ISTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

OPTIMUM DESIGN OF THE R/C FRAMES WITH ROTATION CONSTRAINTS

M.Sc. Thesis by

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Department : Civil Engineering

Programme : Structural Engineering

JULY 2014

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

BETONARME DÜZLEM ÇERÇEVELERİN BİRİNCİ MERTEBE LİMİT YÜKE GÖRE OPTİMUM BOYUTLANDIRILMASINDA DÖNME KISITLAMALARININ GÖZ ÖNÜNE ALINMASI

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Armin-tayyebiazar, a **M.Sc.** student of ITU **Graduate School of structure engineering** student ID **501111006**, successfully defended the **thesis** entitled "**OPTIMUM DESIGN OF R/C FRAMES WITH ROTATION CONSTRAINTS**", which he prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

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Date of Submission : 05 May 2014
Date of Defense : 01 July 2014

To my parents,

FOREWORD

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To my sister and brother in law who has convinced me to go this far and continue my studies in a foreign country.

June 2014

Armin TAYYEBIAZAR
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TABLE OF CONTENTS

ABBREVIATION

AASHTO	**:** American Association of State Highway and Transportation Officials
ACI	**:** American Concrete Institute
AS3600	**:** Australian Standards-Concrete Structure
ATC	**:** Applied Technology Council
FORTRAN	**:** Formula-Translating System
LRFD	**:** Load and Resistance Factor Design
RBDO	**:** Reliability Based Design Optimization
SAP2000	**:** Structural Analysis Program

LIST OF TABLES

Page

LIST OF FIGURES

Page

OPTIMUM DESIGN OF THE R/C FRAMES WITH ROTATION CONSTRAINTS

SUMMARY

In recent years, there have been many improvements in civil and earthquake engineering. Efficient structural design targets to verify the codes and at the same time to make the project be an economical design. One of the way to achieve this goal is structural optimization method.

In this thesis, a successive approximation method is proposed for the optimum design of reinforced concrete plane frame structures according to first order limit load.In the first chapter of this thesis, overall introduction of the subject will be stated. In this chapter, the target andscope of the thesis, the subject matter and conclusions of a literature research are given.

An economical solution in optimum design problem can be achieved by optimizingthe structural weight for a limit or a collapse load. In this kind of problem, structuralweight can be expressed in terms of the section properties that chosen as designvariable. The constraints of optimum design problem may consist of yield conditions, displacements, deformationsand some section properties.

Successive approximation method that each step consists of design and pushover analysis phasesare proposed for optimum design of the reinforced concrete plane frame structures of this thesis. In the proposed method, plastic hinge rotation limitations and some cross-section dimension limitations may considered together with the yield condition constraints, which include equilibrium and geometric compatibility conditions.

In the second chapter, the basis of the successive approximation method, which is adopted to the optimum design of the reinforced concrete frame structure according to first order limit load, are explained.

Assumptions of the method, formulation of yieldconditions, and objective function of the optimizationproblem are explained also in this section.

The basis of the successive approximation method for the optimum design of reinforced concrete space frame structures according to the first order limit loadas cited by Özer (1975) was established firstly using matrix force method, and thenaccording to Orakdöğen (2002), in this method was innovated implementing matrix displacement method for space frame steel structures. After that, in a researchdocument according to Orakdöğen (2002), somesystematic changes was made for the method in order to simplify the methodfor the optimum design of plane frame structures. In this thesis, this method with some modification is developed in order toapply the optimum design procedure to reinforced concrete planeframe structures by using existing structural analysis programs and excel sheets. In order to

use the method for the optimumdesign of reinforced concrete plane frame structures, some additions and newmathematical formulations especially for the yield conditions constraints and sectionproperties formulations are taken into account in this thesis.

Structural weightis the objective functions of the optimum designproblem. The objective function in other word structural weight in this thesis should be expressed in terms of design variables. Here, the structural weight is expressed in terms of plastic moments of the sections.

The weight of the independent r/c cross-sections are expressed by nonlinear functions of plastic moments. For this purpose, geometrical steel ratios and the ratio of d/b are taken as constant and the height of the r/c cross-section is taken as variables.

In this study, bending moments in critical sections are evaluated with superposition of analysis results for external loads and unit plastic hinge rotationsseparately and respectively. The values, which are obtained in this way, are used in proposed successive approximation method in order to achieve the optimal design of the reinforced concrete frame according to first order limit load.

