Discussions on the Seismic Performance Assessment Through a Case Study of "Sosyal Konutlar" Buildings in Famagusta

Hind Mahmood Khudhur

Submitted to the
Institute of Graduate Studies and Research
in partial fulfillment of the requirements for the Degree of

Master of Science
in
Civil Engineering

Eastern Mediterranean University
June 2012
Gazimağusa, North Cyprus

Approval of the Institute of Graduate Studies and Research

Prof. Dr. Elvan Yılmaz
Director

I certify that this thesis satisfies the requirements as a thesis for the degree of Master of Science in Civil Engineering.

Asst. Prof. Dr. Mürüde Çelikağ
Chair, Department of Civil Engineering

We certify that we have read this thesis and that in our opinion it is fully adequate in scope and quality as a thesis for the degree of Master of Science in Civil Engineering.

Asst. Prof. Dr. Serhan Şensoy
Supervisor

Examining Committee

1. Asst. Prof. Dr. Mürüde Çelikağ
2. Asst. Prof. Dr. Serhan Şensoy
3. Asst. Prof. Dr. Giray Ozay

ABSTRACT

This research is to assess the performance of "Sosyal Konutlar Buildings" in Famagusta against earthquakes by using nonlinear static analysis methods. The assessment methodology involves two stages. First, the building data should be prepared such as available drawings, material properties, condition of the structural members etc. At the second stage, the data is used to model the building via the appropriate software.

At the first stage, it was observed that these buildings have corrosion problem especially in the columns. Therefore, this problem and its effect on performance of the buildings have been taken into account. The corrosion effect of reduction in steel area, reduction in concrete strength and slip have been considered for the ground floor columns and their effect on seismic performance has been determined. In this study corrosion level of the ground floor columns have been considered as 5%, 10%, 15% and 20%.

Buildings have been modeled as a two dimensional frame model with three types of loads, uniform lateral loads, triangular lateral loads and the first mode lateral load pattern. Although, the Turkish Earthquake Code design response spectrum has been used, the nonlinear static procedures of FEMA356 and Eurocode8 has been used. Results were obtained for both codes are discussed and comparison between both codes were explained.

Keywords: Earthquake, corrosion, nonlinear static analysis.

ÖZ

Bu araştırma, doğrusal olmayan statik analiz yöntemleri kullanılarak depreme karşı Famagusta "Sosyal Konutlar Binalar" performansını değerlendirmektir.Değerlendirme metodolojisi iki aşamayı kapsar. Birincisi, bina verileri gibi mevcut çizimler, malzeme özellikleri, ikinci aşamada yapı elemanlarının vb koşulu olarak hazırlanmalıdır, verileri uygun yazılım üzerinden bina modellemek için kullanılır.

İlk aşamada, bu binaların özellikle sütunlar korozyon problemi olduğu gözlendi. Bu nedenle, bu sorun ve binaların performans üzerindeki etkisi dikkate alınmıştır. Çelik alanında azalma korozyon etkisi, beton dayanımı ve kayma azalma tespit edilmiştir deprem performansı zemin katta kolon ve onların etkisi dikkate alınmıştır.Zemin Bu çalışmada korozyon seviyede katta kolonlar% 5,% 10,% 15 ve% 20 olarak kabul edilmiştir.

Binalar yükleri, düzgün yatay yükler, üçgen yanal yükler ve ilk modu yatay yük desen üç tip bir iki boyutlu çerçeve modeli olarak modellenmiştir.Türk Deprem Yönetmeliği tasarım spektrumu kullanılmıştır, ancak FEMA356 ve Eurocode8 nonlineer statik prosedürleri kullanılmıştır. Her iki kodları tartışıldı ve hem kodlar arasında karşılaştırma anlatıldı için sonuçlar elde edilmiştir.

Anahtar Kelimeler: Deprem, korozyon, doğrusal olmayan statik analiz.

TO MY FAMILY

ACKNOWLEDGEMENT

First of all, I thank God for granting me the blessing and opportunity of attaining a higher education. It was only through His supernatural guidance, wisdom and peace that I was able to manage this process when it became overwhelming at times.

I would especially like to thank my supervisor Asst. Prof. Dr. Serhan Şensoy, for his willingness to direct this thesis, his insightful recommendations, his patience and support. I’d also like to extend my sincere gratitude to the other members of the Department of Civil Engineering for their helpful guidance, support, and flexibility throughout this research.

Last but not least, I would like to thank my parents, Mahmood, Faten, and my brothers, Ahmed, Zainab, Maha, Ali, Sara, Luma. Thank you for your constant prayers and encouragement. I appreciate all of the times you visited, called, or e-mailed with words of faith and wisdom to carry me through each and every day of this process.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Chapter 1

INTRODUCTION

1.1 Importance of Seismic Performance Assessment

Assessment is a quantitative procedure for checking whether an existing undamaged or damaged Structure will gratify the required level appropriate to the seismic action under thought [1].

Earthquake Engineering Societies was fully percipient of the dominions loss that could be provoked by recurrent seismic events and their supplementary commercial aftermath in the year of 1960. It is not reasonable to avoid any damage under very strong earthquakes, in recognition of this, the Structural Engineers Association of California (SEAOC) adopted the following requirements for seismic design in its 1968 recommendations [2]:

"Structures should, in general, be able to:

- Resist a minor level of earthquake ground motion without damage.
- Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage.
- Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as nonstructural damage."

Major earthquakes that attack industrialized states in the subsequent half of the 1980s and the early half of the 1990s provoked moderately insufficient casualties but extremely colossal damage to possessions and commercial losses. Responding to this, Performance-based earthquake engineering accentuated in the SEAOC Vision 2000 document and industrialized into the solitary most vital believed of present years for seismic design or retrofitting of constructions [2].

Performance-based engineering ponder on the ends, chiefly on the skill of the engineered ability to consummate its intentional intention, alongside the thought of the aftermath of its wreck to encounter it. Acquainted structural design codes, by dissimilarity, are process-guidance, confirmatory the way, namely the prescriptive, facile to apply, but frequently incomprehensible laws that camouflage the pursuance of satisfactory performance. These laws possess been industrialized above period as an appropriate way to furnish safe-side, in supplement frugal resolutions for public combinations of constructing layout, dimensions and materials. They depart manipulated room for the designer to work resolution and innovative and do not furnish a rational basis for innovative sketches that benefit from present advances in knowledge and structural materials [3].

Performance-based earthquake engineering in specific attempts to augmentation the utility from the use of a ability by cutting its anticipated finished price, encompassing the short-term price of the work and the anticipated worth of the defeat in upcoming earthquakes (in words of casualties, price of overhaul or substitute, defeat of use, etc.). Across the design working existence of the utilities one should like to seize into report all probable upcoming seismic events alongside their annual probability and hold out an involvement alongside the corresponding consequences. Though, this is not practical. Therefore, at present performance-based earthquake engineering advocates just substituting the established single-tier design opposing downfall and its prescriptive laws, alongside a transparent multi-tier seismic design, encounter extra than one discrete presentation levels, every single one below a disparate seismic event, recognized across its annual probability of exceedance and termed seismic hazard level [3].

Pairing off all presentation levels believed for a specific case alongside the associated seismic hazard levels is termed, in performance-based earthquake engineering, presentation objective. Every single presentation level is normally recognized alongside a physical condition of the ability, well-described jointly alongside its probable consequences: Probable casualties, injuries and property defeat, endured functionality, price and feasibility of overhaul, anticipated length of disruption of use, price of relocation of occupants [3].

1.2 Methods of Seismic Performance Assessment

Two analysis procedures are obtainable for the performance assessment of buildings: Linear (response spectrum) analysis for the ability level assessment and nonlinear response history analysis for the collapse level assessment. Analysis and assessment have to be gave for the effects of horizontal earthquake shaking [4].

Vertical earthquake shaking must to be believed for vertically flexible constituents of the framing system. Capacity-design principles possess been extensively utilized to guard opposing unwanted failure modes such as shear in reinforced concrete beams, columns and walls. Such principles must to be believed in the proportioning of constituents subjected to deeds that are believed non-ductile (or force-controlled). For example, the needed shear capacity of a reinforced concrete beam must to be established on the computed plastic flexural capacity of the beam so as to guard opposing shear failure. For walls, whereas the shear span is not predetermined, this can be attained by ascertaining the shear demand by response history analysis established on best-estimate strength properties and to design for this demand employing program physical strengths and strength reduction factors [4].

In this study, nonlinear analysis methods will be studied through static procedures for seismic performance assessment of existing buildings. The procedures defined by FEMA 356 and EN 1998-1-3.

1.3 Seismicity of Cyprus Region

Cyprus is placed within the second intensive seismic zone of the earth, that of the Alpine-Himalayan belt. This zone extends from the Atlantic Ocean alongside the Mediterranean bowl across Italy, Greece, Turkey, Iran and India to the Pacific Ocean. The earthquakes that materialize in this zone embody considering 15% of the world seismic attention [5].

Cyprus is located on the southern side of the Anatolian Plate, just north of the African Plate. Its seismicity is attributable to the “Cyprus Arc” that represents the tectonic frontier amid the African and Eurasian lithospheric plates in the span as shown in Figure 1.1. The Cyprus Arc starts from the gulf of Antalya, whereas it joins the Hellenic Arc, going through west and south of Cyprus and extends towards the gulf of Iskenderun in the east whereas it joins the Eastern Obligation of Anatolia [5].

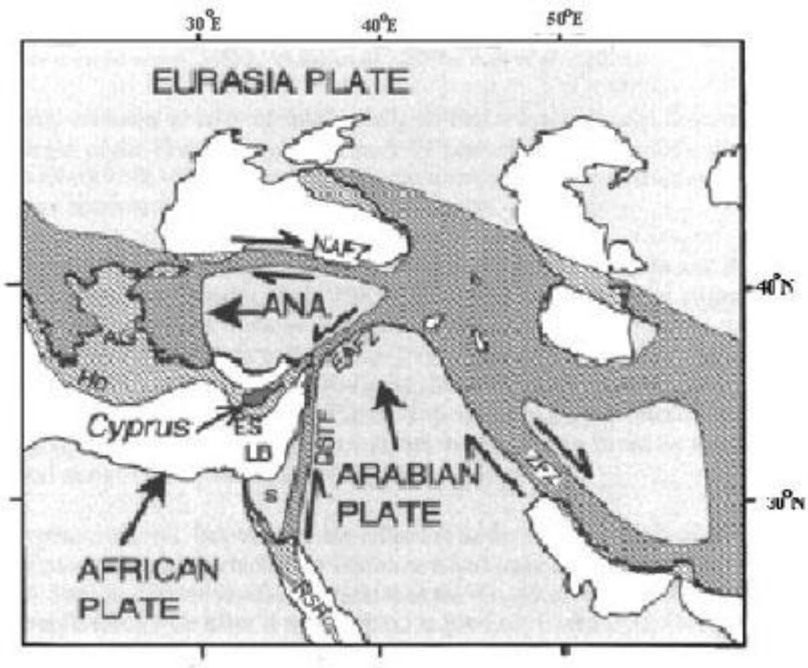


Figure 1.1: The Tectonic Boundary of Cyprus Arc [5]

The tectonic movements alongside Cyprus arc are the cause of countless earthquakes, several of which are strong. Current neotectonic studies by the Geological Survey Department (GSD) display that Cyprus possesses countless alert faults alongside which earthquakes additionally transpire, such as the earthquake of 17th August 1999 that was provoked by a movement on the Greece fault. Therefore, it is seeming that the Cyprus arc seizes up merely portion of the movements of the lithospheric plates and that the remainder is distributed in the rest of Cyprus as distant as the Pentadaktylos range [5].

In this thesis, seismicity has been considered by referring to the Turkish Earthquake Code (2007) where zone 2 with $\boldsymbol{A_0}$=0.3 and soil type ***S3*** has been used.

1.4 Problem Definition

An assessment was performed for the apartment known as "Sosyal Konutlar" existing buildings located in Famagusta city, North Cyprus. These buildings consist of 30 blocks. Some of these buildings have five storey and others have four storey. All these buildings have the same characteristics in terms of area, dimensions of members sections.

It was observed that the five storey buildings have problems of cracks and corrosions in columns sections; hence due to this problem the structure appeared weak at the site visit.

The performance of the building against earthquakes influenced by corrosion problems lead to problems: decrease in concrete strength, decrease in cross sectional area of the reinforcement bars and additional lateral displacements due to slip.

An assessment was requested for the buildings to compute the situation of the structures and decide whether these buildings can be repaired or demolished and reconstructed.

1.5 Purpose of Study

In order to achieve the requirements of buildings assessment, the following objectives have been studied:

1. Calculate the Moment-Curvature relationships for columns and beams sections by using "Response2000" program.
2. Modelling the buildings by using "CSI SAP2000".
3. Perform pushover analysis to evaluate the displacements of the structure.

4. Make comparison between the results obtained from both codes that were used in this study, FEMA356 and Eurocode8.
5. Identify the effect of corrosion according to both codes.
6. Giving an engineering opinion depending on the results obtained.

1.6 An overview on the Chapters

This thesis has been devoted into 6 chapters. Chapter 1 consist of brief discussion on seismic performance assessment and seismicity of Cyprus, i.e. seismic risk of Cyprus. Chapter 2 concentrates on nonlinear static analysis methods according to FEMA356 procedures. Chapter 3 focuses on nonlinear static analysis methods according to Eurocode8 procedures. Chapter 4 gives information on buildings geometry and sections properties and explains the methods to model the buildings. Chapter 5 concentrates on the results obtained according to both codes including discussion of objectives. Finally chapter 6 related to the conclusions in addition to recommendations for probable upcoming studies that can be completed in this scrutiny area.

Chapter 2

NONLINEAR STATIC ANALYSIS

2.1 General

Development and procession of running various pushover analysis which is resemble to a given modal allocation have been used widely in last decades. To estimate the structural responses, the deed results that derived from the modal responses should be integrated. These processes enter within the nonlinear static analysis [6]. The capacity of the structural system ordinarily estimated using static methods whence the deeds and deformations at disparate limit states or performance objectives [7].

There is presently a shove for the evolution and code implementation of displacement or commonly, deformation based design and assessment methods in conjunction alongside the present crusade for performance based seismic engineering. Therefore, subjected to earthquake action, it would seem that applying displacement loading, instead force actions, in pushover procedures would be opportune option for nonlinear static analysis of structures [6].

Amr S. Elnashai & Luigi Di Sarno (2008) says that "static analysis may be viewed as a special case of dynamic analysis when damping and inertia effects are zero or negligible". Nonlinear static analysis (generally called “pushover” analysis) was utilized after the early new-generation guidelines for seismic rehabilitation of existing buildings (ATC 1997) referred to it as the reference method. Since then, because it has attractive simplicity and obviousness and the wide availability of credible and user-friendly, analysis software possess made it the analysis method of choice for seismic assessment and retrofitting of buildings [7].

According to the book of “Seismic Design, Assessment and Retrofitting of Concrete Buildings”, the expansion of the lateral force procedure of static analysis into the nonlinear prescript predominantly known as pushover analysis. It is grasped out under constant gravity loads and gradually increasing lateral loading applied on the masses of the structural model. This loading is meant to emulate inertia forces because of a horizontal component of the seismic action. The engineer can pursue the sluggish progress of plastic hinges, the progress of the plastic mechanism and damage as the requested lateral forces increase in the path of the analysis, as a purpose of the magnitude of the imposed lateral loads and of the emerging displacements [3].

2.2 Nonlinear Static Analysis Procedures According to FEMA 356

2.2.1 Introduction

The structures that have non critical higher mode effects, the nonlinear static analysis procedures (NSP) could be allowable. For the structure using appropriate modes, a modal response spectrum analysis shall be implemented to capture 90% mass involvement, for determining that if the higher mode effects for the structures are critical or not. To theorize only the first mode involvement, a second response spectrum analysis shall also be implemented. If the shear in any story resulting from the modal analysis considering modes required to securing 90% mass involvement exceeds 130% of the conforming story shear considering only the first mode response, higher mode effects shall be considered significant [8].

The nonlinear static analysis procedure (NSP) is mostly a more credible approach to manifesting the performance of a structure than are linear procedures. However, it is not accurate, and cannot precisely report for adjustments in dynamic response as the structure devalues in stiffness or report for higher mode effects. The linear dynamic procedure (LDP) is also recruited to verify the efficiency of the structure design while the (NSP) is applied on a structure that has significant higher mode response. When this approach is taken, less restrained criteria are allowed for the LDP, admission the noticeably progressed knowledge that is obtained by implementing both analysis procedures [8].

2.2.2 Modeling and Analysis Considerations

2.2.2.1 Idealized Force-Displacement Curve

The nonlinear force-displacement relationship between base shear and displacement of the manipulation node will be exchanged with an idealized relationship to calculate the effective lateral stiffness, K_e , and effective yield strength, V_y , of the building as illustrated in Figure 2.1. This relationship shall be bilinear, with initial slope K_e and post-yield slope α. Using a reduplicate graphical procedure that concerning equilibrates the area above and below the idealized force – displacement curve, the line segments on this curve shall be situated. The secant stiffness that measured at a base shear force equal to 60% of the effective yield strength of the structure shall be taken as the effective lateral stiffness, K_e. The post-yield slope, α, shall be identified by a line segment that passes through the substantial curve at the calculated target displacement. The effective yield strength shall not be taken as greater than the maximum base shear force at any point along the actual curve [8].

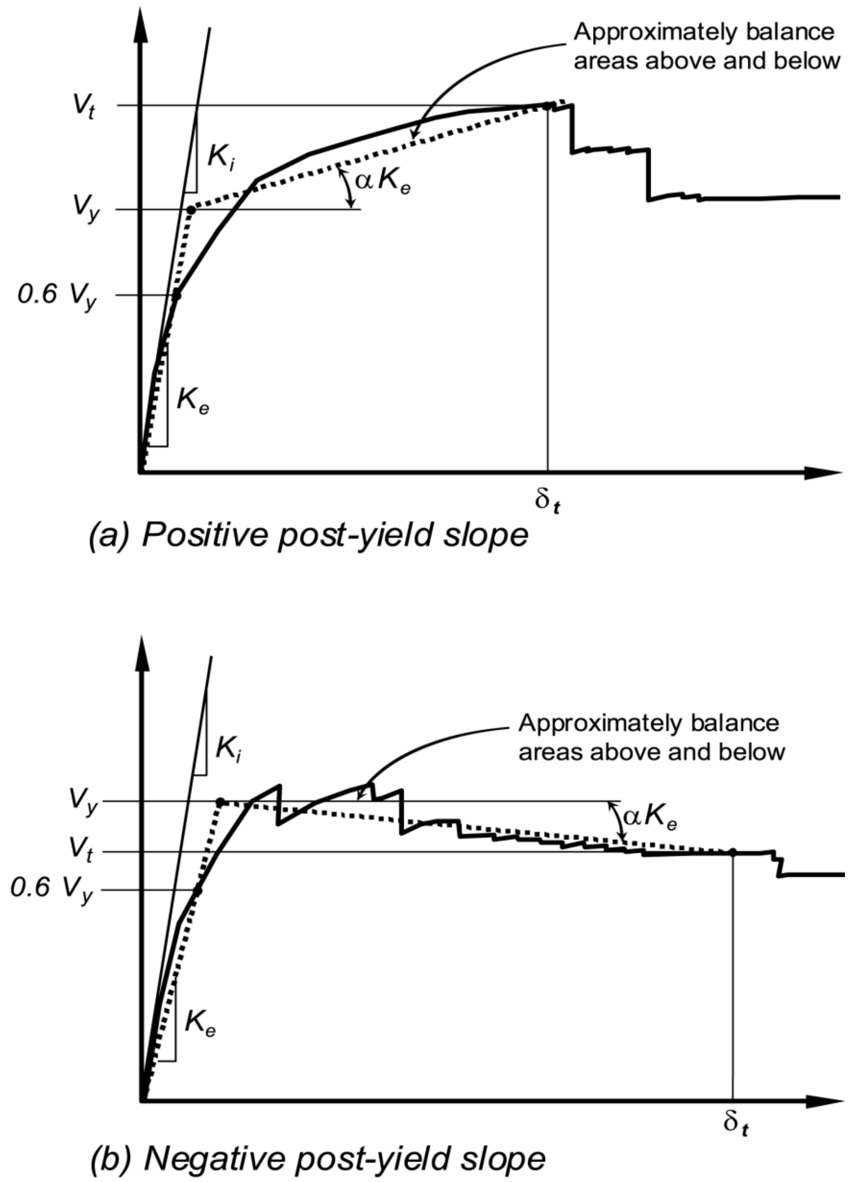


Figure 2.1: Idealized Force-Displacement Curves [8]

2.2.2.2 Period Determination

The effective fundamental period in the direction under consideration shall be based on the idealized force displacement curve. The effective fundamental period, $\boldsymbol{T_e}$, can be calculated according the following equation [8]:

$$\boldsymbol{T_e} = \boldsymbol{T_i}\sqrt{\frac{\boldsymbol{K_i}}{\boldsymbol{K_e}}} \qquad \text{(Eq. 2.1)}$$

Where:

T_i is Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis.

K_i is Elastic lateral stiffness of the building in the direction under consideration.

K_e is Effective lateral stiffness of the building in the direction under consideration.

2.2.3 Determination of Forces and Deformations

2.2.3.1 Target Displacement

The target displacement is intended to embody the maximum displacement probable to be experienced during the design earthquake [8].

The target displacement, δ_t, at each floor level can be calculated according to the following equation[8] .

$$\delta_t = C_O\, C_1\, C_2\, C_3\, S_a \frac{T_e^2}{4\,\pi^2}\, g \qquad \text{(Eq. 2.2)}$$

where:

C_o is Modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system.

C_1 is Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response:

$= 1.0$ for $T_e \geq T_s$

$$= \frac{1.0 + (R-1)\frac{T_s}{T_e}}{R} \quad \text{for} \quad T_e < T_s \qquad \text{(Eq. 2.3)}$$

T_e is The period of the effective fundamental of the building, sec.

T_s is The characteristic period of the response spectrum. This period is defined at the transition region of the spectrum of the constant velocity segment and constant acceleration segment of the spectrum.

R is Ratio of elastic strength demand to calculated yield strength coefficient.

C_2 is Modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response. $C2$ = 1.0 shall be permitted for nonlinear procedures.

C_3 is Modification factor to represent increased displacements due to dynamic $P\text{-}\Delta$ effects. For buildings with positive post-yield stiffness, C_3 shall be set equal to 1.0. For buildings with negative post-yield stiffness.

S_a is Response spectrum acceleration, at the effective fundamental period and damping ratio of the building.

g is acceleration of gravity.

The strength ratio R shall be calculated as follows:

$$R = \frac{S_a}{V_y/W} C_m \qquad \text{(Eq. 2.4)}$$

Where:

Vy is Yield strength calculated using results of the NSP for the idealized nonlinear force displacement curve developed for the building.

W is Effective seismic weight.

Cm is The effective model mass calculated for the fundamental mode using an Eigen value analysis shall be permitted and control node displacement exhibits negative post yield stiffness.

$$C_3 = 1.0 + \frac{|\propto| \ (R-1)^{3/2}}{T_e} \qquad \text{(Eq. 2.5)}$$

Where:

α is Ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force displacement relation shall be characterized by a bilinear relation as shown in Figure 2.1.

2.2.4 Performance Requirement and Acceptance Criteria

The discrete Structural Performance Levels are Immediate Occupancy(IO), Life Safety(LS), Collapse Prevention(CP)[8].

Structural presentation 'Immediate Occupancy' will be described as the post-earthquake damage state that stays harmless to inhabit, vitally retains the pre-earthquake design strength and stiffness of the construction[8].

'Immediate Occupancy', way the post-earthquake damage state in that merely extremely manipulated structural damage possesses occurred. The frank vertical- and lateral-force-resisting arrangements of the constructing retain nearly all of their pre earthquake strength and stiffness. The chance of existence intimidating injury as a consequence of structural damage is extremely low, and even though a little minor structural repairs could be appropriate, these should usually not be needed prior to re occupancy [8] .

Structural performance 'Life Safety', will be described as the post-earthquake damage state that includes damage to structural constituents but retains a margin opposing onset of partial or finished collapse.

'Life Safety', way the post-earthquake damage state in that momentous damage to the construction possesses transpired, but a little margin opposing whichever partial or finished structural collapse remains. A little structural agents and constituents are harshly broken, but this possesses not arose in colossal plummeting debris hazards, whichever inside or beyond the building. Injuries could transpire across the earthquake; though, the finished chance of life-threatening injury as a consequence of structural damage is anticipated to be low. It must to be probable to overhaul the structure; though, for commercial reasons this could not be practical. As the broken construction is not an imminent collapse chance, it should be prudent to apply structural repairs or mount provisional bracing prior to re occupancy [8].

Structural presentation 'Collapse Prevention', will be described as the post-earthquake damage state that includes damage to structural constituents such that the construction endures to prop gravity loads but retains no margin opposing collapse.

'Collapse Prevention', way the post-earthquake damage state in that the constructing is on the verge of partial or finished collapse. Comprehensive damage to the construction possesses transpired, potentially encompassing momentous degradation in the stiffness and strength of the lateral-force challenging arrangement, colossal perpetual lateral deformation of the construction, and to a extra manipulated extent degradation in vertical-load-carrying capacity. Though, all momentous constituents of the gravity load- challenging arrangement have to tolerate to hold their gravity burden demands. Momentous chance of injury because of plummeting hazards from structural debris could exist. The construction could not be technically useful to overhaul and is not harmless for re occupancy, as aftershock attention might instigate collapse [8].

Elements and constituents that alter the lateral stiffness or allocation of force in a construction, or are loaded as a consequence of lateral deformation of the construction, will be categorized as main or secondary, even if they are not portion of the aimed lateral-force-resisting system. Agents and constituents that furnish the capacity of the construction to challenge downfall below seismic powers instigated by earth gesture in each association will be categorized as primary. Supplementary agents and constituents will be categorized as secondary.

Performance requirement for deformation for primary (P)and secondary members (S) described by FEMA 356 is shown in Figure 2.2.

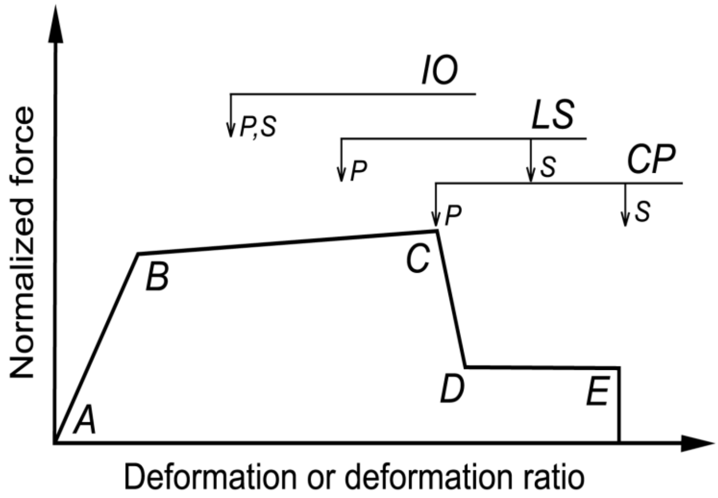


Figure 2.2: Component or Element Deformation Acceptance Criteria [8]

Point A corresponds to the unloaded condition. Point B corresponds to the nominal yield strength. The slop of line BC is normally seized equal to between 0% and 10% of the early slop (line AB). Point C possesses confrontation equal to the nominal strength. Line CD corresponds to early failure of the member. It may be associated alongside phenomena such as fracture of the bending reinforcement, spalling of concrete or shear failure pursuing early yield.

Line DE embodies the residual strength of the member. It could be non-zero in a few cases, or practically zero in others. Point E corresponds to the deformation limit.

Though, usually initial failure at C defines the manipulating deformation, and in that case point E is a point possessing deformation equal to that at C and zero resistance [9].

Chapter 3

NONLINEAR STATIC ANALYSIS PROCEDURE ACCORDING TO EUROCODE8

3.1 Introduction

Pushover analysis is a non-linear static analysis grasped out below conditions of steady gravity loads and monotonically rising horizontal loads. It could be requested to confirm the structural performance of continuing constructions to guesstimate the anticipated plastic mechanisms and the allocation of damage, and to assess the structural performance of continuing or retrofitted constructions [1].

At least two vertical allocations of the lateral loads must to be requested, uniform pattern, established on lateral force, and modal pattern, proportional to lateral forces consistent alongside the lateral force allocation in the association below thought ambitious in flexible analysis [10].

3.2 Target Displacement

The target displacement shall be defined as the seismic demand derived from the elastic response spectrum in terms of the displacement of an equivalent single-degree-of-freedom system. The target displacement is determined from the elastic response spectrum. The following equations used to find target displacement:

$$\boldsymbol{F_i = m_i\, \Phi_i} \quad \text{(Eq. 3.1)}$$

where $\boldsymbol{m}_\mathbf{i}$ is the mass in the i-th storey, Φ_i is the roof displacement for each storey

The mass of an equivalent SDOF system $\boldsymbol{m^*}$ is determined as:

$$\boldsymbol{m^* = \sum m_i\, \Phi_i = \sum F_i} \quad \text{(Eq. 3.2)}$$

and the transformation factor is given by:

$$\boldsymbol{\Gamma} = \frac{\boldsymbol{m}^*}{\sum \boldsymbol{m_i}\ \Phi_{\boldsymbol{i}}^{\boldsymbol{2}}} = \frac{\sum \boldsymbol{F_i}}{\sum[\frac{\boldsymbol{F_i^2}}{\boldsymbol{m_i}}]} \qquad \text{(Eq. 3.3)}$$

The force $\boldsymbol{F^*}$ and displacement $\boldsymbol{d^*}$ of the equivalent SDOF system are computed as:

$$\boldsymbol{F}^* = \frac{\boldsymbol{F_b}}{\Gamma} \qquad \text{(Eq. 3.4)}$$

$$\boldsymbol{d}^* = \frac{\boldsymbol{d_n}}{\Gamma} \qquad \text{(Eq. 3.5)}$$

where $\boldsymbol{F_b}$ and $\boldsymbol{d_n}$ are, respectively, the base shear force and the control node displacement of the Multi Degree of Freedom (MDOF) system.

The yield force $\boldsymbol{F_y}^*$, which represents also the ultimate strength of the idealized system, is equal to the base shear force at the formation of the plastic mechanism. Figure 3.1 shows the initial stiffness of the idealized curve which is determined in such a method that the areas below the actual and the idealized force curves are equal. The yield displacement of the idealized SDOF system $\boldsymbol{d_y^*}$ is given by:

$$\boldsymbol{d_y^*} = 2\ [\boldsymbol{d_m^*} - \frac{\boldsymbol{E_m^*}}{\boldsymbol{F_y^*}}] \qquad \text{(Eq. 3.6)}$$

where $\boldsymbol{E_m}^*$ is the actual deformation energy up to the formation of the plastic mechanism.

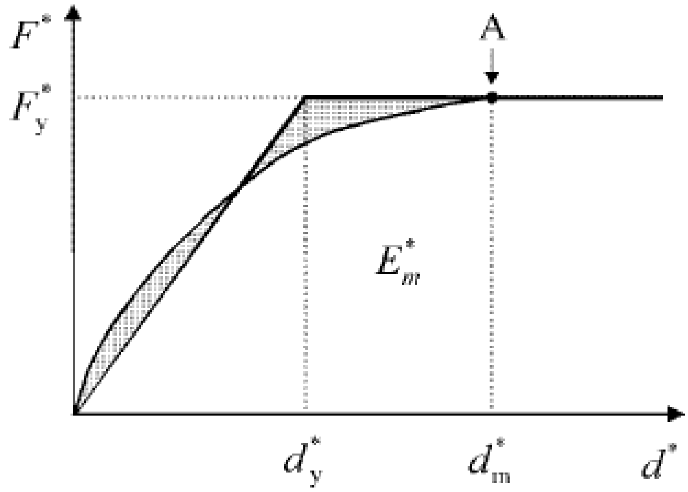


Figure 3.1: Determination of the Idealized Force – Displacement Relationship [1]

The period T^* of the idealized equivalent SDOF system is determined by:

$$T^* = 2\pi \sqrt{\frac{m^*\, d_y^*}{F_y^*}} \qquad \text{(Eq. 3.7)}$$

The target displacement of the structure is given by:

$$d^*_{et} = S_e\,(T^*)\ [\frac{T^*}{2\pi}]^2 \qquad \text{(Eq. 3.8)}$$

where ***Se(T*)*** is the elastic acceleration response spectrum at the period ***T****.

For the determination of the target displacement d_t* for structures in the short-period range and for structures in the medium and long-period ranges different the following expressions should be used :

a) $T^* < T_C$ (short period range)

If $F^*_y\ /\ m^* \geq S_e\,(T^*)$, the response is elastic, therefore $d^*_t = d^*_{et}$

If $F^*_y\ /\ m^* < S_e\,(T^*)$, the response is nonlinear, so

$$d^*_t = \frac{d^*_{et}}{q_u}\ [\,1 + (q_u - 1)\,\frac{T_C}{T^*}\,] \geq d^*_{et} \qquad \text{(Eq. 3.9)}$$

where q_u is the ratio between the acceleration in the structure with unlimited elastic behavior ***Se(T*)*** and in the structure with limited strength ***Fy* / m****.

$$q_u = \frac{S_e\,(T^*)\,m^*}{F_y^*} \tag{Eq. 3.10}$$

b) $T^* \geq T_C$ (medium and long period range)

$$d^*_t = d^*_{et} \tag{Eq. 3.11}$$

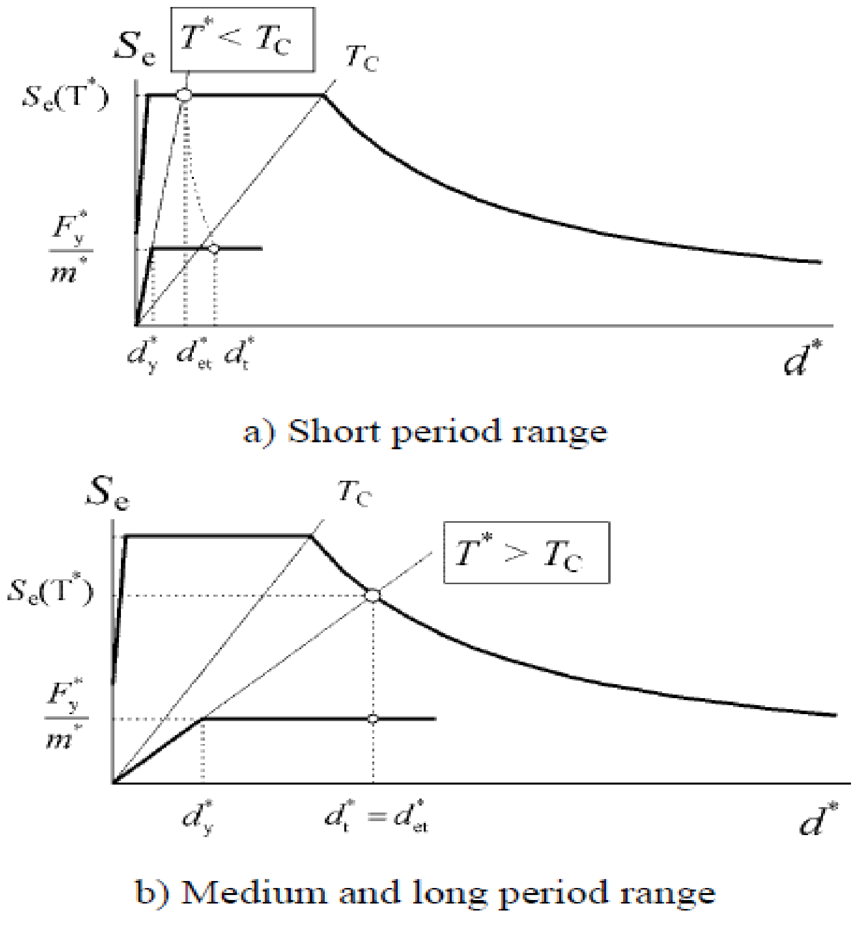


Figure 3.2: Determination of the Target Displacement for the Equivalent SDOF System [1]

The target displacement of the MDOF system is given by:

$$d_t = \Gamma\, d^*_t \tag{Eq. 3.12}$$

3.3 Performance Requirement and Acceptance Criteria

An adequate degree of reliability opposite unacceptable damage will be safeguarded by fulfilling the deformation limits. The structural arrangement will be confirmed to safeguard that the construction own adequate confrontation and stiffness to uphold the intention of the vital services in the abilities for a seismic event associated alongside an appropriate revisit period [1].

The deformation capacity of beams, columns and walls, is defined on the one hand the chord rotation $\boldsymbol{\theta}$. The chord rotation is also equal to the member drift ratio, the deflection at the end of the shear span with respect to the tangent to the axis at the yielding end, divided by the shear span.

3.3.1 Near Collapse Level (NC)

The value of the total chord rotation capacity (elastic plus inelastic part) at ultimate, $\boldsymbol{\theta_u}$, of concrete members under cyclic loading may be evaluated from the following equation:

$$\boldsymbol{\theta_{um}} = \frac{1}{\gamma_{e1}} 0.016(0.3)^{v}\left[\frac{max(0.01;\omega')}{max(0.01;\omega)} fc\right]^{0.225}\left(\frac{L_v}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{fc}\right)}(1.25)^{100\rho_d}$$

(Eq. 3.13)

where:

γ_{e1} is equal to 1.5 for primary seismic elements and to 1.0 for secondary seismic elements.

h is the depth of cross-section.

$\boldsymbol{L_V = M/V}$ (Eq. 3.14)

M is the moment, ***V*** is the shear at the end section.

$\boldsymbol{v = N / bh\ fc}$ (Eq. 3.15)

b is width of compression zone, ***N*** is axial force positive for compression.

ω*,* ω' is the mechanical reinforcement ratio of the tension (including the web reinforcement) and compression, respectively, longitudinal reinforcement.

fc and ***fyw*** are the concrete compressive strength (MPa) and the stirrup yield strength (MPa), respectively.

$\rho_{sx} = A_{sx}/b_w$ (Eq. 3.16)

sh is the ratio of transverse steel parallel to the direction *x* of loading (*sh* = stirrup spacing).

ρ_d is the steel ratio of diagonal reinforcement.

α is the confinement effectiveness factor, that may be taken equal to:

$$\alpha = \left(1- \frac{s_h}{2b_0}\right)\left(1- \frac{s_h}{2h_0}\right)\left(1- \frac{\sum b_i^2}{6h_0 b_0}\right) \quad \text{(Eq. 3.17)}$$

where:

bo and ***ho*** is the dimension of confined core to the centre line of the hoop.

bi is the centerline spacing of longitudinal bars laterally restrained by a stirrup corner or a cross-tie along the perimeter of the cross-section.

The value of the plastic part of the chord rotation capacity of concrete elements under cyclic loading may be computed from the following equation:

$$\theta_{um}^{p1} = \theta_{um} - \theta_y = \frac{1}{\gamma_{e1}}\ 0.0145\ (0.25)^{v}\ \left[\frac{max(0.01;\omega')}{max(0.01;\omega)}\right]^{0.3} fc^{0.2} \left(\frac{L_v}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{fyw}{fc}\right)}.(1.275)^{100\rho_d} \quad \text{(Eq. 3.18)}$$

Where:

θ_y is the chord rotation at yielding, γ_{e1} is equal to 1.8 for primary seismic elements and to 1.0 for secondary seismic elements.

For the evaluation of the ultimate chord rotation capacity an alternative expression can be used:

$$\boldsymbol{\theta_{um}} = \frac{1}{\gamma_{e1}} \{\theta_y + (\varphi_u - \varphi_y)\, L_{p1}[1- \frac{0.5\, L_{p1}}{L_V}]\} \qquad \text{(Eq. 3.19)}$$

Where:

θ_y is the chord rotation at yield.

φ_u is the ultimate curvature at the end section.

φ_y is the yield curvature at the end section.

The value of the length L_{p1} of the plastic hinge depends on how the enforcement of strength and deformation capacity of concrete due to imprisonment is taken into account in the computation of the ultimate curvature of the end section. L_{p1} may be computed from the following expression:

$$L_{p1} = 0.1\, L_v + 0.17h + 0.24\, \frac{d_{bL}\, f_y\, (MPa)}{\sqrt{fc\, (MPa)}} \qquad \text{(Eq. 3.20)}$$

where h is the depth of the member and d_{bL} is the diameter of the tension reinforcement.

3.3.2 Significant Damage Level (SD)

The chord rotation capacity approved for significant damage θ_{SD} may be assumed to be 0.75 of the ultimate chord rotation θ_u of limit state of near collapse (NC)

3.3.3 Damage Limitation Level (DL)

The capacity for this limit state used in the investigations is the yielding bending moment under the design value of the axial load. Chord rotation at yielding θ_y can be evaluated as:

For beams and columns:

$$\theta_y = \emptyset_y\, \frac{L_V + a_V\, Z}{3} + 0.0013\, (1+1.5\, \frac{h}{L_v}) + \frac{\varepsilon_y}{d-d'}\, \frac{d_b\, f_y}{6\sqrt{f_c}} \qquad \text{(Eq. 3.21)}$$

For walls of rectangular, T- or bar belled section:

$$\boldsymbol{\theta_y} = \emptyset_y \frac{L_V + a_V Z}{3} + 0.002 \left(1\text{- } 0.135 \frac{L_V}{h} \right) + \frac{\varepsilon_y}{d-d'} \frac{d_b f_y}{6\sqrt{f_c}} \quad \text{(Eq. 3.22)}$$

or from the alternative expressions for beams and columns:

$$\boldsymbol{\theta_y} = \emptyset_y \frac{L_V + a_V Z}{3} + 0.0013 \left(1+1.5 \frac{h}{L_V}\right) + 0.13 \, \emptyset_y \frac{d_b f_y}{\sqrt{f_c}} \quad \text{(Eq. 3.23)}$$

and for walls of rectangular, T- or bar belled section:

$$\boldsymbol{\theta_y} = \emptyset_y \frac{L_V + a_v Z}{3} + 0.002 \left(1\text{- } 0.125 \frac{L_V}{h}\right) + 0.13 \, \emptyset_y \frac{d_b f_y}{\sqrt{f_c}} \quad \text{(Eq. 3.24)}$$

where:

φ_y is the yield curvature of the end section.

$\alpha_v z$ is the tension shift of the bending moment diagram.

z is length of internal lever arm, taken equal to ***d-d*** ˙ in beams, columns, or walls with bar belled or T-section, or to 0.8*h* in walls with rectangular section.

$\alpha_v = 1$ if shear cracking is expected to precede flexural yielding at the end section ; otherwise $\alpha_v = 0$.

***f*y** and ***f*c** are the steel yield stress and the concrete strength, respectively both in MPa.

ε_y is equal to ***f*y/*E*s**.

d and ***d*** ˙ are the depths to the tension and compression reinforcement, respectively.

Chapter 4

METHODOLOGY

4.1 Buildings Data

In this research "Sosyal Konutlar" existing buildings were studied for seismic performance assessment. These buildings are collection of 30 blocks, some of the buildings consisting of 5 storey while others have 4 storey. All buildings have an area of 16 x 15.3 m^2 and a height of 3 m per floor.

All buildings have the same characteristics in terms of the area, dimensions of columns and beams, distance between columns in length and width of building, type of stairs and type of foundation. The building plans are shown in Appendix G. For the purpose of this study an assessment was done on one building since all the buildings are similar in characteristics.

Figure 4.1: Side View of Buildings

4.2 Problems Observed in the Buildings

The problems of corrosion may be a direct effect on the performance of the building against earthquake, or be one of the causes of damage to the building during earthquakes. Therefore, the problems of corrosion must be taken into consideration when determining the performance of the building. The problems of corrosion have different effects on the structure, these effects could be a decrease in the sectional area of reinforcing steel bar, internal cracks in the members of the structure, reduction in concrete strength and an additional lateral displacements due to slip [11].

It has been observed that the five-storey buildings have corrosion problems in the columns as it appears in Figure 4.2. Therefore, these problems have been identified and studied for both ends of the first floor columns with estimated rates of corrosion which was 5%, 10%, 15% and 20%. Methods of finding these effects have been discussed in the sections below.

