E411	√	E511	√
F104	√	F204	√	F304	√	F404	√	F504	√
F110	√	F210	√	F310	√	F410	√	F510	√
TOTAL # OF NONCONFIRMING COLUMNS	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***	**TOTAL # OF NONCONFIRMING COLUMNS**	***0***
Wall	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**	**Wall**	**Acceptability**
SWDA-1	√	SWDA-2	√	SWDA-3	√	SWDA-4	√	SWDA-5	√
SWDB-1	√	SWDB-2	√	SWDB-3	√	SWDB-4	√	SWDB-5	√
SWEC-1	√	SWEC-2	√	SWEC-3	√	SWEC-4	√	SWEC-5	√
SWED-1	√	SWED-2	√	SWED-3	√	SWED-4	√	SWED-5	√
SWF-1	√	SWF-2	√	SWF-3	√	SWF-4	√	SWF-5	√
SWGA-1	√	SWGA-2	√	SWGA-3	√	SWGA-4	√	SWGA-5	√
SWGB-1	√	SWGB-2	√	SWGB-3	√	SWGB-4	√	SWGB-5	√
TOTAL # OF NONCONFIRMING SHEAR WALLS	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***	**TOTAL # OF NONCONFIRMING SHEAR WALLS**	***0***