For this purpose, the system is analyzed for external loads and for unit values of plasticrotations at the plastic sections by SAP2000, respectively. Then the results at the criticalsections superposed using the suggested superposition formulations in order toconstruct the yield condition constraints.

Numerical examples to illustrate the successive approximation method for the optimumdesign of reinforced concrete plane frame structures arepresented in the third chapter. A single-story, single-bay reinforced concrete plane frame, a two-story, two-bay reinforced concrete plane frame and a two-story, single-bay reinforced concrete plane frame are analyzed in this study. Theseexamples are studied to design for different optimum design approaches by using theproposed method.

The last chapter of the thesis includes conclusions. Results of the numericalexamples of the proposed successive approximation method are concluded.

BETONARME DÜZLEM ÇERÇEVELERİN BİRİNCİ MERTEBE LİMİT YÜKE GÖRE OPTİMUM BOYUTLANDIRILMASINDA DÖNME KISITLAMALARININ GÖZ ÖNÜNE ALINMASI

ÖZET

Son yıllarda yapı ve deprem mühendisliğindeki gelişmeler yapı sistemlerinin davranışını gerçeğe daha yakın olarak dikkate alabilme ve modelleyebilme konusunda birçok yeni yaklaşımı beraberinde getirmiştir. Yapı tasarımında amaçlanan, yönetmeliklerin gerektirdiği seviyede güvenli ve aynı zamanda ekonomik yapı tasarımıdır. Bu doğrultuda yapılan araştırma konularından biri de yapısal optimizasyon yöntemleridir.

Doğrusal olmayan malzemelerden yapılmış olan sistemlerde artan yükler altında iç kuvvetler artıyor ve iç kuvvetler doğrusal elastik sınırlarına ulaştığınızda, doğrusal olmayan plastik deformasyonlar oluşabilir. Doğrusal olmayan deformasyonlar genelde tüm sisteme yayılır. Fakat toplam deformasyonun doğrusal deformasyona oranının yüksek olduğu sünek malzemelerden yapılmış sünek yapı sisteminde, doğrusal olmayan deformasyonlar plastik mafsalla adlandırılan bölümlerde toplanmış olduğu kabul edilebilir ve sistemin diğer bölümlerinde doğrusal bir davranış olduğu varsayılır. Bu varsayım "plastik mafsal hipotezi" ve bu hipotezin geçerli olduğu birinci mertebe teorisine göre sistem analizinde, sistemin tümünü veya bir kismini mekanizma durumuna neden olan yüke "birinci mertebe limit yük" denilir.

Optimizasyon yöntemleri genel olarak, matematik programlama teknikleri ve optimumluk kriteri teknikleri olarak iki gruba ayrılmaktadır. Matematik programlama teknikleri de kendi içinde lineer programlama problemleri ve lineer olmayan programlama problemleri olarak ikiye ayrılır. Burada lineer olup olmama durumu, problemin kısıtlamalarının ve amaç fonksiyonunun tasarım değişkenleri cinsinden lineer bağıntılarla ifade edilip edilmemesine bağlıdır.

Bu çalışmada önerilen optimizasyon yönteminde, yapının ağırlığı kesitlerinin plastik momentine bağlı olan amaç fonksiyonu olarak belirtilmiştir. Bu nedenle, betonarme kirişlerin ve sütunlarin boyut özellikleri arasındaki korelasyonlar ayrıntılı olarak ifade edilir ve genişlik, yükseklik, en kesit alanı, ve atalet momenti kesitlerinin plastik momenti ile ilgili bir parametre olarak ifade edilmıştir. Seçilen ile ilgili betonarme yapının boyutları arasındaki ilişki elde edilip, ve gelecek bölümlerde belirtilmiştir.

Lineer olmayan programlamaya dayanan yapı optimizasyonu problemlerinde, tasarım değişkenleri sayısının artması problemin çözümünün yakınsaklık hızını ve güvenilirliğini olumsuz yönde etkilemektedir. Bu olumsuzluğu gidermek amacı ile optimumluk kriteri yöntemleri geliştirilmiştir. Ardışık yaklaşımla optimum çözümün arandığı bu yöntemler, hem değişken sayısından bağımsızdır hem de basit bir algoritma ile ifade edilebilmektedir.