Figure 4.2: Corrosion Problem in Columns

4.2.1 Reduction in Concrete Strength

The reduction in concrete strength for columns has been calculated for four different ratios of corrosion by using the formula below. In addition the reduction in cross sectional area of reinforcement bars has been taken into account, which can be found easily by subtracting the rates of corrosion from the rebar diameter. Equations used are as follows [12]:

$$\boldsymbol{f_c^*} = \frac{\boldsymbol{f_c^`}}{\mathbf{1}+\boldsymbol{k\varepsilon_1/\varepsilon_{c0}}} \qquad \text{(Eq. 4.1)}$$

Where:

$\boldsymbol{f_c^*}$: the reduced concrete strength

$\boldsymbol{f_c^`}$: the concrete compressive strength

$\boldsymbol{k}$: the coefficient related to the bar roughness and diameter

$\boldsymbol{\varepsilon_1}$: the average tensile strain in the cracked concrete at right angles to the direction of the applied compression

$\boldsymbol{\varepsilon_{co}}$: the strain at the peak compressive stress

$$\boldsymbol{\varepsilon_1} = \frac{\boldsymbol{b_f - b_0}}{\boldsymbol{b_o}} \qquad \text{(Eq. 4.2)}$$

Where:

$\boldsymbol{b_f}$: the width increased by corrosion cracking

$\boldsymbol{b_o}$: the section width in the virgin state

$$\boldsymbol{b_f - b_o = n_{bars}\, w_{cr}} \qquad \text{(Eq. 4.3)}$$

Where:

$\boldsymbol{n_{bars}}$: the number of bars in the top layer (compressed bars)

$\boldsymbol{w_{cr}}$: the total crack width for a given corrosion level

$$\boldsymbol{w_{cr}} = \frac{\mathbf{4}\,\boldsymbol{\pi\, d_s(t)}}{(\mathbf{1}-\boldsymbol{v_c})/(\boldsymbol{a/b})^{\sqrt{\alpha}} + (\mathbf{1}+\boldsymbol{v_c})(\boldsymbol{b/a})^{\sqrt{\alpha}}} - \frac{\mathbf{2}\,\boldsymbol{\pi\, b\, f_t}}{\boldsymbol{E_{eff}}} \qquad \text{(Eq. 4.4)}$$

Where:

$\boldsymbol{d_s(t)}$: the thickness of the corrosion product form

$\boldsymbol{f_t}$: the tensile strength of concrete

$\boldsymbol{E_{eff}}$: the effective elastic modulus of concrete

$\boldsymbol{\upsilon_c}$: the Poisson's ratio of concrete

$\boldsymbol{\alpha}$: the tangential stiffness reduction factor

a : the inner radii of the thick-wall cylinder $\{a = (d_b + 2d_o)/2\}$, d_b is the diameter of the reinforcement bars, d_o is the thickness of the annular layer of concrete pores

b : the outer radii of the thick-wall cylinder $(b = S/2)$, ***S*** is the rebar spacing

$$d_s(t) = \frac{w_{rust}(t)}{\pi\,(d_b + 2d_o)} \left(\frac{1}{\rho_{rust}} - \frac{\alpha_{rust}}{\rho_{st}} \right) \qquad \text{(Eq. 4.5)}$$

Where:

$\boldsymbol{w_{rust}(t)}$: the mass of rust per unit length of rebar

$\boldsymbol{\alpha_{rust}}$: the coefficient related to the type of rust

$\boldsymbol{\rho_{rust}}$: the density of rust

$\boldsymbol{\rho_{st}}$: the density of steel

$$w_{rust}(t) = \left[2 \int_0^t 0.105 \left(\frac{1}{\alpha_{rust}}\right) \pi\, d_b\, i_{corr}(t)\; d_t \right]^{1/2} \qquad \text{(Eq. 4.6)}$$

$\boldsymbol{i_{corr}(t)}$: corrosion current density (µA/cm^2) which is a measure of corrosion rate

The effective elastic modulus under sustained loading will be reduced over time due to the effect of creep. The following equations used to find the effective elastic modulus [13]:

$$E_{eff} = E_{c28} / (1+\varphi) \qquad \text{(Eq. 4.7)}$$

Where:

$\boldsymbol{E_{c28}}$: elastic modulus, $\boldsymbol{\varphi}$: creep coefficient

$$E_{c28} = 1.05\ E_{cm} \qquad \text{(Eq. 4.8)}$$

Where:

E_{cm} : 28 days secant modulus

$$E_{cm} = 22[f_{cm}/10]^{0.3} \quad \text{(Eq. 4.9)}$$

Where:

f_{cm} : mean compressive strength

$$f_{cm} = f_c' + 8 \quad \text{(Eq. 4.10)}$$

To find creep coefficient following equation was used [13]:

$$\varphi(t,t_o) = \varphi_o \,.\, \beta_c(t,t_o) \quad \text{(Eq. 4.11)}$$

Where:

φ_o : the notional creep coefficient

$$\varphi_o = \varphi_{RH} \,.\, \beta(f_{cm}) \,.\, \beta(t_o) \quad \text{(Eq. 4.12)}$$

Where:

φ_{RH} : a factor to allow for the effect of relative humidity on the notional creep coefficient

$$\varphi_{RH} = 1 + \frac{1-RH/100}{0.1\sqrt[3]{h_0}} \qquad \text{for } f_{cm} \leq 35\ MPa \quad \text{(Eq. 4.13)}$$

Where:

RH : the relative humidity of the ambient environment in %

$\beta(f_{cm})$: a factor to allow for the effect of concrete strength on the notional creep coefficient

$$\beta(f_{cm}) = \frac{16.8}{\sqrt{f_{cm}}} \quad \text{(Eq. 4.14)}$$

$\beta(t_o)$: a factor to allow for the effect of concrete age at loading on the notional creep coefficient

$$\beta(t_o) = \frac{1}{(0.1 + t_o^{0.2})} \quad \text{(Eq. 4.15)}$$

h_o : the notional size of the member in mm

$h_o = 2A_c / u$ (Eq. 4.16)

Where:

A_c : the cross – sectional area of the member

u : the perimeter of the member in contact with the atmosphere

$\beta_c(t,t_o)$: A coefficient to describe the development of creep with time after loading

$$\beta_c(t,t_o) = [\frac{t - t_o}{(\beta_H + t - t_o)}]^{0.3} \quad \text{(Eq. 4.17)}$$

Where:

t : the age of concrete in days at the moment considered

t_o : the age of concrete at loading in days

$t - t_o$: the non – adjusted duration of loading in days

β_H : a coefficient depending on the relative humidity (RH in %) and the notional member size (h_o in mm)

$$\beta_H = 1.5\,[1 + (0.012\,RH)^{18} h_o + 250 \quad \leq 1500 \quad \text{for} \quad f_{cm} \leq 35\ Mpa \quad \text{(Eq. 4.18)}$$

All these results are shown in Appendix B. After calculating of reduction in concrete strength, moment-curvature relationships are drawn by using "Response 2000" program for each corrosion rate. Moment-Curvature relations are shown in Appendix A.

4.2.2 Additional Displacement Due to Slip

The reduction of the reinforcement bar diameter and the loss of bonding between the steel and concrete have a relevant influence on the performance level of reinforced concrete structures. To calculate slip rotation the following equation are used [14]:

$$\theta_s = \frac{\varepsilon_s\, f_s\, d_b}{8\, u_b (d - c)} \quad \text{for} \quad \varepsilon_s \leq \varepsilon_y \quad \text{(Eq. 4.19)}$$

$$\theta_s = \frac{d_b}{8\, u_{b\,(d-c)}}\, (\varepsilon_y f_y + 2\,(\varepsilon_s + \varepsilon_y)\,(f_s - f_y)) \qquad \text{for} \quad \varepsilon_s > \varepsilon_y \qquad \text{(Eq. 4.20)}$$

Where:

ε_s : the strain in the reinforcement bar

f_s : the stress in the reinforcement bar

f_y : the yield stress in the reinforcement bar

d : the section depth, c : the neutral axis depth

u : bond strength

To calculate the bond strength relation, following equations are used [15]:

$$\frac{U}{\sqrt{f'_c}} = 0.63 - 0.041\, X \qquad \text{(Eq. 4.21)}$$

X : the percent of mass loss of steel bar in the end regions

$$X = 2\,\frac{\Delta D_i}{D_i}\,.\,100\% \qquad \text{(Eq. 4.22)}$$

D_i : the diameter of the bar

Moment-Reinforcement strain relations are shown in Appendix C. After determining the slip rotation, the following equation was used to determine curvature to draw moment-curvature relationship for each corrosion rate, the equation was:

$$\theta_s = \varphi\, L_{p1} \qquad \text{(Eq. 4.23)}$$

L_{p1} : plastic hinge length, it was calculated according to FEMA356 and Eurocode8 as described in chapter 2 & 3

4.3 Building Modeling

For the purpose of determining the disposal of the structure against seismic loads, characteristics of sections for the structural members must be defined. In this study, properties of materials and structural members have been identified in "IDE CAD Structural" program to find the weight of the structure and the axial loads acting on columns sections. Figure 4.3 show 3D computer model of the building.

The simplest way to analyze building and perform seismic assessment is to study on a frame model, where in this research structure frame consisting beams, and columns are modeled in structural analysis program namely CSI SAP2000 14.0.0 program. By using SAP2000 program, the geometry of the structure can be drawn then assigning properties and loads to the members of the structure to completely define the model. For the two dimensional sectional analysis of beams and columns "Response-2000" program is used to calculate Moment-Curvature relationships for columns and beams sections in order to define hinge properties in the frame model.

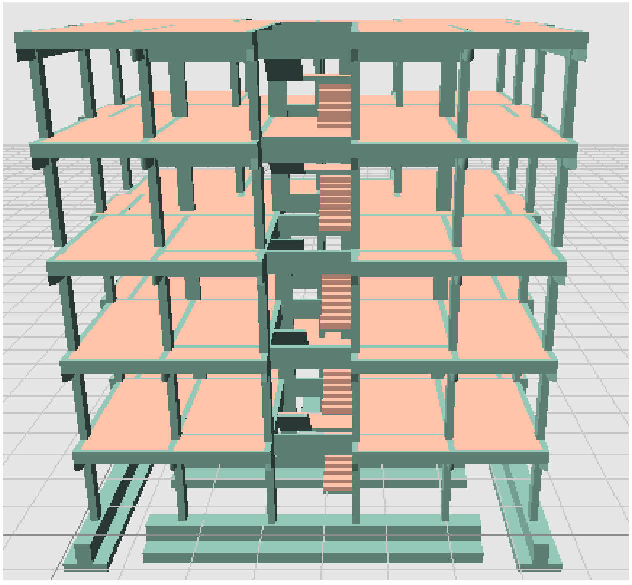


Figure 4.3: 3D Computer Model of the Building

4.3.1 Structural Members Sections Properties

Characteristics for sections of columns and beams for frame structural members shown in tables below.

Table 4.1: Beams sections properties

Sections	Dim.(cm)	Top	Bent-up	Bottom	Straight	Stirrup	W(kN/m)
Beam	20 X 70	2Φ14	3Φ14	2Φ14	2Φ10	Φ8/20	3.5

(cover = 3 cm, $\boldsymbol{f_c}$ = 19 MPa, $\boldsymbol{f_y}$ = 191 MPa)

Table 4.2: Columns sections properties

Sections	Dim.(cm)	Major	Minor	Middle layer 1	Middle layer 2	Stirrup	W(kN/m)
Column	25 X 50	2Φ18	2Φ18	2Φ14	---------	Φ8/17	3.125
Column	25 X 70	2Φ18	2Φ18	2Φ14	2Φ14	Φ8/17	4.375

(cover = 3cm, $\boldsymbol{f_c}$ = 15 MPa for the first floor, $\boldsymbol{f_c}$ = 19 MPa for other floors, $\boldsymbol{f_y}$ = 191 MPa, unit weight of concrete = 25 kN/m^3)

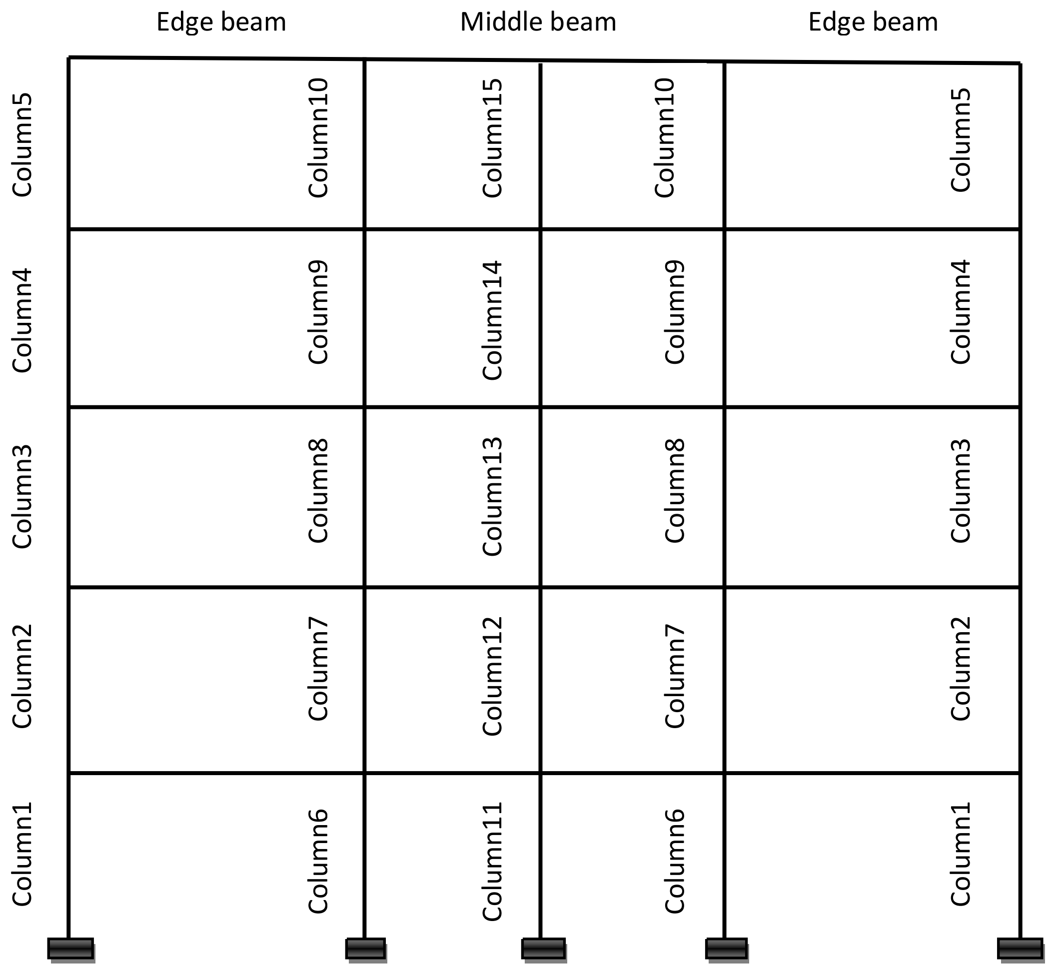


Figure 4.4: Frame Model of Building

4.3.2 External Disturbances on the Building

After delineating the geometric properties of the structure in computer electronic program and conception sections and hinges for every single conclude of frame members, loads replacing on frames have to be identified.

1.5 kN/m^2 are identified as additional dead load for all members over the direction of gravity. 2 kN/m^2 are identified as live load in the direction of gravity. Self weights of slabs are found for 17 cm thickness as 4.25 kN/m^2 distributed in the gravity direction. 4 kN/m^2 are defined as wall load for 1 m^2 distributed along the beams over the direction of gravity.

All sections of the structure were defined by "IDE CAD Structural" program to calculate self weights for all sections and self weight of the whole structure. Also this program was used to calculate the axial load acting on columns by conducting a linear static analysis to the structure after defining sections of columns, beams, slabs, foundations, stairs and their characteristics.

4.4 Sectional Analysis of Reinforced Concrete Members

Sectional analysis approach is a method of analysis to calculate the strength and deformation in terms of moments, shears and curvatures for reinforced concrete members. The problem of determining the response of a reinforced concrete structure to applied loads is assuming that the structure remains linearly elastic, so by using a specific programs such as "Response-2000" program that was used in this study, the non-linear characteristics of cracked reinforced concrete sections are taken into account. "Evan C. Bentz (2000)" state that "Response-2000 program is believed to be the most immediately useful program. It will calculate strengths and deformations for beams and columns subjected to axial load, moment and shear" [16].

In order to perform sectional analysis for columns and beams sections, sections were modeled as a two dimensional sections in analysis computer program "Response-2000". After defining sections characteristics and material properties, axial loads are applied as constant loads that were calculated from "IDE CAD Structural" program.
Reinforcement bars were defined in each section either by selecting diameter or calculating area of these bars.

"Response-2000" provides an intensive sectional analysis for the full member behavior for a prismatic section which helps to identify the deformation of the sections. Also this program allows user to calculate the deflection at any shear force until reaching to the maximum deflection under maximum shear force. This will allow engineer to observe the full member behavior. Moment-Curvature relationships for all columns and beams sections are illustrated in Appendix A.

4.4.1 Reinforced Concrete Sectional Analysis Steps

In order to perform sectional analysis following steps must be taken:

1- Creating columns and beams sections according to their characteristics.

2- Defining reinforcement bars for columns and beams sections.

3- Assigning axial loads that were taken from "IDE CAD Structural" program for each column section.

4- Running member response analysis and using output data of moment-curvature curves to modify plastic hinges properties.

4.5 Non-Linear Static Pushover Analysis

Pushover analysis represent a static approximation of the response of the structure under earthquake loading. The nonlinear static analysis method components of applying a vertical distributed of monotonically increasing lateral loads to a model which appropriates the material non-linearities of the structure [17].

For the purpose of implementation pushover analysis, building was modeled as a two dimensional frame system. "SAP 2000" structural analysis program has been used for this purpose. Dead loads and live loads have been assigned as uniformly distributed loads. In addition to 1st Mode lateral load pattern, rectangular and triangular shapes are used for the lateral load pattern as shown in Figures 4.5 & 4.6. Reinforced concrete sections and their material characteristics were identified and assigned to the related member.

"SAP 2000" provides an effective nonlinear static analysis preference which helps to determine the failure modes of structure. Plastic hinges may insert at both end of clear length of frame element. Each hinge represents concentrated post-yield behavior in one or more degrees of freedom. In this study, properties of created hinges were taken from Moment-Curvature relationship that created by "Response-2000" program for columns and beams sections.

To calculate any response of the structure caused by the load patterns, load cases must be defined. P-Δ effects were taken into account. "Mehmet Inel, Hayri Baytan Ozmen (2006)" said that " In pushover analysis, the behavior of the structure is characterized by a capacity curve that represents the relationship between the base shear force and the displacement of the roof. This is a very convenient representation in practice, and can be visualized easily by the engineer" [18].

In this research, pushover analysis have been carried out on the results of the vertical load analysis. Loads imposed must be carried out up to the displacement of the control joint specified before by the user. Structural models in this research were analyzed through static nonlinear procedures according to both case, with and without effect of corrosion. Results will be depending on FEMA356 and Eurocode8. Comparison of results obtained from both analysis are illustrated in chapter 5.

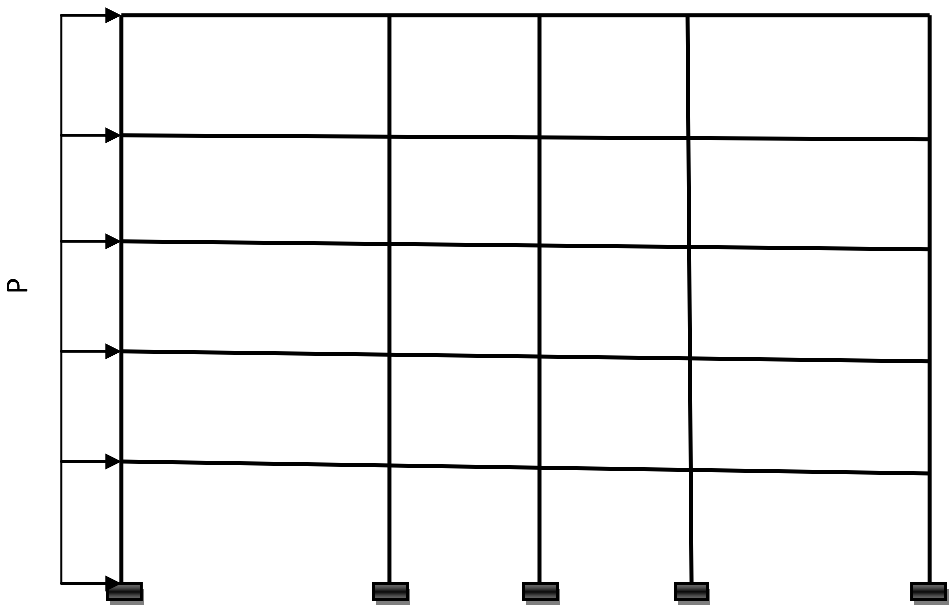


Figure 4.5: Frame Model With Uniform Lateral Loads

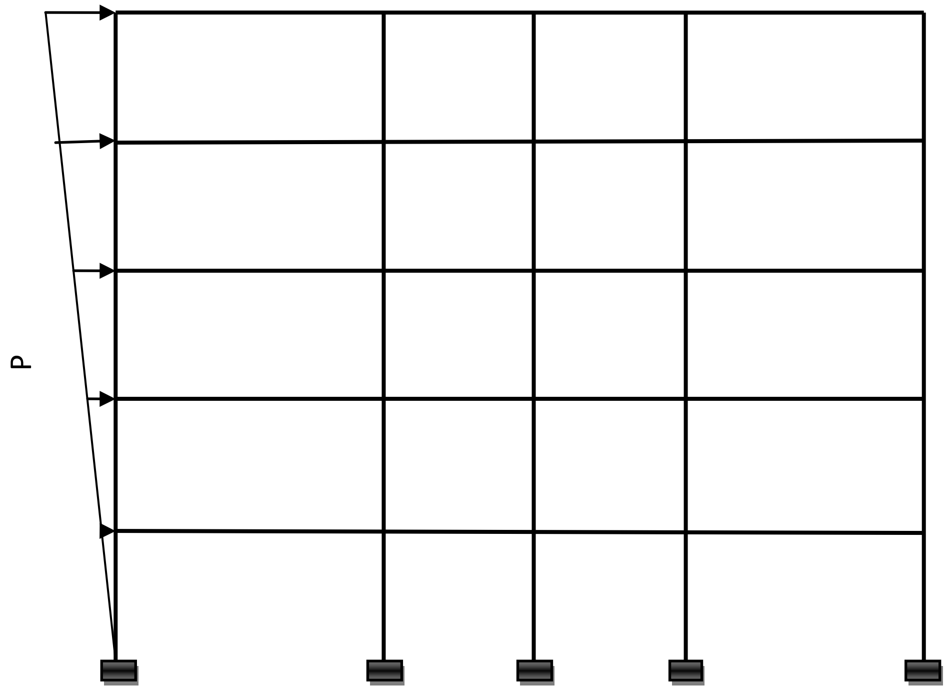


Figure 4.6: Frame Model With Triangular Lateral Loads

4.5.1 Pushover Analysis Steps

To find out the performance of nonlinear static pushover analysis following steps must be taken:

1- Create elements that represent beams and columns of frame structure.

2- Identify material properties and section characteristics of beams and columns.

3- Assign defined sections of beams and columns to the frame structure.

4- Define load patterns of dead and live loads. Rectangular and triangularly distributed shapes are used for the lateral load pattern.

5- Assign dead and live loads as gravity distributed load, lateral loads will be assigned as joint loads on frame structure.

6- Define plastic hinge properties that will be taken from Moment-Curvature relationship calculated by "Response-2000" program for beams and columns sections. Plastic hinge length defined according to FEMA356 and Eurocode8.

7- Assign plastic hinges at both ends of each member of frame model.

8- Pushover load case must be defined; Nonlinear static analysis was identified, P-Δ effects were taken into account.

9- Run analysis and use output data to emphasize the capacity curve and distinguish the performance of the structure.

Chapter 5

RESULTS AND DISCUSSION

5.1 Pushover Analysis Results According to FEMA 356

Nonlinear static pushover analysis has been conducted on two dimensional frame models of buildings. Since these buildings have corrosion problems, nonlinear static analysis was performed with estimated 5%, 10%, 15%, 20% different corrosion level appeared in columns of the ground floor. Results acquired from the analysis have been discussed in sections below.

5.1.1 Force-Displacement Curves

Force-Displacement curve or "Capacity Curve" is delineating the displacement of the control node because of the lateral loads replacing alongside the height of the construction. Lateral loads were requested in one direction (U_x) for the two dimensional frame model and the related displacements in the same direction were recorded.

Pushover curves of two dimensional frame model of buildings for corroded and non-corroded cases are shown in following sections.

5.1.1.1 Non-Corroded Case

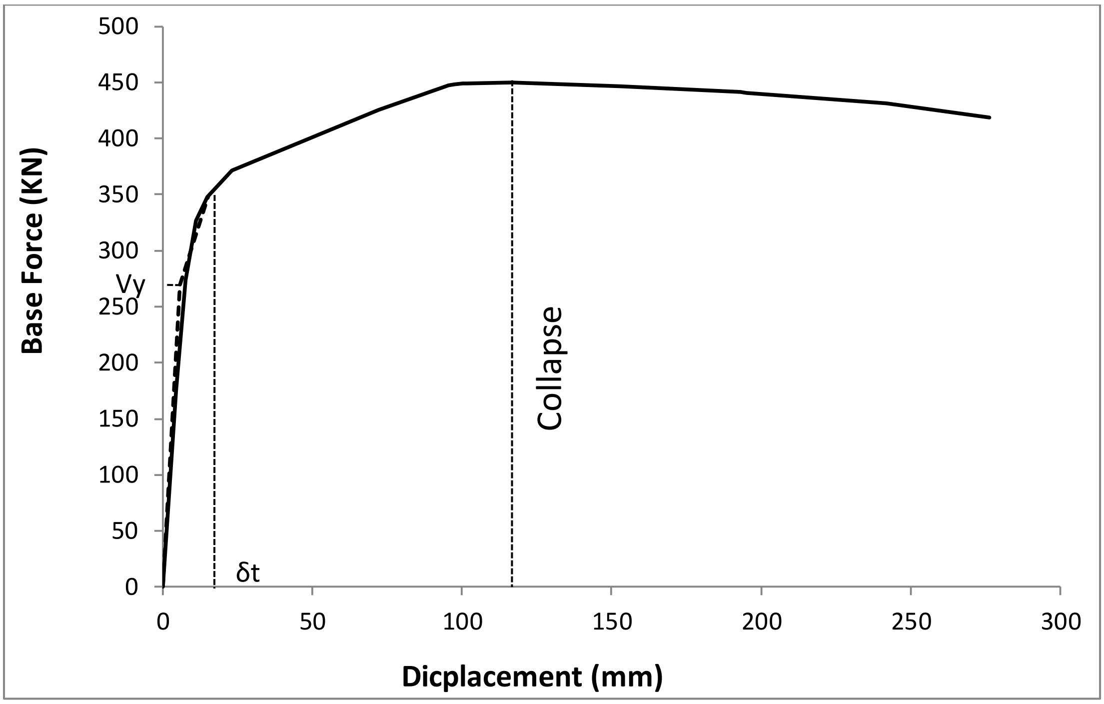


Figure 5.1: Capacity Curve for Frame Model With 1st Mode Lateral Load (Non-Corroded Case)

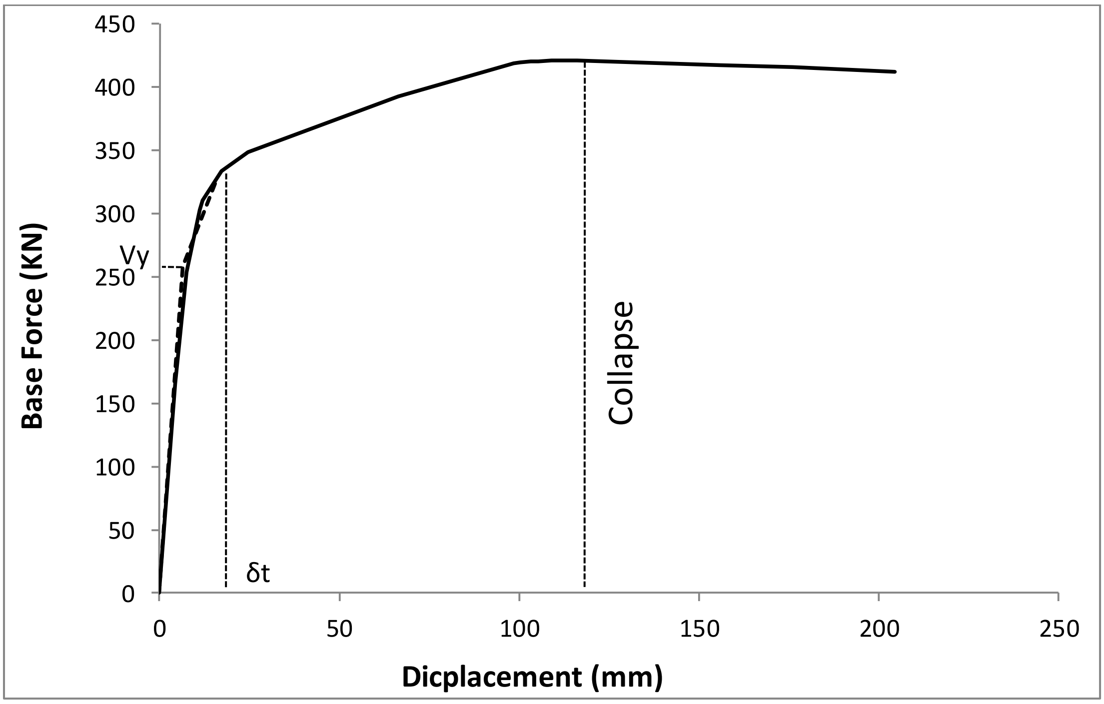


Figure 5.2: Capacity Curve for Frame Model With Uniform Lateral Loads (Non-Corroded Case)

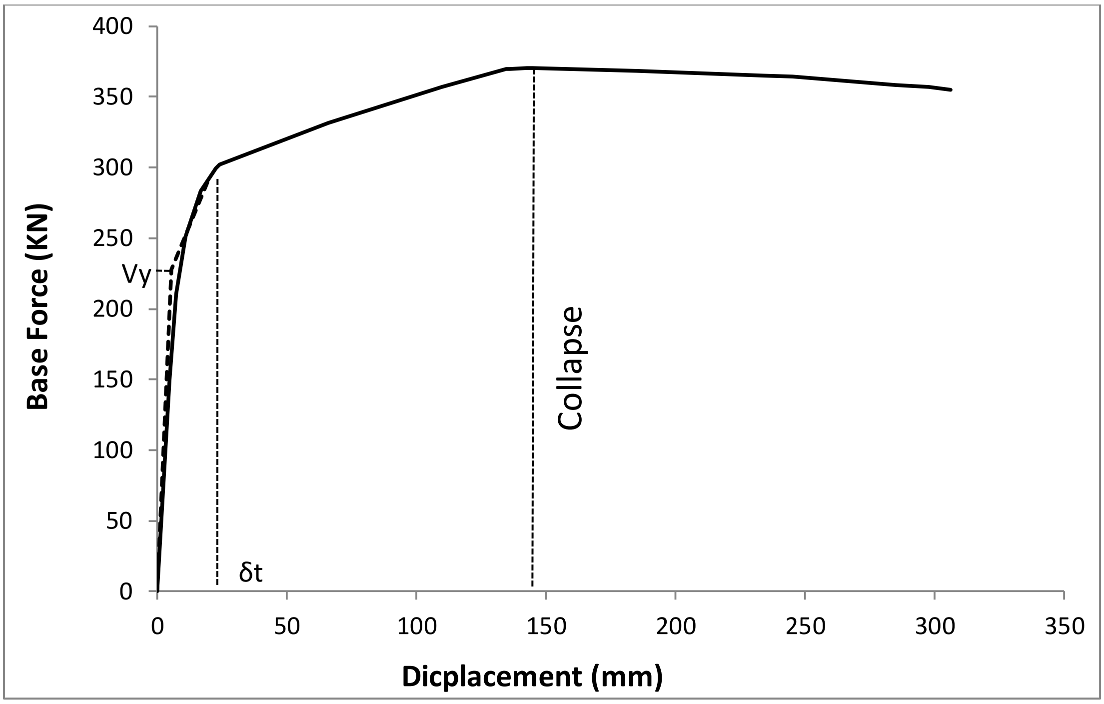


Figure 5.3: Capacity Curve for Frame Model With Triangular Lateral Loads (Non-Corroded Case)

FEMA 356 procedure is applied to determine the target displacement and the corresponding base shear force as discussed in chapter 2. Figure 5.1 show the capacity curve for two dimensional frame model with 1st mode lateral load where target displacement is evaluated as $\boldsymbol{\delta_t}$ = 15.042 mm, base shear force provoking this displacement is equal to 349.035 kN and effective yield strength $\boldsymbol{V_y}$ = 269.531 kN. Capacity curve for frame model with uniform lateral load pattern is shown in Figure 5.2. According to the FEME356 procedure the target displacement calculated as $\boldsymbol{\delta_t}$ = 16.281 mm where the corresponding base shear force is 328.997 kN. It can be seen that the effective yield strength, $\boldsymbol{V_y}$, in this case is equal to 258.202 kN. Figure 5.3 show the capacity curve for the frame model with triangular lateral load pattern where target displacement is evaluated as $\boldsymbol{\delta_t}$ = 19.134 mm, base shear force provoking this displacement is equal to 289.291 kN and effective yield strength $\boldsymbol{V_y}$ = 227.503 kN.

From the obtained results it can be perceived that both 1st mode lateral load and frame model with triangular lateral load have convergent values of roof displacement at yield point that equal to 5.79 and 5.61 mm respectively, while the frame model with uniform lateral load has roof displacement equal to 6.51 mm at yield point.

Collapse happens at 116.5 mm roof displacement for both 1st mode lateral load and frame with uniform lateral load pattern. For the frame with triangular lateral load pattern collapse occurs at 144.99 mm roof displacement. At this level of displacement demand, it can be observed that the structural model will be in mechanism.

5.1.1.2 (5%) Corroded

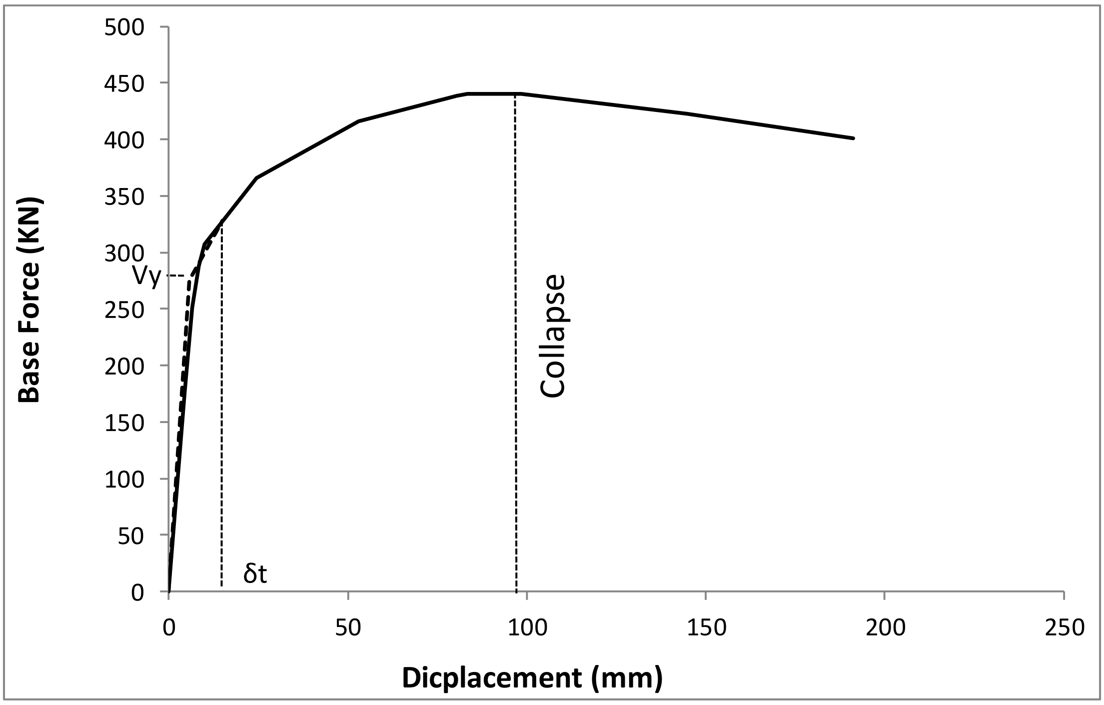


Figure 5.4: Capacity Curve for Frame Model With 1st Mode Lateral Load (5% Corroded)

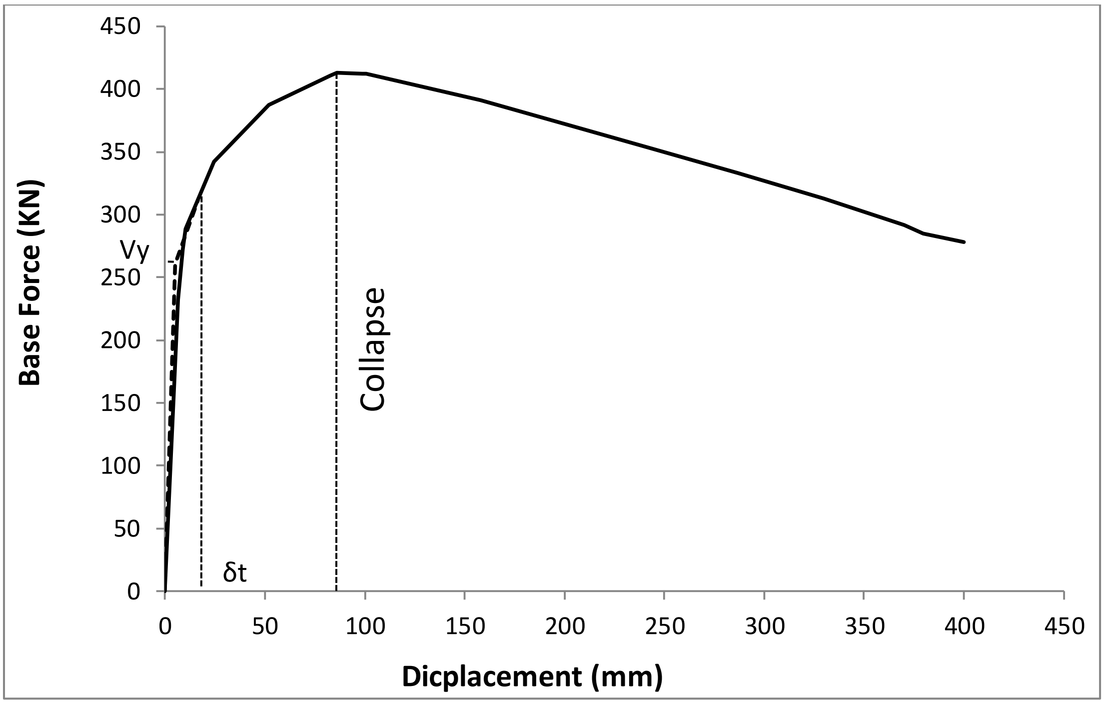


Figure 5.5: Capacity Curve for Frame Model With Uniform Lateral Loads (5% Corroded)

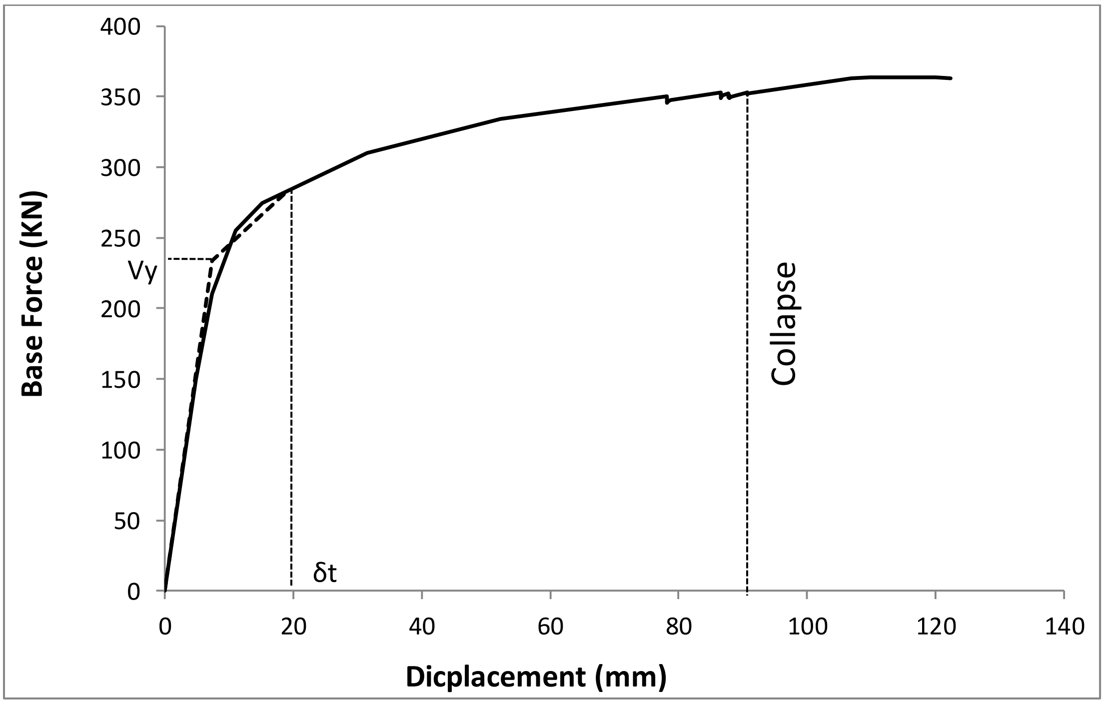


Figure 5.6: Capacity Curve for Frame Model With Triangular Lateral Loads (5% Corroded)

Figure 5.4 show the 1st mode lateral load capacity curve, where target displacement is evaluated as δ_t = 15.025 mm, base shear force provoking this displacement is equal to 327.878 kN and effective yield strength V_y = 275.768 kN. Capacity curve for frame model with uniform lateral load pattern is shown in Figure 5.5. According to the FEME356 procedure the target displacement calculated as δ_t = 16.251 mm where the corresponding base shear force is 311.585 kN. It can be seen that the effective yield strength, V_y, in this case is equal to 260.984 kN. Figure 5.6 shows the capacity curve for the frame model with triangular lateral load pattern where target displacement is evaluated as δ_t = 19.124 mm, base shear force provoking this displacement is equal to 283.27 kN and effective yield strength V_y = 233.64 kN.

From the obtained results it can be perceived that both 1st mode lateral load and frame model with uniform lateral load have convergent values of roof displacement at yield point that equal to 5.68 and 5.168 mm respectively, while the frame model with triangular lateral load has roof displacement equal to 7.39 mm at yield point.

Collapse happens at 96.35 mm roof displacement for 1st mode lateral load and for frame with uniform lateral load pattern collapse occurs at 85.92 mm roof displacement. For the frame with triangular lateral load pattern collapse occurs at 87.79 mm roof displacement.

5.1.1.3 (10%) Corroded

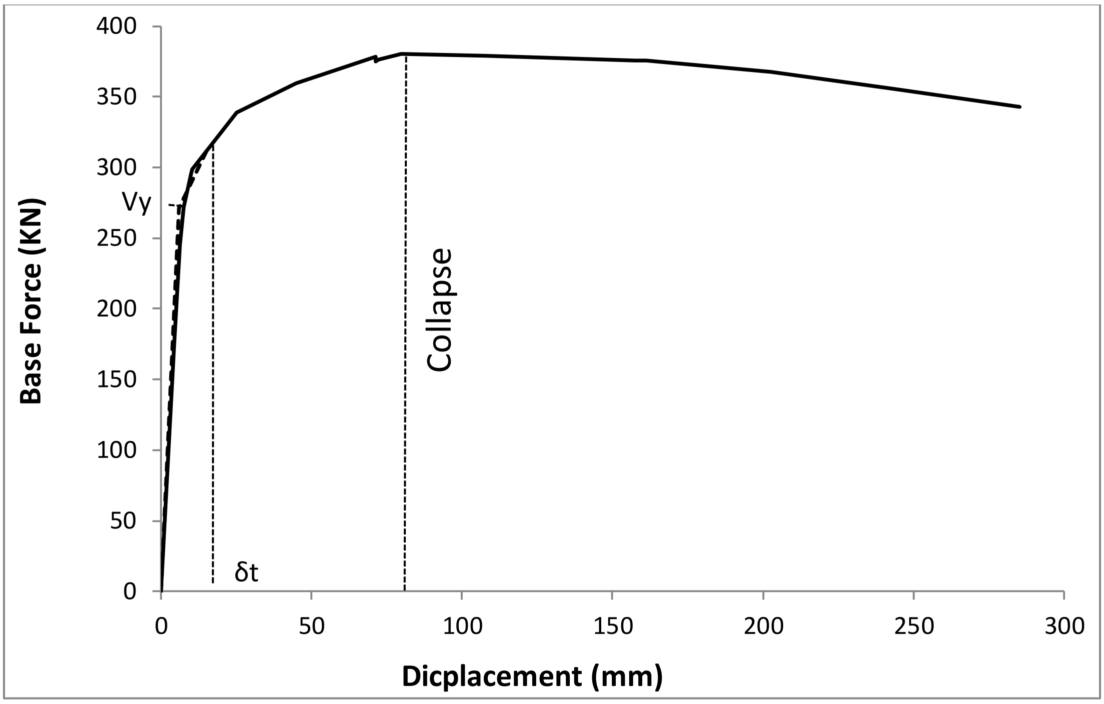


Figure 5.7: Capacity Curve for Frame Model With 1st Mode Lateral Load (10% Corroded)

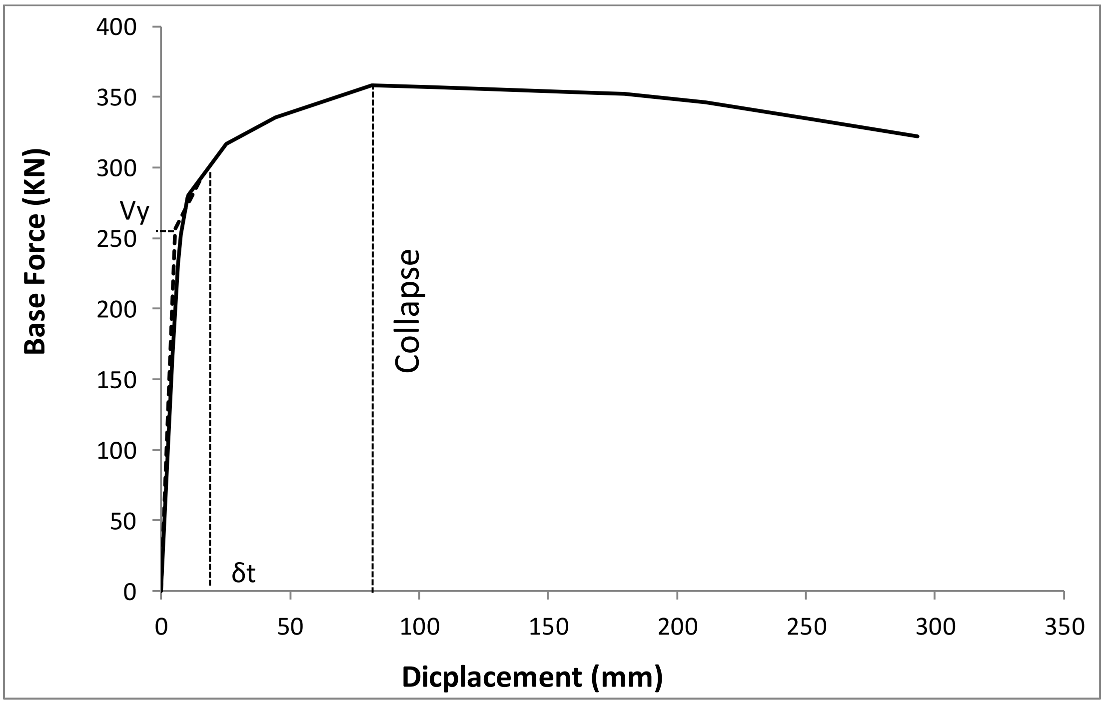


Figure 5.8: Capacity Curve for Frame Model With Uniform Lateral Loads (10% Corroded)

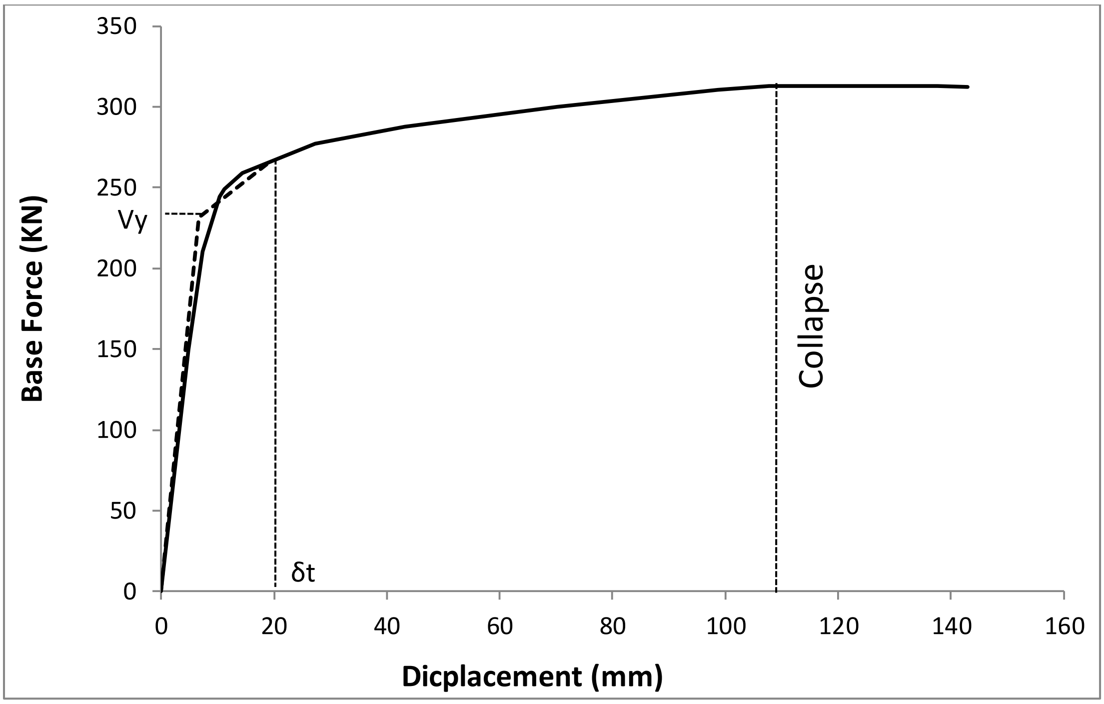


Figure 5.9: Capacity Curve for Frame Model With Triangular Lateral Loads (10% Corroded)

Figure 5.7 show the capacity curve for 1st mode lateral load where target displacement is computed as $\boldsymbol{\delta_t}$ = 15.01 mm, base shear force provoking this displacement is equal to 311.431 kN and effective yield strength $\boldsymbol{V_y}$ = 271.09 kN. Capacity curve for frame model with uniform lateral load pattern is shown in Figure 5.8. According to the FEME356 procedure the target displacement calculated as $\boldsymbol{\delta_t}$ = 16.228 mm where the corresponding base shear force is 294.453 kN. It can be seen that the effective yield strength, $\boldsymbol{V_y}$, in this case is equal to 256.56 kN. Figure 5.9 shows the capacity curve for the frame model with triangular lateral load pattern where target displacement is calculated as $\boldsymbol{\delta_t}$ = 19.09 mm, base shear force causing this displacement is equal to 265.682 kN and effective yield strength $\boldsymbol{V_y}$ = 231.261 kN.

From the obtained results it can be perceived that both 1st mode lateral load and frame model with uniform lateral load have convergent values of roof displacement at yield point that equal to 5.99 and 5.43 mm respectively, while the frame model with triangular lateral load has roof displacement equal to 6.59 mm at yield point.

Collapse happens at 79.29 mm roof displacement for 1st mode lateral load and for frame with uniform lateral load pattern collapse occurs at 81.56 mm roof displacement. For the frame with triangular lateral load pattern collapse occurs at 108.89 mm roof displacement.

5.1.1.4 (15%) Corroded

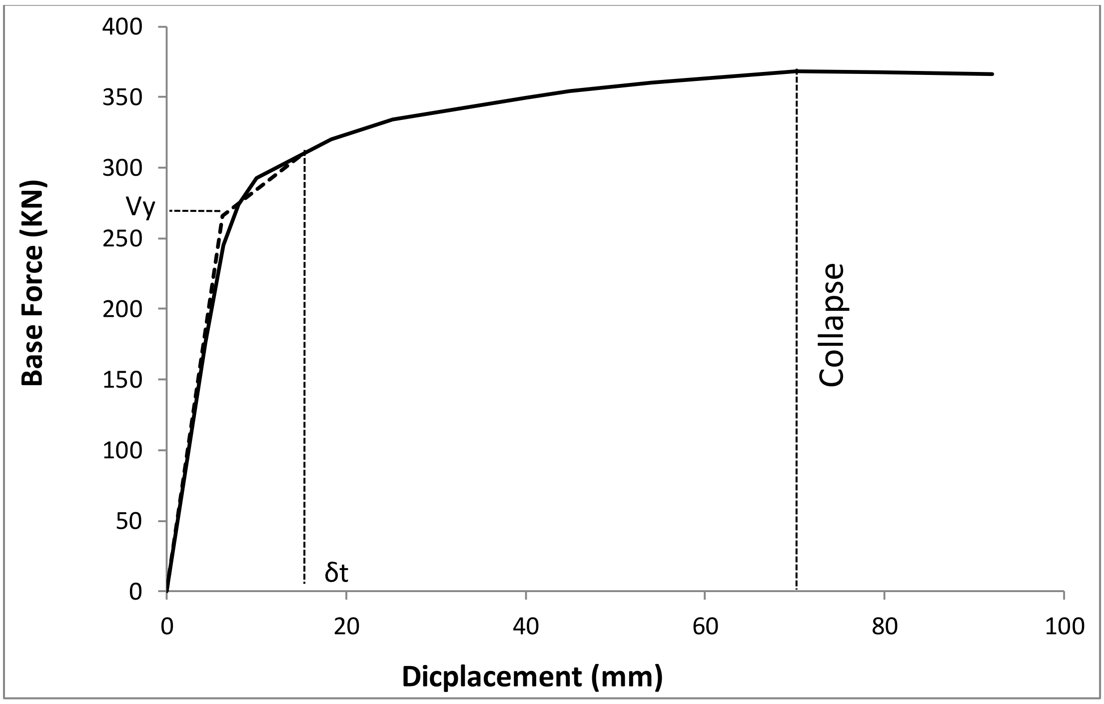


Figure 5.10: Capacity Curve for Frame Model With 1st mode lateral load (15% Corroded)

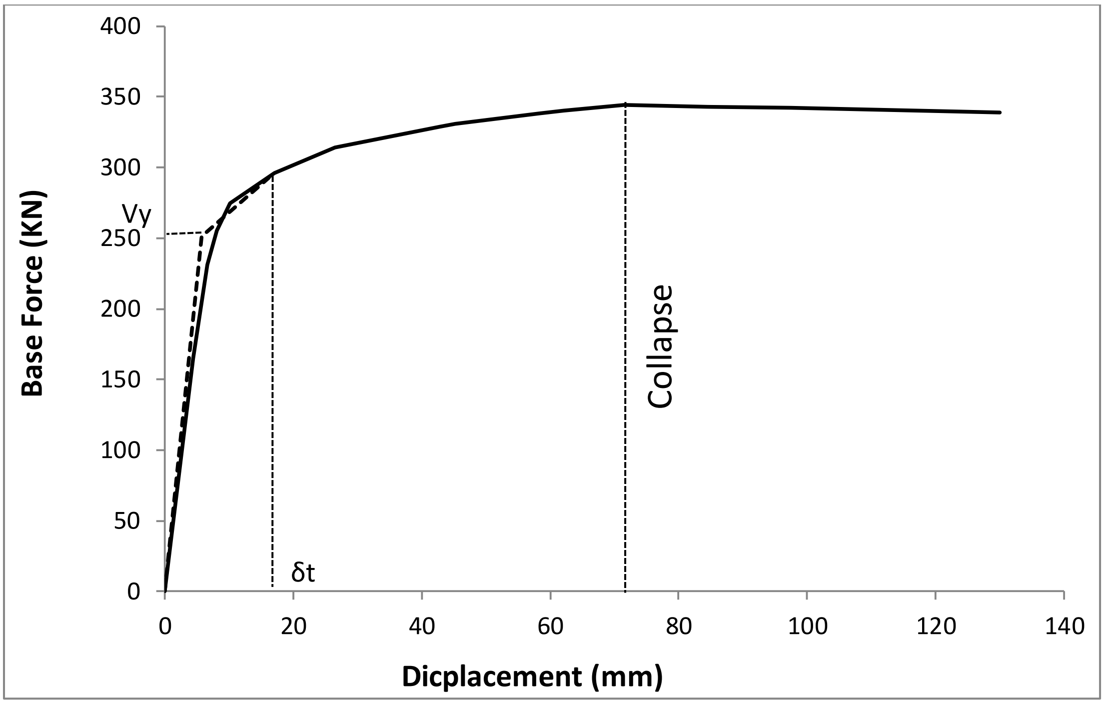


Figure 5.11: Capacity Curve for Frame Model With Uniform Lateral Loads (15% Corroded)

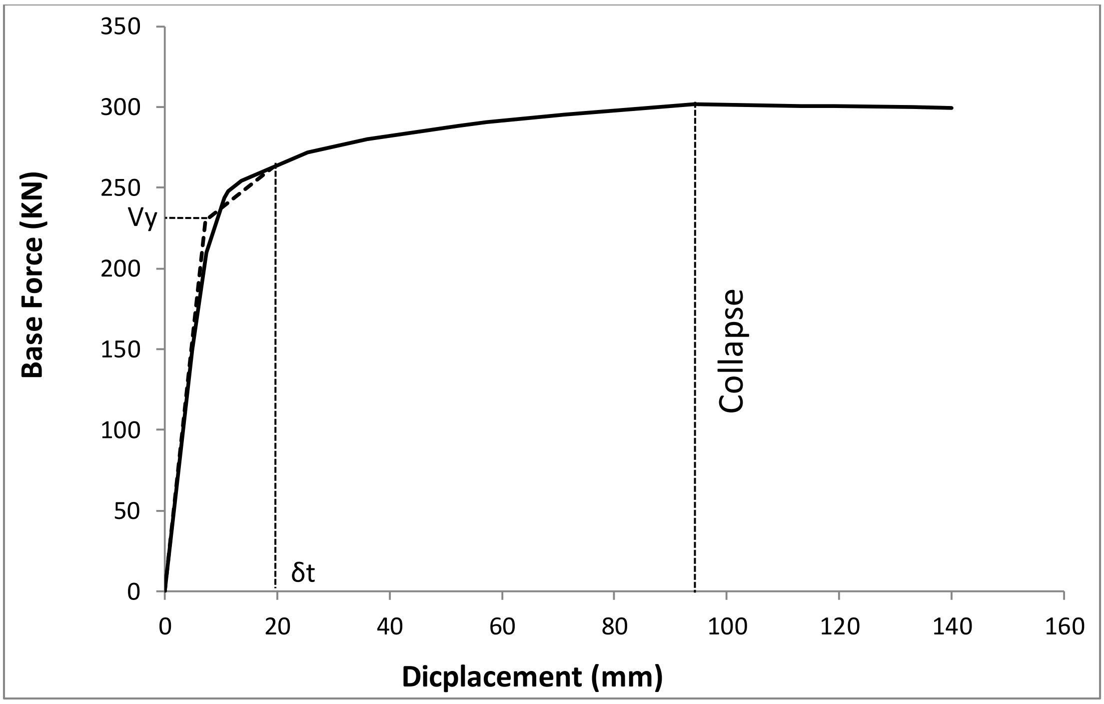


Figure 5.12: Capacity Curve for Frame Model With Triangular Lateral Loads (15% Corroded)

Figure 5.10 shows the 1st mode lateral load capacity curve, where target displacement is computed as $\boldsymbol{\delta_t}$ = 14.961 mm, base shear force causing this displacement is equal to 309.041 kN and effective yield strength $\boldsymbol{V_y}$ = 266.05 kN. Capacity curve for frame model with uniform lateral load pattern is shown in Figure 5.11. According to the FEME356 procedure the target displacement calculated as $\boldsymbol{\delta_t}$ = 16.151 mm where the corresponding base shear force is 292.959 kN. It can be seen that the effective yield strength, $\boldsymbol{V_y}$, in this case is equal to 251.104 kN. Figure 5.12 show the capacity curve for the frame model with triangular lateral load pattern where target displacement is computed as $\boldsymbol{\delta_t}$ = 19.086 mm, base shear force provoking this displacement is equal to 262.715 kN and effective yield strength $\boldsymbol{V_y}$ = 229.564 kN.

From the obtained results it can be perceived that both 1st mode lateral load and frame model with uniform lateral load have values of roof displacement at yield point that equal to 6.18 and 5.71 mm respectively, while the frame model with triangular lateral load has roof displacement equal to 7.24 mm at yield point.

Collapse happens at 70.23 mm roof displacement for 1st mode lateral load and for frame with uniform lateral load pattern collapse occurs at 71.52 mm roof displacement. For the frame with triangular lateral load pattern collapse occurs at 94.26 mm roof displacement.

5.1.1.5 (20%) Corroded

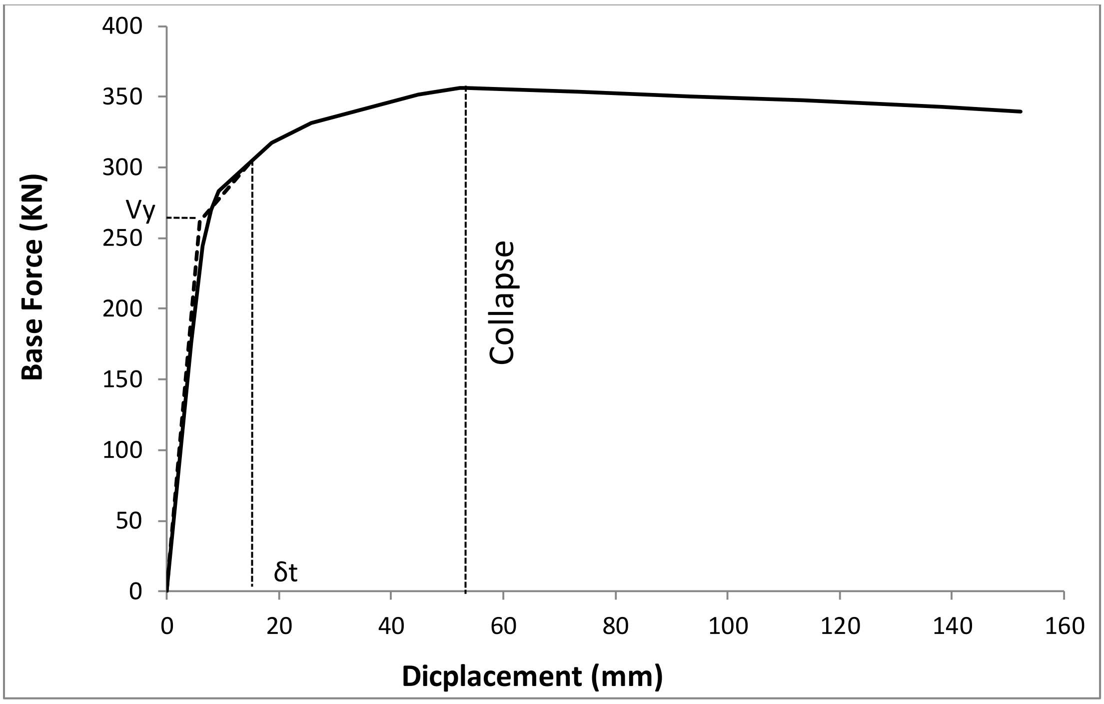


Figure 5.13: Capacity Curve for Frame Model With 1st Mode Lateral Load (20% Corroded)

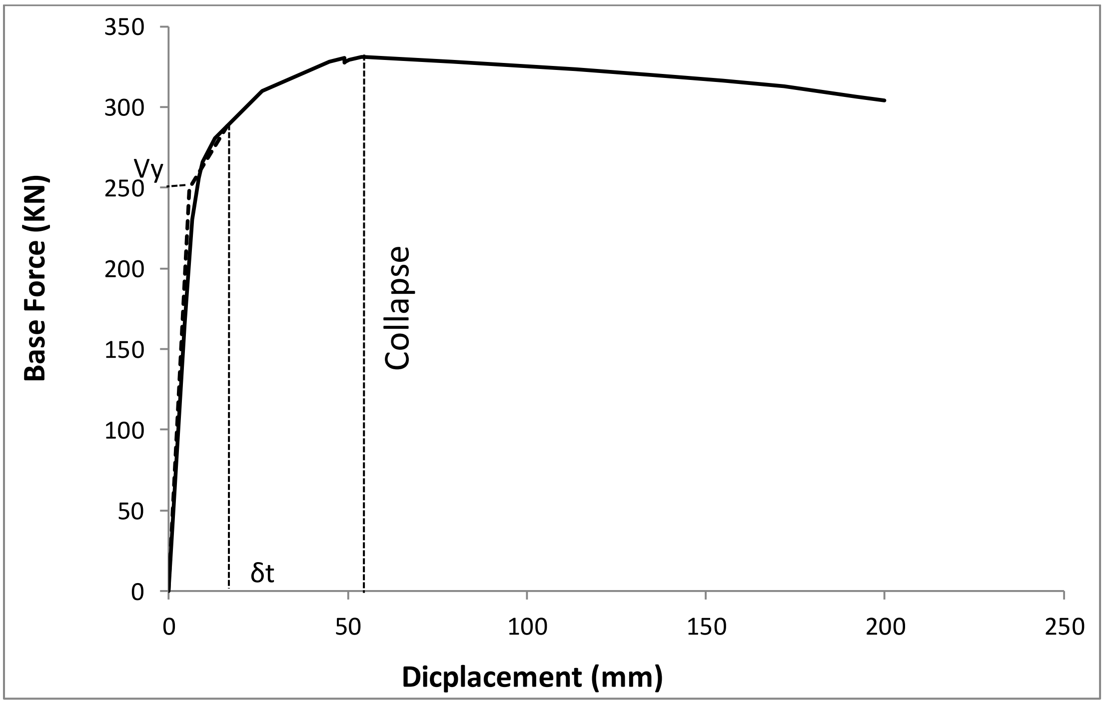


Figure 5.14: Capacity Curve for Frame Model With Uniform Lateral Loads (20% Corroded)

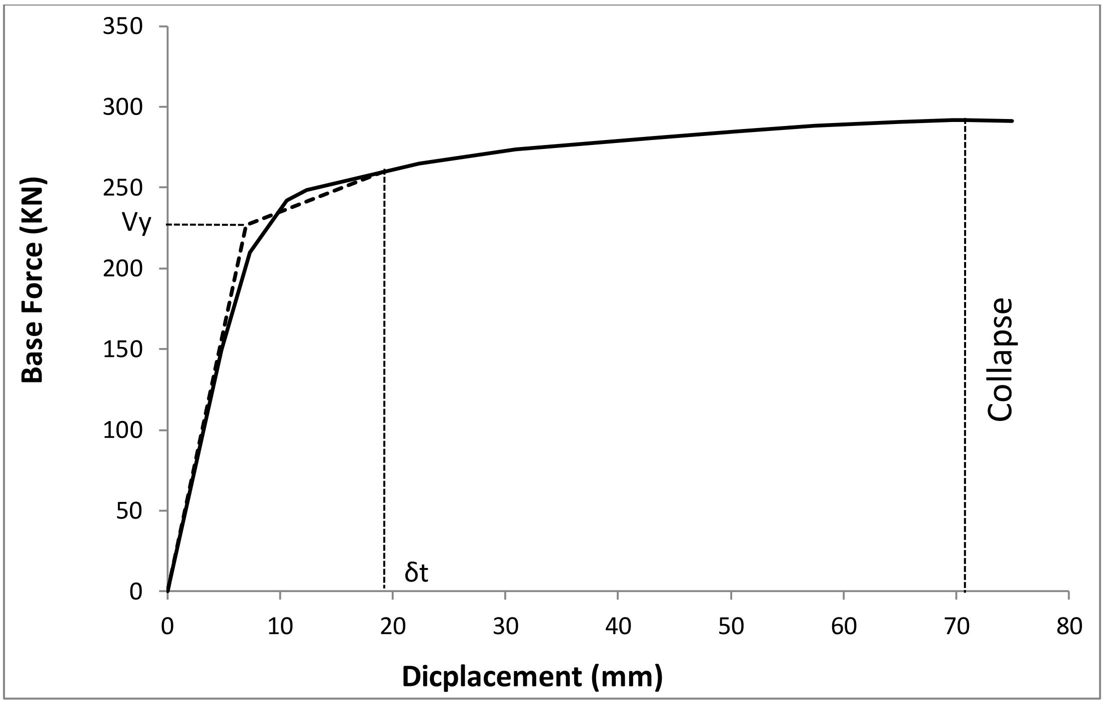


Figure 5.15: Capacity Curve for Frame Model With Triangular Lateral Loads (20% Corroded)

Figure 5.13 shows the capacity curve for 1st mode lateral load where target displacement is evaluated as $\boldsymbol{\delta_t}$ = 14.956 mm, base shear force provoking this displacement is equal to 303.803 kN and effective yield strength $\boldsymbol{V_y}$ = 261.639 kN. Capacity curve for frame model with uniform lateral load pattern is shown in Figure 5.14. According to the FEME356 procedure the target displacement calculated as $\boldsymbol{\delta_t}$ = 16.157 mm where the corresponding base shear force is 288.143 kN. It can be seen that the effective yield strength, $\boldsymbol{V_y}$, in this case is equal to 249.519 kN. Figure 5.15 show the capacity curve for the frame model with triangular lateral load pattern where target displacement is calculated as $\boldsymbol{\delta_t}$ = 19.068 mm, base shear force provoking this displacement is equal to 259.506 kN and effective yield strength $\boldsymbol{V_y}$ = 226.727 kN.

From the obtained results it can be perceived that both 1st mode lateral load and frame model with uniform lateral load have convergent values of roof displacement at yield point that equal to 5.79 and 5.68 mm respectively, while the frame model with triangular lateral load has roof displacement equal to 6.98 mm at yield point.

Collapse happens at 53.44 mm roof displacement for 1st mode lateral load and for frame with uniform lateral load pattern collapse occurs at 54.51 mm roof displacement. For the frame with triangular lateral load pattern collapse occurs at 70.76 mm roof displacement.

5.1.2 Performance Limit States

Base shear force and displacement results have been given in tables together with the plastic hinge distributed at each stage. Target displacement that was obtained from capacity curve for each frame model state represent a turning point of structure to the case of inelasticity, in the other words, the structure will start to resist moments formed due to an increase of lateral forces and changes the level of performance to be up to the collapse state. FEMA356 coefficient method parameters are shown in Appendix D.

The sections below discuss the level of performance of structure in the absence of corrosion and corrosion rates for the other. Through the results of pushover analysis steps, it can be observed that when increasing lateral loads the displacements are increased gradually up to the maximum displacement of the structure, then the structure will collapse, where the failure occurs for the first plastic hinge. After collapsing of the structure, displacement increasing with decreasing lateral forces, where lower force cause large displacement. To identify the displacement of the structure depending on its height this displacement may be taken as a ratio of the building height as shown in sections below.

5.1.2.1 Non-Corroded Case

For non-corroded case of building three different frame models were identified which was frame model with 1st mode lateral load, frame model with uniform lateral loads and frame model with triangular lateral loads. Results obtained from pushover analysis steps were discussed in sections below.

5.1.2.1.1 Frame Model With 1st Mode Lateral Load

Table 5.1: Pushover steps for frame model with 1st mode lateral load -non corroded case

Step	Displacement (mm)	Base force (kN)	Roof drift (%)
1	4.26	175.892	0.028
2	7.50	273.303	0.05
3	11.16	326.448	0.074
4	14.74	348.188	0.098
5	23.01	371.198	0.15
6	72.17	425.8	0.48
7	95.33	447.252	0.64
8	97.23	448.183	0.65
9	100.24	448.974	0.67
10	101.43	449.157	0.68
11	116.81	449.728	0.78
12	153.08	446.542	1.02
13	193.08	441.159	1.29
14	195.18	440.876	1.30
15	241.91	431.381	1.61
16	276.39	419.137	1.84

Table 5.1 shows pushover analysis steps for 1st mode lateral load for non-corroded structure case. The results were obtained show that the building start to yield at 5.79 mm displacement. Plastic hinges for ground columns and beams of two storey change their level to immediate occupancy at step 4. According to FEMA356 at this level the chance of existence intimidating injury as a consequence of structural damage is extremely low, and even though a little minor structural repairs could be appropriate. The structure has 14.74 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 97.23 mm and

performance level change to life safety. This level includes damage to structural components but retains a margin opposing onset of partial or finished collapse. All ground column collapse at step 11 where displacement is equal to 116.81 mm. Table 5.2 shows performance of the structure. Figure 5.16 shows structure's pushover curve with performance level.

Table 5.2: Performance level for frame model with 1st mode lateral load -non corroded case

Performance level	**Start (mm)**	**End (mm)**
Yield	5.79	-
IO	5.79	14.74
LS	14.74	97.23
CP	97.23	116.81
Collapse	116.81	276.39

FEMA 356 procedure yield the target displacement as 15.042 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.1%. Plastic hinges resist moments and change their limit to life safety level. Therefore the structure is expected to have major damages and cannot be used anymore.

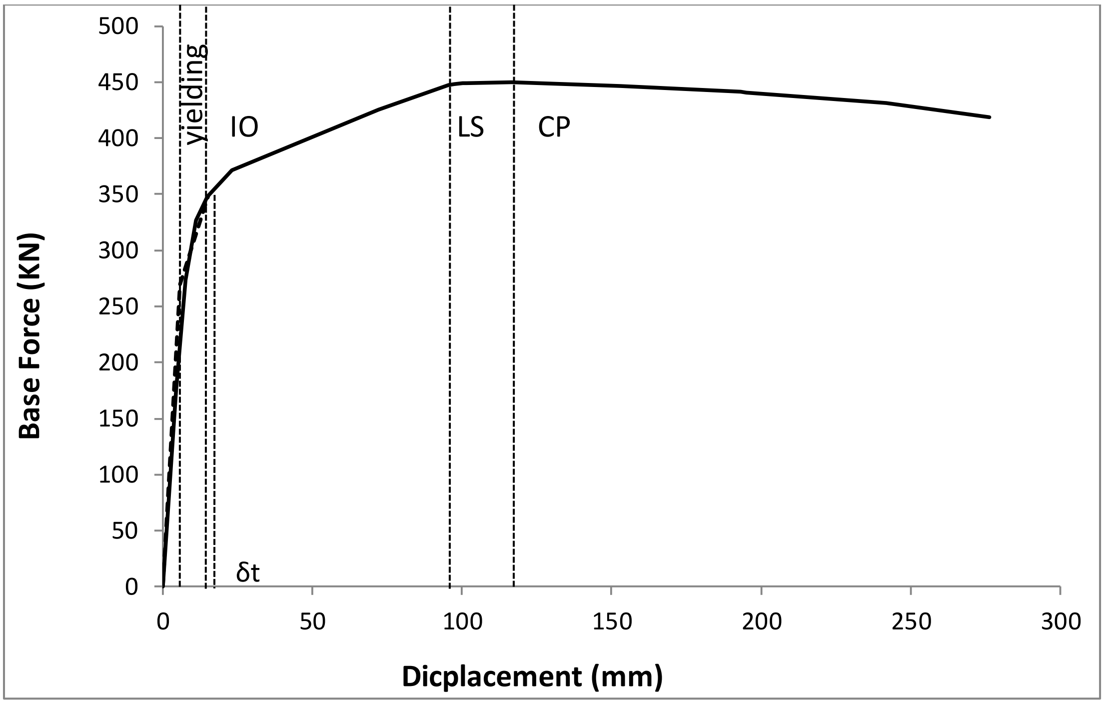


Figure 5.16: Performance Level for Frame Model With 1st Mode Lateral Load (Non-Corroded Case)

5.1.2.1.2 Frame Model With Uniform Lateral Loads

Table 5.3: pushover steps for frame model with uniform lateral loads-non corroded case

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.39	165.045	0.03
2	7.57	253.523	0.05
3	11.22	303.09	0.07
4	12.09	310.523	0.08
5	17.32	333.557	0.12
6	24.47	348.377	0.16
7	66.50	392.119	0.44
8	98.61	418.427	0.66
9	100.43	419.247	0.67
10	103.33	419.987	0.69
11	105.28	420.301	0.70
12	109.06	420.533	0.73
13	116.05	420.765	0.77
14	156.05	417.407	1.04
15	176.11	415.724	1.17
16	204.59	412.09	1.36
17	204.59	412.09	1.36

Results obtained from pushover analysis show that the building start to yield at 6.51 mm displacement. Table 5.3 shows pushover analysis steps for frame model with uniform lateral loads. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 5. The building has 17.32 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 24.47 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for tow storey collapse at step 13 where displacement is equal to 116.05 mm. Table 5.4 shows performance level of the structure. Figure 5.17 shows structure's pushover curve with performance level.

Table 5.4: Performance level for frame model with uniform lateral loads-non corroded case

Performance level	Start (mm)	End (mm)
Yield	6.51	-
IO	6.51	17.32
LS	17.32	24.47
CP	24.47	116.05
Collapse	116.05	204.59

FEMA 356 procedure yield the target displacement as 16.281 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.109%. Plastic hinges resist moments but they are still in immediate occupancy level. Therefore the building under uniform lateral loads expected to have low damages and remain safe to use.

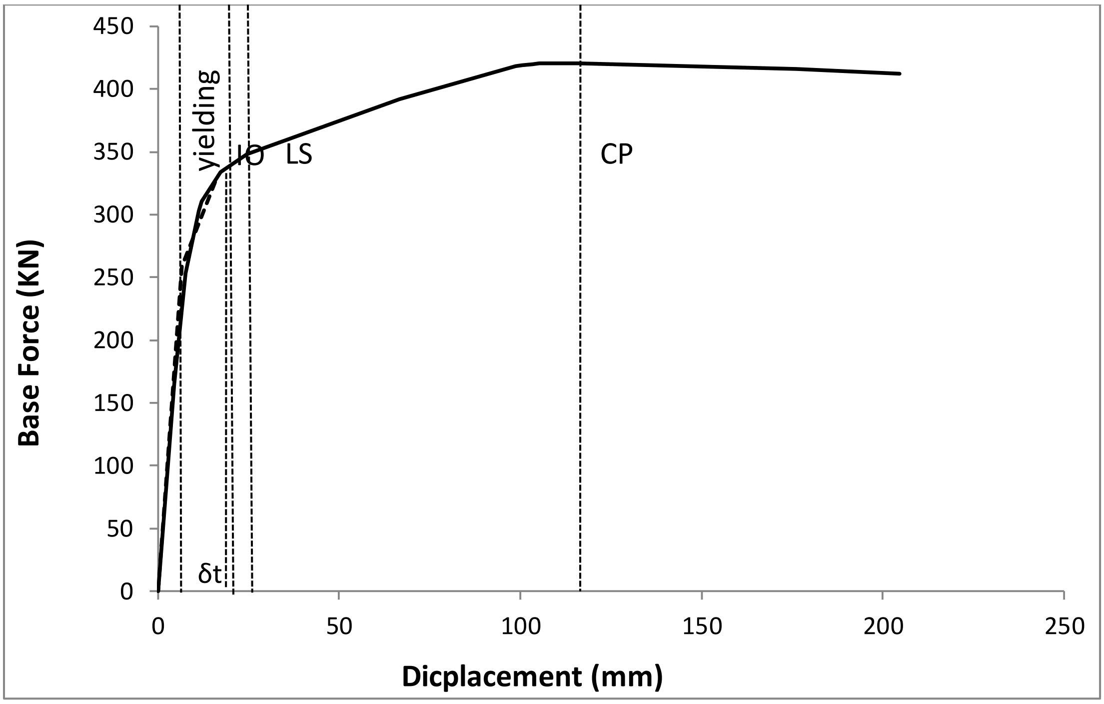


Figure 5.17: Performance Level for Frame Model With Uniform Lateral Loads (Non-Corroded Case)

5.1.2.1.3 Frame Model With Triangular Lateral Loads

Table 5.5: Pushover steps for frame model with triangular lateral loads-non corroded case

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.84	149.752	0.03
2	7.34	210.492	0.05
3	10.79	251.115	0.07
4	16.93	283.163	0.11
5	22.68	299.13	0.15
6	24.22	301.745	0.16
7	66.02	331.656	0.44
8	110.04	357.13	0.73
9	134.84	369.575	0.89
10	136.05	369.854	0.91
11	142.86	370.462	0.95
12	144.99	370.503	0.97
13	184.99	368.202	1.23
14	233.86	365.043	1.56
15	245.28	364.291	1.64
16	285.28	358.592	1.90
17	298.06	356.704	1.99
18	306.47	354.642	2.04

Results obtained from pushover analysis show that the building start to yield at 5.61 mm displacement. Table 5.5 shows pushover analysis steps for frame model with triangular lateral loads. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 5. The building has 22.68 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 110.04 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for tow storey collapse at step 12 where displacement is equal to 144.99 mm. Table 5.6 shows performance level of the structure. Figure 5.18 shows structure's pushover curve with performance level.

Table 5.6: Performance level for frame model with triangular lateral loads-non corroded case

Performance level	Start (mm)	End (mm)
Yield	5.61	-
IO	5.61	22.68
LS	22.68	110.04
CP	110.04	144.99
Collapse	144.99	306.47

FEMA 356 procedure yield the target displacement as 19.134 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.13%. Plastic hinges resist moments but they are still in immediate occupancy level. Therefore the building under triangular lateral loads expected to have low damages and remain safe to use.

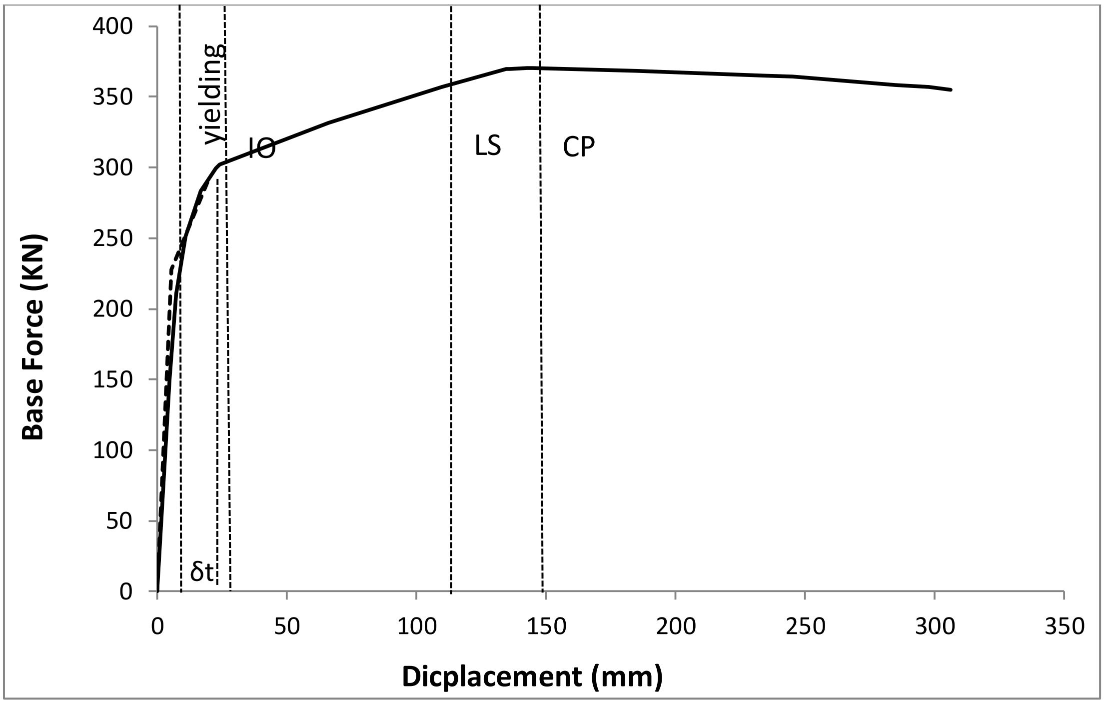


Figure 5.18: Performance Level for Frame Model With Triangular Lateral Loads (Non-Corroded Case)

5.1.2.2 (5%) Corroded

For 5% corroded case of building three different frame models were identified which was 1st mode lateral load, frame model with uniform lateral loads and frame model with triangular lateral loads. Results obtained from pushover analysis steps were discussed in sections below.

5.1.2.2.1 Frame Model With 1st Mode Lateral Load

Table 5.7: Pushover steps for frame model with 1st mode lateral load -5% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.26	175.892	0.03
2	6.50	251.115	0.04
3	8.33	288.232	0.06
4	10.01	307.462	0.07
5	24.34	365.842	0.16
6	52.98	415.993	0.35
7	80.41	438.899	0.54
8	83.36	440.493	0.56
9	96.35	440.524	0.64
10	98.38	440.219	0.66
11	144.67	422.833	0.96
12	191.23	400.805	1.27

Results obtained from pushover analysis show that the building start to yield at 5.68 mm displacement. Table 5.7 shows pushover analysis steps for 1st mode lateral load. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 4. The building has 10.01 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 52.98 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for tow storey collapse at step 9 where displacement is equal to 96.35 mm. Table 5.8 shows performance level of the structure. Figure 5.19 shows structure's pushover curve with performance level.

Table 5.8: Performance level for frame modal with gravity loads-5% corroded

Performance level	Start (mm)	End (mm)
Yield	5.68	-
IO	5.68	10.01
LS	10.01	52.98
CP	52.98	96.35
Collapse	96.35	191.2

FEMA 356 procedure yield the target displacement as 15.025 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.1%. Plastic hinges resist moments and change their limit to life safety level. Therefore the structure is expected to have major damages and cannot be used anymore.

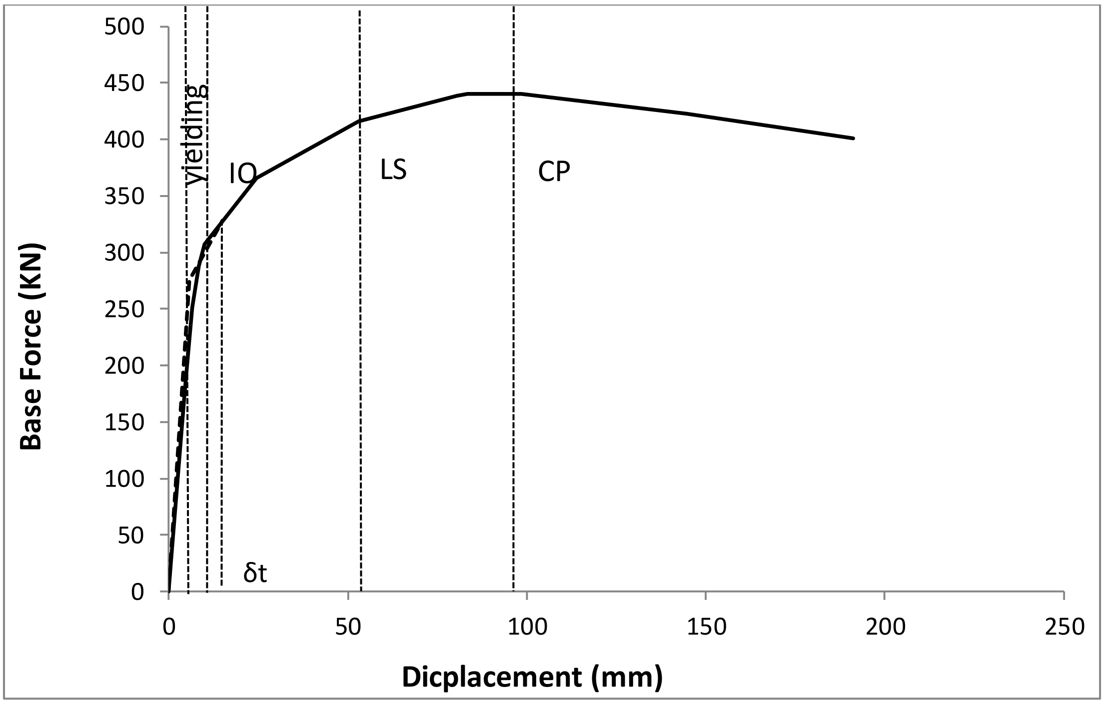


Figure 5.19: Performance Level for Frame Model With 1st Mode Lateral Load (5% Corroded)

5.1.2.2.2 Frame Model With Uniform Lateral Loads

Table 5.9: Pushover steps for frame model with uniform lateral loads-5% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.38	165.045	0.03
2	6.53	232.255	0.04
3	8.71	272.264	0.06
4	10.25	288.957	0.07
5	24.31	342.003	0.16
6	52.02	387.642	0.35
7	82.77	410.957	0.55
8	85.92	412.51	0.57
9	98.62	412.494	0.66
10	100.75	412.188	0.67
11	158.03	391.171	1.05
12	198.03	373.226	1.32
13	242.03	353.39	1.61
14	286.98	332.811	1.91
15	329.76	312.763	2.19
16	370.19	291.391	2.47
17	379.38	285.074	2.53
18	399.99	278.383	2.67

Results obtained from pushover analysis show that the building start to yield at 5.168 mm displacement. Table 5.9 shows pushover analysis steps for frame model with uniform lateral loads. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 6. The building has 52.02 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 82.92 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for tow storey collapse at step 9 where displacement is equal to 98.62 mm. Table 5.10 shows performance level of the structure. Figure 5.20 shows structure's pushover curve with performance level.

Table 5.10: Performance level for frame model with uniform lateral loads-5% corroded

Performance level	Start (mm)	End (mm)
Yield	5.168	-
IO	5.168	52.02
LS	52.02	82.92
CP	82.92	98.62
Collapse	98.62	399.99

FEMA 356 procedure yield the target displacement as 16.251 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.11%. Plastic hinges resist moments but they are still in immediate occupancy level. Therefore the building under uniform lateral loads expected to have low damages and remain safe to use.

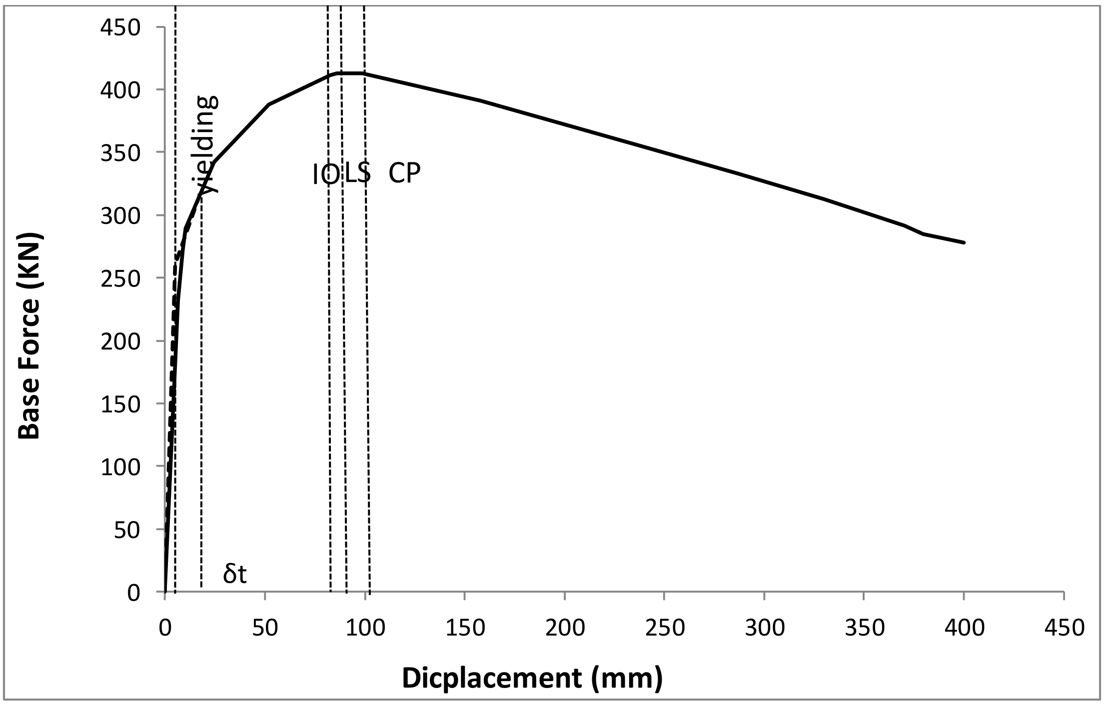


Figure 5.20: Performance Level for Frame Model With Uniform Lateral Loads (5% Corroded)

5.1.2.2.3 Frame Model With Triangular Lateral Loads

Table 5.11: Pushover steps for frame model with triangular lateral loads-5% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.84	149.752	0.03
2	7.34	210.465	0.05
3	10.99	255.121	0.07
4	15.13	274.55	0.10
5	31.53	310.363	0.21
6	52.19	334.34	0.35
7	78.19	350.125	0.52
8	78.19	345.635	0.52
9	78.42	346.816	0.523
10	78.72	347.426	0.525
11	86.58	353.033	0.58
12	86.59	348.727	0.58
13	86.71	350.067	0.58
14	86.88	350.785	0.58
15	87.27	351.466	0.58
16	87.79	352.07	0.59
17	87.80	349.648	0.59
18	87.94	350.754	0.59
19	87.95	349.591	0.59
20	87.95	348.731	0.59
21	88.08	349.426	0.59
22	88.49	350.414	0.59
23	90.66	352.603	0.60
24	90.66	351.343	0.60
25	90.89	352.4	0.61
26	106.86	362.629	0.71
27	109.82	363.727	0.73
28	117.53	363.817	0.78
29	120.05	363.612	0.80
30	122.42	362.958	0.82

Results obtained from pushover analysis show that the building start to yield at 7.39 mm displacement. Table 5.11 shows pushover analysis steps for frame model with triangular lateral loads. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 4. The building has 15.02 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 52.19 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for tow storey collapse at step 28 where displacement is equal to 117.53 mm. Table 5.12 shows performance of the structure. Figure 5.21 shows structure's pushover curve with performance level.

Table 5.12: Performance level for frame model with triangular lateral loads-5% corroded

Performance level	Start (mm)	End (mm)
Yield	7.39	-
IO	7.39	15.02
LS	15.02	52.19
CP	52.19	117.53
Collapse	117.53	122.42

FEMA 356 procedure yield the target displacement as 19.124 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.13%. Plastic hinges resist moments and change their limit to life safety level. Therefore the structure under triangular loads is expected to have major damages and cannot be used anymore.

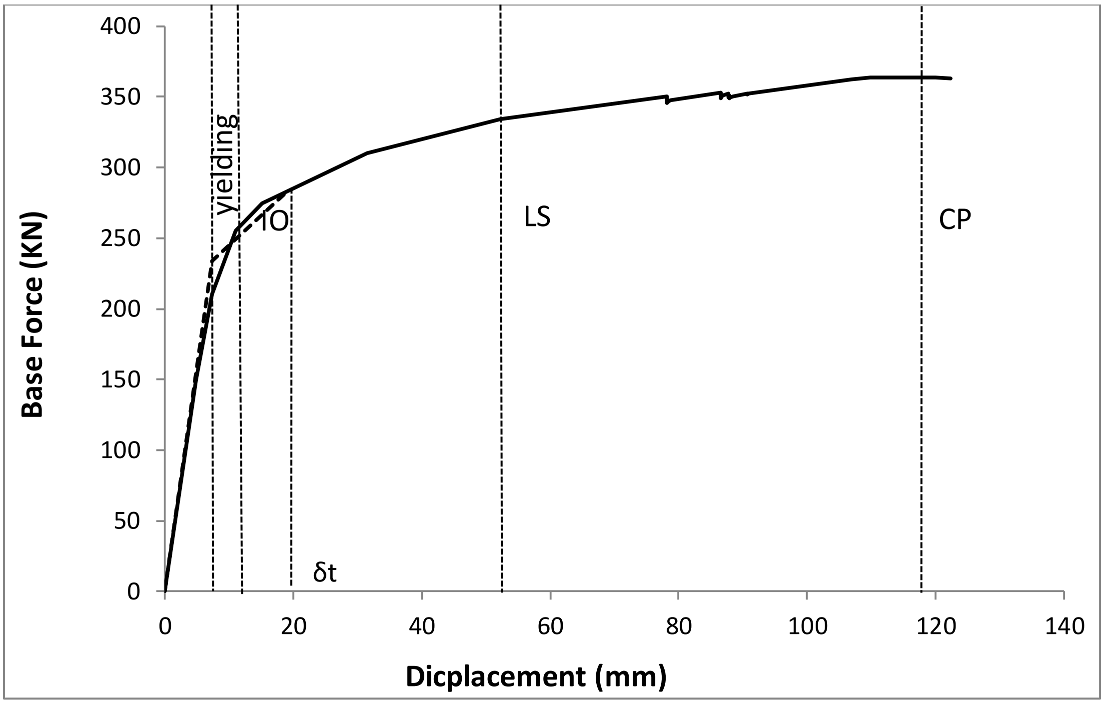


Figure 5.21: Performance Level for Frame Model With Triangular Lateral Loads (5% Corroded)

5.1.2.3 (10%) Corroded

For 10% corroded case of building three different frame models were identified which was 1st mode lateral load, frame model with uniform lateral loads and frame model with triangular lateral loads. Results obtained from pushover analysis steps were discussed in sections below.