Geometri değişimlerinin denge denklemlerine etkisi terk edilir ve akma koşulu iç kuvvetlerin lineer bir fonksiyonu olarak ifade edilirse, limit yük için minimum ağırlıklı boyutlandırma amaçlı optimizasyon problemi, lineer programlama problem olarak çözülebilmektedir. Bunun sebebi, bu durumda birinci mertebe limit yükün mekanizma yüküne eşit olması ve sistemin mekanizma durumuna gelmesine neden olan iç kuvvet durumunun sadece denge denklemleri kullanılarak hesaplanabilmesidir.

Yapı mühendisliği problemlerinde uygulamada işletme ve hesap yükleri altında, düğüm noktalarının yer değiştirmelerinin ve plastik kesitlerdeki plastik şekil değiştirme parametrelerinin söz konusu yükler için öngörülen sınır değerleri aşmaması, enkesit karakteristiklerinin verilen sınır değerlerden küçük olmaması veya enkesit karakteristikleri ile ilgili birtakım özel kısıtlamalar (alt kat kolonlarının enkesit boyutlarının üst kat kolonlarının enkesit boyutlarından küçük olmaması gibi) istenebilir

Bu tez çalışmasında, betonarme düzlem çerçevelerini birinci mertebe limit yüke göre optimum boyutlandıran bir ardışık yaklaşım yöntemi önerilmiştir. Göçme yüküne veya limit yüke göre amaç fonksiyon olarak seçilen yapı ağırlığını optimum yapan çözümün en ekonomik çözüm olarak kabul edildiği bir optimum boyutlandırma probleminde yapı ağırlığı, tasarım değişkeni olarak seçilen enkesit karakteristiklerinden biri cinsinden ifade edilebilir. Çözümün sağlaması gereken akma koşulları, denge koşulları, geometrik uygunluk koşulları ile yer değiştirmeler, şekil değiştirmeler ve enkesit karakteristiklerine ait sınırlamalar optimum boyutlandırma probleminin kısıtlamalarını oluştururlar.

Geliştirilen yöntemde, betonarme yapıların taşıyıcı sistem tasarımı ile ilgili istenen kısıtlamalara ilave olarak, ulusal ve uluslararası deprem yönetmeliklerinde öngörülen hasar ve performans seviyelerine bağlı olarak plastik şekil değiştirme kısıtlamaları da göz önüne alınarak optimum boyutlandırma yapılabilmektedir.

Bu çalışmada, kritik kesitlerde eğilme momentleri ayrı ayrı ve sırasıyla dış yükler ve birim plastik dönmeler için analiz sonuçları süperpozisyon ile değerlendirilir. Bu şekilde elde edilen değerler, betonarme çerçevesinin birinci mertebe limit yüke göre en iyi tasarımı elde etmek için önerilen ardışık yaklaşım yönteminde kullanılır.

Bu tezin betonarme düzlem çerçeve yapıların optimum tasarımı için önerilen Ardışık yaklaşım yönteminın her adımı tasarımı ve itme analizi aşamasından oluşmaktadır. bu amaç için, sistem SAP2000 programıyla sırasıyla dış yükler ve birim plastik dönmeler için analiz edilir. Sonra kritik bölümlerdekı sonuçları akma koşulları kısıtlamalarını oluşturmak amacıyla önerilen süperpozisyon formülasyonlar kullanılarak süperpoze edilecektır. önerilen boyutlandırma yönteminde, kritik kesitlerdeki büyüklükler ve eğilme momentleri, düzlem çerçeve sisteminin sırasıyla dış yükler ve plastik kesitlerdeki birim plastik şekil değiştirmeler için ayrı ayrı analiz edilerek bulunmaktadır.

Tezin giriş bölümünü oluşturan ilk bölümünde konu tanıtılmış, konu ile ilgili yapılan literatür araştırmasının sonuçlarına yer verilmiş ve çalışmanın amacı ile kapsamından bahsedilmiştir. Tezin ikinci bölümünde, betonarme yapıların birinci mertebe limit yüke göre optimum boyutlandırılması için önerilen ardışık yaklaşım yönteminin esasları açıklanmış, yöntemde yapılan varsayımlar anlatılmış, problemin kısıtlamaları, akma ve denge koşulları, optimize edilecek amaç fonksiyon kavramları açıklanmıştır.