5.1.2.3.1 Frame Model With 1st Mode Lateral Load

Table 5.13: Pushover steps for frame model with 1st mode lateral load -10% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.26	175.892	0.03
2	6.28	245.355	0.04
3	7.60	271.791	0.05
4	9.94	295.927	0.07
5	10.39	298.971	0.07
6	25.12	338.746	0.17
7	44.78	359.963	0.29
8	71.18	378.198	0.47
9	71.18	374.741	0.47
10	71.35	375.099	0.48
11	72.37	376.297	0.48
12	79.92	380.139	0.53
13	108.03	379.021	0.72
14	157.40	375.713	1.05
15	161.14	375.421	1.07
16	202.54	367.975	1.35
17	243.39	355.761	1.62
18	285.45	342.732	1.90

Results obtained from pushover analysis show that the building start to yield at 5.99 mm displacement. Table 5.13 shows pushover analysis steps for 1st mode lateral load. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 4. The building has 9.94 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 71.18 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for tow storey collapse at step 12 where displacement is equal to 79.92 mm. Table 5.14 shows performance level of the structure. Figure 5.22 shows structure's pushover curve with performance level.

Table 5.14: Performance level for frame model with 1st mode lateral load -10% corroded

Performance level	Start (mm)	End (mm)
Yield	5.99	-
IO	5.99	9.94
LS	9.94	71.18
CP	71.18	79.92
Collapse	79.92	285.45

FEMA 356 procedure yield the target displacement as 15.01 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.1%. Plastic hinges resist moments and change their limit to life safety level. Therefore the structure is expected to have major damages and cannot be used anymore.

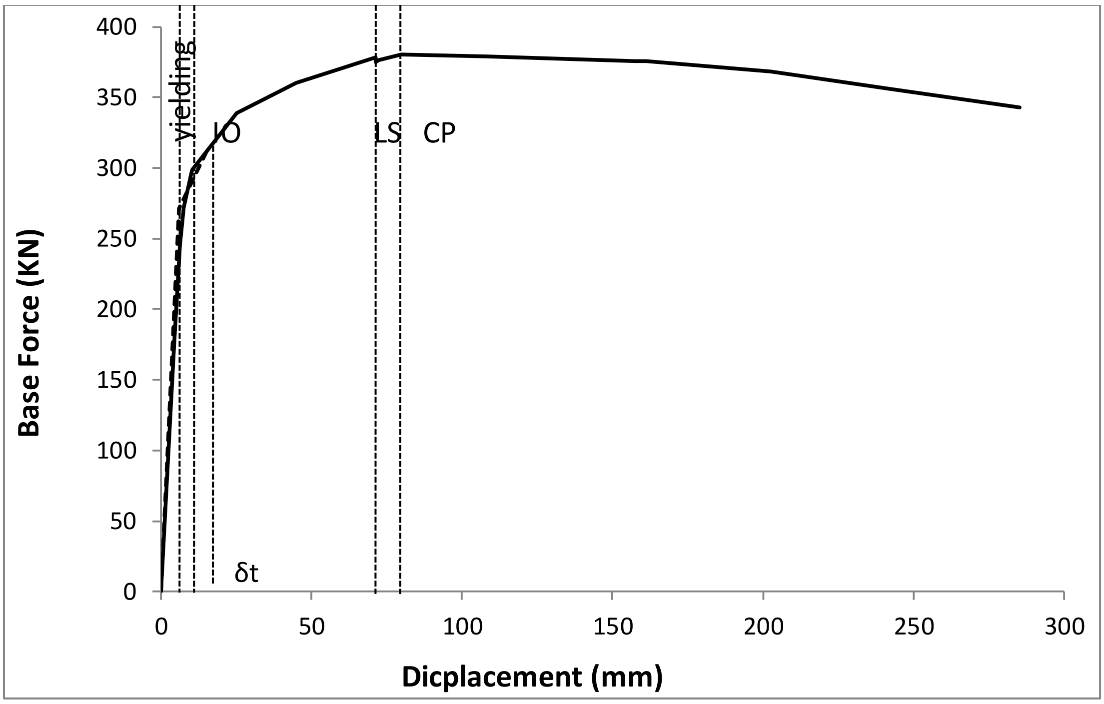


Figure 5.22: Performance Level for Frame Model With 1st Mode Lateral Load (10% Corroded)

5.1.2.3.2 Frame Model With Uniform Lateral Loads

Table 5.15: Pushover steps for frame model with uniform lateral loads-10% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.39	165.045	0.03
2	6.52	231.571	0.04
3	7.58	252.184	0.05
4	10.18	278.272	0.07
5	10.61	280.759	0.07
6	25.25	316.417	0.17
7	44.31	335.345	0.29
8	81.56	358.003	0.54
9	105.93	357.204	0.71
10	167.89	353.142	1.12
11	179.67	351.879	1.19
12	211.02	346.284	1.41
13	252.56	334.389	1.68
14	293.24	322.334	1.95

Results obtained from pushover analysis show that the building start to yield at 5.43 mm displacement. Table 5.15 shows pushover analysis steps for frame model with uniform lateral loads. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 6. The building has 25.25 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 44.31 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for tow storey collapse at step 8 where displacement is equal to 81.56 mm. Table 5.16 shows performance level of the structure. Figure 5.23 shows structure's pushover curve with performance level.

Table 5.16: Performance level for frame model with uniform lateral loads-10% corroded

Performance level	Start (mm)	End (mm)
Yield	5.43	-
IO	5.43	25.25
LS	25.25	44.31
CP	44.31	81.56
Collapse	81.56	293.24

FEMA 356 procedure yield the target displacement as 16.228 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.11%. Plastic hinges resist moments but they are still in immediate occupancy level. Therefore the building under uniform lateral loads expected to have low damages and remain safe to use.

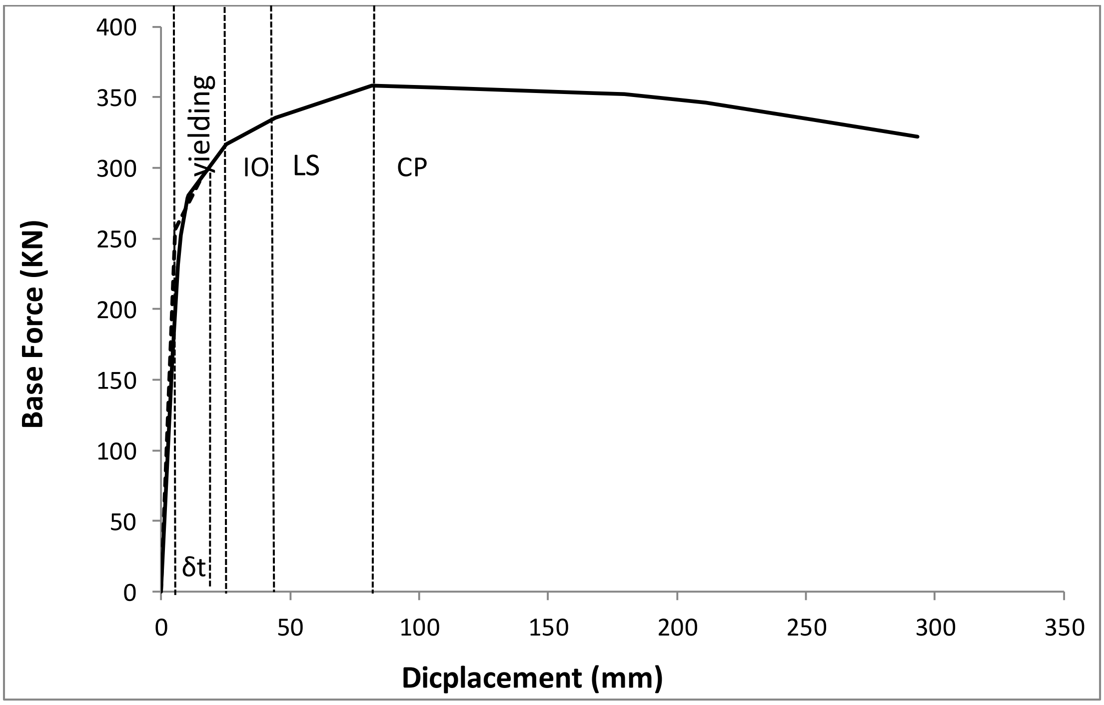


Figure 5.23: Performance Level for Frame Model With Uniform Lateral Loads (10% Corroded)

5.1.2.3.3 Frame Model With Triangular Lateral Loads

Table 5.17: Pushover steps for frame model with triangular lateral loads-10% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.85	149.761	0.03
2	7.34	210.383	0.05
3	10.35	244.209	0.07
4	11.14	248.844	0.07
5	14.38	258.992	0.09
6	27.34	277.399	0.18
7	43.29	287.666	0.29
8	70.17	299.836	0.47
9	84.47	305.111	0.56
10	98.77	310.385	0.66
11	107.70	312.902	0.72
12	108.89	313.061	0.73
13	114.75	313.035	0.77
14	129.05	312.917	0.86
15	137.67	312.847	0.92
16	142.99	312.613	0.95

Results obtained from pushover analysis show that the building start to yield at 6.59 mm displacement. Table 5.17 shows pushover analysis steps for frame model with triangular lateral loads. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 3. The building has 10.35 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 27.34 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for tow storey collapse at step 12 where displacement is equal to 108.89 mm. Table 5.18 shows performance level of the structure. Figure 5.24 shows structure's pushover curve with performance level.

Table 5.18: Performance level for frame model with triangular lateral loads-10% corroded

Performance level	Start (mm)	End (mm)
Yield	6.59	-
IO	6.59	10.35
LS	10.35	27.34
CP	27.34	108.89
Collapse	108.89	142.99

FEMA 356 procedure yield the target displacement as 19.09 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.13%. Plastic hinges resist moments and change their limit to life safety level. Therefore the structure is expected to have major damages and cannot be used anymore.

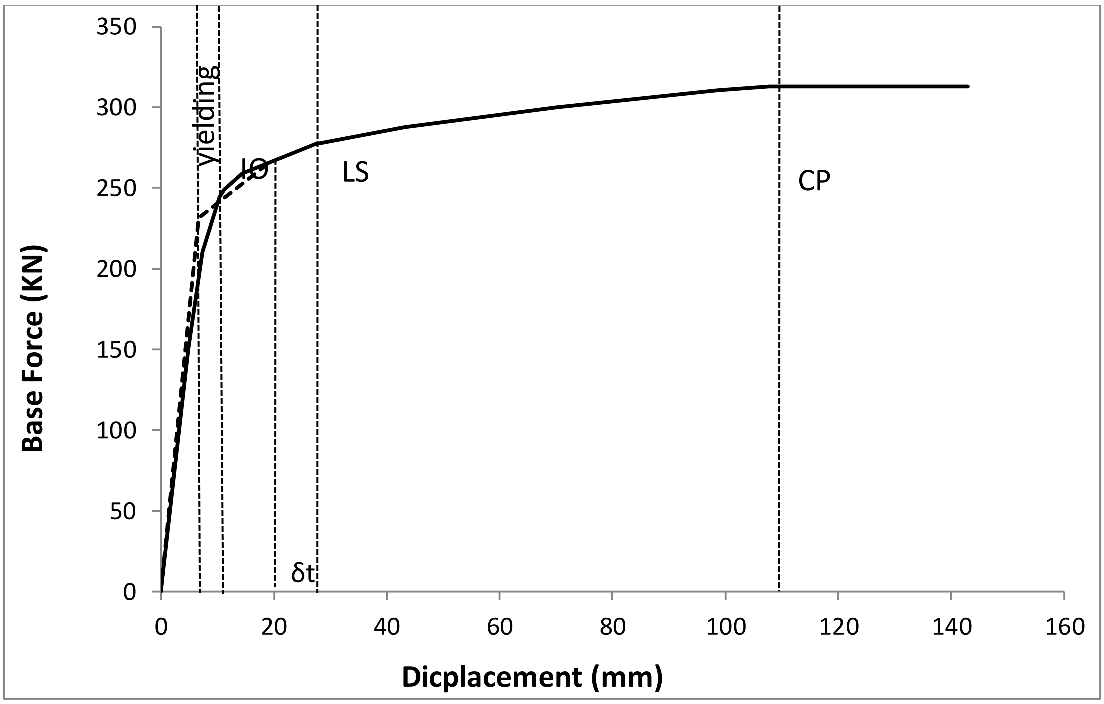


Figure 5.24: Performance Level for Frame Model With Triangular Lateral Loads (10% Corroded)

5.1.2.4 (15%) Corroded

For 15% corroded case of building three different frame models were identified which was 1st mode lateral load, frame model with uniform lateral loads and frame model with triangular lateral loads. Results obtained from pushover analysis steps were discussed in sections below.

5.1.2.4.1 Frame Model With 1st Mode Lateral Load

Table 5.19: Pushover steps for frame model with 1st mode lateral load -15% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.26	175.899	0.03
2	6.28	244.839	0.04
3	7.97	274.044	0.05
4	9.94	292.401	0.07
5	18.26	319.984	0.12
6	25.15	334.218	0.17
7	40.02	349.766	0.27
8	44.85	354.449	0.29
9	54.05	360.164	0.36
10	70.23	368.238	0.47
11	79.43	367.431	0.53
12	91.99	366.243	0.61

Results obtained from pushover analysis show that the building start to yield at 6.18 mm displacement. Table 5.19 shows pushover analysis steps for 1st mode lateral load. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 4. The building has 9.94 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 25.15 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for tow storey collapse at step 10 where displacement is equal to 70.23 mm. Table 5.20 shows performance level of the structure. Figure 5.25 shows structure's pushover curve with performance level.

Table 5.20: Performance level for frame model with 1st mode lateral load -15% corroded

Performance level	Start (mm)	End (mm)
Yield	6.18	-
IO	6.18	9.94
LS	9.94	25.15
CP	25.15	70.23
Collapse	70.23	91.99

FEMA 356 procedure yield the target displacement as 14.961 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.099%. Plastic hinges resist moments and change their limit to life safety level. Therefore the structure is expected to have major damages and cannot be used anymore.

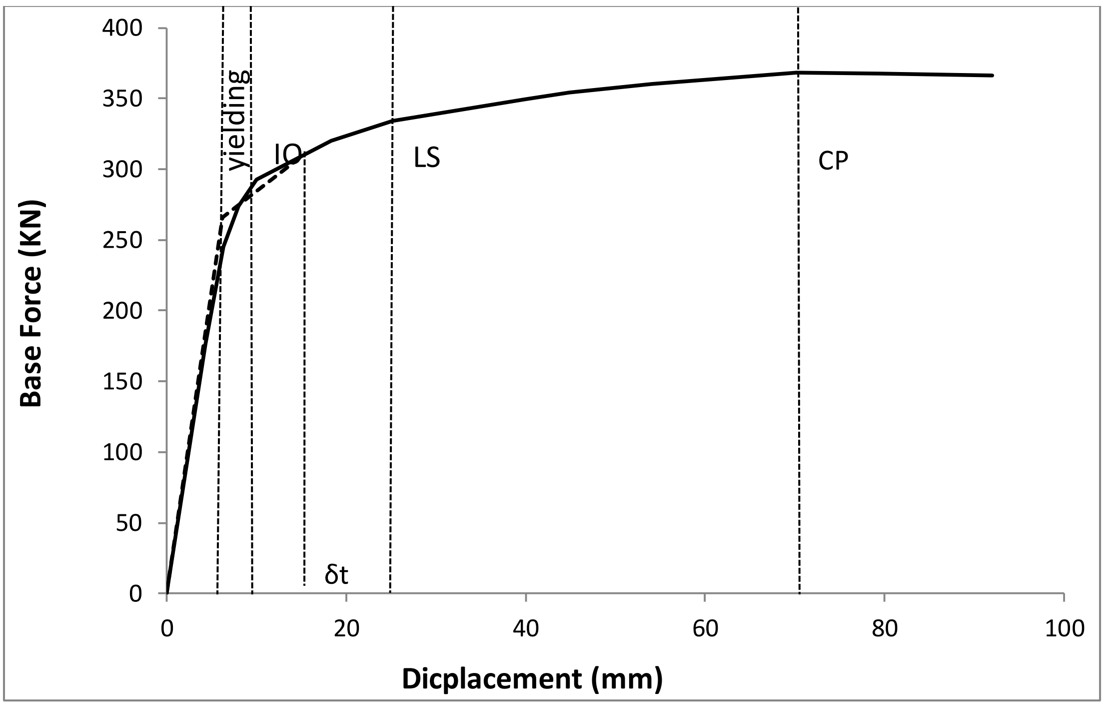


Figure 5.25: Performance Level for Frame Model With 1st Mode Lateral Load (15% Corroded)

5.1.2.4.2 Frame Model With Uniform Lateral Loads

Table 5.21: Pushover steps for frame model with uniform lateral loads-15% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.39	165.047	0.03
2	6.51	230.904	0.04
3	8.02	255.157	0.05
4	10.14	274.42	0.07
5	17.07	295.804	0.11
6	26.53	313.797	0.18
7	42.84	328.689	0.29
8	45.22	330.839	0.30
9	58.22	338.324	0.39
10	62.09	340.553	0.41
11	71.52	344.096	0.48
12	84.52	343.019	0.56
13	97.52	341.942	0.65
14	110.52	340.716	0.74
15	123.52	339.492	0.82
16	129.99	338.881	0.87

Results obtained from pushover analysis show that the building start to yield at 5.71 mm displacement. Table 5.21 shows pushover analysis steps for frame model with uniform lateral loads. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 6. The building has 26.53 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 58.22 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for tow storey collapse at step 11 where displacement is equal to 71.52 mm. Table 5.22 shows performance level of the structure. Figure 5.26 shows structure's pushover curve with performance level.

Table 5.22: Performance level for frame model with uniform lateral loads-15% corroded

Performance level	Start (mm)	End (mm)
Yield	5.71	-
IO	5.71	26.53
LS	26.53	58.22
CP	58.22	71.52
Collapse	71.52	129.99

FEMA 356 procedure yield the target displacement as 16.151 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.11%. Plastic hinges resist moments but they are still in immediate occupancy level. Therefore the building under uniform lateral loads expected to have low damages and remain safe to use.

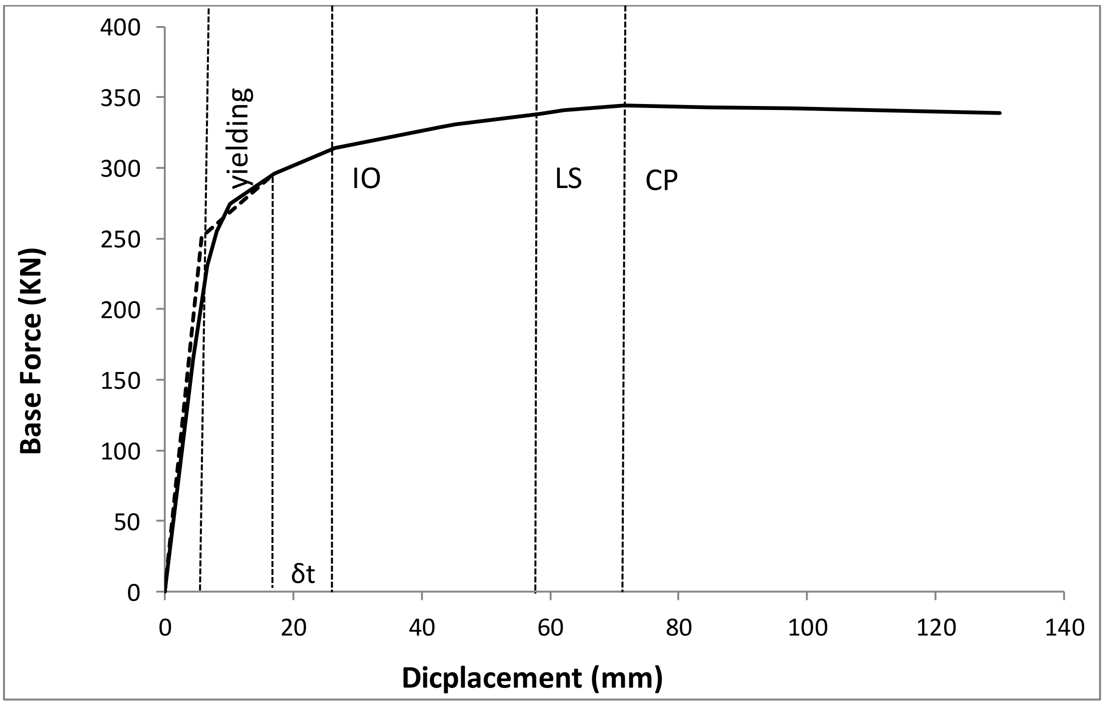


Figure 5.26: Performance Level for Frame Model With Uniform Lateral Loads (15% Corroded)

5.1.2.4.3 Frame Model With Triangular Lateral Loads

Table 5.23: Pushover steps for frame model with triangular lateral loads-15% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.85	149.761	0.03
2	7.33	210.06	0.05
3	10.48	243.537	0.07
4	11.24	247.716	0.07
5	13.61	254.518	0.09
6	25.36	272.104	0.17
7	35.91	280.21	0.24
8	52.22	288.402	0.35
9	57.18	290.876	0.38
10	71.18	295.369	0.47
11	94.26	301.471	0.63
12	113.19	300.853	0.75
13	119.13	300.655	0.79
14	133.13	299.726	0.89
15	139.99	299.27	0.93

Results obtained from pushover analysis show that the building start to yield at 7.24 mm displacement. Table 5.23 shows pushover analysis steps for frame model with triangular lateral loads. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 5. The building has 13.61 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 52.22 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for tow storey collapse at step 11 where displacement is equal to 94.26 mm. Table 5.24 shows performance level of the structure. Figure 5.27 shows structure's pushover curve with performance level.

Table 5.24: Performance level for frame model with triangular lateral loads-15% corroded

Performance level	Start (mm)	End (mm)
Yield	7.24	-
IO	7.24	13.61
LS	13.61	52.22
CP	52.22	94.26
Collapse	94.26	139.99

FEMA 356 procedure yield the target displacement as 19.086 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.13%. Plastic hinges resist moments and change their limit to life safety level. Therefore the structure is expected to have major damages and cannot be used anymore.

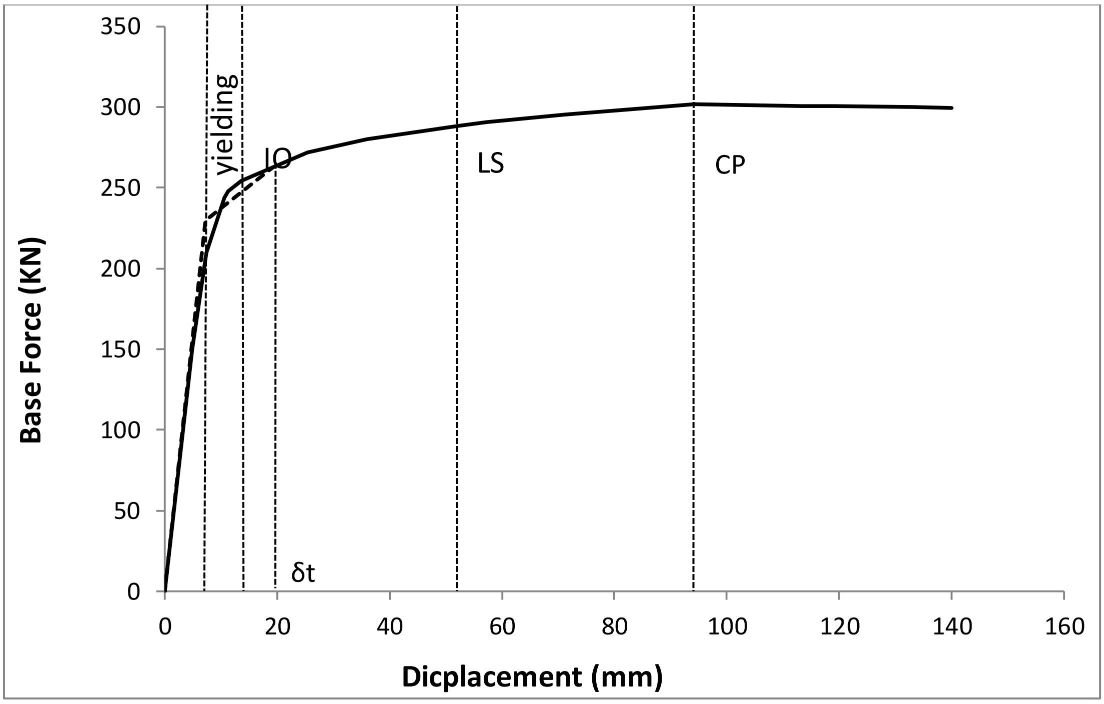


Figure 5.27: Performance Level for Frame Model With Triangular Lateral Loads (15% Corroded)

5.1.2.5 (20%) Corroded

For 20% corroded case of building three different frame models were identified which was 1st mode lateral load, frame model with uniform lateral loads and frame model with triangular lateral loads. Results obtained from pushover analysis steps were discussed in sections below.

5.1.2.5.1 Frame Model With 1st Mode Lateral Load

Table 5.25: Pushover steps for frame model with 1st mode lateral load -20% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.26	175.896	0.03
2	6.27	244.305	0.04
3	7.82	269.647	0.05
4	9.22	283.127	0.06
5	18.68	317.214	0.12
6	25.69	331.755	0.17
7	44.74	351.79	0.29
8	52.23	356.419	0.35
9	53.44	356.437	0.36
10	73.44	353.609	0.49
11	93.44	350.576	0.62
12	113.44	347.544	0.76
13	138.32	342.59	0.92
14	152.33	339.666	1.02

Results obtained from pushover analysis show that the building start to yield at 5.79 mm displacement. Table 5.25 shows pushover analysis steps for 1st mode lateral load. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 4. The building has 9.21 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 25.69 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for three storey collapse at step 9 where displacement is equal to 53.44 mm. Table 5.26 shows performance level of the structure. Figure 5.28 shows structure's pushover curve with performance level.

Table 5.26: Performance level for frame with 1st mode lateral load -20% corroded

Performance level	Start (mm)	End (mm)
Yield	5.79	-
IO	5.79	9.21
LS	9.21	25.69
CP	25.69	53.44
Collapse	53.44	152.33

FEMA 356 procedure yield the target displacement as 14.956 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.099%. Plastic hinges resist moments and change their limit to life safety level. Therefore the structure is expected to have major damages and cannot be used anymore.

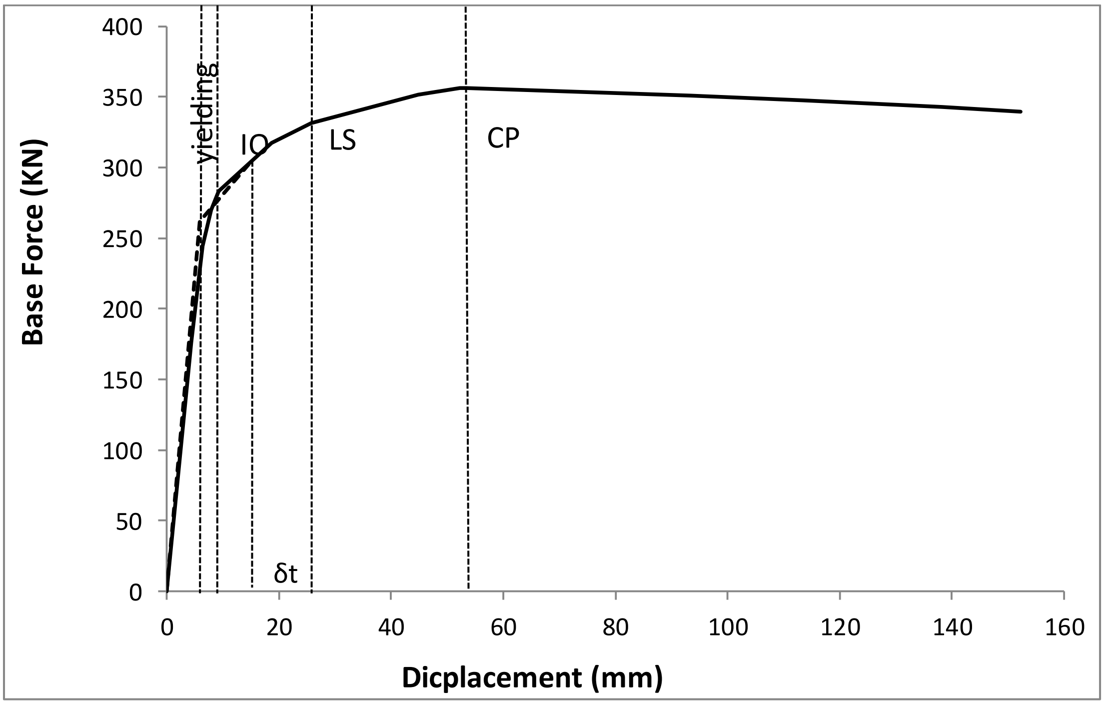


Figure 5.28: Performance Level for Frame Model With 1st Mode Lateral Load (20% Corroded)

5.1.2.5.2 Frame Model With Uniform Lateral Loads

Table 5.27: Pushover steps for frame model with uniform lateral loads-20% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.39	165.047	0.03
2	6.50	230.479	0.04
3	8.29	256.115	0.06
4	9.43	266.027	0.06
5	12.92	280.877	0.09
6	25.92	310.082	0.17
7	44.77	328.157	0.29
8	49.09	330.675	0.33
9	49.09	327.643	0.33
10	49.26	328.255	0.33
11	50.27	329.27	0.34
12	53.81	331.291	0.36
13	54.51	331.298	0.36
14	79.05	328.049	0.53
15	113.63	323.203	0.76
16	134.73	320.1	0.89
17	154.73	316.159	1.03
18	171.85	312.789	1.15
19	191.85	306.709	1.28
20	199.99	304.232	1.33

Results obtained from pushover analysis show that the building start to yield at 5.68 mm displacement. Table 5.27 shows pushover analysis steps for frame model with uniform lateral loads. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 5. The building has 12.92 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 49.09 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for three storey collapse at step 13 where displacement is equal to 54.51 mm. Table 5.28 shows performance level of the structure. Figure 5.29 shows structure's pushover curve with performance level.

Table 5.28: Performance level for frame with uniform lateral loads-20% corroded

Performance level	Start (mm)	End (mm)
Yield	5.68	-
IO	5.68	12.92
LS	12.92	49.09
CP	49.09	54.51
Collapse	54.51	199.99

FEMA 356 procedure yield the target displacement as 16.157 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.11%. Plastic hinges resist moments and change their limit to life safety level. Therefore the structure is expected to have major damages and cannot be used anymore.

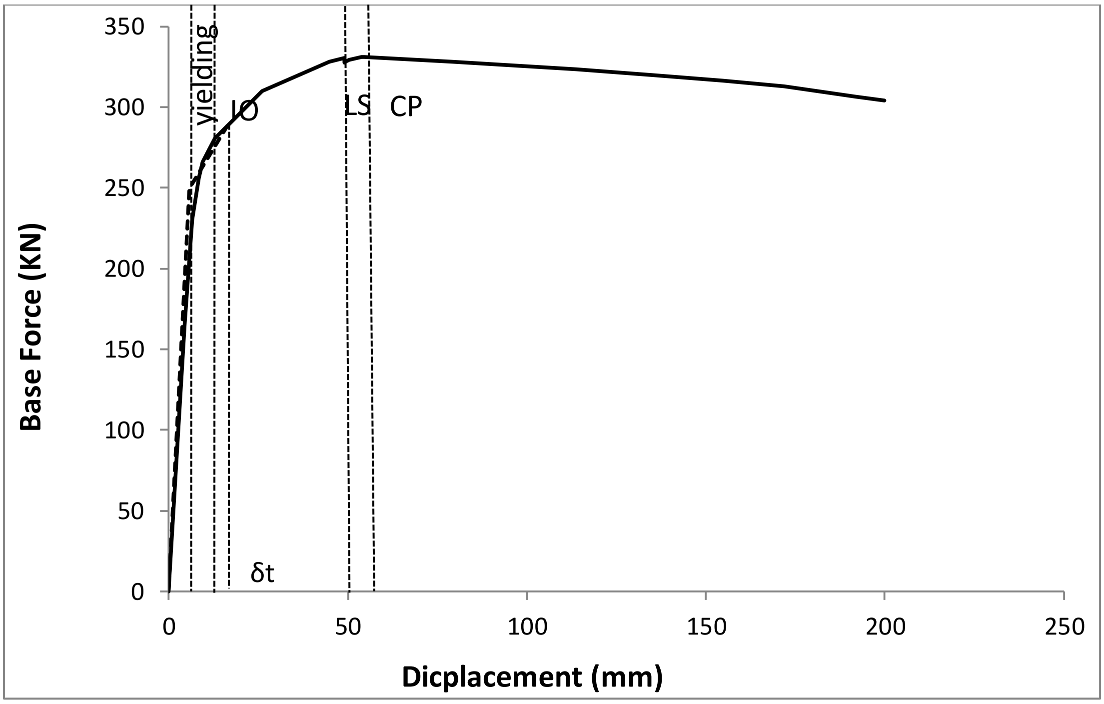


Figure 5.29: Performance Level for Frame Model With Uniform Lateral Loads (20% Corroded)

5.1.2.5.3 Frame Model With Triangular Lateral Loads

Table 5.29: Pushover steps for frame model with triangular lateral loads-20% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.85	149.763	0.03
2	7.32	209.618	0.05
3	10.57	242.249	0.07
4	12.39	248.381	0.08
5	22.33	264.938	0.15
6	30.89	273.924	0.21
7	43.06	281.008	0.29
8	50.56	284.752	0.34
9	57.42	288.127	0.38
10	64.92	290.404	0.43
11	69.63	291.835	0.46
12	70.76	291.844	0.47
13	74.99	291.535	0.49

Results obtained from pushover analysis show that the building start to yield at 6.98 mm displacement. Table 5.29 shows pushover analysis steps for frame model with triangular lateral loads. Plastic hinges for ground columns and interior beams for two storey change their level to immediate occupancy at step 4. The building has 12.39 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 57.42 mm and performance level change to life safety. All ground columns, two interior beams and two exterior beams for three storey collapse at step 9 where displacement is equal to 70.76 mm. Table 5.30 performance level of the structure. Figure 5.30 shows structure's pushover curve with performance level.

Table 5.30: Performance level for frame model with triangular lateral loads-20% corroded

Performance level	Start (mm)	End (mm)
Yield	6.98	-
IO	6.98	12.39
LS	12.39	57.42
CP	57.42	70.76
Collapse	70.76	74.99

FEMA 356 procedure yield the target displacement as 19.068 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.13%. Plastic hinges resist moments and change their limit to life safety level. Therefore the structure is expected to have major damages and cannot be used anymore.

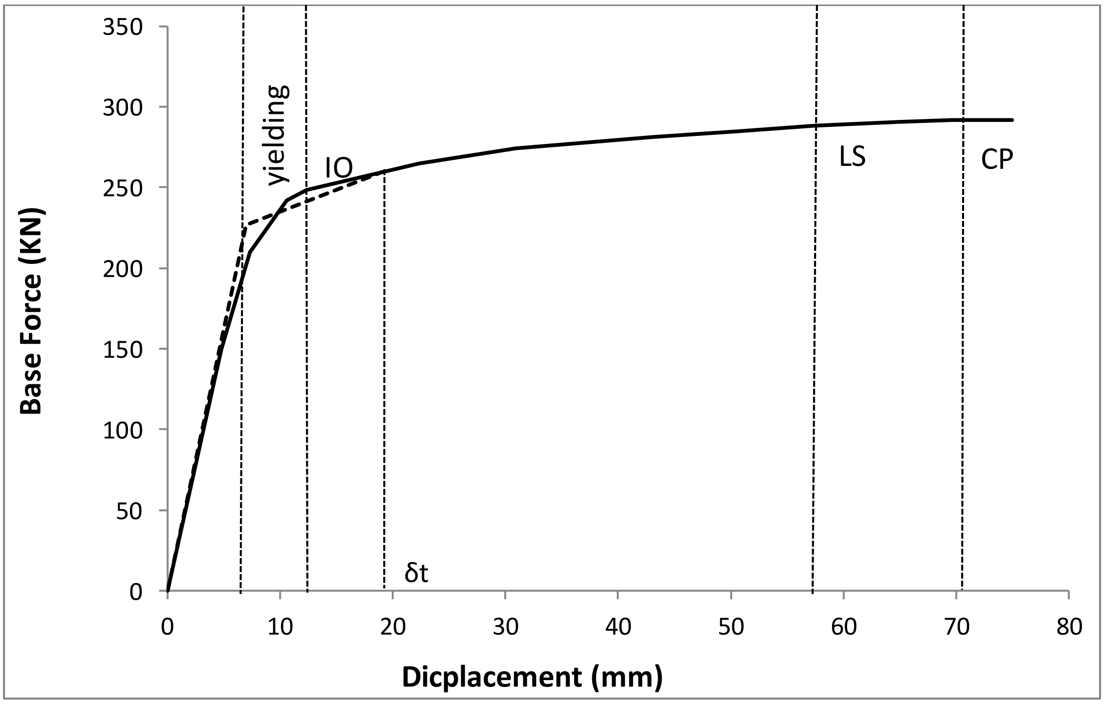


Figure 5.30: Performance Level for Frame Model With Triangular Lateral Loads (20% Corroded)

5.2 Pushover Analysis Results According to Eurocode8

Nonlinear static pushover analysis has been performed on two dimensional frame models of buildings. As these constructions possess corrosion problems, nonlinear static analysis was given alongside approximated 5%, 10%, 15%, 20% disparate corrosion case materialized in columns of the ground floor. Results acquired from the analysis have been debated in sections below.

5.2.1 Capacity Curves And Performance Limits

Force-displacement relationships of two dimensional frame model of buildings for corroded and non corroded cases are shown in pursuing sections.Eurocode8 parameters to find target displacement are shown in Appendix F.

5.2.1.1 Non-Corroded Case

For non-corroded case of building three disparate frame models were defined that was 1^{st} mode lateral load, frame model with uniform lateral loads and frame model with triangular lateral loads. Results obtained from pushover analysis steps were discussed in sections below.

5.2.1.1.1 Frame Model With 1st Mode Lateral Load

Table 5.31: Pushover steps for frame model with 1st mode lateral load -non corrode case

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.26	175.892	0.03
2	7.50	273.302	0.05
3	11.16	326.437	0.07
4	14.72	347.985	0.09
5	22.15	368.781	0.15
6	77.55	425.768	0.52
7	109.74	450.271	0.73
8	115.88	451.531	0.77
9	131.97	452.559	0.88
10	136.25	452.48	0.91
11	160.49	449.901	1.07
12	233.19	435.181	1.55
13	240.79	433.378	1.61
14	274.24	420.051	1.83
15	315.29	395.055	2.10
16	360.41	363.131	2.40
17	373.50	353.416	2.49
18	373.50	353.416	2.49
19	373.50	353.416	2.49
20	378.31	350.176	2.52

The state of damage in a structure is defined in EN 1998-3, Eurocode8, by three limit states. Applying limit states of EN 1998-3 cannot be done automatically using SAP2000 program. Nevertheless some critical plastic hinges in columns and beams have been chosen and the rotation values calculated by the program at each deformation step will be used to compare it with the rotation values calculated by EN 1998-3 for each limit state. The roof displacement can be read with the force causing this displacement. Table 5.31 shows pushover analysis steps for non-corroded structure case for 1st mode lateral load. The results were obtained show that the building start to yield at 7.45 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 6. According to Eurocode8 at this level the structure has only slight damage. The structure has 77.55 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 109.74 mm and performance level change to significant damage (SD). At this level the structure is significantly damaged. Ground columns collapse at step 9 where displacement is equal to 131.97 mm. Table 5.32 shows performance level of the structure. Figure 5.31 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 11.4 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.08%. Plastic hinges change their limit to damage limitation. Therefore the structure is expected to have light damage and be safe to use.

Table 5.32: Performance level for frame model with 1st mode lateral load -non corroded case

Performance level	Start (mm)	End (mm)
Yield	7.45	-
DL	7.45	77.55
SD	77.55	109.74
NC	109.74	131.97
Collapse	131.97	378.31

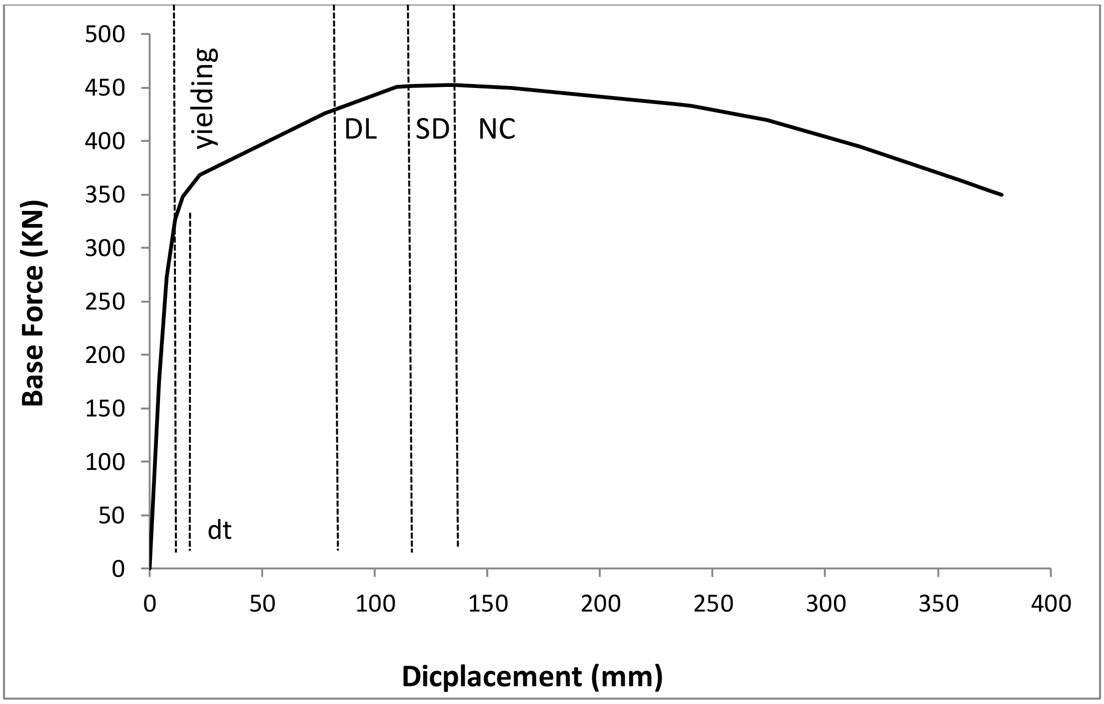


Figure 5.31: Capacity Curve for Frame Model With 1st Mode Lateral Load (Non-Corroded Case)

5.2.1.1.2 Frame Model With Uniform Lateral Loads

Table 5.33: Pushover steps for frame model with uniform lateral loads-non corroded case

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.39	165.045	0.03
2	7.57	253.522	0.05
3	11.22	303.083	0.07
4	12.08	310.494	0.08
5	17.28	333.237	0.12
6	22.59	344.861	0.15
7	43.42	369.981	0.29
8	99.47	413.556	0.66
9	113.61	421.555	0.76
10	115.29	421.871	0.77
11	135.61	423.047	0.90
12	140.77	422.948	0.94
13	180.77	418.895	1.21
14	184.16	418.551	1.23
15	244.35	406.51	1.63
16	248.37	405.61	1.66
17	281.96	393.281	1.88
18	326.05	367.932	2.17
19	367.26	340.804	2.45
20	381.93	330.565	2.55
21	381.93	330.565	2.55
22	381.93	330.565	2.55
23	395.24	322.099	2.63

Table 5.33 shows pushover analysis steps for non-corroded structure case with uniform lateral loads. The results were obtained show that the building start to yield at 7.45 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 7. The structure has 43.46 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 99.47 mm and performance level change to significant damage (SD). Ground columns collapse at step 11 where displacement is equal to 135.61 mm. Table 5.34 shows performance level of the structure. Figure 5.32 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 11.7 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.08%. Plastic hinges change their limit to damage limitation (DL). Therefore the structure is expected to have light damage and be safe to use.

Table 5.34: Performance level for frame model with uniform lateral loads-non corroded case

Performance level	Start (mm)	End (mm)
Yield	7.45	-
DL	7.45	43.46
SD	43.46	99.47
NC	99.47	135.61
Collapse	135.61	395.24

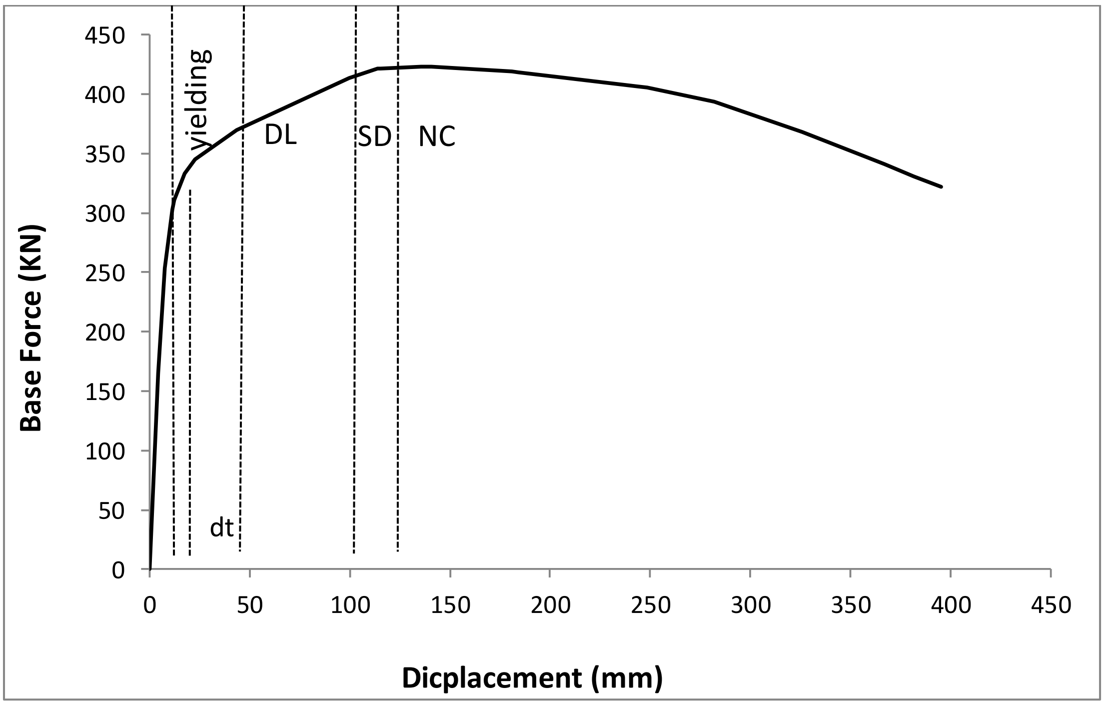


Figure 5.32: Capacity Curve for Frame Model With Uniform Lateral Loads (Non-Corroded Case)

5.2.1.1.3 Frame Model With Triangular Lateral Loads

Table 5.35: Pushover steps for frame model with triangular lateral loads-non corroded case

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.84	149.752	0.03
2	7.34	210.492	0.05
3	10.79	251.106	0.07
4	16.93	283.14	0.11
5	21.83	297.368	0.15
6	24.15	301.483	0.16
7	64.52	329.021	0.43
8	116.43	357.502	0.78
9	152.29	370.529	1.02
10	177.65	371.896	1.18
11	183.74	371.856	1.22
12	245.52	367.118	1.64
13	285.52	360.807	1.90
14	298.79	358.712	1.99
15	303.89	357.443	2.03

Table 5.35 shows pushover analysis steps for non-corroded structure case with triangular lateral loads. The results were obtained show that the building start to yield at 10.21 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 7. The structure has 64.52 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 116.43 mm and performance level change to significant damage (SD). Ground columns collapse at step 10 where displacement is equal to 177.65 mm Table 5.36 shows performance level of the structure. Figure 5.33 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 17.1 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.11%.

Plastic hinges change their limit to damage limitation (DL). Therefore the structure is expected to have light damage and be safe to use.

Table 5.36: Performance level for frame with triangular lateral loads-non corroded case

Performance level	Start (mm)	End (mm)
Yield	10.21	-
DL	10.21	64.52
SD	64.52	116.43
NC	116.43	177.65
Collapse	177.65	303.89

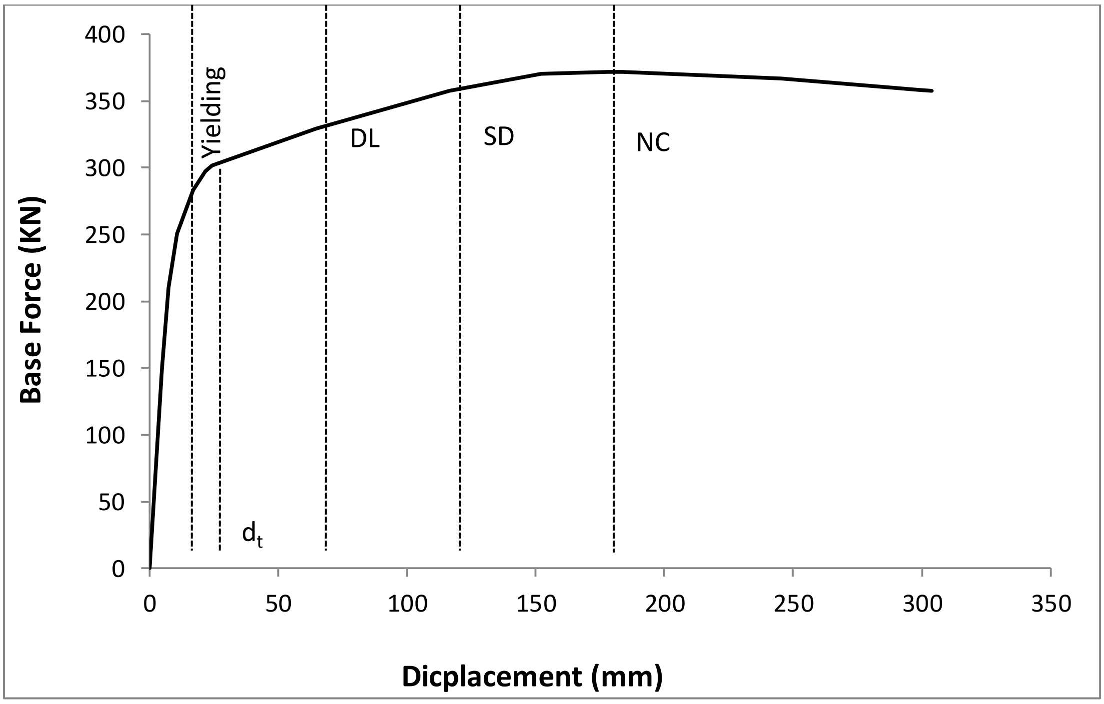


Figure 5.33: Capacity Curve for Frame Model With Triangular Lateral Loads (Non-Corroded Case)

5.2.1.2 (5%) Corroded

For 5% corroded case of building three disparate frame models were defined that was 1st mode lateral load, frame model with uniform lateral loads and frame model with triangular lateral loads. Results obtained from pushover analysis steps were discussed in sections below.

5.2.1.2.1 Frame Model With 1st Mode Lateral Load

Table 5.37: Pushover steps for frame model with 1st mode lateral load -5% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.26	175.892	0.03
2	6.50	251.115	0.04
3	8.33	288.212	0.06
4	10.02	307.321	0.07
5	20.93	353.948	0.14
6	39.16	392.773	0.26
7	63.33	419.642	0.42
8	97.72	442.146	0.65
9	98.56	442.209	0.66
10	108.62	441.676	0.72
11	113.08	440.304	0.75

Table 5.37 shows pushover analysis steps for 5% corroded structure case for 1st mode lateral load. The results were obtained show that the building start to yield at 5.89 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 5. The structure has 20.93 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 63.33 mm and performance level change to significant damage (SD). Ground columns collapse at step 9 where displacement is equal to 98.56 mm Table 5.38 shows performance level of the structure. Figure 5.34 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 11.2 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.07%.

Plastic hinges change their limit to damage limitation (DL). Therefore the structure is expected to have light damage and be safe to use.

Table 5.38: Performance level for frame model with 1st mode lateral load -5% corroded

Performance level	Start (mm)	End (mm)
Yield	5.89	-
DL	5.89	20.93
SD	20.93	63.33
NC	63.33	98.56
Collapse	98.56	113.08

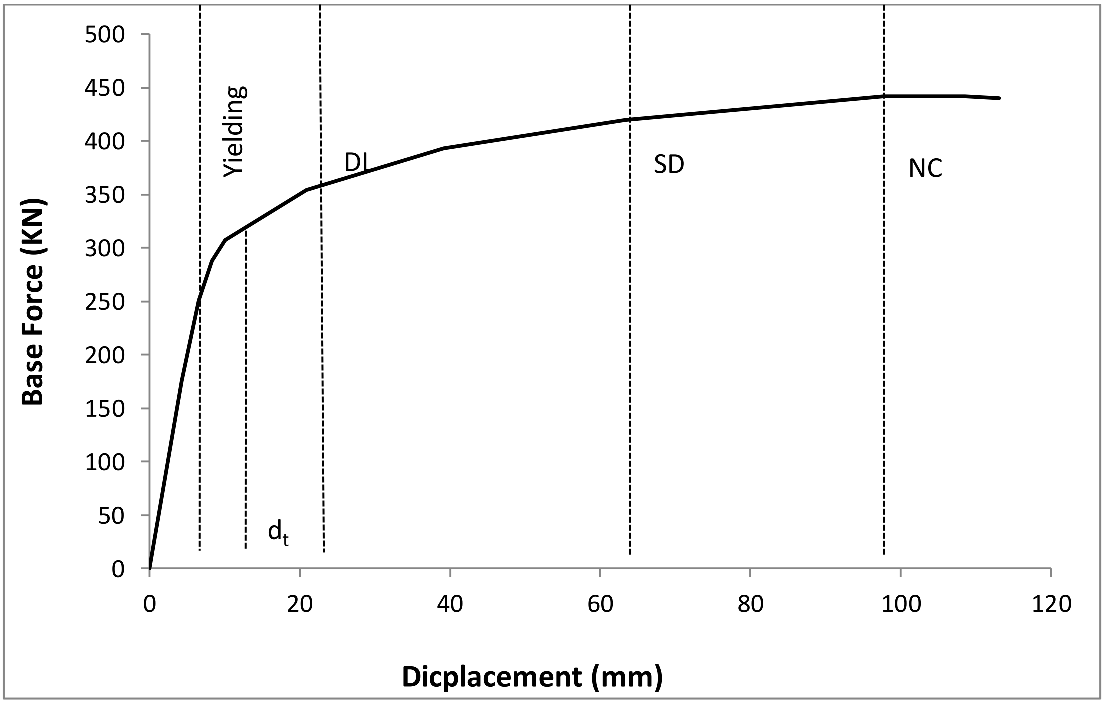


Figure 5.34: Capacity Curve for Frame Model With 1st Mode Lateral Load (5% Corroded)

5.2.1.2.2 Frame Model With Uniform Lateral Loads

Table 5.39: Pushover steps for frame model with uniform lateral loads-5% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.39	165.045	0.03
2	6.53	232.255	0.04
3	8.70	272.246	0.06
4	10.26	288.866	0.07
5	24.59	340.651	0.16
6	50.64	380.934	0.34
7	91.26	408.694	0.61
8	100.27	413.908	0.67
9	110.65	413.366	0.74
10	112.20	413.107	0.75
11	125.87	409.123	0.84

Table 5.39 shows pushover analysis steps for 5% corroded structure case with uniform lateral loads. The results were obtained show that the building start to yield at 6.14 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 5. The structure has 24.59 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 50.64 mm and performance level change to significant damage (SD). Ground columns collapse at step 8 where displacement is equal to 100.27 mm Table 5.40 shows performance level of the structure. Figure 5.35 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 11.4 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.08%. Plastic hinges change their limit to damage limitation (DL). Therefore the structure is expected to have light damage and be safe to use.

Table 5.40: Performance level for frame model with uniform lateral loads-5% corroded

Performance level	Start (mm)	End (mm)
Yield	6.14	-
DL	6.14	24.59
SD	24.59	50.64
NC	50.64	100.27
Collapse	100.27	125.87

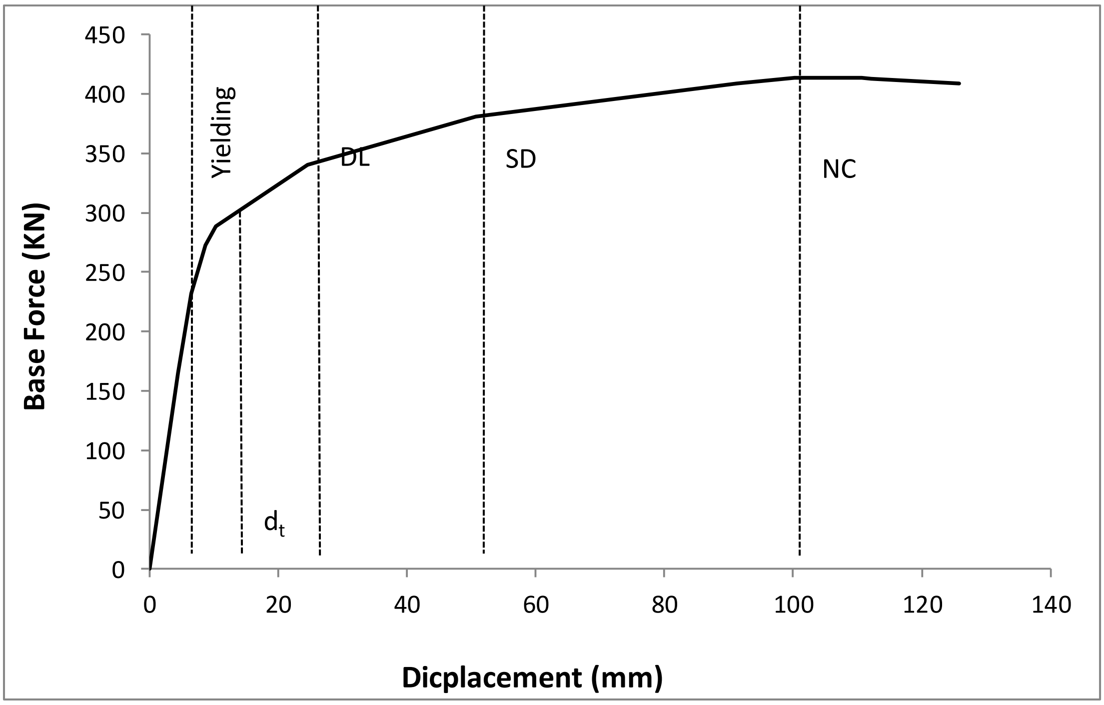


Figure 5.35: Capacity Curve for Frame Model With Uniform Lateral Loads (5% Corroded)

5.2.1.2.3 Frame Model With Triangular Lateral Loads

Table 5.41: Pushover steps for frame model with triangular lateral loads-5% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.84	149.752	0.03
2	7.34	210.465	0.05
3	10.99	255.118	0.07
4	15.09	274.232	0.10
5	27.88	303.527	0.19
6	48.25	328.437	0.32
7	69.52	341.389	0.46
8	69.52	341.389	0.46
9	69.52	337.158	0.46
10	69.71	338.43	0.46
11	69.90	338.879	0.47
12	92.74	351.617	0.62
13	92.74	351.618	0.62
14	92.74	351.601	0.62

Table 5.41 shows pushover analysis steps for 5% corroded structure case with triangular lateral loads. The results were obtained show that the building start to yield at 5.45 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 7. The structure has 69.51 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 69.90 mm and performance level change to significant damage (SD). Ground columns collapse at step 13 where displacement is equal to 92.74 mm Table 5.42 shows performance level of the structure. Figure 5.36 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 16.53 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.11%. Plastic hinges change their limit to damage limitation (DL). Therefore the structure is expected to have light damage and be safe to use.

Table 5.42: Performance level for frame model with triangular lateral loads-5% corroded

Performance level	Start (mm)	End (mm)
Yield	5.45	-
DL	5.45	69.51
SD	69.51	69.90
NC	69.90	92.74
Collapse	92.74	-

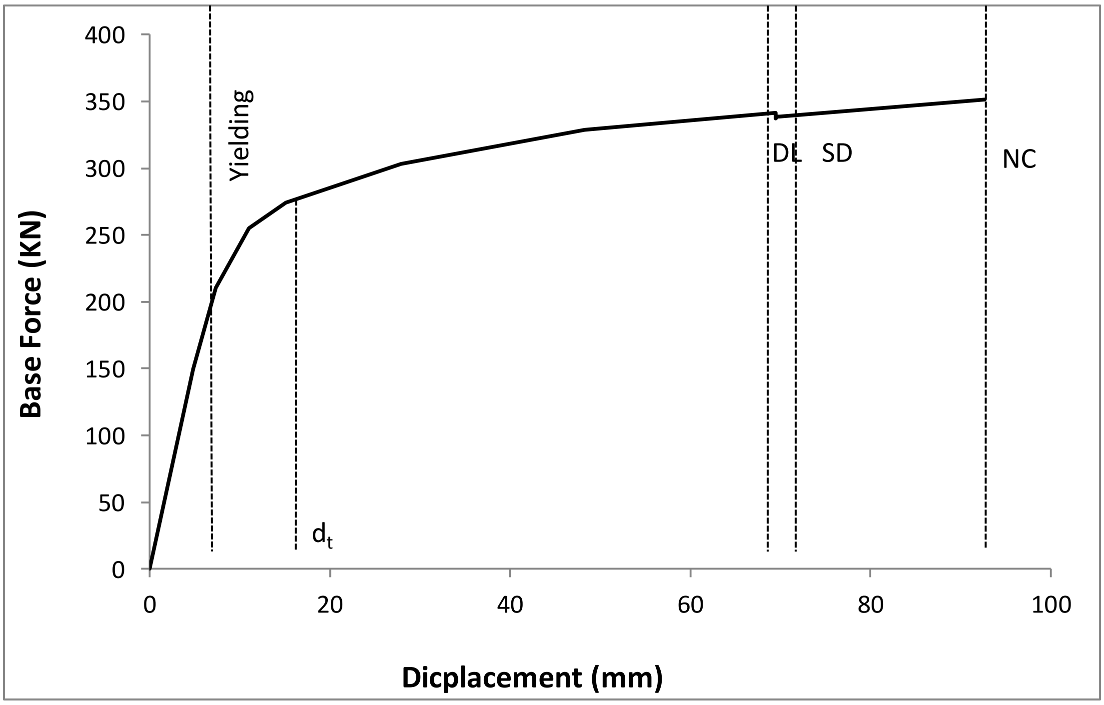


Figure 5.36: Capacity Curve for Frame Model With Triangular Lateral Loads (5% Corroded)

5.2.1.3 (10%) Corroded

For 10% corroded case of building three disparate frame models were defined that was 1st mode lateral load, frame model with uniform lateral loads and frame model with triangular lateral loads. Results obtained from pushover analysis steps were discussed in sections below.

5.2.1.3.1 Frame Model With 1st Mode Lateral Load

Table 5.43: Pushover steps for frame model with 1st mode lateral load -10% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.26	175.892	0.03
2	6.28	245.353	0.04
3	7.60	271.779	0.05
4	9.94	295.737	0.07
5	10.29	298.143	0.07
6	17.94	322.093	0.12
7	25.43	336.986	0.17
8	45.57	356.828	0.30
9	79.20	377.182	0.53
10	79.21	374.525	0.53
11	79.38	374.968	0.53
12	80.49	376.048	0.54
13	91.71	382.792	0.61
14	97.68	383.835	0.65
15	141.16	380.359	0.94
16	165.67	378.318	1.10
17	206.46	369.82	1.38
18	207.96	369.485	1.39
19	249.49	353.128	1.66
20	284.42	339.247	1.89

Table 5.43 shows pushover analysis steps for 10% corroded structure case for 1st mode lateral load. The results were obtained show that the building start to yield at 5.81 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 8. The structure has 45.57 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 79.21 mm and performance level change to significant damage (SD). Ground columns collapse at step 14 where displacement is equal to 97.68 mm Table 5.44 shows performance level of the structure. Figure 5.37 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 10.3 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.07%.

Plastic hinges change their limit to damage limitation (DL). Therefore the structure is expected to have light damage and be safe to use.

Table 5.44: Performance level for frame model with 1st mode lateral load -10% corroded

Performance level	Start (mm)	End (mm)
Yield	5.81	-
DL	5.81	45.57
SD	45.57	79.21
NC	79.21	97.68
Collapse	97.68	284.42

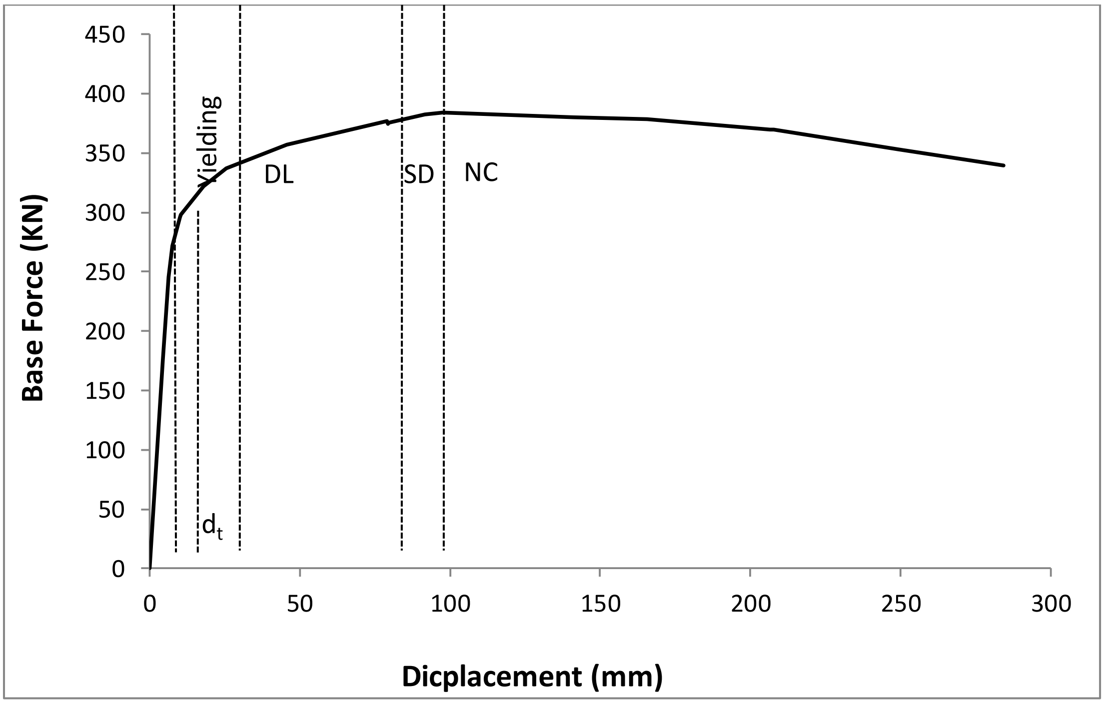


Figure 5.37: Capacity Curve for Frame Model With 1st Mode Lateral Load (10% Corroded)

5.2.1.3.2 Frame Model With Uniform Lateral Loads

Table 5.45: Pushover steps for frame model with uniform lateral loads-10% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.39	165.045	0.03
2	6.52	231.569	0.04
3	7.58	252.171	0.05
4	9.47	272.861	0.06
5	10.59	280.445	0.07
6	16.76	298.423	0.11
7	26.26	316.075	0.18
8	45.45	333.017	0.30
9	85.45	355.48	0.57
10	93.29	359.883	0.62
11	98.92	360.793	0.66
12	100.15	360.867	0.67
13	104.52	360.666	0.69
14	170.86	355.392	1.14
15	179.84	354.643	1.19
16	188.55	353.192	1.26

Table 5.45 shows pushover analysis steps for 10% corroded structure case with uniform lateral loads. The results were obtained show that the building start to yield at 5.03 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 7. The structure has 26.27 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 85.45 mm and performance level change to significant damage (SD). Ground columns collapse at step 12 where displacement is equal to 100.15 mm Table 5.46 shows performance level of the structure. Figure 5.38 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 10.3 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.07%. Plastic hinges change their limit to damage limitation (DL). Therefore the structure is expected to have light damage and be safe to use.

Table 5.46: Performance level for frame model with uniform lateral loads-10% corroded

Performance level	Start (mm)	End (mm)
Yield	5.03	-
DL	5.03	26.27
SD	26.27	85.45
NC	85.45	100.15
Collapse	100.15	188.55

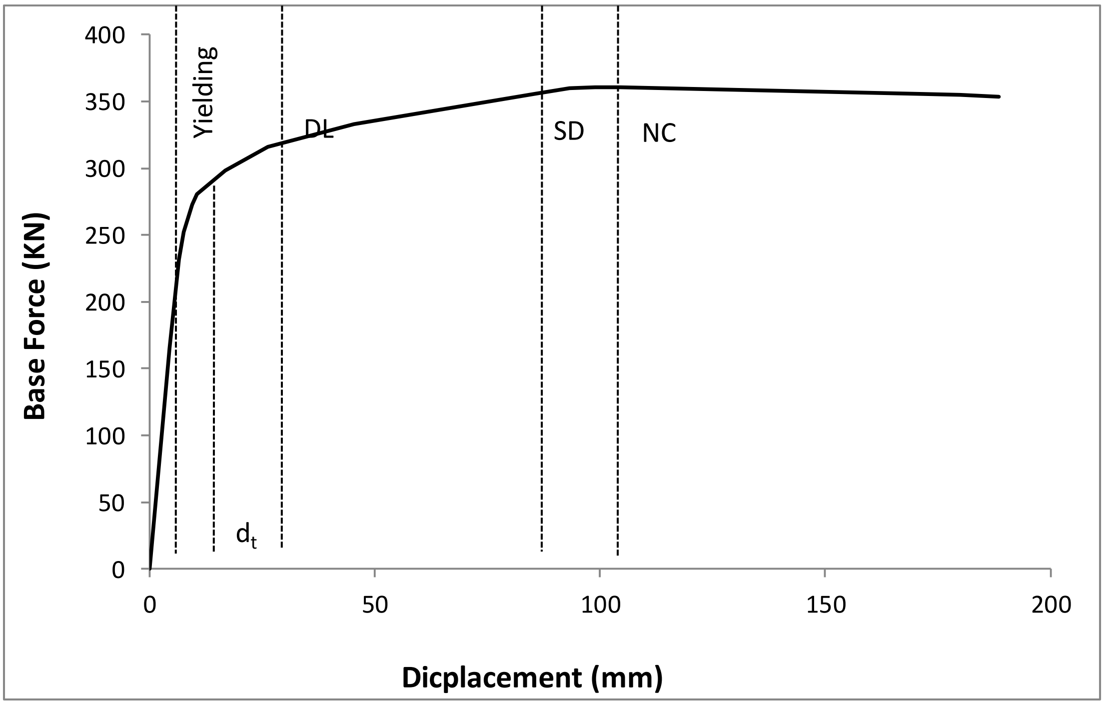


Figure 5.38: Capacity Curve for Frame Model With Uniform Lateral Loads (10% Corroded)

5.2.1.3.3 Frame Model With Triangular Lateral Loads

Table 5.47: Pushover steps for frame model with triangular lateral loads-10% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.85	149.761	0.03
2	7.34	210.383	0.05
3	10.35	244.204	0.07
4	11.14	248.824	0.07
5	14.32	258.583	0.09
6	27.44	276.291	0.18
7	45.15	286.744	0.30
8	57.46	292.889	0.38
9	71.76	297.602	0.48
10	86.06	302.315	0.57
11	99.12	306.619	0.66
12	99.13	300.105	0.66
13	99.29	301.993	0.66
14	99.55	302.98	0.66
15	101.83	306.165	0.68
16	101.83	306.154	0.68
17	103.25	306.831	0.69
18	117.55	311.255	0.78
19	123.89	313.22	0.83
20	131.12	313.96	0.87
21	132.55	314.025	0.88
22	142.99	313.873	0.95

Table 5.47 shows pushover analysis steps for 10% corroded structure case with triangular lateral loads. The results were obtained show that the building start to yield at 5.88 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 8. The structure has 57.46 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 99.12 mm and performance level change to significant damage (SD). Ground columns collapse at step 21 where displacement is equal to 132.55 mm Table 5.48 shows performance level of the structure. Figure 5.39 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 15.96 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.11%. Plastic hinges change their limit to damage limitation (DL). Therefore the structure is expected to have light damage and be safe to use.

Table 5.48: Performance level for frame model with triangular lateral loads-10% corroded

Performance level	Start (mm)	End (mm)
Yield	5.88	-
DL	5.88	57.46
SD	57.46	99.12
NC	99.12	132.55
Collapse	132.55	142.99

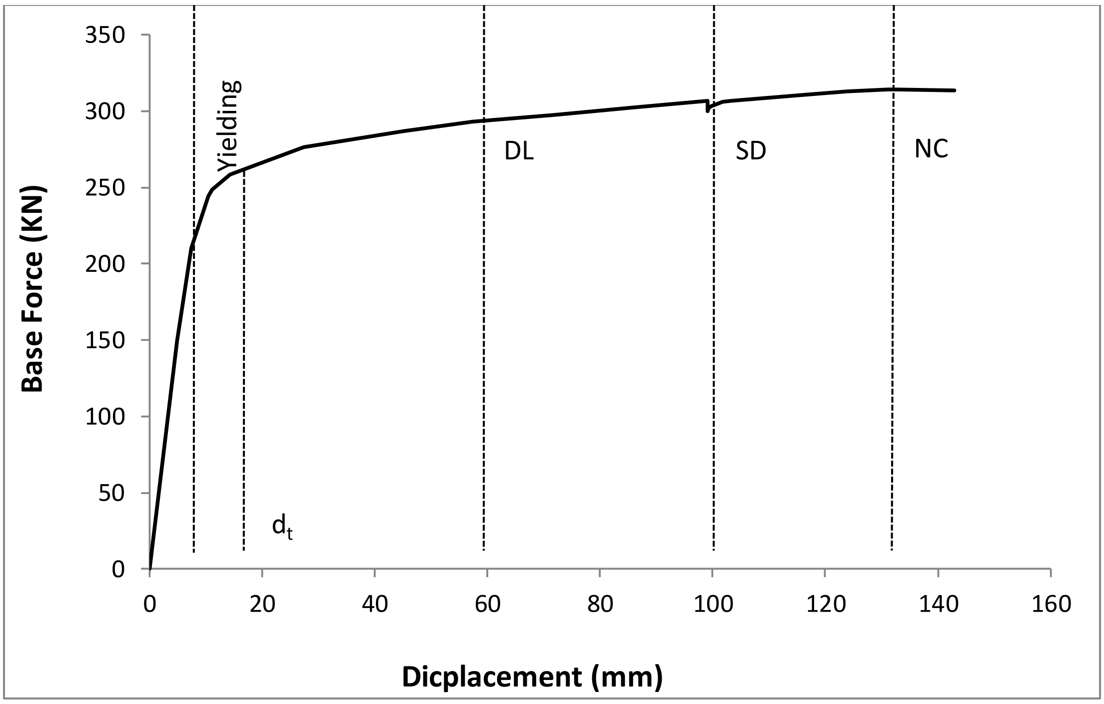


Figure 5.39: Capacity Curve for Frame Model With Triangular Lateral Loads (10% Corroded)

5.2.1.4 (15%) Corroded

For 15% corroded case of building three disparate frame models were defined that was 1st mode lateral load, frame model with uniform lateral loads and frame model with triangular lateral loads. Results obtained from pushover analysis steps were discussed in sections below.

5.2.1.4.1 Frame Model With 1st Mode Lateral Load

Table 5.49: Pushover steps for frame model with 1st mode lateral load -15% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.26	175.899	0.03
2	6.28	244.835	0.04
3	7.97	274.023	0.05
4	9.94	292.199	0.07
5	18.43	319.12	0.12
6	25.67	333.127	0.17
7	41.26	348.029	0.28
8	45.59	351.869	0.30
9	54.79	356.882	0.37
10	63.99	361.895	0.43
11	73.19	366.908	0.49
12	79.27	370.221	0.53
13	88.47	369.284	0.59

Table 5.49 shows pushover analysis steps for 15% corroded structure case for 1st mode lateral load. The results were obtained show that the building start to yield at 3.83 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 7. The structure has 41.26 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 54.79 mm and performance level change to significant damage (SD). Ground columns collapse at step 12 where displacement is equal to 79.27 mm Table 5.50 shows performance level of the structure. Figure 5.40 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 9.7 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.06%. Plastic hinges change their limit to damage limitation (DL). Therefore the structure is expected to have light damage and be safe to use.

Table 5.50: Performance level for frame model with 1st mode lateral load -15% corroded

Performance level	Start (mm)	End (mm)
Yield	3.83	-
DL	3.83	41.26
SD	41.26	54.79
NC	54.79	79.27
Collapse	79.27	88.47

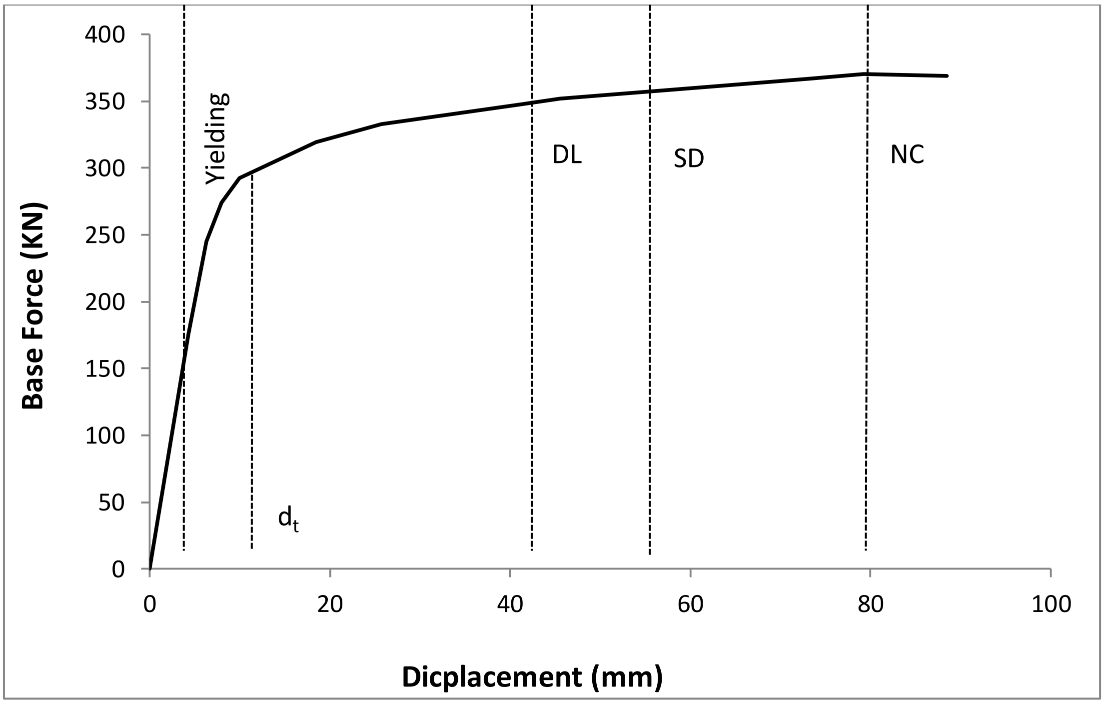


Figure 5.340: Capacity Curve for Frame Model With 1st Mode Lateral Load (15% Corroded)

5.2.1.4.2 Frame Model With Uniform Lateral Loads

Table 5.51: Pushover steps for frame with uniform lateral loads-15% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.39	165.047	0.03
2	6.51	230.9	0.04
3	8.02	255.138	0.05
4	10.12	274.117	0.07
5	17.23	295.186	0.11
6	26.87	312.497	0.18
7	43.63	326.508	0.29
8	45.72	328.228	0.30
9	51.33	331.066	0.34
10	51.33	328.673	0.34
11	51.49	329.395	0.34
12	52.75	330.578	0.35
13	52.75	329.266	0.35
14	52.91	329.756	0.35
15	53.87	330.654	0.36
16	72.11	339.924	0.48
17	80.71	344.255	0.54
18	93.71	342.995	0.62
19	117.16	340.567	0.78
20	129.99	339.169	0.87

Table 5.51 shows pushover analysis steps for 15% corroded structure case with uniform lateral loads. The results were obtained show that the building start to yield at 3.87 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 3. The structure has 8.02 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 45.72 mm and performance level change to significant damage (SD). Ground columns collapse at step 17 where displacement is equal to 80.71 mm Table 5.52 shows performance level of the structure. Figure 5.41 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 9.7 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.06%.

Plastic hinges change their limit to significant damage (SD). Therefore the structure is expected to have considerable damage and cannot be safe to use.

Table 5.52: Performance level for frame with uniform lateral loads-15% corroded

Performance level	Start (mm)	End (mm)
Yield	3.87	-
DL	3.87	8.02
SD	8.02	45.72
NC	45.72	80.71
Collapse	80.71	129.99

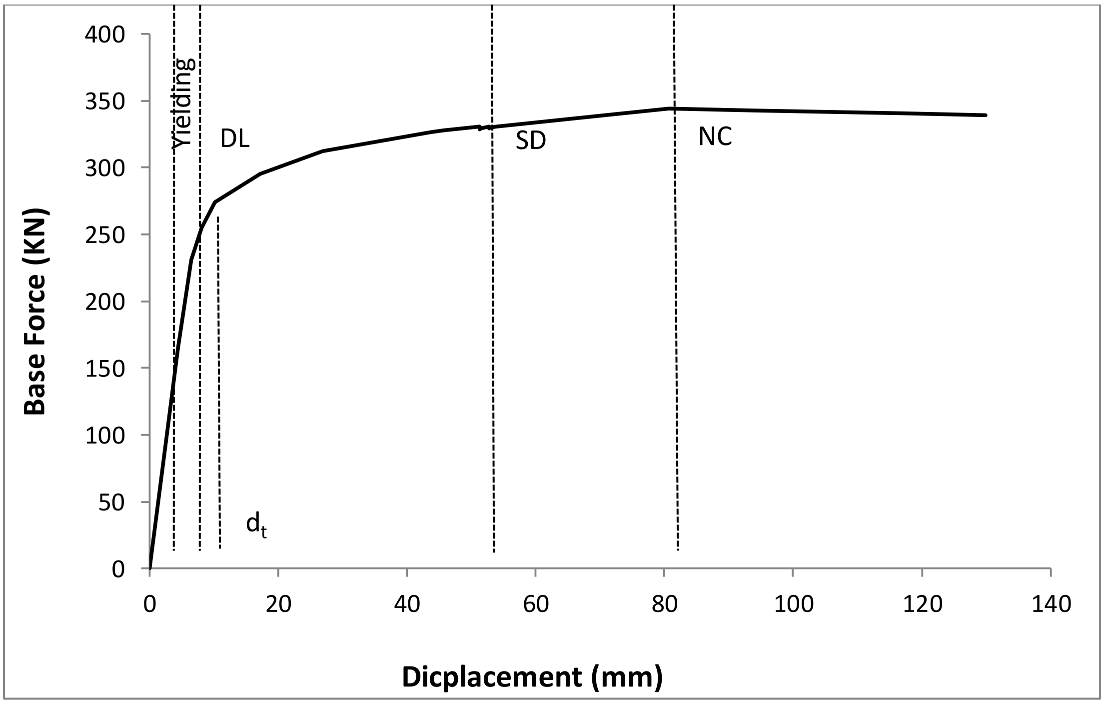


Figure 5.41: Capacity Curve for Frame Model With Uniform Lateral Loads (15% Corroded)

5.2.1.4.3 Frame Model With Triangular Lateral Loads

Table 5.53: Pushover steps for frame model with triangular lateral loads-15% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.85	149.761	0.03
2	7.33	210.059	0.05
3	10.48	243.514	0.07
4	11.24	247.654	0.07
5	13.57	254.271	0.09
6	25.45	271.224	0.17
7	36.63	279.203	0.24
8	54.03	287.189	0.36
9	58.64	289.29	0.39
10	72.64	293.292	0.48
11	86.64	297.294	0.58
12	100.64	301.295	0.67
13	106.34	302.923	0.71
14	129.46	302.064	0.86
15	139.99	301.27	0.93

Table 5.53 shows pushover analysis steps for 15% corroded structure case with triangular lateral loads. The results were obtained show that the building start to yield at 5.11 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 6. The structure has 25.45 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 58.64 mm and performance level change to significant damage (SD). Ground columns collapse at step 13 where displacement is equal to 106.34 mm Table 5.54 shows performance level of the structure. Figure 5.42 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 15.39 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.1%. Plastic hinges change their limit to damage limitation (DL). Therefore the structure is expected to have light damage and be safe to use.

Table 5.54: Performance level for frame model with triangular lateral loads-15% corroded

Performance level	Start (mm)	End (mm)
Yield	5.11	-
DL	5.11	25.45
SD	25.45	58.64
NC	58.64	106.34
Collapse	106.34	139.99

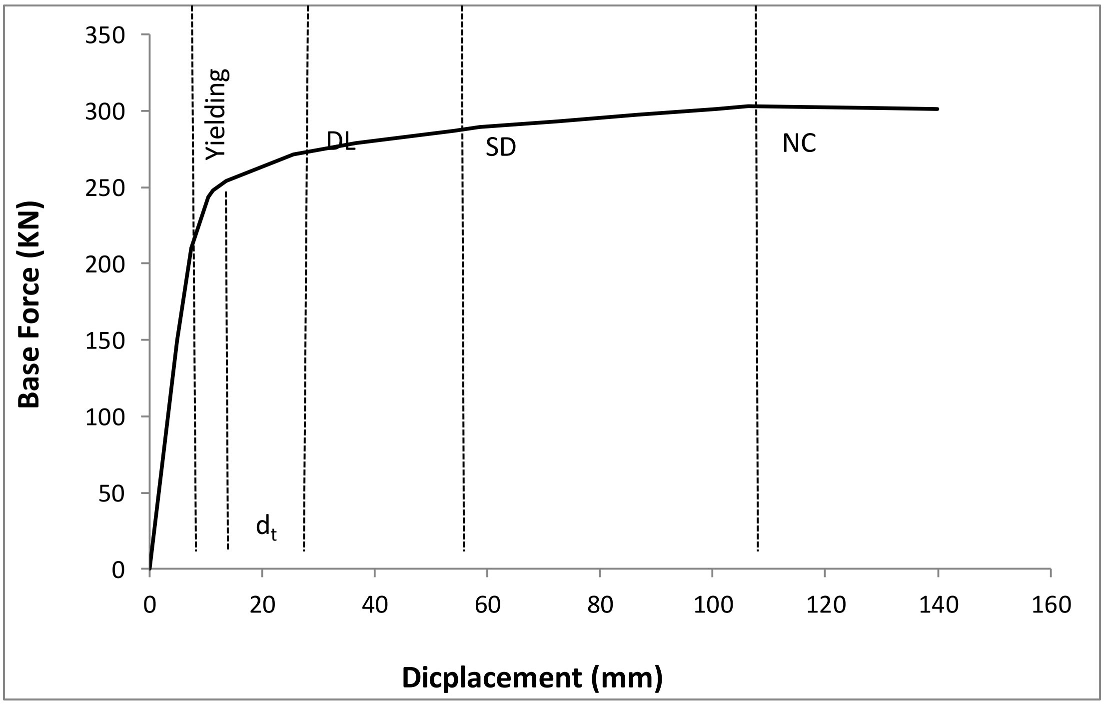


Figure 5.42: Capacity Curve for Frame Model With Triangular Lateral Loads (15% Corroded)

5.2.1.5 (20%) Corroded

For 20% corroded case of building three disparate frame models were defined that was 1st mode lateral load, frame model with uniform lateral loads and frame model with triangular lateral loads. Results obtained from pushover analysis steps were discussed in sections below.

5.2.1.5.1 Frame Model With 1st Mode Lateral Load

Table 5.55: Pushover steps for frame model with 1st mode lateral load -20% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.26	175.896	0.03
2	6.27	244.299	0.04
3	7.81	269.616	0.05
4	9.22	282.946	0.06
5	18.38	315.285	0.12
6	25.91	330.249	0.17
7	45.34	349.129	0.30
8	57.42	355.783	0.38
9	66.07	356.559	0.44
10	66.71	356.541	0.44
11	98.44	351.424	0.66
12	118.44	348.12	0.79
13	145.14	342.417	0.97
14	160.58	338.541	1.07
15	180.58	330.894	1.20
16	199.99	323.463	1.33

Table 5.55 shows pushover analysis steps for 20% corroded structure case for 1st mode lateral load. The results were obtained show that the building start to yield at 3.4 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 5. The structure has 18.38 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 45.34 mm and performance level change to significant damage (SD). Ground columns collapse at step 9 where displacement is equal to 66.07 mm Table 5.56 shows performance level of the structure. Figure 5.43 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 9.12 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.06%. Plastic hinges change their limit to damage limitation (DL). Therefore the structure is expected to have light damage and be safe to use.

Table 5.56: Performance level for frame model with 1st mode lateral load -20% corroded

Performance level	Start (mm)	End (mm)
Yield	3.4	-
DL	3.4	18.38
SD	18.38	45.34
NC	45.34	66.07
Collapse	66.07	199.99

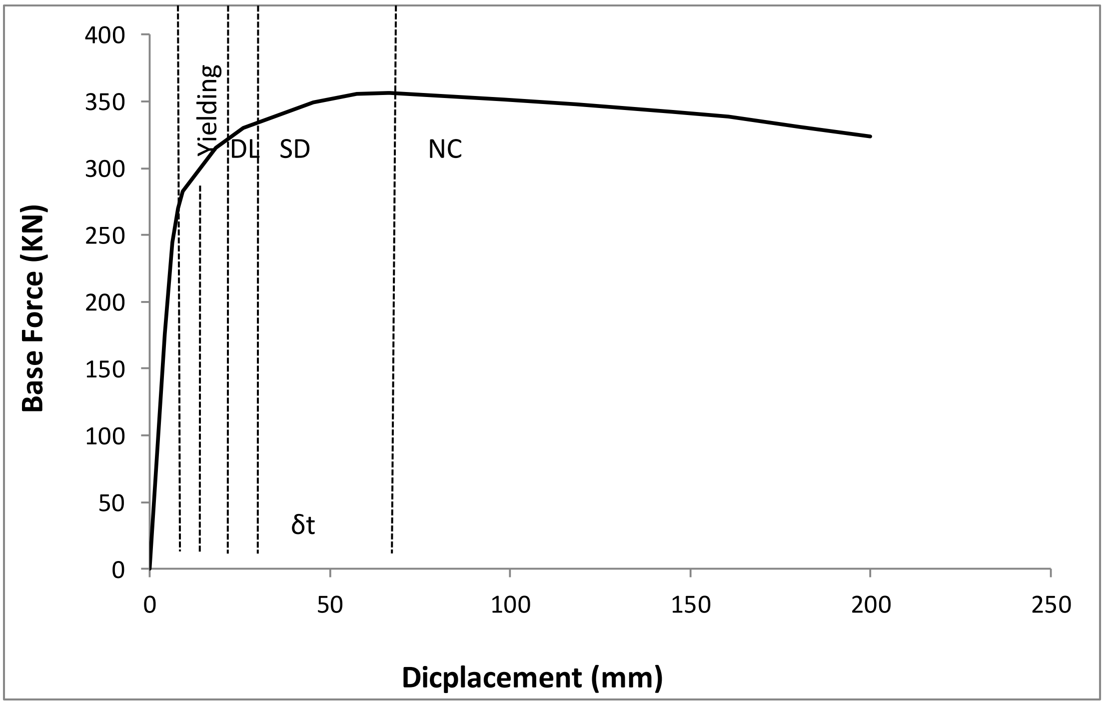


Figure 5.43: Capacity Curve for Frame Model With 1st Mode Lateral Load (20% Corroded)

5.2.1.5.2 Frame Model With Uniform Lateral Loads

Table 5.57: Pushover steps for frame model with uniform lateral loads-20% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.39	165.047	0.03
2	6.50	230.474	0.04
3	8.29	255.987	0.06
4	9.41	265.72	0.06
5	12.99	280.41	0.09
6	25.84	308.237	0.17
7	27.13	309.943	0.18
8	45.02	325.212	0.30
9	57.02	331.352	0.38
10	57.02	329.245	0.38
11	57.27	329.613	0.38
12	58.25	330.464	0.39
13	59.02	330.701	0.39
14	67.72	331.411	0.45
15	87.72	328.505	0.58
16	114.76	324.379	0.77
17	137.99	320.748	0.92
18	157.99	316.489	1.05
19	173.16	313.157	1.15

Table 5.57 shows pushover analysis steps for 20% corroded structure case with uniform lateral loads. The results were obtained show that the building start to yield at 3.21 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 3. The structure has 8.29 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 45.02 mm and performance level change to significant damage (SD). Ground columns collapse at step 14 where displacement is equal to 67.72 mm Table 5.58 shows performance level of the structure. Figure 5.44 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 9.12 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.06%.

Plastic hinges change their limit to significant damage (SD). Therefore the structure is expected to have considerable damage and cannot be safe to use.