Tezin üçüncü bölümünde, geliştirilen ardışık yaklaşım yönteminin sayısal uygulamalarına yer verilmiş olup, tek katlı betonarme düzlem çerçeve ve çok katlı çok açıklıklı betonarme düzlem çerçeve örnekleri üzerinde farklı yaklaşımlar ile çeşitli optimum boyutlandırma hesapları yapılmış ve ulaşılan sonuçlar açıklanmıştır.

Tezin son bölümü olan dördüncü bölümde, sonuçlara yer verilmiş olup, tez kapsamında geliştirilen ardışık yaklaşım yöntemi ve bu yöntemin sayısal uygulamaları neticesinde ulaşılan sonuçlar yorumlanmıştır.

Bu çalışmada önerilen optimizasyon yönteminde, yapının ağırlığı kesitlerinin plastik momentine bağlı olan amaç fonksiyonu olarak belirtilmiştir. Bu nedenle, betonarme kirişlerin ve sütunlarin boyut özellikleri arasındaki korelasyonlar ayrıntılı olarak ifade edilir ve genişlik, yükseklik, en kesit alanı, ve atalet momenti kesitlerinin plastik momenti ile ilgili bir parametre olarak ifade edilmiştir. Seçilen ile ilgili betonarme yapının boyutları arasındaki ilişki elde edilip, ve gelecek bölümlerde belirtilmiştir.

1. INTRODUCTION

In recent years, there have been many improvements in civil and earthquake engineering. Efficientstructural design targets to verify the codes and at the same time make the project be an economical design.One of the ways to achieve this goal is structural optimization method.

In general, optimization methods divide into mathematic programming technics and optimality criteria technics.Mathematic programming technics also divide into linear and nonlinear programming problems that depend on constrains of problems and design variables of objective function expressed in linear or nonlinear terms.

According to the first order elementary theory of plasticity in which the plastic hinge concept is valid, as the structure is statically determinateat the incipient collapse state, internal forces of the frame can be calculated by applying only the equilibrium equations.

The best economical result in an optimal design problem is achieved when the weight of structure is minimum and the collapse load is equal to limit load or mechanism load. Thus, weight of structure can be stated as one of chosen design variables such as cross section characteristics. If the yield constrains and the weight of structure are expressed linearly in terms of the design variables, the minimum weight design process can be converted into a linear programming problem that result minimum weight design of structure.

In a structural system, nonlinear behaviors are generally due to two reasons:

1. As the materials are not linear elastic, stress-strain relations are nonlinear.
2. Because of geometrical changes element axes due to the second order (P-Δ) effects, the equilibrium equations are nonlinear.

Systems that are made of nonlinear materials, under excessive loads, internal forces are increasing, and when internal forces reach the linear elastic limits, nonlinear plastic deformations can occur. Nonlinear deformations generally spreads overall system, but in ductile structural system that made of ductile materials as the ratio of total deformations to lineardeformationis high, it can be assumed that nonlinear deformations are gathered in sections so called plastic hinges, out of these areas system assumed to have linear behavior. This assumption is called “plastic hinge hypothesis” andstructural analysis according to first-order theory in which this hypothesis is valid, the load that causes the total system or just a part of it to become mechanism system stateis called“first-order limit load”.

While effects of changes in geometry to equilibrium equation are abandoned and yield constraintsare expressed as linear functions, equilibrium equation and weight of structure can be solved as linear programing.

Regarding optimal design of reinforced concrete frame with rotation targets to state the weight of structure as a parameter related to the cross section plastic moment (M_P) and constraintsconform to ATC-40 requirements. Thus, in plane structural systems made of columns and beams, cross sectional characteristics such as width (b), height (h), section area (F),and moment of inertia (I),are stated as parameters related to cross section plastic moment (M_P).

1.1 Purpose of Thesis

This study targets to develop a method for optimum design of reinforced concrete plane frames according to first-order limit load with and rotation constraints.

In this study, the matrix displacement method that has been proposedaccording toÖzer (1975), and the minimum weigh design method for plane frame systems, which has been stated byOrakdöğen (2002), is implemented to develop a method for optimum design of reinforced concrete plane frames according to the first-order limit load.

In the developed method, in addition to equilibrium, geometric compatibility and yield conditions, the plastic rotation limitation for different performance levels, whichare given in the earthquake codes, are also considered.

This study attempts to extend the existing formulation with respect to first-order limit load of reinforced concrete structures.