Table 5.58: Performance level for frame model with uniform lateral loads-20% corroded

Performance level	Start (mm)	End (mm)
Yield	3.21	-
DL	3.21	8.29
SD	8.29	45.02
NC	45.02	67.72
Collapse	67.72	173.16

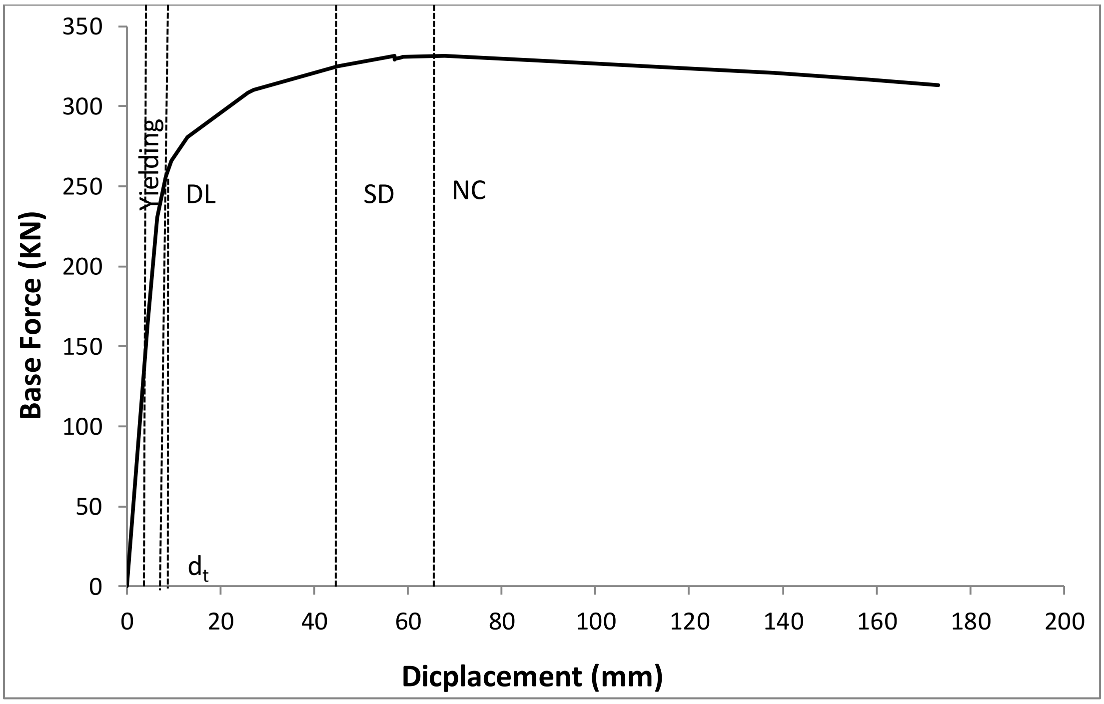


Figure 5.44: Capacity Curve for Frame Model With Uniform Lateral Loads (20% Corroded)

5.2.1.5.3 Frame Model With Triangular Lateral Loads

Table 5.59: Pushover steps for frame model with triangular lateral loads-20% corroded

Step	Displacement (mm)	Base Force (kN)	Roof drift (%)
1	4.85	149.763	0.03
2	7.32	209.616	0.05
3	10.58	242.203	0.07
4	12.37	248.161	0.08
5	19.48	260.197	0.13
6	27.82	270.272	0.19
7	37.38	276.73	0.25
8	44.38	279.938	0.29
9	51.38	283.135	0.34
10	58.89	286.556	0.39
11	65.89	288.47	0.44
12	69.99	289.582	0.47

Table 5.59 shows pushover analysis steps for 20% corroded structure case with triangular lateral loads. The results were obtained show that the building start to yield at 3.12 mm displacement plastic hinges for ground columns and beams of two storey change their level to damage limitation (DL) at step 4. The structure has 12.37 mm displacement at this level, then the column hold to raise this level to the next step where displacement equal to 44.38 mm and performance level change to significant damage (SD). Ground columns collapse at step 12 where displacement is equal to 69.99 mm Table 5.60 shows performance level of the structure. Figure 5.45 shows structure's pushover curve with performance level.

Eurocode8 procedure yield the target displacement as 14.82 mm which indicated that the structure yield. The calculated roof drift ratio at that displacement is 0.099%. Plastic hinges change their limit to significant damage (SD). Therefore the structure is expected to have considerable damage and cannot be safe to use.

Table 5.60: Performance level for frame model with triangular lateral loads-20% corroded

Performance level	Start (mm)	End (mm)
Yield	3.21	-
DL	3.21	12.37
SD	12.37	44.38
NC	44.38	69.99
Collapse	69.99	-

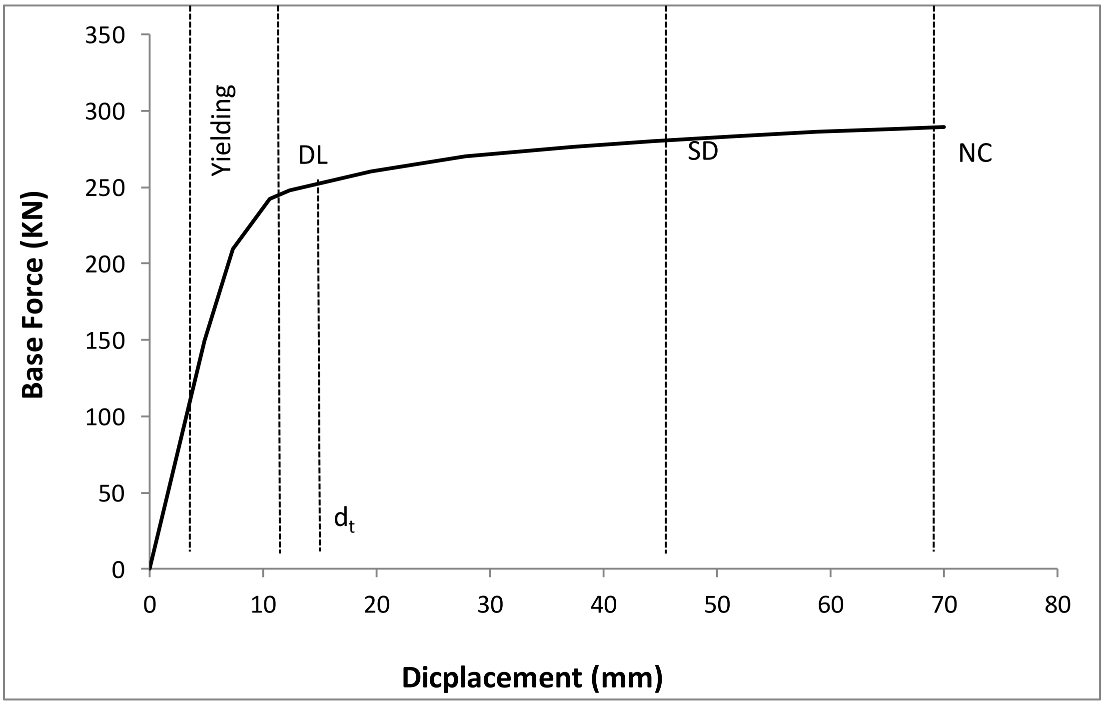


Figure 5.45: Capacity Curve for Frame Model With Triangular Lateral Loads (20% Corroded)

5.3 Comparison of Results Between Both Codes

In this section, a comparison of results between two codes, FEMA356 and Eurocode8 were done for the level of corrosion used in this research.

5.3.1 Eurocode8 Results

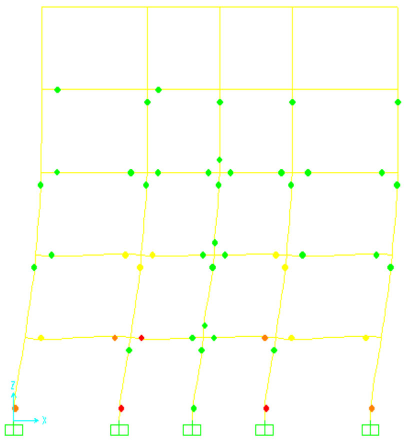
Table 5.61: Performance of buildings according to Eurocode8 limit

Case	**Load type**	**Eurocode8 limit**	**V_y/W**	**$V_{collapse}$ /W**
Non-corroded	1^{st} mode lateral load	DL	54%	91%
	uniform pattern	DL	52%	85%
	triangular pattern	DL	46%	75%
5% corroded	1^{st} mode lateral load	DL	56%	89%
	uniform pattern	DL	53%	84%
	triangular pattern	DL	47%	71%
10% corroded	1^{st} mode lateral load	DL	54%	77%
	uniform pattern	DL	52%	73%
	triangular pattern	DL	47%	63%
15% corroded	1^{st} mode lateral load	DL	54%	75%
	uniform pattern	SD	51%	69%
	triangular pattern	DL	46%	61%
20% corroded	1^{st} mode lateral load	DL	53%	72%
	uniform pattern	SD	50%	67%
	triangular pattern	SD	46%	58%

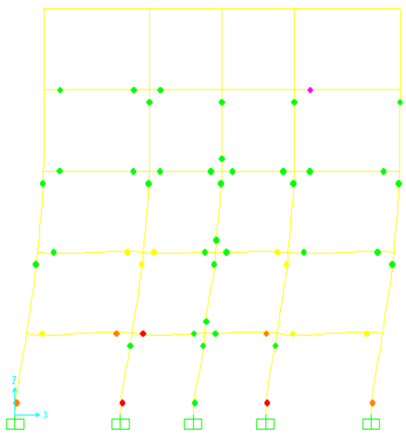
5.3.1.1 Non-Corroded Case

Figure 5.46 shows deformed shape and plastic hinge generation for non-corroded case of building, where pink color represent the yielding level, blue color represent DL level, turquoise color represent SD level, green color represent NC level, yellow color represent collapse level, orange and red color represent points after collapse as described in chapter 3.

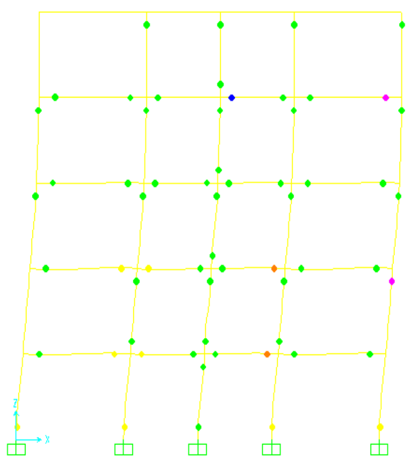
Results show that the building start to yield when base shear reaches 54, 52 and 46 percent of structure's weight for the three types of loads as shown in Table 5.61. First column collapse that column in the first floor then all ground columns collapse. The construction display good confrontation as yielding till collapse of columns. Also it displays good flexible confrontation to loading. Load type was effect on plastic hinge mechanism where triangular lateral load model shows more roof displacement than others. Choosing appropriate strengthens methods for weak columns will help structure to improve its ductility during earthquake.



(a) 1st Mode Lateral Load



(b) Uniform Lateral Loads

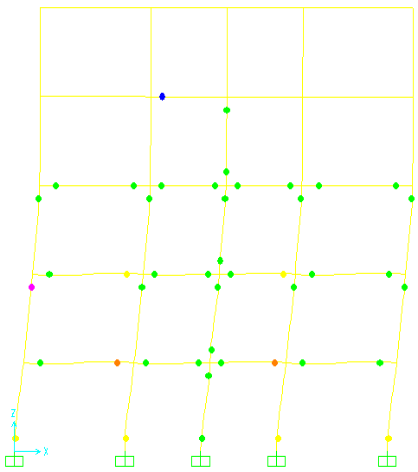


(c) Triangular Lateral Loads

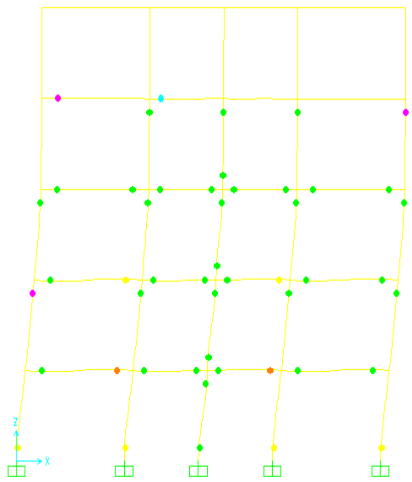
Figure 5.46: Deformed Shape and Plastic Hinge Generation (Non-Corroded Case)

5.3.1.2 (5%) Corroded

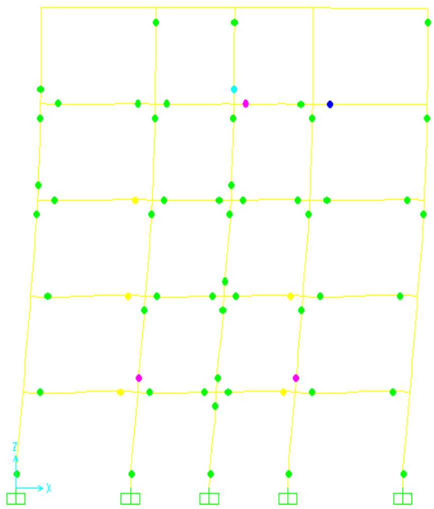
Figure 5.47 shows deformed shape and plastic hinge generation for 5% corroded case of building. Results show that the building start to yield when base shear reaches 89, 84 and 71 percent of structure's weight for the three types of loads as shown in Table 5.61. First column collapse that column in the first floor then all ground columns collapse. The construction display good confrontation as yielding till collapse of columns. Also it displays good flexible confrontation to loading. Load type was effect on plastic hinge mechanism where triangular lateral load model shows more roof displacement than others. Choosing appropriate strengthens methods for weak columns will help structure to improve its ductility during earthquake.



(a) 1st Mode Lateral Load



(b) Uniform Lateral Loads

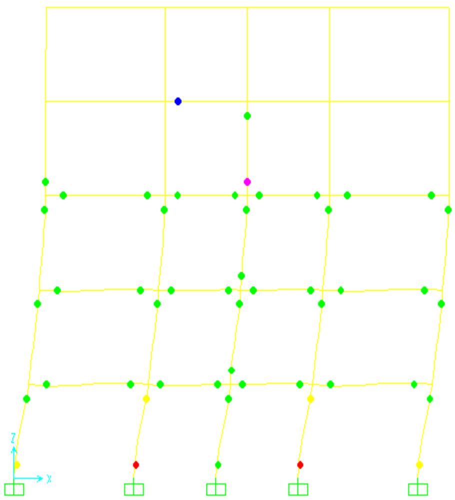


(c) Triangular Lateral Loads

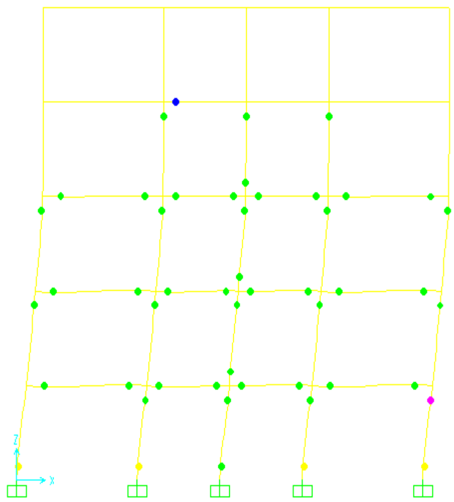
Figure 5.47: Deformed Shape and Plastic Hinge Generation (5% Corroded)

5.3.1.3 (10%) Corroded

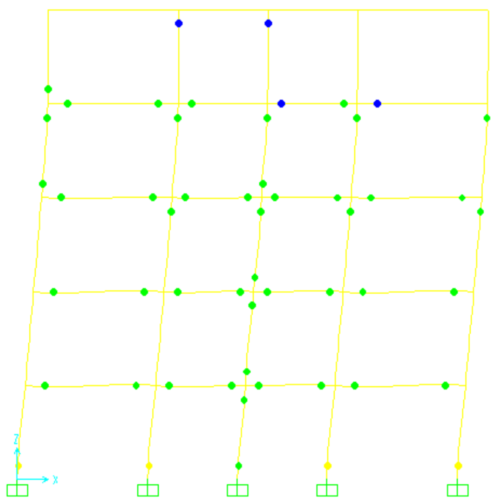
Figure 5.48 shows deformed shape and plastic hinge generation for 10% corroded case of building. Results show that the building start to yield when base shear reaches 77, 73 and 63 percent of structure's weight for the three types of loads as shown in Table 5.61. First column collapse that column in the first floor then all ground columns collapse. The construction display good confrontation as yielding till collapse of columns. Also it displays good flexible confrontation to loading. Load type was effect on plastic hinge mechanism where triangular lateral load model shows more roof displacement than others. Choosing appropriate strengthens methods for weak columns will help structure to improve its ductility during earthquake.



(a)1st Mode Lateral Load



(b) Uniform Lateral Loads

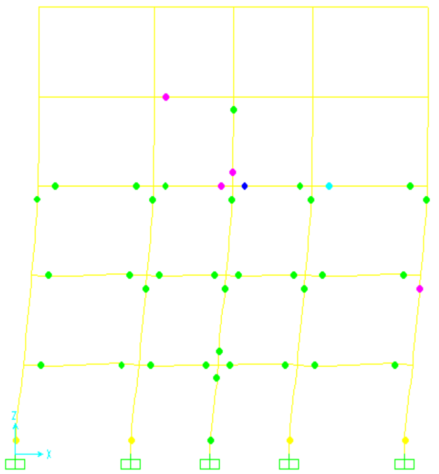


(c) Triangular Lateral Loads

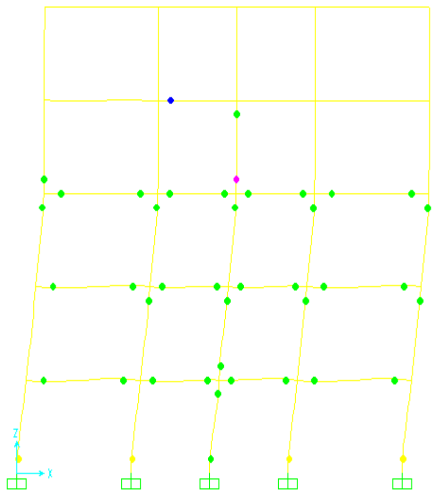
Figure 5.48: Deformed Shape and Plastic Hinge Generation (10% Corroded)

5.3.1.4 (15%) Corroded

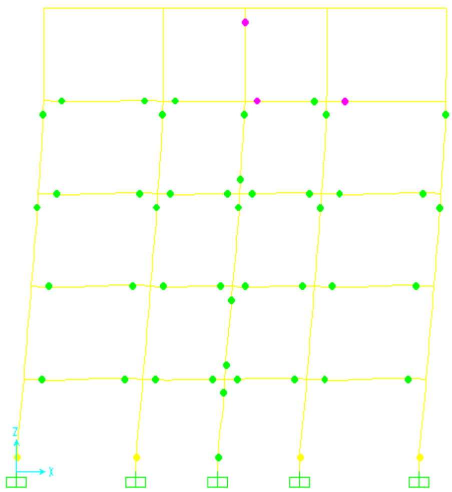
Figure 5.49 shows deformed shape and plastic hinge generation for 15% corroded case of building. Results show that the building start to yield when base shear reaches 75, 69 and 61 percent of structure's weight for the three types of loads as shown in Table 5.61. First column collapse that column in the first floor then all ground columns collapse. The construction display good confrontation as yielding till collapse of columns. Also it displays good flexible confrontation to loading. Load type was effect on plastic hinge mechanism where triangular lateral load model shows more roof displacement than others. Choosing appropriate strengthens methods for weak columns will help structure to improve its ductility during earthquake. Uniform lateral loads model have significant damage level, the building at this level is expected to have big damage and not be safe to use.



(a) 1st Mode Lateral Load



(b) Uniform Lateral Loads

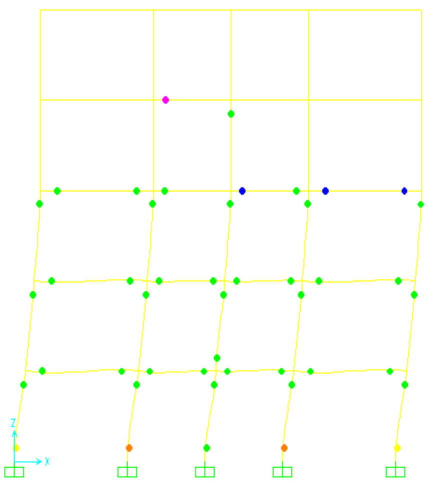


(c) Triangular Lateral Loads

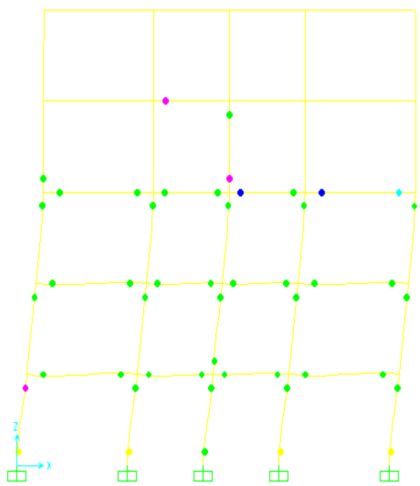
Figure 5.49: Deformed Shape and Plastic Hinge Generation (15% Corroded)

5.3.1.5 (20%) Corroded

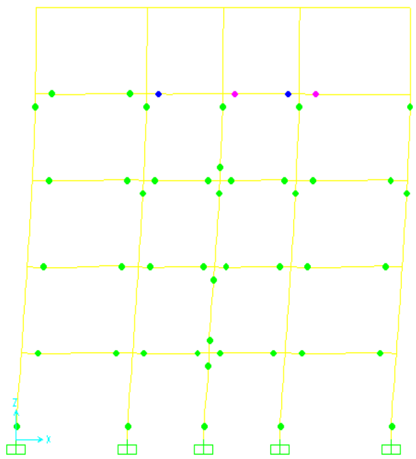
Figure 5.50 shows deformed shape and plastic hinge generation for 20% corroded case of building. Results show that the building start to yield when base shear reaches 72, 67 and 58 percent of structure's weight for the three types of loads as shown in Table 5.61. First column collapse that column in the first floor then all ground columns collapse. The construction display good confrontation as yielding till collapse of columns. Also it displays good flexible confrontation to loading. Load type was effect on plastic hinge mechanism where triangular lateral load model shows more roof displacement than others. Choosing appropriate strengthens methods for weak columns will help structure to improve its ductility during earthquake. Uniform and triangular lateral loads model have significant damage level, the building at this level is expected to have big damage and not be safe to use.



(a) 1st Mode Lateral Load



(b) Uniform Lateral Loads



(c) Triangular Lateral Loads

Figure 5.50: Deformed Shape and Plastic Hinge Generation (20% Corroded)

5.3.2 FEMA 356 Results

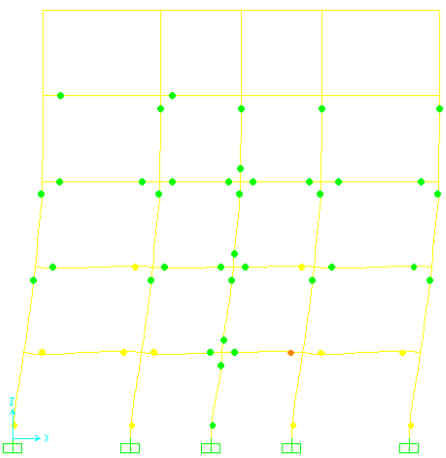
Table 5.62: Performance of buildings according to FEMA356 limit

Case	Load type	FEMA 356 limit	V_y/W	$V_{collapse}$ /W
Non-corroded	1st mode lateral load	LS	54%	91%
	uniform pattern	IO	52%	85%
	triangular pattern	IO	46%	75%
5% corroded	1st mode lateral load	LS	56%	89%
	uniform pattern	IO	53%	83%
	triangular pattern	LS	47%	73%
10% corroded	1st mode lateral load	LS	55%	77%
	uniform pattern	IO	52%	72%
	triangular pattern	LS	47%	63%
15% corroded	1st mode lateral load	LS	54%	74%
	uniform pattern	IO	51%	70%
	triangular pattern	LS	46%	61%
20% corroded	1st mode lateral load	LS	53%	72%
	uniform pattern	LS	50%	67%
	triangular pattern	LS	46%	59%

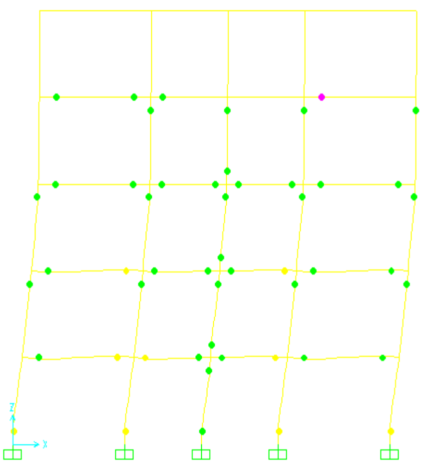
5.3.2.1 Non-Corroded Case

Figure 5.51 shows deformed shape and plastic hinge generation for non-corroded case of building, where pink color represent the yielding level, blue color represent IO level, turquoise color represent LS level, green color represent CP level, yellow color represent collapse level, orange and red color represent points D, E after collapse as described in chapter 2.

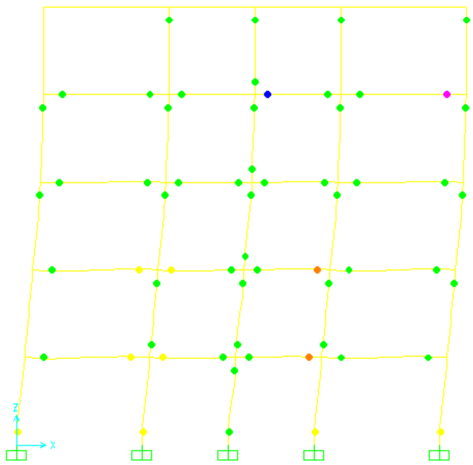
Results show that the building start to yield when base shear reaches 54, 52 and 46 percent of structure's weight for the three types of loads as shown in Table 5.62. First column collapse that column in the first floor then all ground columns collapse. The construction display good confrontation as yielding till collapse of columns. Also it displays good flexible confrontation to loading. Load type was effect on plastic hinge mechanism where triangular lateral load model shows more roof displacement than others. Choosing appropriate strengthens methods for weak columns will help structure to improve its ductility during earthquake. Building with 1st mode lateral load has life safety limit. According to this limit its expected that building has a big damage and cannot be safe to use.



(a)1st Mode Lateral Load



(b) Uniform Lateral Loads

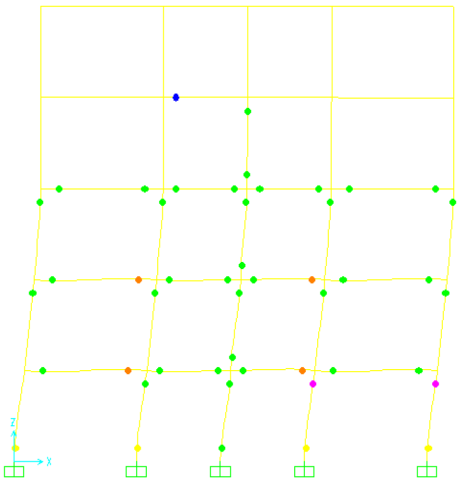


(c) Triangular Lateral Loads

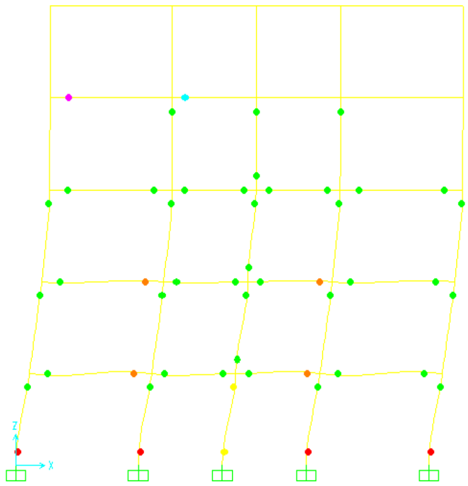
Figure 5.51: Deformed Shape and Plastic Hinge Generation (Non-Corroded Case)

5.3.2.2 (5%) Corroded

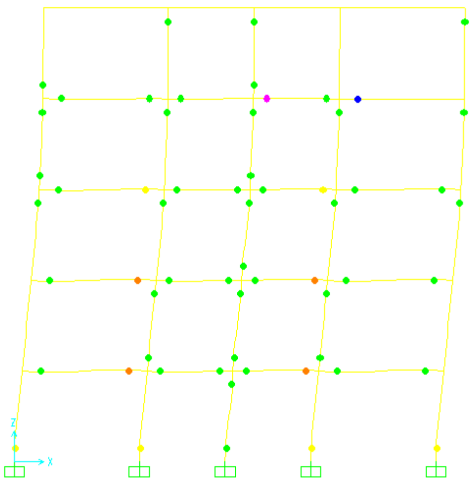
Figure 5.52 shows deformed shape and plastic hinge generation for 5% corroded case of building. Results show that the building start to yield when base shear reaches 56, 53 and 47 percent of structure's weight for the three types of loads as shown in Table 5.62. First column collapse that column in the first floor then all ground columns collapse. The construction display good confrontation as yielding till collapse of columns. Also it displays good flexible confrontation to loading. Load type was effect on plastic hinge mechanism where triangular lateral load model shows more roof displacement than others. Choosing appropriate strengthens methods for weak columns will help structure to improve its ductility during earthquake. Building with 1st mode lateral load and triangular lateral loads has life safety limit. According to this limit its expected that building has a big damage and cannot be safe to use.



(a) 1st Mode Lateral Load



(b) Uniform Lateral Loads

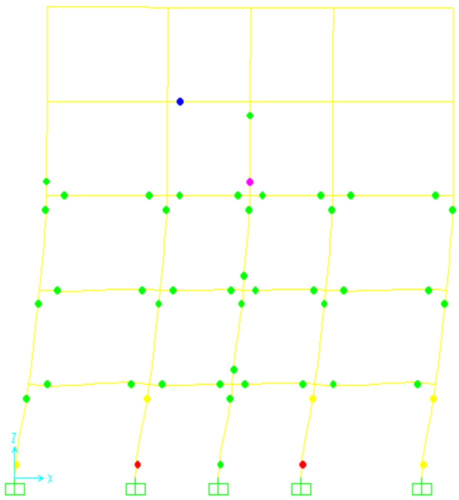


(c) Triangular Lateral Loads

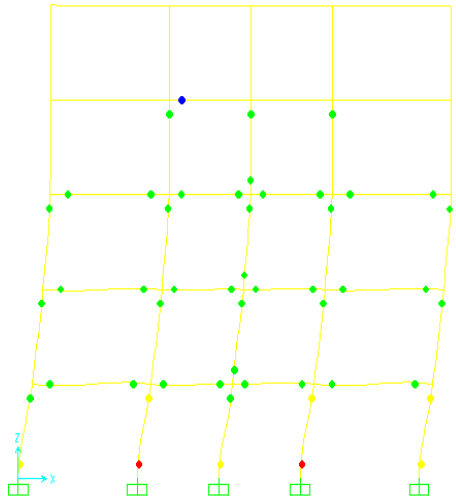
Figure 5.52: Deformed Shape and Plastic Hinge Generation (5% Corroded)

5.3.2.3 (10%) Corroded

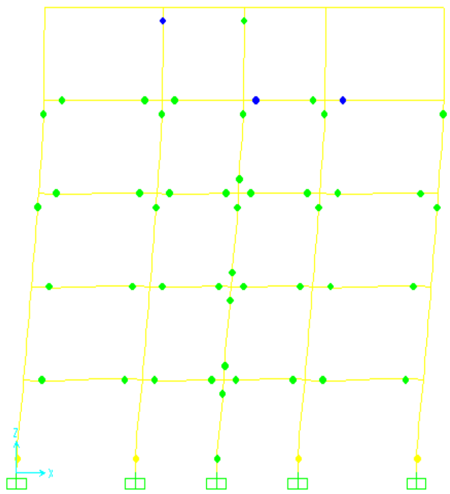
Figure 5.53 shows deformed shape and plastic hinge generation for 10% corroded case of building. Results show that the building start to yield when base shear reaches 55, 52 and 47 percent of structure's weight for the three types of loads as shown in Table 5.62. First column collapse that column in the first floor then all ground columns collapse. The construction display good confrontation as yielding till collapse of columns. Also it displays good flexible confrontation to loading. Load type was effect on plastic hinge mechanism where triangular lateral load model shows more roof displacement than others. Choosing appropriate strengthens methods for weak columns will help structure to improve its ductility during earthquake. Building with 1st mode lateral load and triangular lateral loads has life safety limit. According to this limit its expected that building has a big damage and cannot be safe to use.



(a) 1st Mode Lateral Load



(b) Uniform Lateral Loads

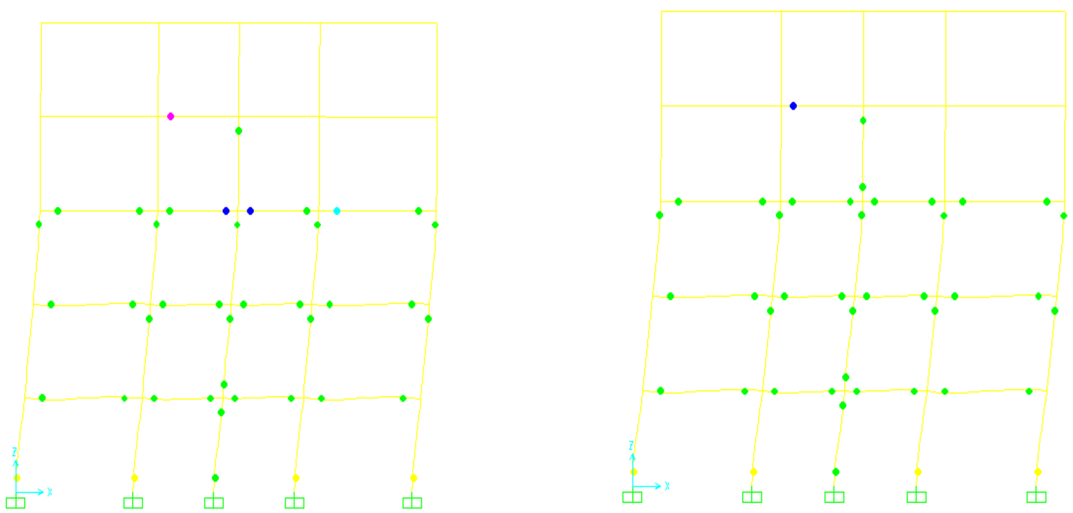


(c) Triangular Lateral Loads

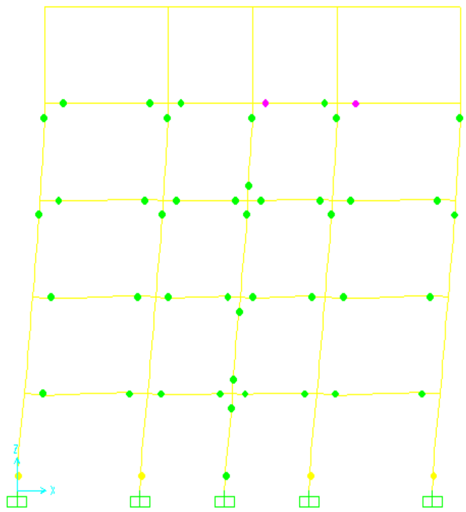
Figure 5.53: Deformed Shape and Plastic Hinge Generation (10% Corroded)

5.3.2.4 (15%) Corroded

Figure 5.54 shows deformed shape and plastic hinge generation for 15% corroded case of building. Results show that the building start to yield when base shear reaches 54, 51 and 46 percent of structure's weight for the three types of loads as shown in Table 5.62. First column collapse that column in the first floor then all ground columns collapse. The construction display good confrontation as yielding till collapse of columns. Also it displays good flexible confrontation to loading. Load type was effect on plastic hinge mechanism where triangular lateral load model shows more roof displacement than others. Choosing appropriate strengthens methods for weak columns will help structure to improve its ductility during earthquake. Building with 1st mode lateral load and triangular lateral loads has life safety limit. According to this limit its expected that building has a big damage and cannot be safe to use.



(a) 1st Mode Lateral Load (b) Uniform Lateral Loads

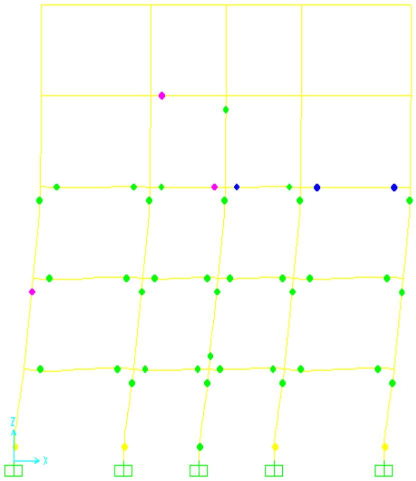


(c) Triangular Lateral Loads

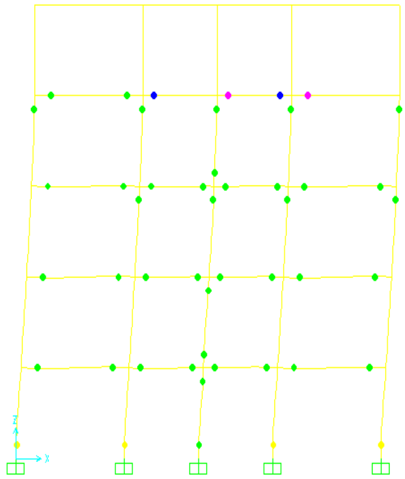
Figure 5.54: Deformed Shape and Plastic Hinge Generation (15% Corroded)

5.3.2.5 (20%) Corroded

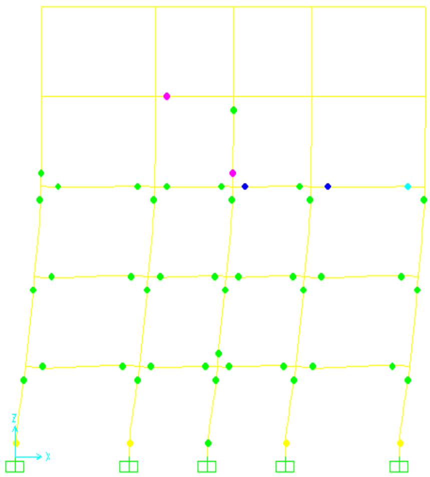
Figure 5.55 shows deformed shape and plastic hinge generation for 20% corroded case of building. Results show that the building start to yield when base shear reaches 53, 50 and 46 percent of structure's weight for the three types of loads as shown in table 6.2. First column collapse that column in the first floor then all ground columns collapse. The construction display good confrontation as yielding till collapse of columns. Also it displays good flexible confrontation to loading. Load type was effect on plastic hinge mechanism where triangular lateral load model shows more roof displacement than others. Building at 20% corrosion rate has life safety level under three types of loads 1st mode lateral load, uniform lateral loads and triangular lateral loads. At level building has a big damage and cannot be safe to use anymore.



(a) 1st Mode Lateral Load



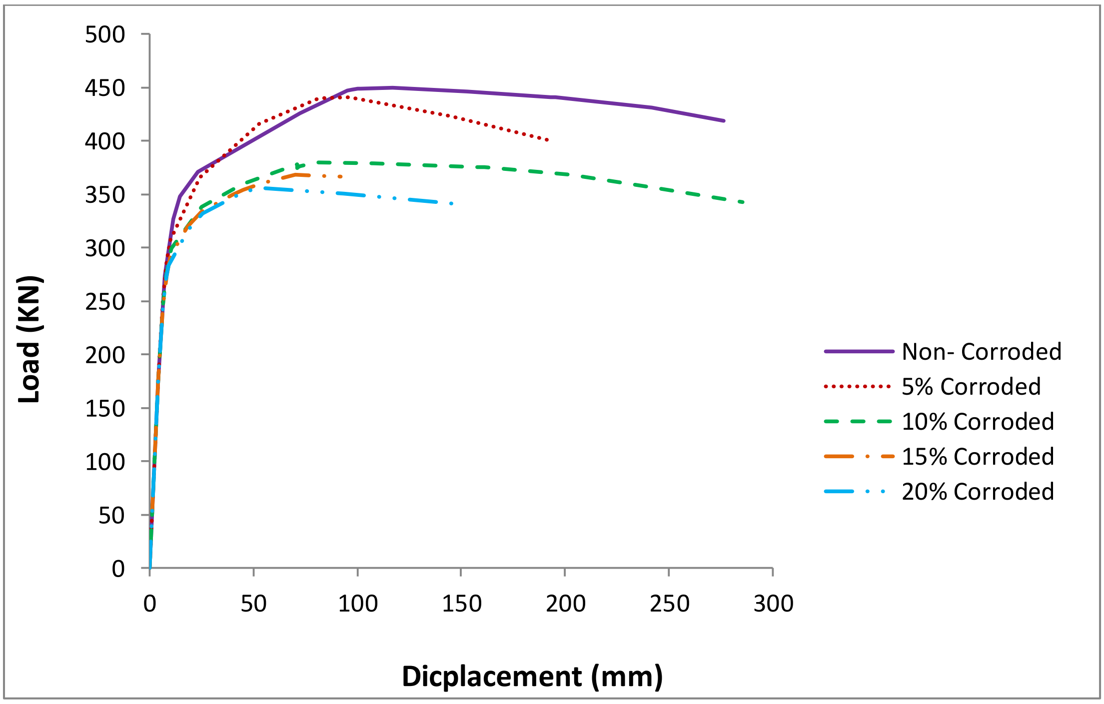
(b) Uniform Lateral Loads



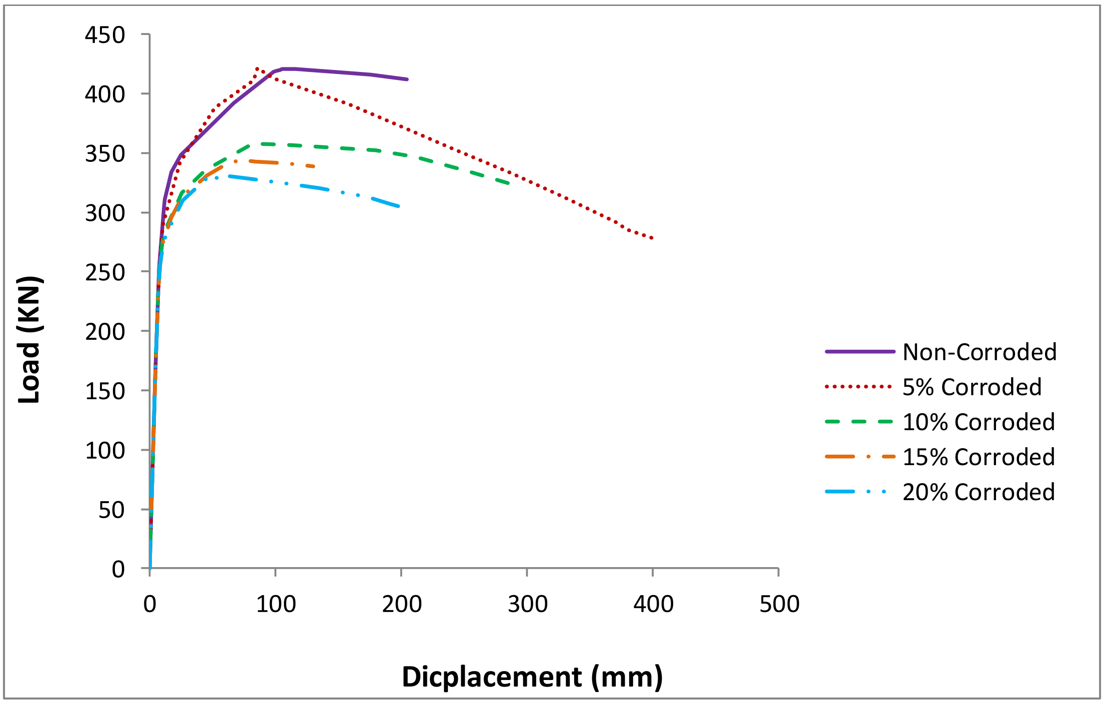
(c) Triangular Lateral Loads

Figure 5.55: Deformed Shape and Plastic Hinge Generation (20% Corroded)

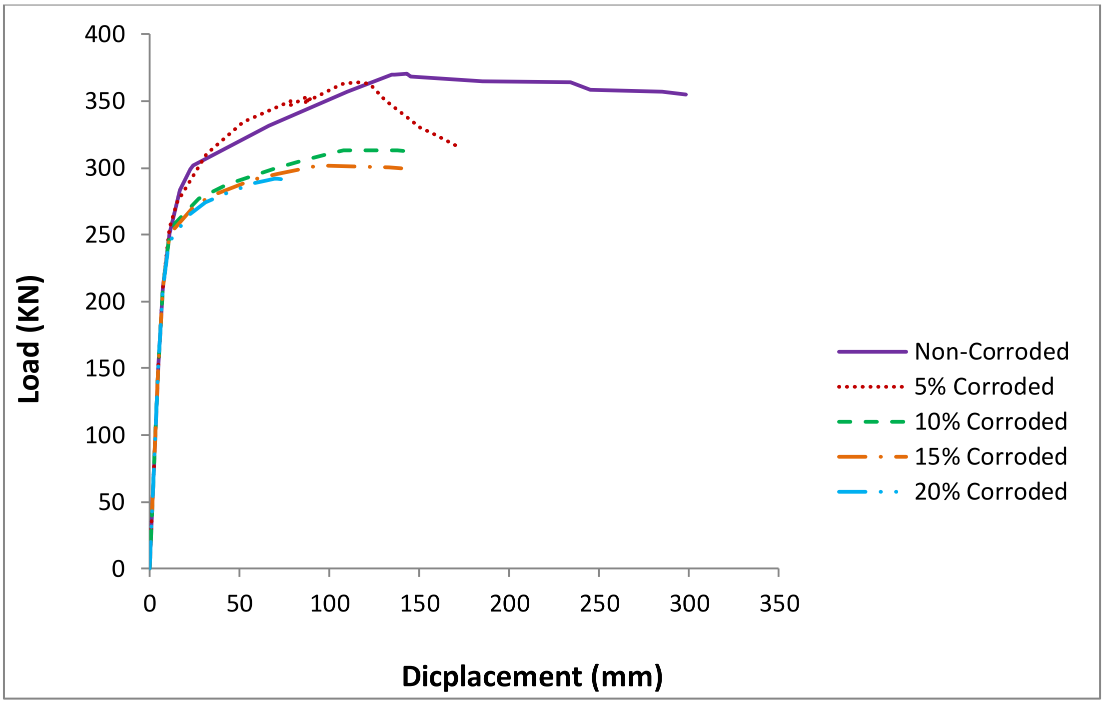
From the results obtained from both codes it can be observed that performance of building under uniform and triangular lateral pattern reach to the life safety level according to FEMA356, while according to Eurocode8 building under triangular pattern reaches to the significant damage level with 20% corrosion rate. These differences in results between both codes depend on the plastic hinges mechanism. In addition the performance of building varies according to the type of loads, where in this research three different types of loads were used. Triangular lateral loads occur displacements of building more than other types.



(a) 1st Mode Lateral Load

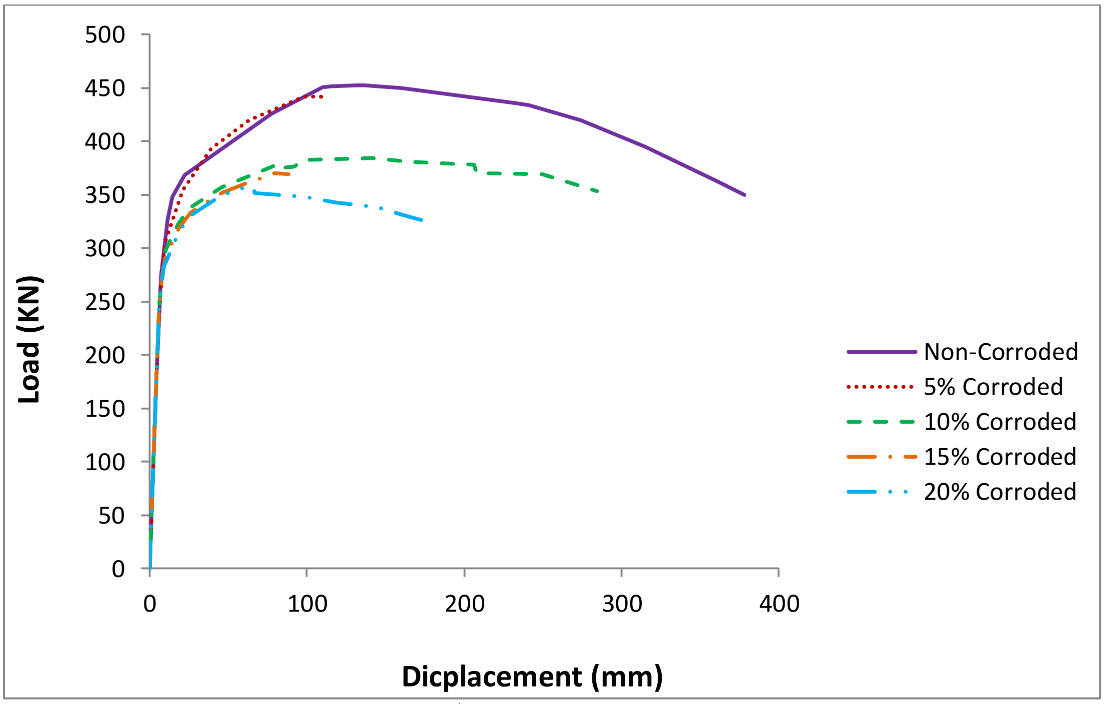


(b) Uniform Lateral Loads

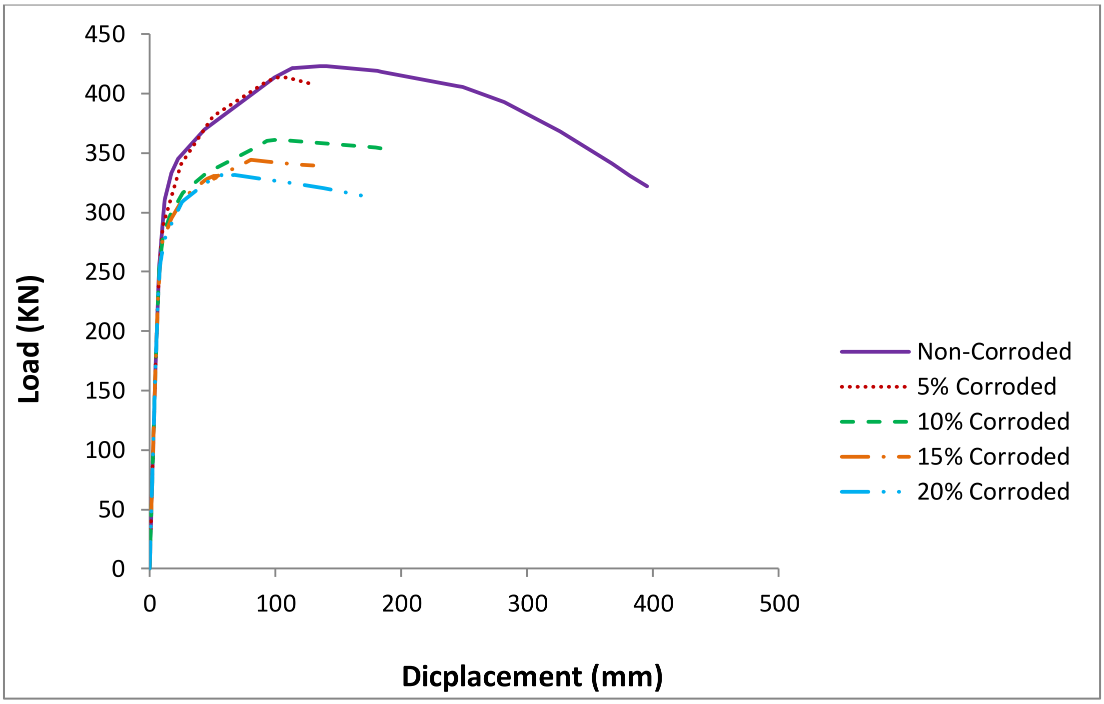


(c) Triangular Lateral Loads

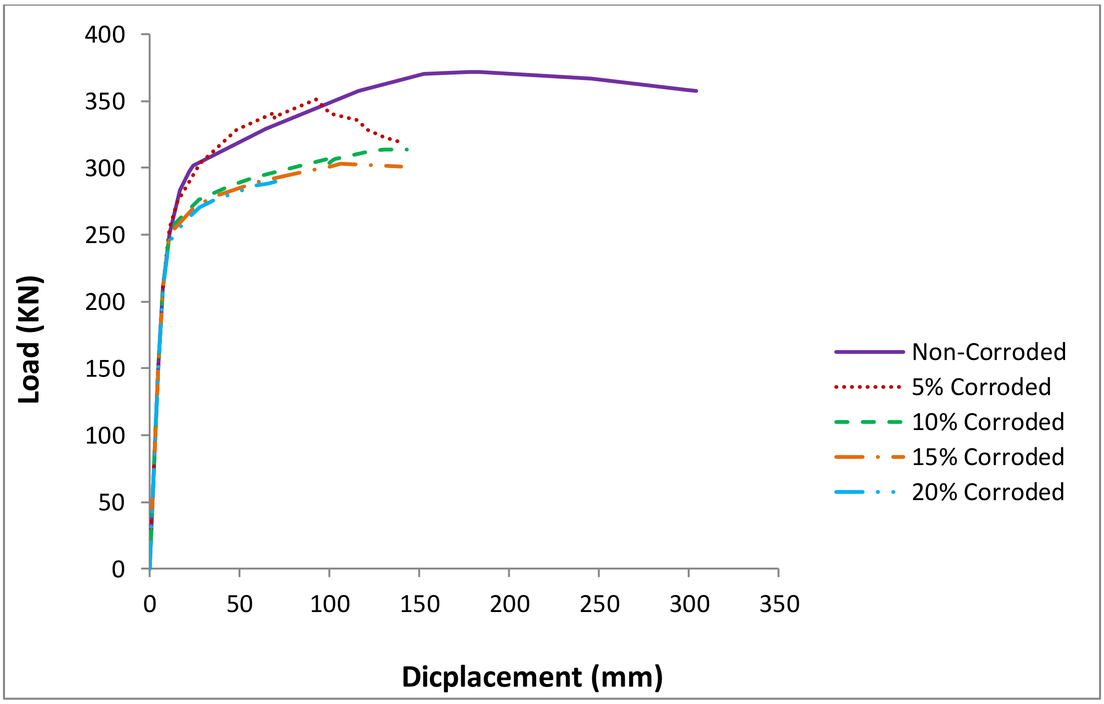
Figure 5.56: Capacity Curve With Corrosion Levels According to FEMA356



(a) 1st Mode Lateral Load



(b) Uniform Lateral Loads



(c) Triangular Lateral Loads

Figure 5.57: Capacity Curve With Corrosion Levels According to Euroced8

Figures 5.56 & 5.57 show the effect of corrosion on the capacity curve of building according to both codes. It is clear that the high percentage of corrosion cause largest displacement with low force, that's mean the corrosion problem possess sensitive effect on building capacity in terms of reduction in concrete strength and additional displacement due to slip. Capability of the building decreases approximately by 30% from non-corrode case to 20% corroded. 5% corroded occur a few effect on structure.

Chapter 6

CONCLUSIONS

Through this study, the nonlinear static analysis method has been explained in accordance with two codes, FEMA356 and Eurocode8. An assessment was performed for the "Sosyal Konutlar" buildings in Famagusta to calculate buildings capability against earthquakes and their displacements due to three types of loading, 1^{st} mode lateral load pattern, uniform lateral load and triangular lateral load.

The modeling has been carried out by using "CSI SAP2000" structural analysis program. Moment-Curvature relationships were constructed for columns and beams sections of building.

The investigations have been conducted on the buildings showed that the columns sections were suffered from corrosion problems. Therefore, corrosion problems have been taken into account in this research with estimated rates of 5%, 10%, 15% and 20% to see the effect of corrosion on the performance of the buildings through two different procedures, namely FEMA 356 and Eurocode 8.

Results obtained for each case of corrosion and each type of load show that the buildings are not safe if the corrosion level exceeds 20%. It is observed that the buildings capacity dropped significantly and the post yielding stiffness has been decreased and become negative as the corrosion increased. On the other hand, such a reduction in the capacity for these buildings (more than 20% in some cases) may be dangerous and remedial measures should be taken for these buildings. It should be noted that, the evaluation procedure carried out in this study is limited with the static procedures. On the other hand, dynamic procedures may be applied for detailed studies.

The nonlinear static analysis method is simplified method to evaluate building capacity and seismic performance of reinforced concrete frame structures as discussed in chapter 2 & 3 according to both codes, but it needs more research to prove whether these methods are appropriate for assessment especially for the buildings considered in this study. Therefore, some further studies may be recommended such as nonlinear dynamic analysis method.

In some cases of corrosion, it can be observed that post-yielding of the structure may cause ***"negative stiffness"*** in which under dynamic excitation behavior might be very complicated due to instability of the structure during the excitation.

One of the most important modes of failure of a structure is the defeat of stability. The progression of failure from loss of stability is invariably a dynamic procedure wherein the motion of the structure, normally to no good end [19].

In this study, three effects of corrosion (i.e. reduction in steel area, slip and reduction in concrete strength) all considered and it has been observed that effect of corrosion may cause both reduction in the strength level of the building and decrease in post-yield stiffness.

It should be noted that recent observations on these buildings have been showed that some columns have corrosion problems more than 20% percent as shown in Figures 6.1 & 6.2. Therefore, further future studies can be done with high levels of corrosion in columns sections of buildings.

Figure 6.1: High Corrosion Levels in Columns

Figure 6.2: Extinction of Concrete Cover Due to High Levels of Corrosion

REFERENCES

[1] EC8. (2005). Design of Structures for Earthquake Resistance, Part3: Assessment and Retrofitting of Buildings. Brussels: European Committee for Standardization.

[2] SEAOC. (1995). Performance Based Seismic Engineering of Buildings. Sacramento: Structural Engineers Association of California.

[3] Michael. N. F. (2009). Seismic Design, Assessment and Retrofitting of Concrete Buildings. New York: Springer Dordrecht Heidelberg, vol. 8, p. 302.

[4] Antony. W. (2008). Recommendations for the Seismic Design of High-Rise Buildings. The Council on Tall Buildings and Urban Habitat, p. 09.

[5] Cyprus Geological Heritage Educational Tool. (2004). The Seismicity of the Cyprus Region. Retrieved December 15, 2011 from the World Wide Web: http//www.cyprusgeology.org/english 5-2-seismicity.htm.

[6] Alain. P. (2007). Advanced Earthquake Engineering Analysis. New York: Springer Wien, p. 63-91.

[7] Amr S. E. and Luigi D. S. (2008). Fundamentals of Earthquake Engineering. United Kingdom: John Wiley & Sons Ltd, p. 220.

[8] A.S.C.E. (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings. Virginia: Federal Emergency Management Agency & American Society of Civil Engineering.

[9] Ioannis. P. G. (2009). Seismic Assessment of a RC Building According to FEMA356 and Eurocode8. *TEE,ETEK*, vol. 21, p. 3-8.

[10] EC8. (2004). Design of Structures for Earthquake Resistance, Part1: General Rules, Seismic Actions and Rules for Buildings. Brussels: European Committee for Standardization.

[11] Hakan. Y, Serhan. S and Ozgur. E. (2011). Time Dependent Seismic Performance Assessment of a Single-Degree-of-Freedom Frame Subject to Corrosion. *Engineering Failure Analysis*, vol. 19, p. 1-5.

[12] C. Q. Li, J. J. Zheng, W. Lawanwisut and R. E. Melchers. (2007). Concrete Delimitation Caused by Steel Reinforcement Corrosion. *Journal of Materials in Civil Engineering*, p. 3-7.

[13] EC2. (2004). Design of Concrete Structures, Part1: General Rules and Rules of Buildings. Brussels: European Committee for Standardization.

[14] Halil. S. and Eric. J. S. (2008). Reinforcement Slip in Reinforced Concrete Columns. *ACI Structural Journal: Technical Paper*, vol. 105, p. 03.

[15] Kyle. S, R. D. Hooton and S. J. Pantazopoulou. (1999). Corrosion Effects on Bond Strength in Reinforced Concrete. *ACI Structural Journal: Technical Papers,* vol. 96, p. 07.

[16] Evan. C. B. (2000). Sectional Analysis of Reinforced Concrete Members. Canada: Acquisitions and Bibliographic, p. 16.

[17] R. Martino, E. Spacone and G. Kingsley. (2004). Nonlinear Pushover Analysis of RC Structures. *Structures 2000*, p. 02.

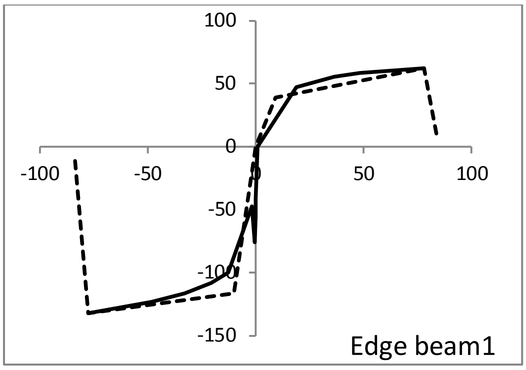
[18] Mehmet. I. and Hayri. B. O. (2006). Effects of Plastic Hinge Properties in Nonlinear Analysis of Reinforced Concrete Buildings. *Engineering Structures,* vol. 28, p. 08.

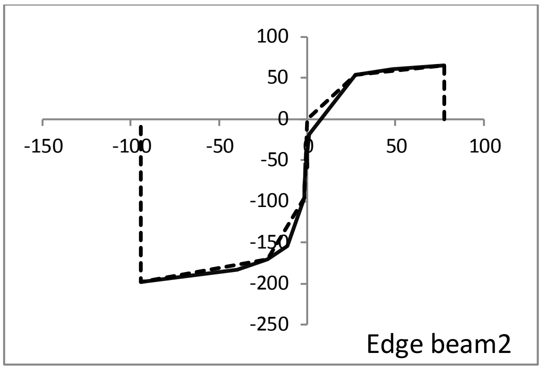
[19] K. D. Hjelmstad and E. B. Williamson. (1998). Dynamic Stability of Structural Systems Subjected to Base Excitation. *Engineering Structures*, vol. 20, p. 01.

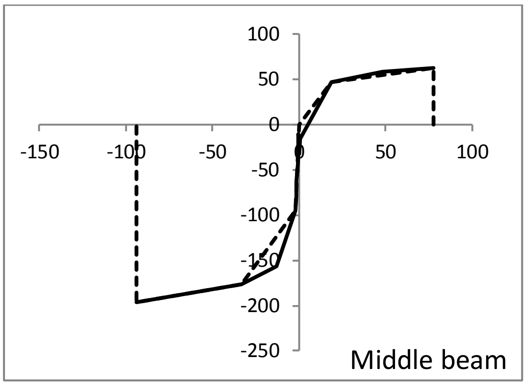
APPENDICES

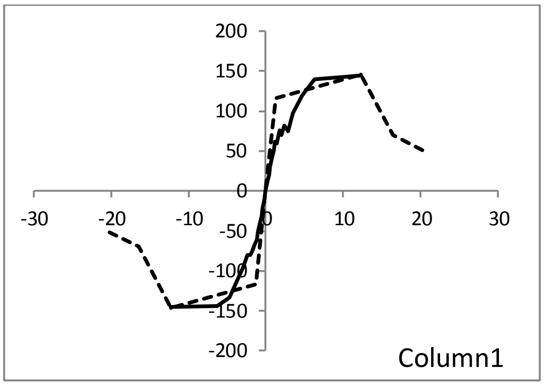
Appendix A: Moment-Curvature Relationships For Members Sections of Building

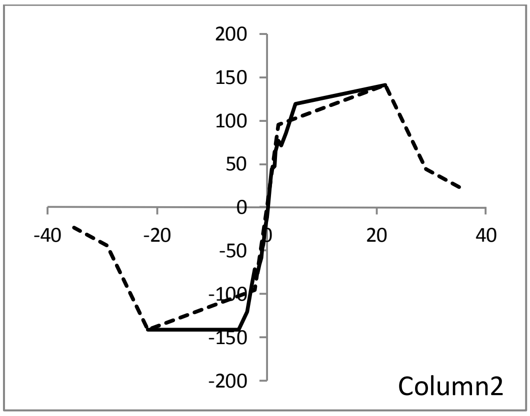
Horizontal axis represents curvature in (rad/km) and vertical axis represents moment in (kN.m), — represents curve,---- represents idealized curve.

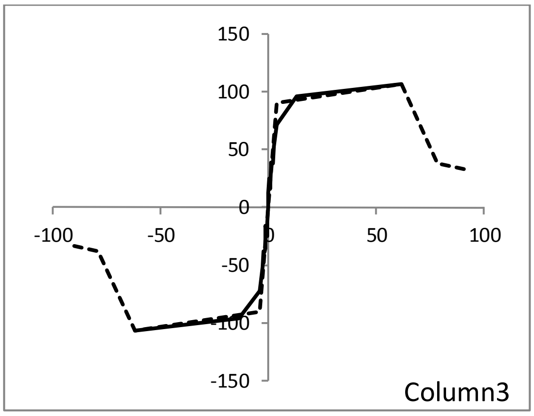


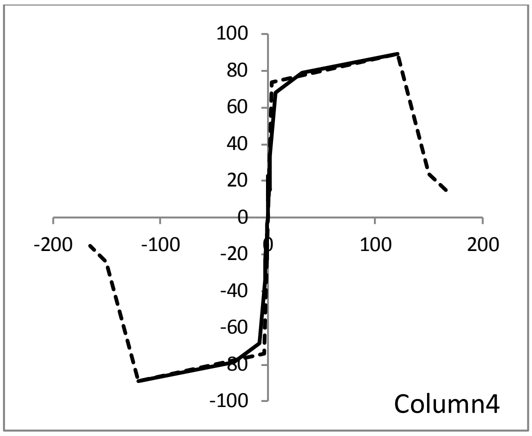


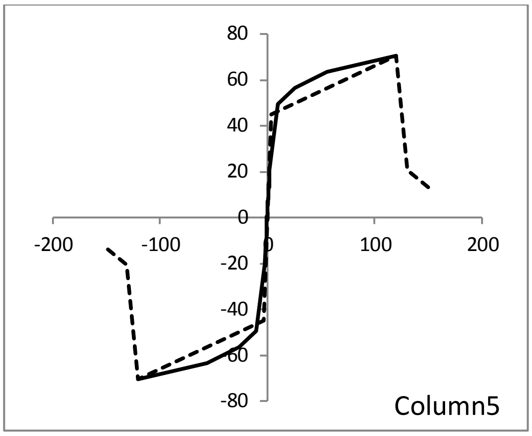


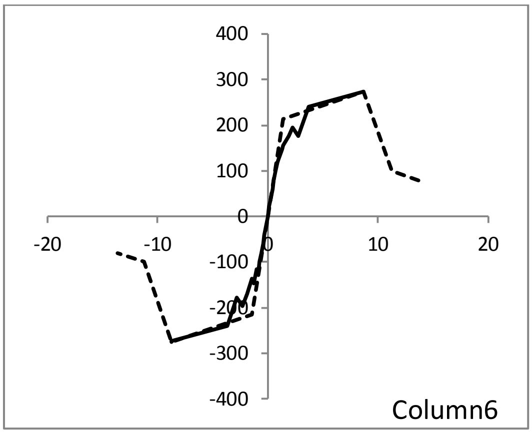


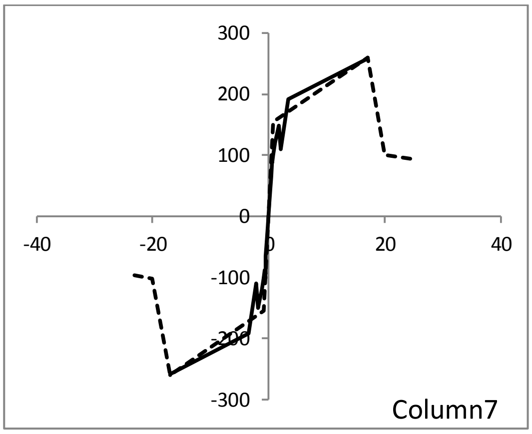


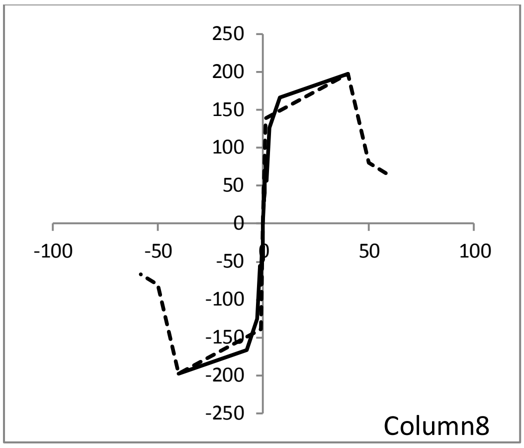


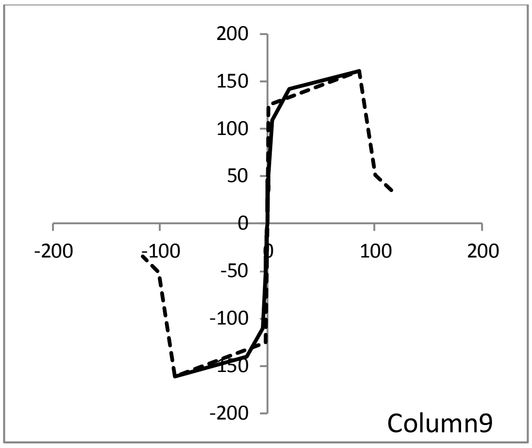

100
80
60
40
20
0
-20
-40
-60
-80
-100
-200
-100
0
100
200
Column4

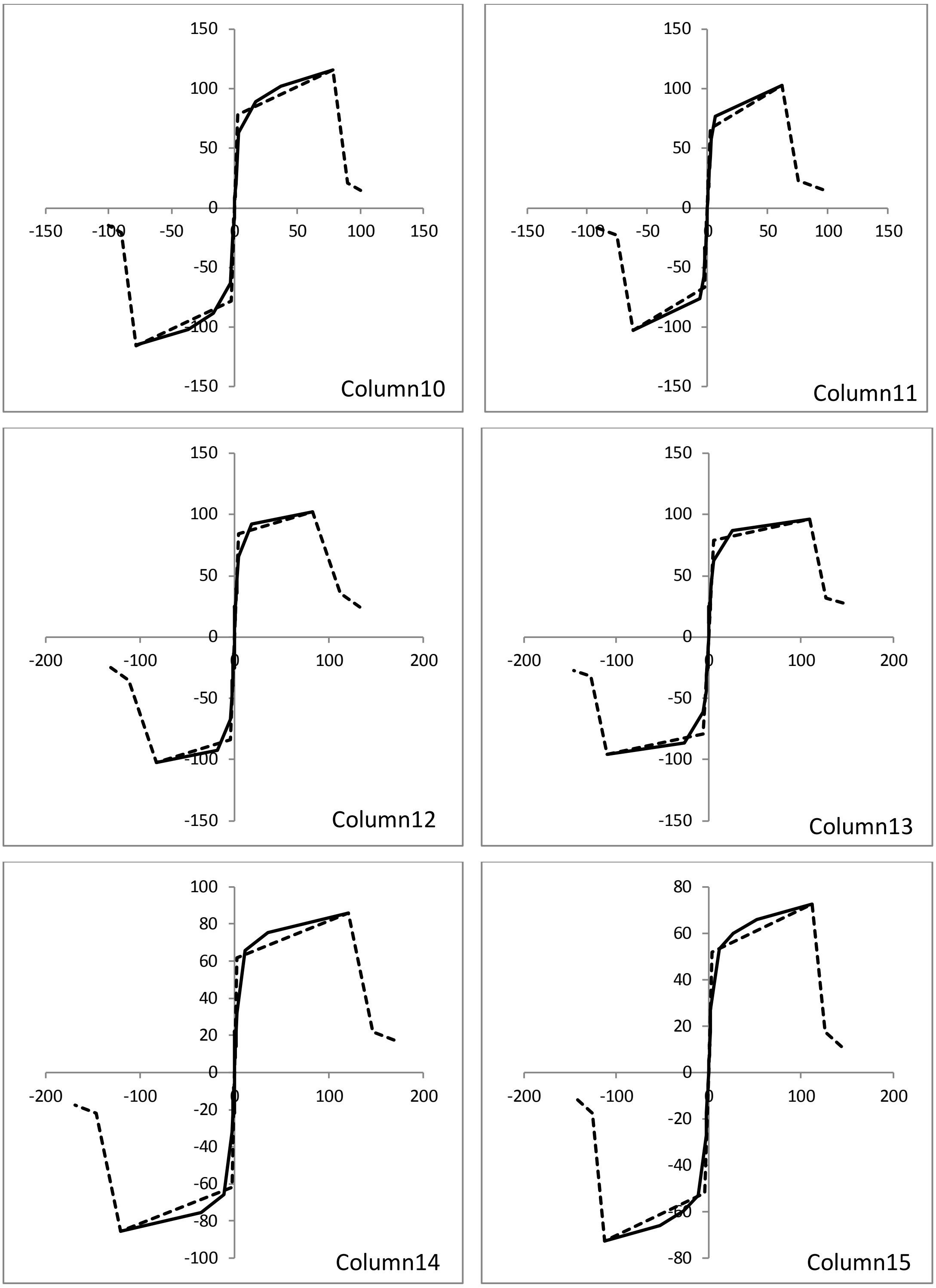

80
60
40
20
0
-20
-40
-60
-80
-200
-100
0
100
200
Column5

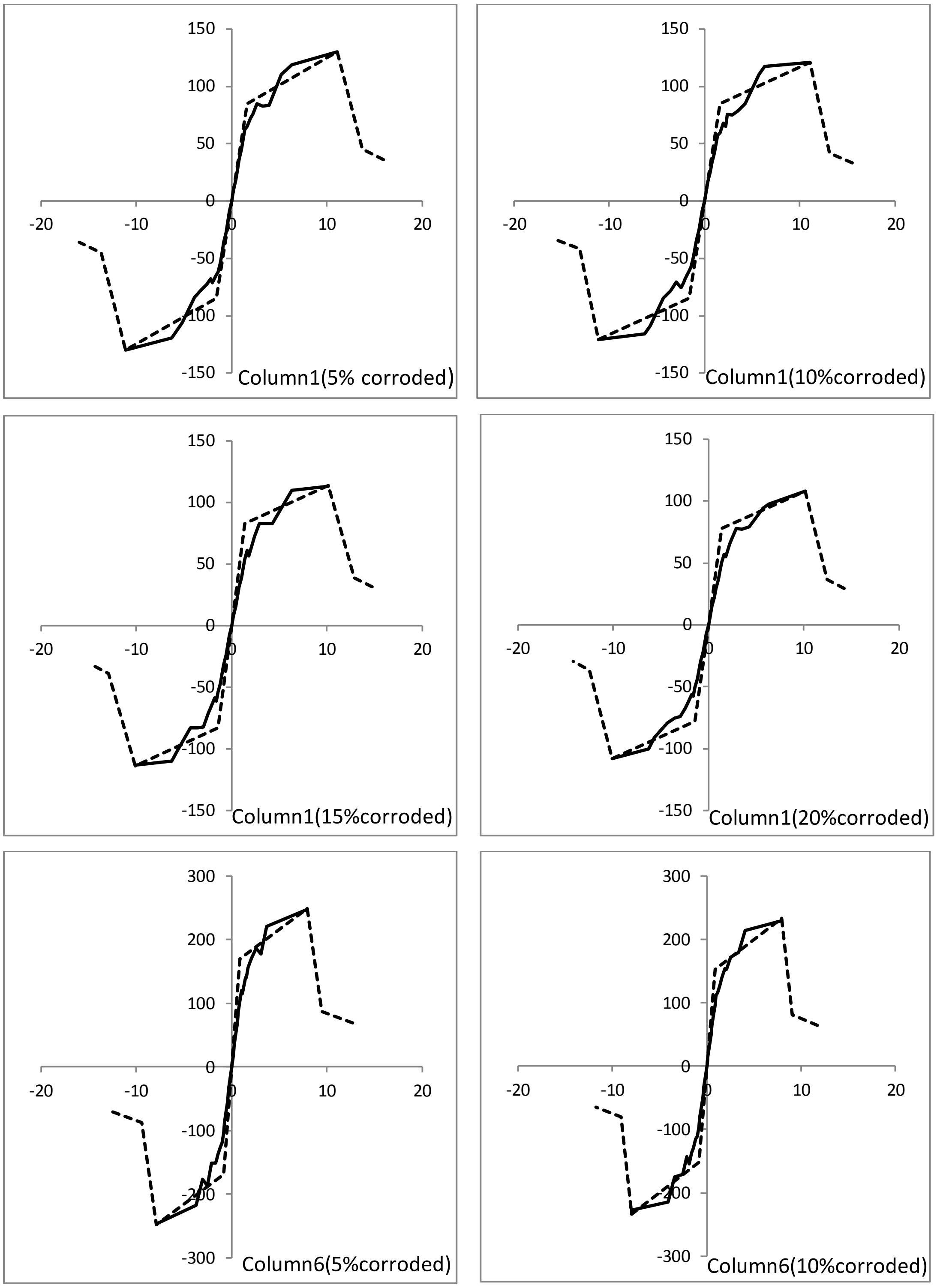

400
300
200
100
0
-100
-200
-300
-400
-20
-10
0
10
20
Column6

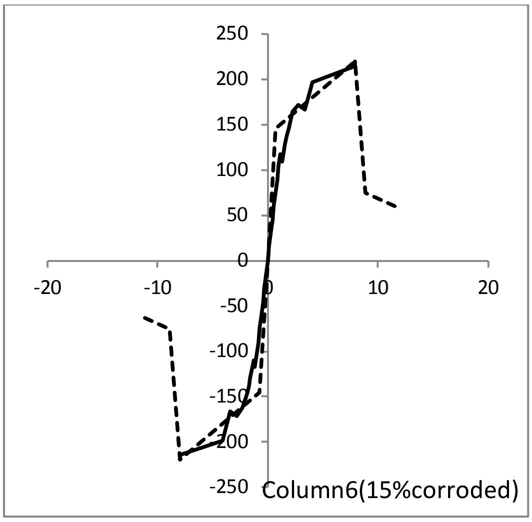

300
200
100
0
-100
-200
-300
-40
-20
0
20
40
Column7

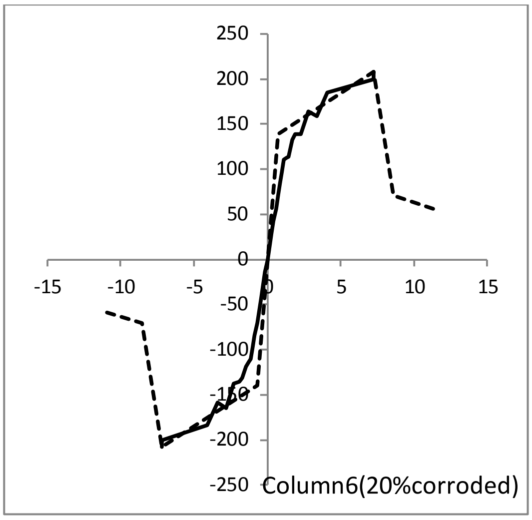

250
200
150
100
50
0
-50
-100
-150
-200
-250
-100
-50
0
50
100
Column8

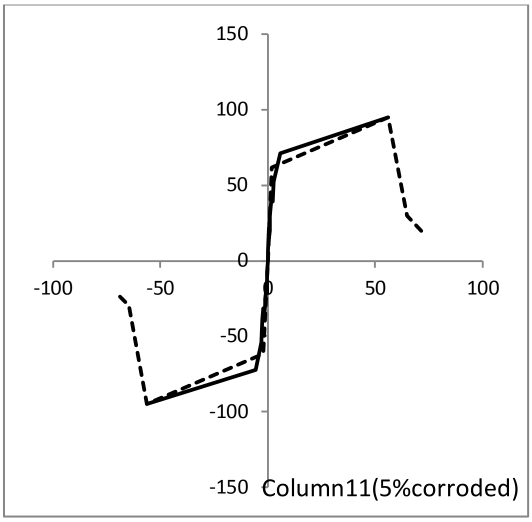

200
150
100
50
0
-50
-100
-150
-200
-200
-100
0
100
200
Column9

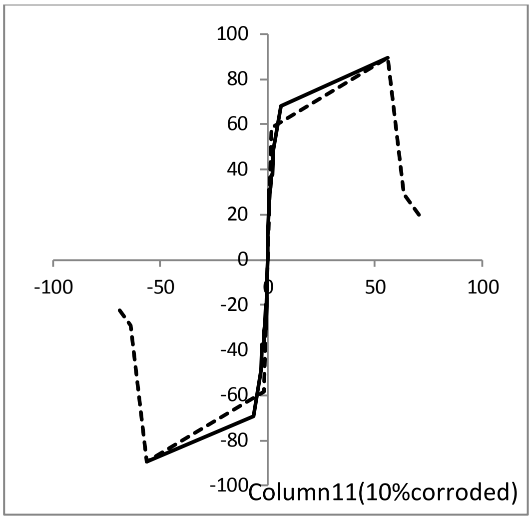

Column10
Column11
Column12
Column13
Column14
Column15

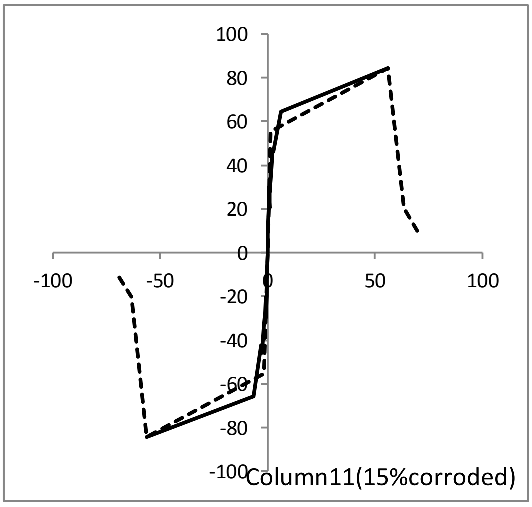

150
100
50
0
-20
-10
0
10
20
-50
-100
-150
Column1(5% corroded)
Column1(10%corroded)
Column1(15%corroded)
Column1(20%corroded)
300
200
100
-200
-300
Column6(5%corroded)
Column6(10%corroded)

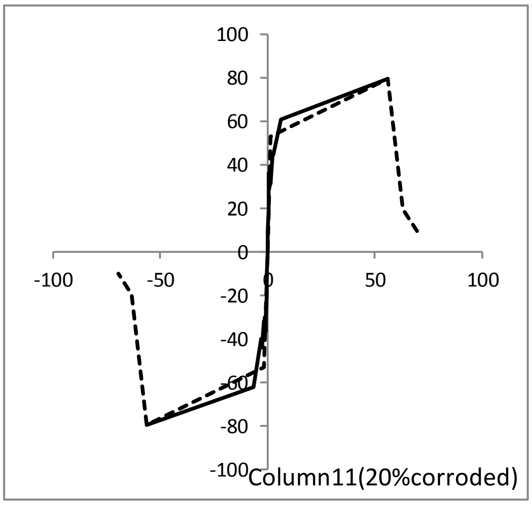

250
200
150
100
50
0
-20 -10 0 10 20
-50
-100
-150
-200
-250
Column6(15%corroded)


250
200
150
100
50
0
-15 -10 -5 0 5 10 15
-50
-100
-150
-200
-250
Column6(20%corroded)


150
100
50
0
-100 -50 0 50 100
-50
-100
-150
Column11(5%corroded)


100
80
60
40
20
0
-100 -50 0 50 100
-20
-40
-60
-80
-100
Column11(10%corroded)


100
80
60
40
20
0
-100 -50 0 50 100
-20
-40
-60
-80
-100
Column11(15%corroded)


100
80
60
40
20
0
-100 -50 0 50 100
-20
-40
-60
-80
-100
Column11(20%corroded)

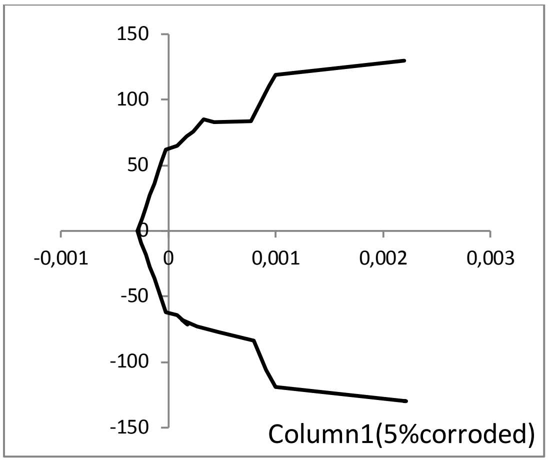
Appendix B: Parameters to Calculate Reduction in Concrete Strength Duo to Corrosion

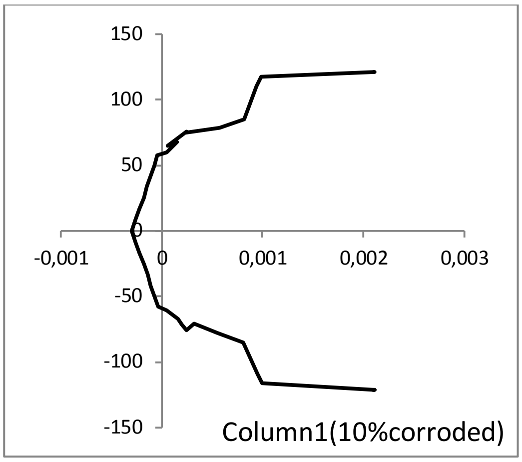
Table B.1: Values of basic variables

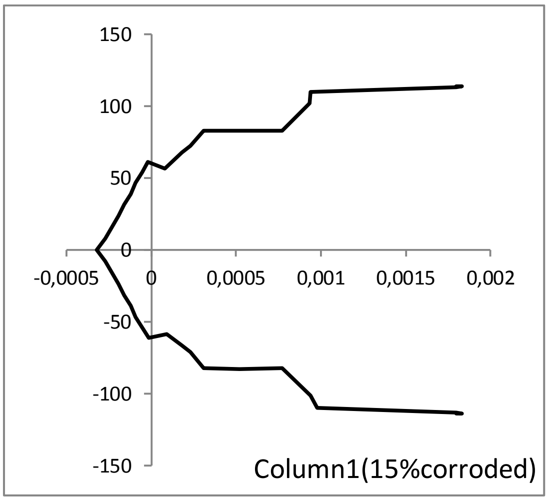
Symbol	Value	Source
υ_c	0.2	[11]
f_t	1.36 MPa	Present study
ρ_{rust}	3600 kg/m^3	[11]
ρ_{st}	7850 kg/m^3	[11]
α_{rust}	0.57	[11]
d_o	12.5 μm	[11]
φ_{cr}	1.39	Present study
α	0.1	[11]
E_{eff}	12480 MPa	Present study
$i_{corr}(5\%)$	0.127 μA/cm^2	Present study
$i_{corr}(10\%)$	0.25 μA/cm^2	Present study
$i_{corr}(15\%)$	0.38 μA/cm^2	Present study
$i_{corr}(20\%)$	0.509 μA/cm^2	Present study
f_c^* *(5% corroded)*	12.58 MPa	Present study
f_c^* *(10% corroded)*	12MPa	Present study
f_c^**(15% corroded)*	11.41 MPa	Present study
f_c^**(20% corroded)*	10.98 MPa	Present study

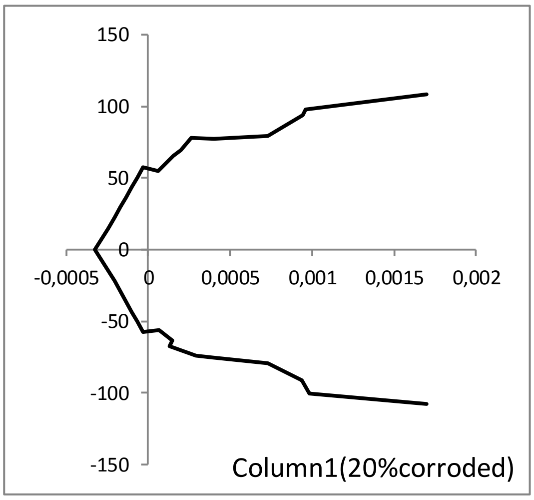
Appendix C: Moment-Reinforcement Strain Relationships for Corroded Case of Columns Sections

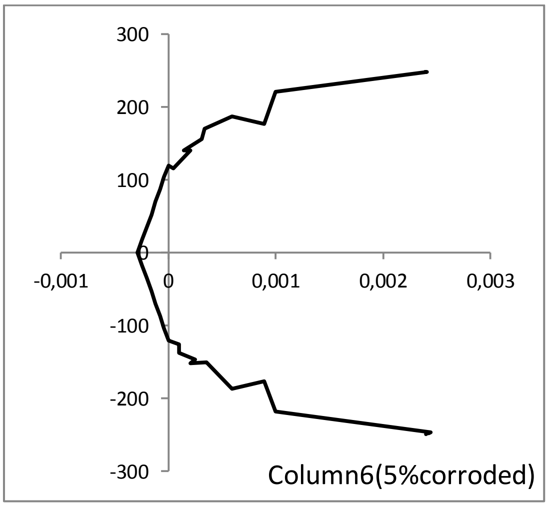
Horizontal axis represents reinforcement strain and vertical axis represents moment in (kN.m).

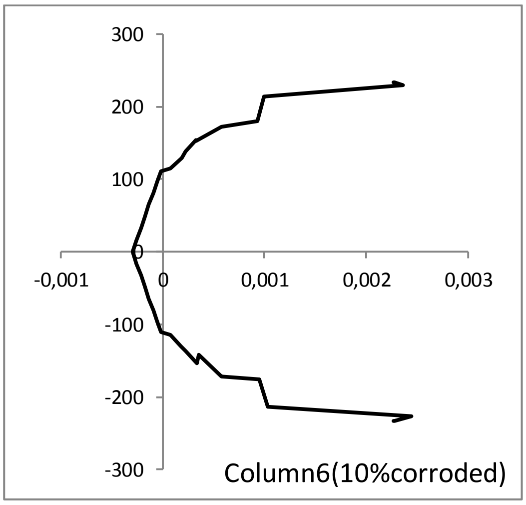


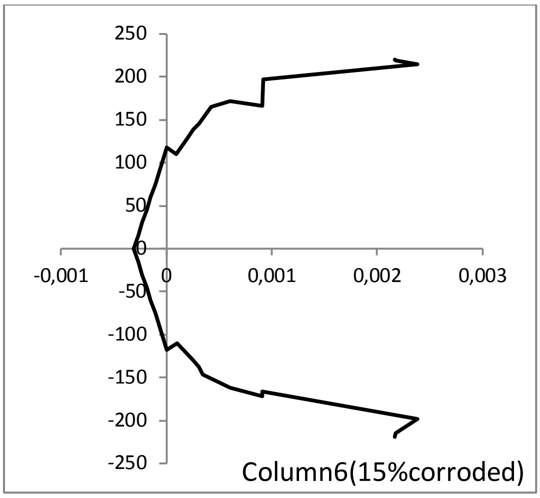


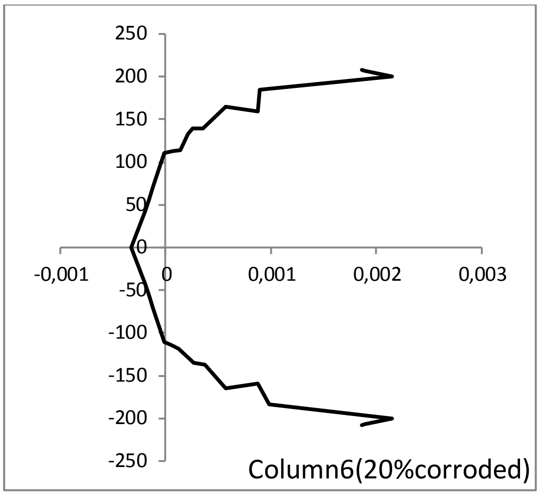


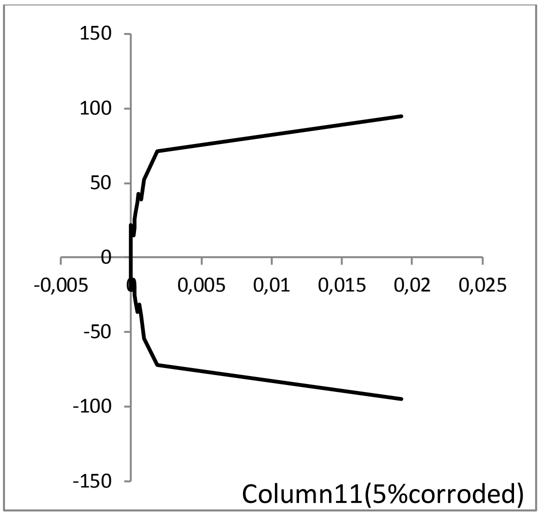


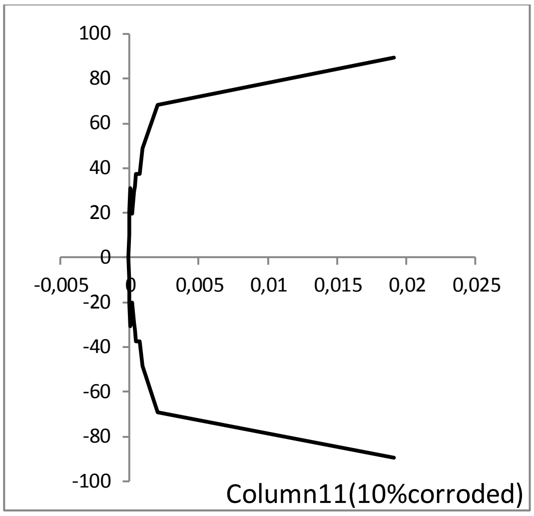


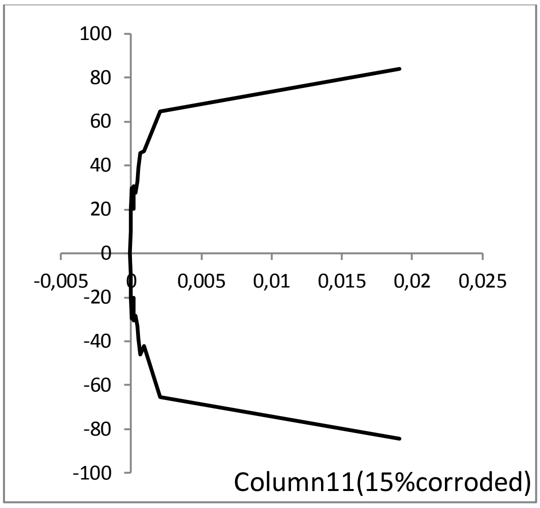


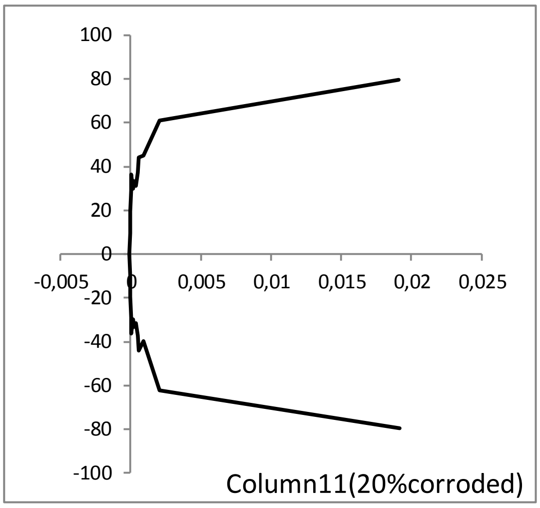

250
200
150
100
50
0
-50
-100
-150
-200
-250
-0,001
0
0,001
0,002
0,003
Column6(15%corroded)


250
200
150
100
50
0
-50
-100
-150
-200
-250
-0,001
0
0,001
0,002
0,003
Column6(20%corroded)


150
100
50
0
-50
-100
-150
-0,005
0,005
0,01
0,015
0,02
0,025
Column11(5%corroded)


100
80
60
40
20
0
-20
-40
-60
-80
-100
-0,005
0
0,005
0,01
0,015
0,02
0,025
Column11(10%corroded)


100
80
60
40
20
0
-20
-40
-60
-80
-100
-0,005
0
0,005
0,01
0,015
0,02
0,025
Column11(15%corroded)


100
80
60
40
20
0
-20
-40
-60
-80
-100
-0,005
0
0,005
0,01
0,015
0,02
0,025
Column11(20%corroded)

Appendix D: FEMA356 Parameter to Calculate Target Displacement

Table D.1: FEMA356 parameters for 1st mode lateral load (non-corroded case)

Item	Value
C_o	1.1593
C_1	1.4127
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.1841
T_i	0.1841
K_i	41267.74
K_e	41267.74
Alpha	0.2264
R	2.0219
V_y	269.5314
weight	495.4146
C_m	1.0

Table D.2: FEMA356 parameters for parameters with uniform lateral loads (non-corroded case)

Item	Value
C_o	1.1679
C_1	1.4052
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.1913
T_i	0.1913
K_i	37630.54
K_e	37630.54
Alpha	0.1698
R	2.3954
V_y	227.5033
weight	495.4146
C_m	1.0

Table D.3: FEMA parameters for frame model with triangular lateral loads (non-corroded)

Item	Value
C_o	1.1793
C_1	1.3882
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.2077
T_i	0.2077
K_i	30908.868
K_e	30908.868
Alpha	0.1698
R	2.3954
V_y	227.5033
weight	495.4146
C_m	1.0

Table D.4: FEMA356 parameters for 1st mode lateral load (5% corroded)

Item	Value
C_o	1.1577
C_1	1.4127
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.1841
T_i	0.1841
K_i	41267.74
K_e	41267.74
Alpha	0.1514
R	1.9761
V_y	275.7684
weight	495.4146
C_m	1.0

Table D.5: FEMA356 parameters for frame model with uniform lateral loads (5% corroded)

Item	Value
C_o	1.1653
C_1	1.4052
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.1913
T_i	0.1913
K_i	37630.54
K_e	37630.54
Alpha	0.1444
R	2.0881
V_y	260.984
weight	495.4146
C_m	1.0

Table D.6: FEMA356 parameters for frame model with triangular lateral loads (5% corroded)

Item	Value
C_o	1.1784
C_1	1.3882
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.2077
T_i	0.2077
K_i	30908.868
K_e	30908.868
Alpha	0.1388
R	2.3325
V_y	233.6408
weight	495.4146
C_m	1.0

Table D.7: FEMA356 parameters for 1st mode lateral load (10% corroded)

Item	Value
C_o	1.1564
C_1	1.4127
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.1841
T_i	0.1841
K_i	41267.74
K_e	41267.74
Alpha	0.1158
R	2.0102
V_y	271.0932
weight	495.4146
C_m	1.0

Table D.8: FEMA356 parameters for frame model with uniform lateral loads (10% corroded)

Item	Value
C_o	1.1637
C_1	1.4052
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.1913
T_i	0.1913
K_i	37630.54
K_e	37630.54
Alpha	0.107
R	2.1241
V_y	256.5559
weight	495.4146
C_m	1.0

Table D.9: FEMA356 parameters for frame model with triangular lateral loads (10% corroded)

Item	Value
C_o	1.1762
C_1	1.3882
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.2077
T_i	0.2077
K_i	30908.867
K_e	30908.867
Alpha	0.0959
R	2.3565
V_y	231.261
weight	495.4146
C_m	1.0

Table D.10: FEMA356 parameters for 1st mode lateral load (15% corroded)

Item	Value
C_o	1.1523
C_1	1.4127
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.1841
T_i	0.1841
K_i	41267.74
K_e	41267.74
Alpha	0.1224
R	2.0483
V_y	266.0503
weight	495.4146
C_m	1.0

Table D.11: FEMA356 parameters for frame model with uniform lateral loads (15% corroded)

Item	Value
C_o	1.1574
C_1	1.4052
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.1913
T_i	0.1913
K_i	37630.54
K_e	3763.54
Alpha	0.1174
R	2.1702
V_y	251.1048
weight	495.4146
C_m	1.0

Table D.12: FEMA356 parameters for frame model with triangular lateral loads (15% corroded)

Item	Value
C_o	1.1759
C_1	1.3882
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.2077
T_i	0.2077
K_i	30908.867
K_e	30908.867
Alpha	0.092
R	2.3739
V_y	229.5635
weight	495.4146
C_m	1.0

Table D.13: FEMA356 parameters for 1st mode lateral load (20% corroded)

Item	Value
C_o	1.1519
C_1	1.4127
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.1841
T_i	0.1841
K_i	41267.74
K_e	41267.74
Alpha	0.1186
R	2.0828
V_y	261.6399
weight	495.4146
C_m	1.0

Table D.14: FEMA356 parameters for frame model with uniform lateral loads (20% corroded)

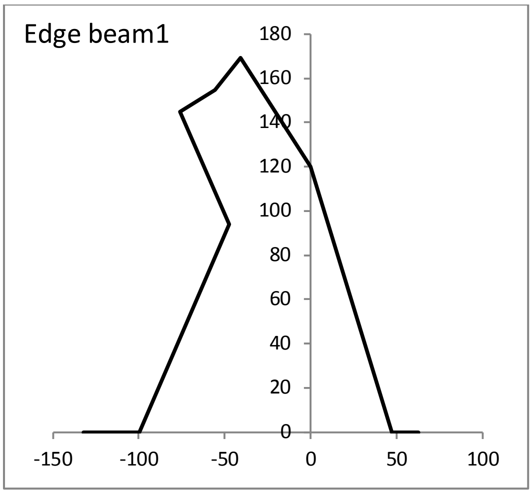
Item	Value
C_o	1.1586
C_1	1.4052
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.1913
T_i	0.1913
K_i	37630.54
K_e	37630.54
Alpha	0.1077
R	2.184
V_y	249.5194
weight	495.4146
C_m	1.0

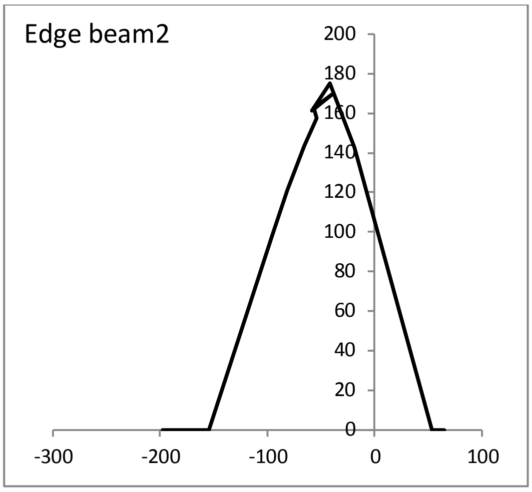
Table D.15: FEMA356 parameters for frame model with triangular lateral loads (20% corroded)

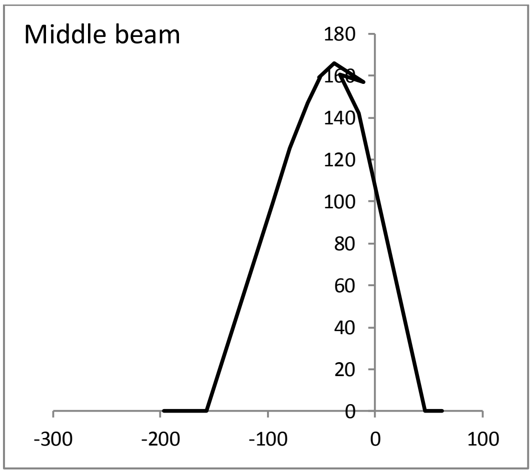
Item	Value
C_o	1.1747
C_1	1.3882
C_2	1.0
C_3	1.0
S_a	1.1
T_e	0.2077
T_i	0.2077
K_i	30908.867
K_e	30908.867
Alpha	0.0909
R	2.4036
V_y	226.7272
weight	495.4146
C_m	1.0

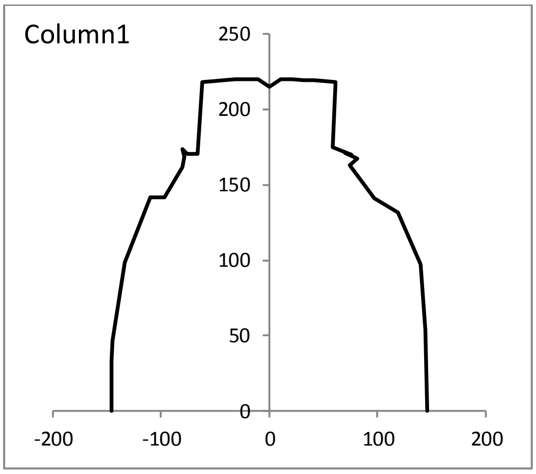
Appendix E: Moment-Shear Relationships to Calculate Shear Length for Sections

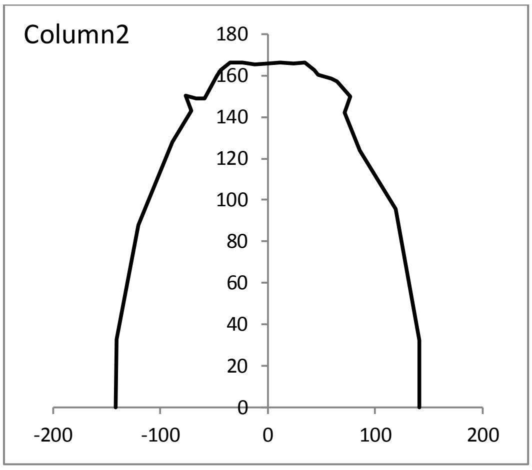
Horizontal axis represents moment in (kN.m) and vertical axis represents shear in (kN).

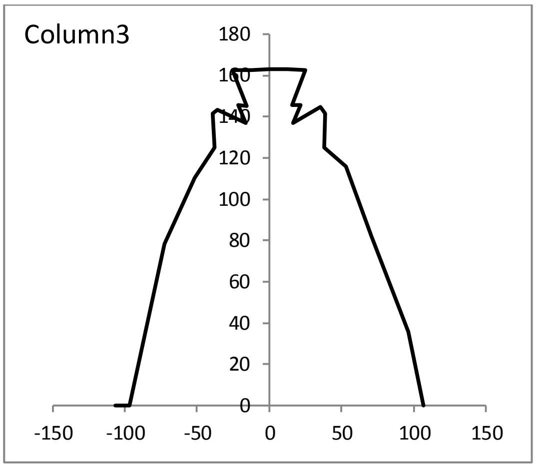


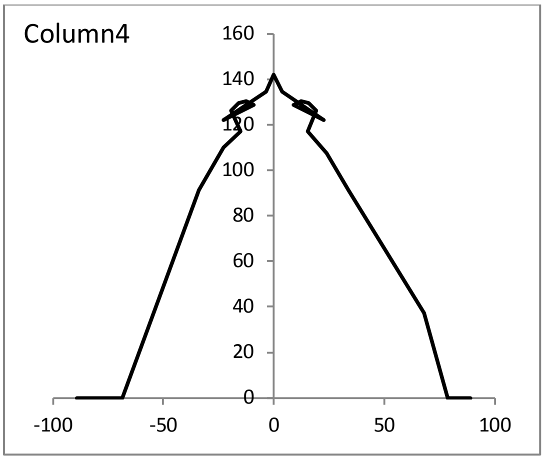


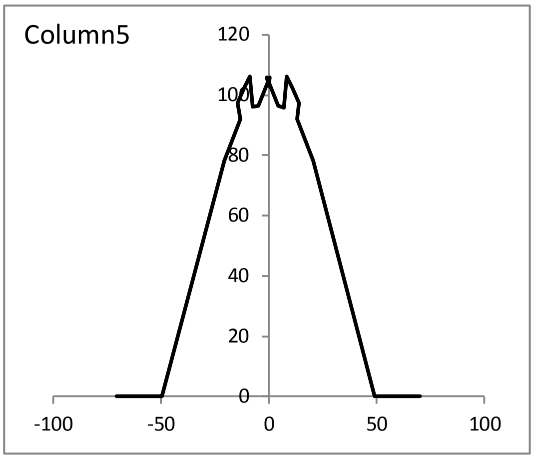


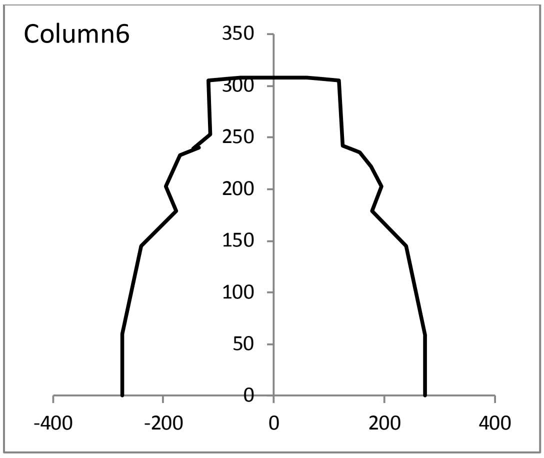


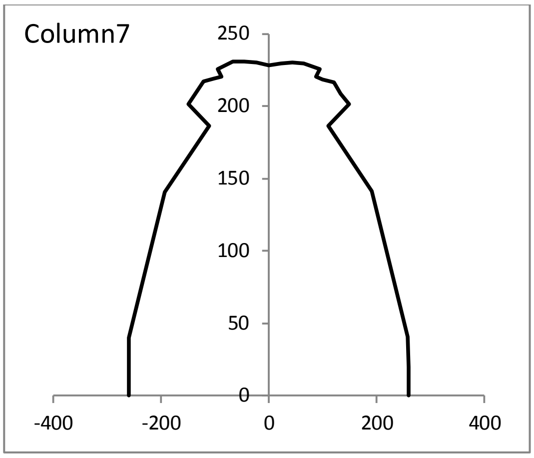


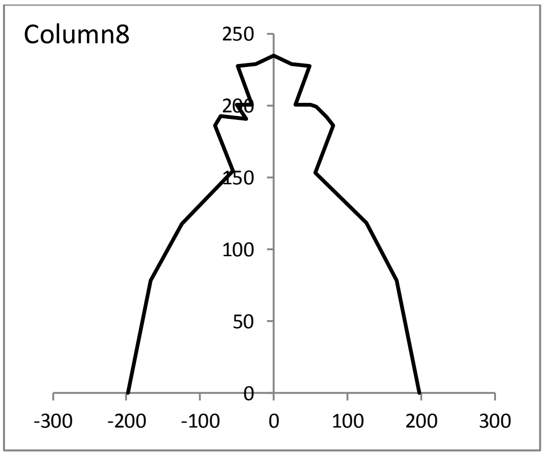


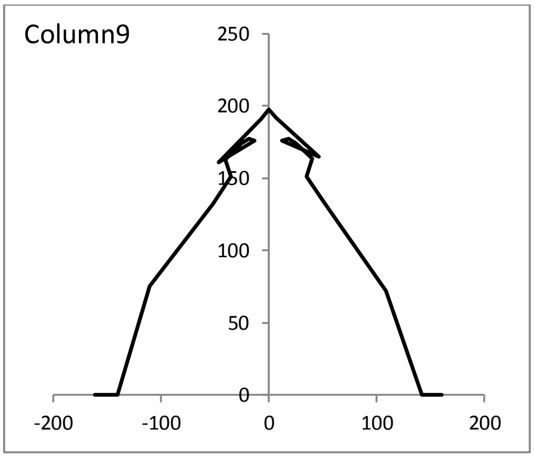

Column4
160
140
120
100
80
60
40
20
0
-100
-50
0
50
100

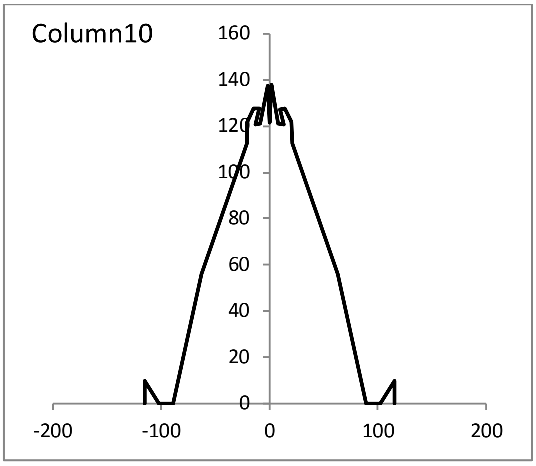

Column5
120
100
80
60
40
20
0
-100
-50
0
50
100

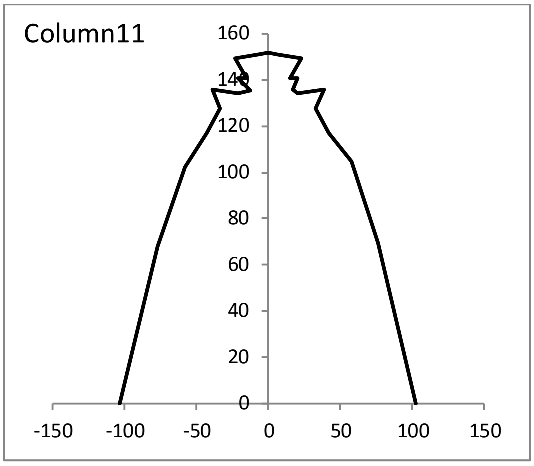

Column6
350
300
250
200
150
100
50
0
-400
-200
0
200
400

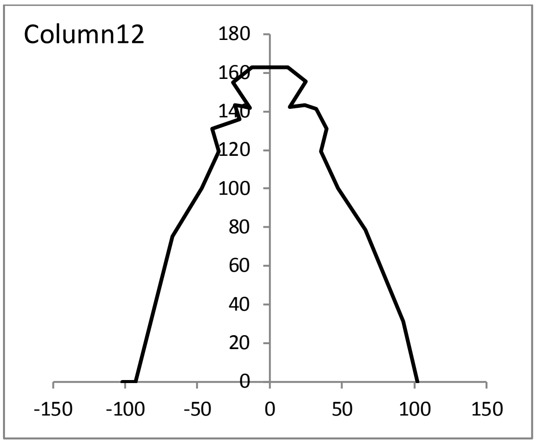

Column7
250
200
150
100
50
0
-400
-200
0
200
400

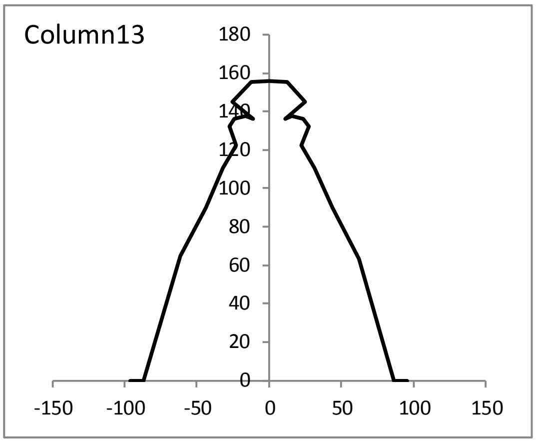

Column8
250
200
150
100
50
0
-300
-200
-100
0
100
200
300

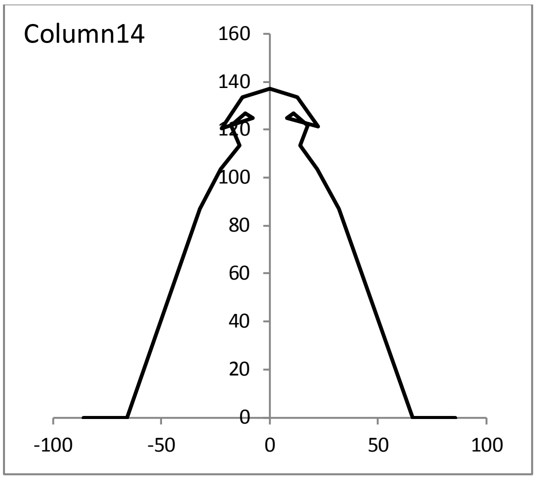

Column9
250
200
150
100
50
0
-200
-100
0
100
200

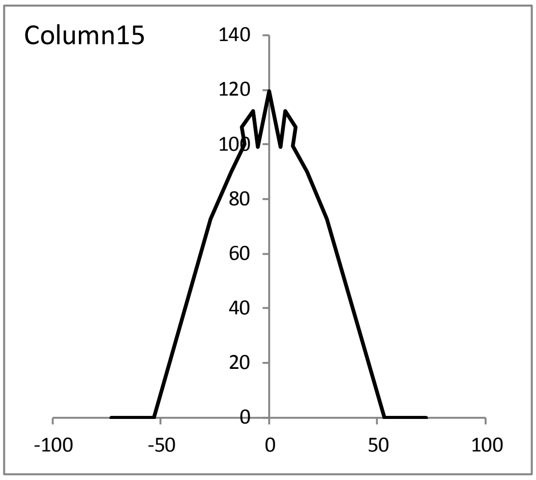

Column10
160
140
120
100
80
60
40
20
0
-200
-100
0
100
200

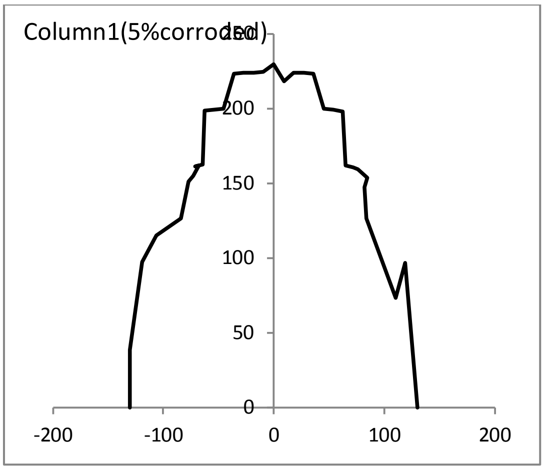

Column11
160
140
120
100
80
60
40
20
0
-150
-100
-50
0
50
100
150

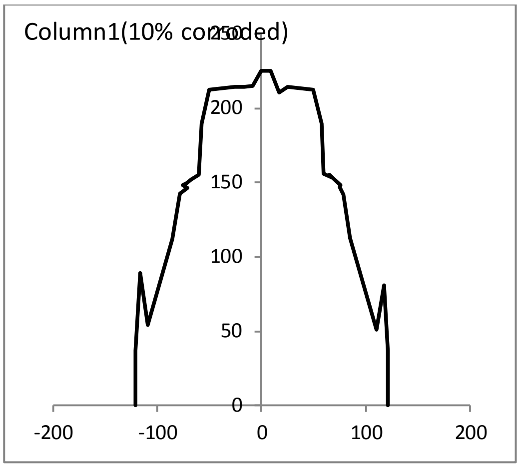

Column12
180
160
140
120
100
80
60
40
20
0
-150
-100
-50
0
50
100
150

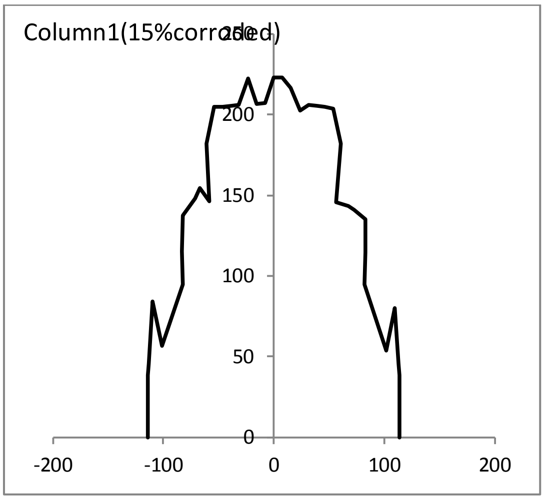

Column13
180
160
140
120
100
80
60
40
20
0
-150
-100
-50
0
50
100
150

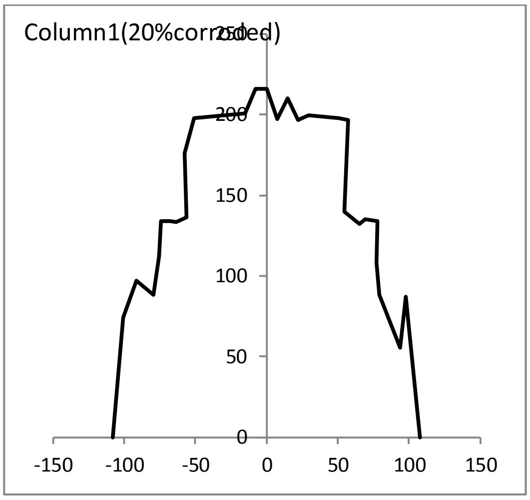

Column14
160
140
120
100
80
60
40
20
0
-100
-50
0
50
100

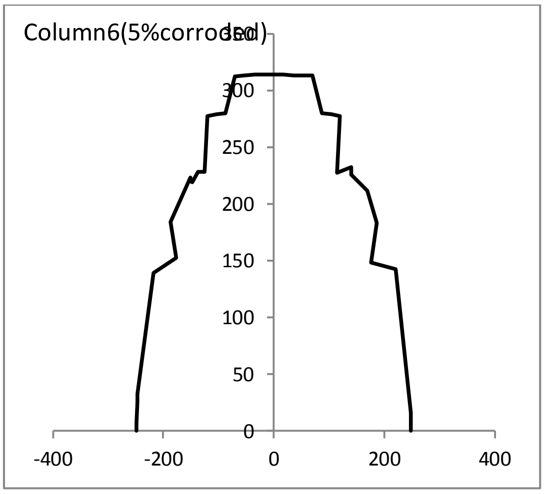

Column15
140
120
100
80
60
40
20
0
-100
-50
0
50
100

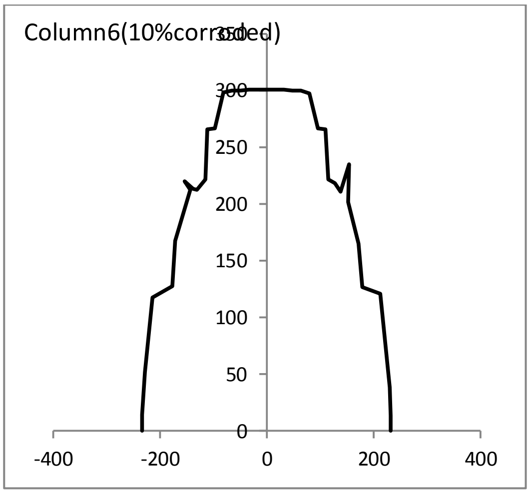

Column1(5%corroded)
250
200
150
100
50
0
-200
-100
0
100
200

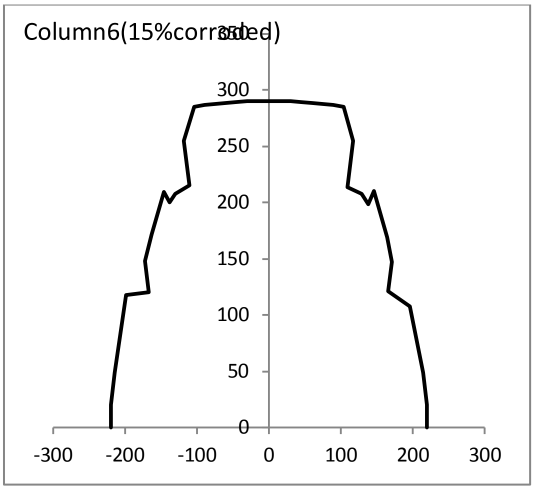

Column1(10% corroded)
250
200
150
100
50
0
-200
-100
0
100
200

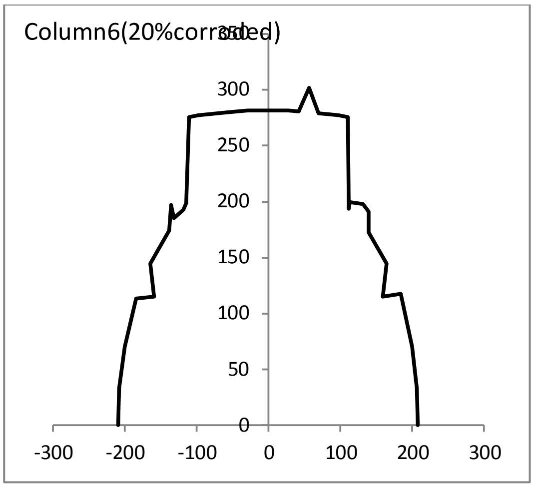

Column1(15%corroded)
250
200
150
100
50
0
-200
-100
0
100
200

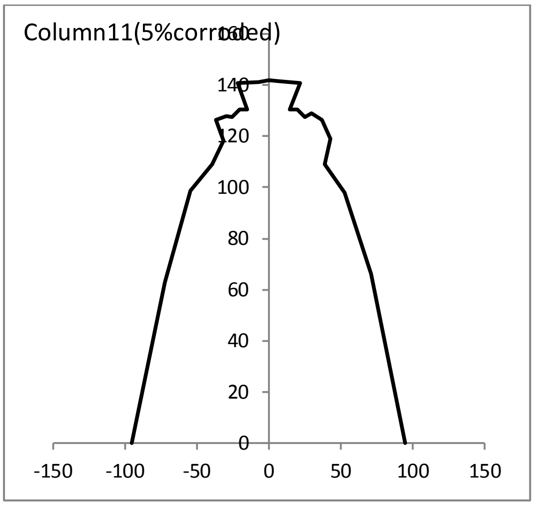

Column1(20%corroded)
250
200
150
100
50
0
-150
-100
-50
0
50
100
150

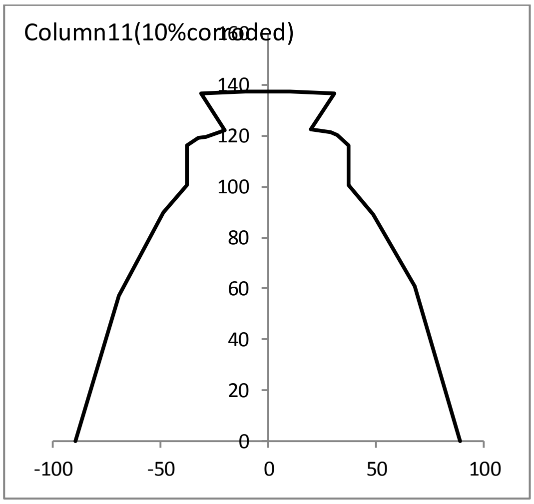

Column6(5%corroded)
350
300
250
200
150
100
50
0
-400
-200
0
200
400

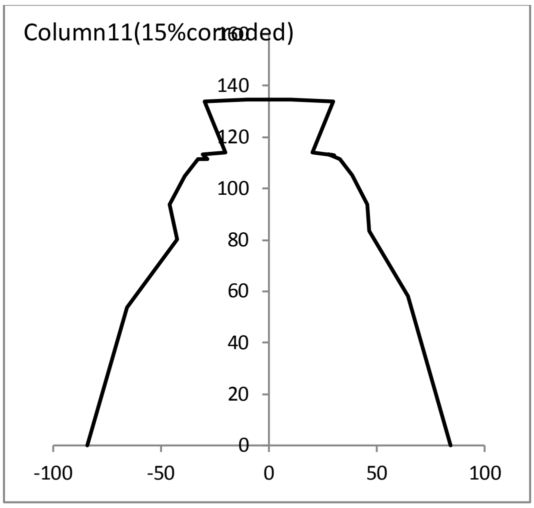

Column6(10%corroded)
350
300
250
200
150
100
50
0
-400
-200
0
200
400

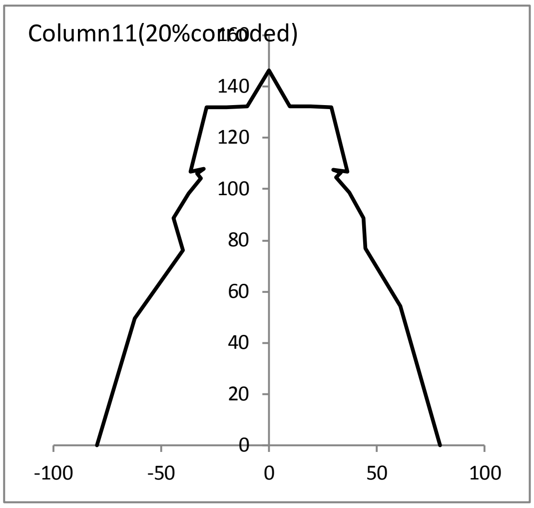

Column6(15%corroded)
350
300
250
200
150
100
50
0
-300
-200
-100
0
100
200
300


Column6(20%corroded)
350
300
250
200
150
100
50
0
-300
-200
-100
0
100
200
300


Column11(5%corroded)
160
140
120
100
80
60
40
20
0
-150
-100
-50
0
50
100
150

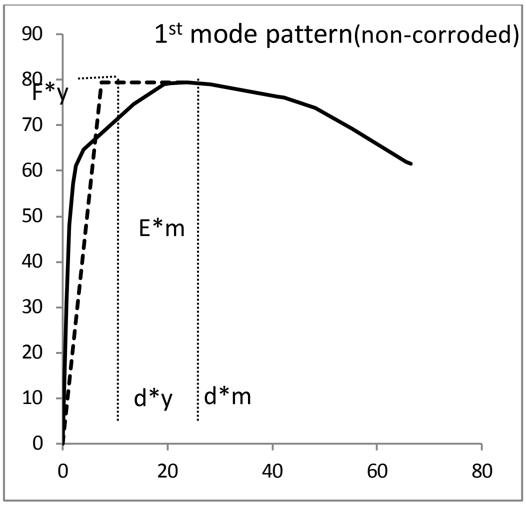

Column11(10%corroded)
160
140
120
100
80
60
40
20
0
-100
-50
0
50
100

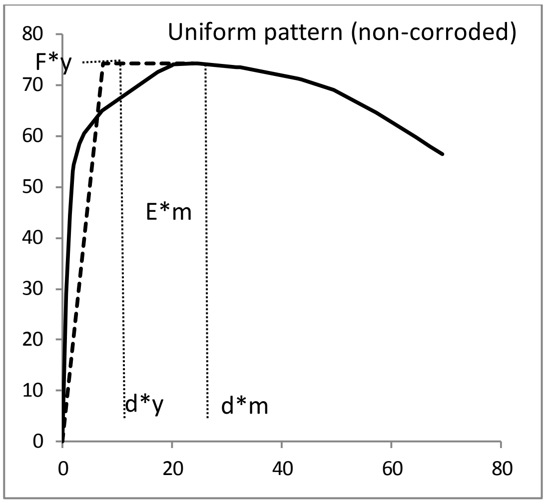

Column11(15%corroded)
160
140
120
100
80
60
40
20
0
-100
-50
0
50
100

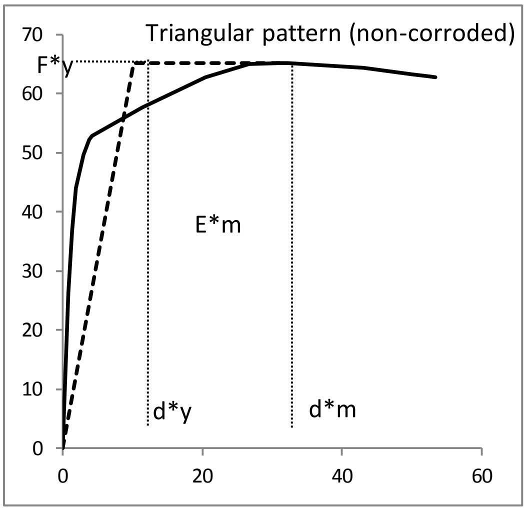

Column11(20%corroded)
160
140
120
100
80
60
40
20
0
-100
-50
0
50
100

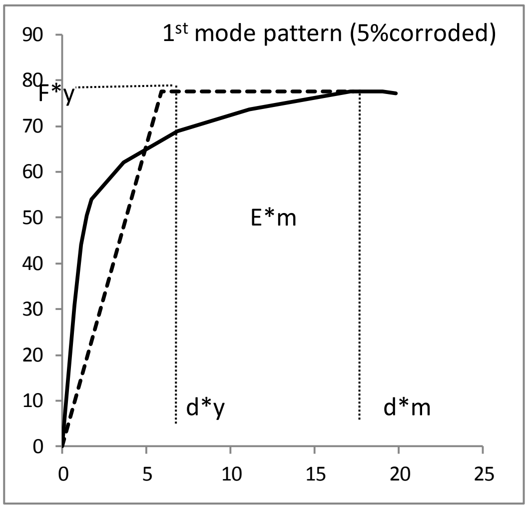
Appendix F: Idealized Force-Displacement Curves to Calculate Target Displacement According to Eurocode8

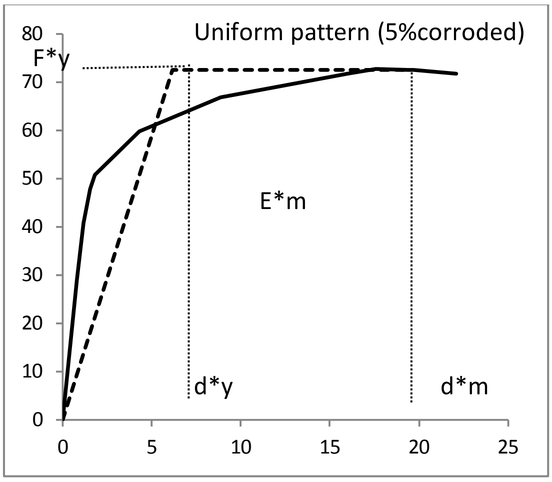
Horizontal axis represents displacement in (mm) and vertical axis represents force in (kN).

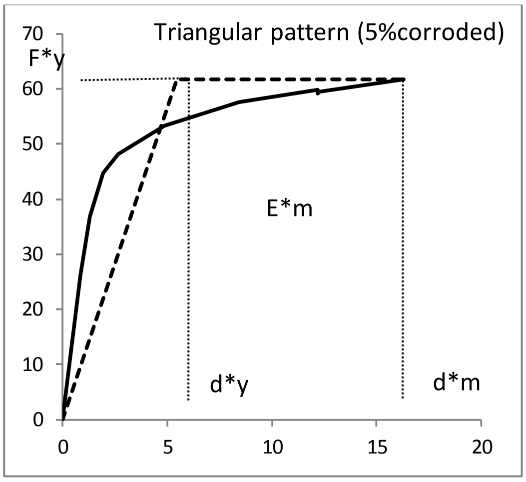


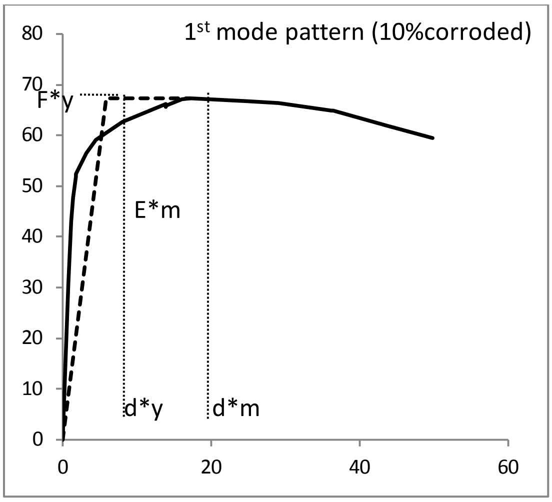


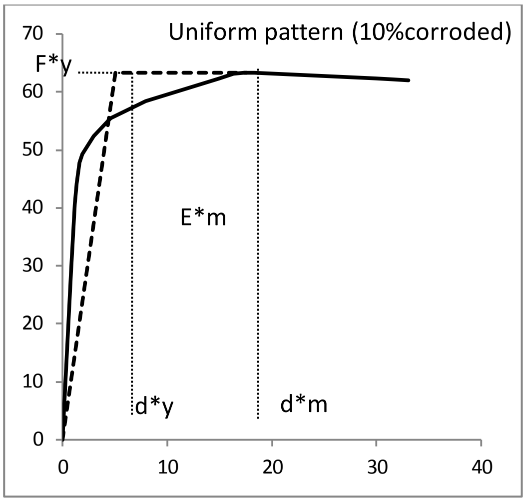


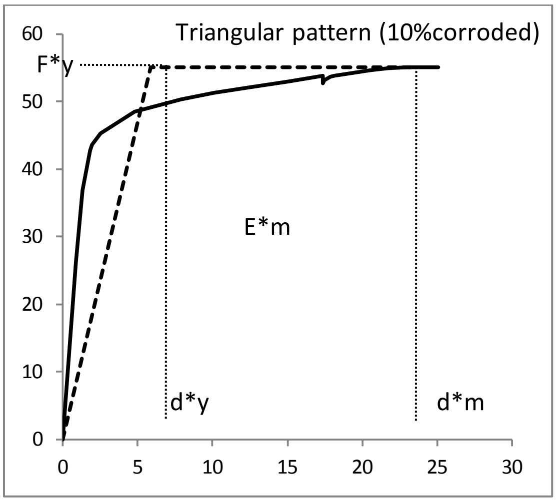


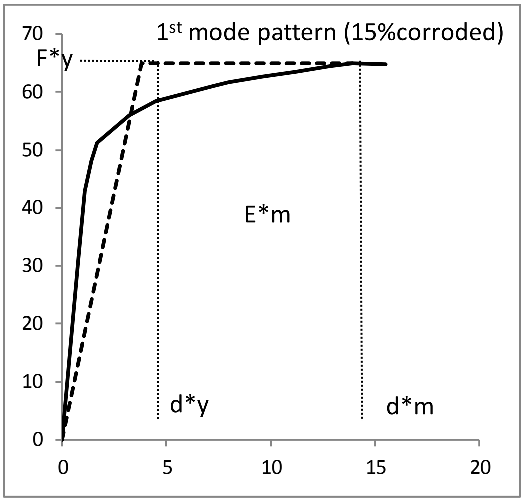


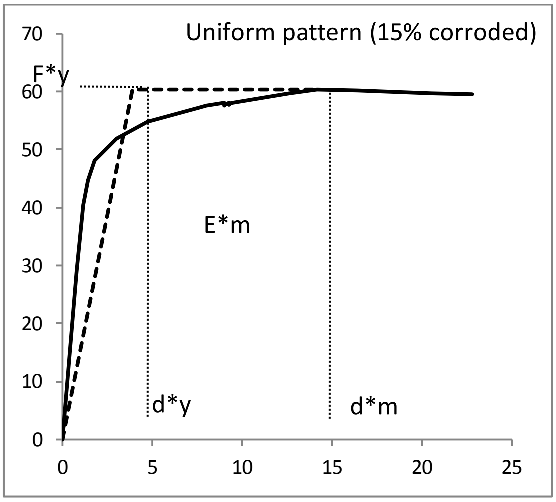


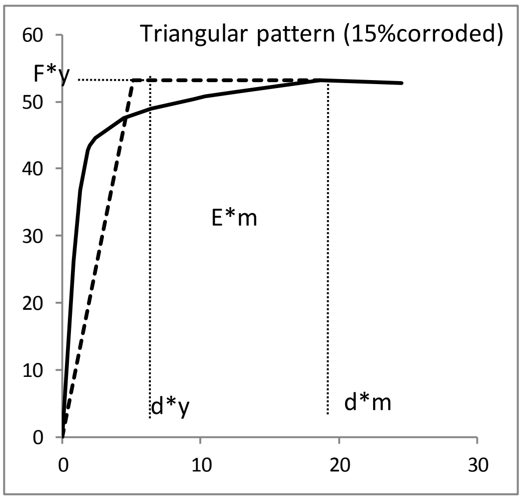

1st mode pattern (10%corroded)
F*y
E*m
d*y
d*m
0
10
20
30
40
50
60
70
80
0
20
40
60

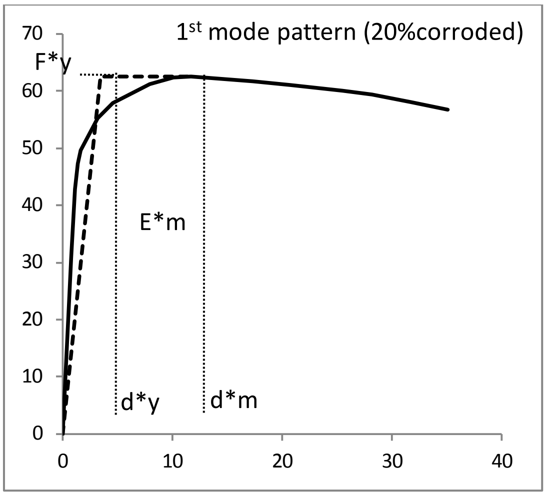

Uniform pattern (10%corroded)
F*y
E*m
d*y
d*m
0
10
20
30
40
50
60
70
0
10
20
30
40

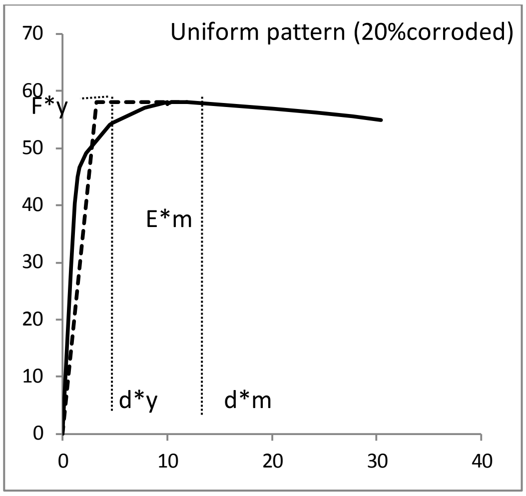

Triangular pattern (10%corroded)
F*y
E*m
d*y
d*m
0
10
20
30
40
50
60
0
5
10
15
20
25
30

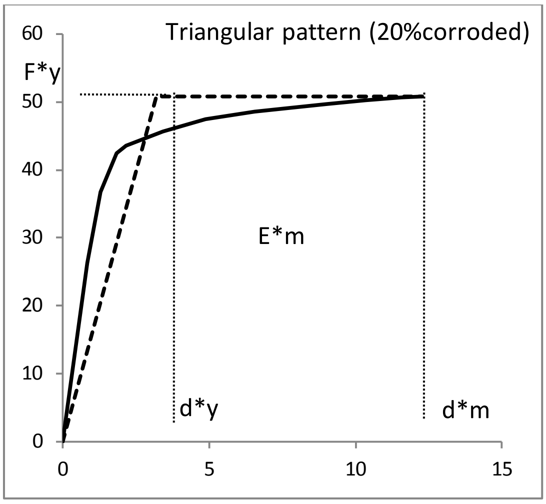

1st mode pattern (15%corroded)
F*y
E*m
d*y
d*m
0
10
20
30
40
50
60
70
0
5
10
15
20


Uniform pattern (15% corroded)
F*y
E*m
d*y
d*m
0
10
20
30
40
50
60
70
0
5
10
15
20
25


Triangular pattern (15%corroded)
F*y
E*m
d*y
d*m
0
10
20
30
40
50
60
0
10
20
30


1st mode pattern (20%corroded)
F*y
E*m
d*y
d*m


Uniform pattern (20%corroded)
F*y
E*m
d*y
d*m


Triangular pattern (20%corroded)
F*y
E*m
d*y
d*m

Appendix G: The Building Plans