In the study, nonlinear weight function is expressed in terms of plastic moments of the r/c sections and constants of the problem are constructed by SAP2000. In design phases, an excel macro sheet is modified for the structural optimization and it is used for the linear programming problem. In pushover analysis phase, again, SAP2000 structural analysis program is used for obtaining the plastic hinge pattern. The successive approximation process is ended when similar cross-section and plastic hinge pattern is obtained in two successive step.

For this purpose, width (b), height (h), cross section (F) and moments of inertia (I) as cross-sectional characteristics of columns and beams are expressed depending on plastic section moment (M_P). Constraints of problem including plasticrotation limitationconform to ATC-40 requirements.

Since steel is a homogenous material, using analytical methods, the objective function can be established by using plastic moments easily. However, in reinforced concrete structures, it is essential to consider both concrete and reinforcement to construct a relationship between F and M_P. Compared to steel structures, this condition is slightly more complex, and the primary problem is to solve the section capacity arisen by reinforced concrete members. The relationships between cross-sectional areas and plastic moments for r/c members are given in section 2.3.

1.2 Literature Review

In this chapter, literature that summarizes the results of previously conducted research about optimum design of reinforcement concrete structures will be discussed.

According to Hassanian (1992), for concrete frames optimization, concrete dimension for columns and beams are the design variables. For each story, the design variables pertaining to o the concrete sections are linked, meaning that the column widths are assigned same design variables as well as each of the column depths, beam widths, and beam depts. The objective function is the sum of all the costs for each column and beam. Constraints consist of requirements of the ACI Code, and explicit bounds on the design variables.

Özer (1975) developed a method for optimum weight design of structure based on second-order limit load in hisassociate professor thesis. The technique that has been utilized in his method will be implemented also in this study.

KanagasundaranandKarihalaoo (1991) developed an optimization method for optimal and most economical design of reinforced concrete plane frame according to constraints consist of requirements of the Australian design standards (AS3600-1988). Total structure cost consists of reinforcement of concrete and cost of mold are chosen as objective function.

Ganzerli, Pantelides and Reaveley (2000) utilized performance-based design method for reinforced concrete plane frame minimum cost optimization. Cross sections of beams and columns and reinforcement percentage are chosen as design variables. Constrains are chosen according to plastic rotations of ends of beams and columns that relate to chosen performance level.

Zou, Chan, Liand Wang (2007) examined multi-purpose optimizations of performance-based designs of reinforced concrete frames. They targeted to develop a method to optimize the total cost of construction of building life, which includes cost of materials in structural design and cost of the damages due to possible earthquakes along buildinglifetime.

Lee andAhn (2003) utilized generic algorithm advantages for optimization method of reinforced concrete plane frame under horizontal and vertical loads. To determinethe ratio between dimension of beams and columns cross sections and percentage of reinforcement, they chose the values that are used commonly in practice.

Camp, Pezeshkand Hansson (2003) developed adesign procedure implementing a genetic algorithm for discrete optimization of reinforced concrete frames. The design procedure conforms to the American Concrete Institute (ACI) Building Code and Commentary. The objective of the procedure is to minimize the material and construction costs of reinforced concrete structural elements subjected to serviceability and strength requirements described by the ACI Code.

According to Orakdöğen (1994), in order to minimize the weight in the second-order limited load of the space truss system a sequential approach method has been developed. In this method, every optimization step consists of both design and analysis. Each of non-linear optimization problems is simplified into a linear problem. In this proposed method, estimated axial forces, which are multiplied by load coefficients, made of design load and intensely depended on external loads, are used to linearize equilibrium equations, whilst yield constraints are linearized by assuming yield section flat and even. While sectional properties of the sequential approach in their every step are taken of the previous step, plastic hinges and plastic deformation vectors are determined by non-linear analysis of the designed system in the previous step. When the design variables or sequential structure weight get close to each other sufficiently, optimal solution is gained and the account is terminated. Using FORTRAN programing language in the numerical applications of the developed method, a computer program has been prepared. Developing the methodology, steel space frames have been taken into consideration.

Based on the researches of Hajek and Frangopol (1991), in the study of weight optimization of the shear wall systems; a computer program has been developed by using folded plate theory. In this program, tensions and displacement are considered as constraints and width of the shear walls are chosen as design variables.

According to DinnoandMekha (1993), in the study, which targets to optimize reinforced concrete plane frame with nonlinear materials, the amount of cross-sectional dimensions and reinforcement have been chosen as design variables of the problem. In this study minimum cost designare chosen as objective, function and a sequential unconstrained minimization technique as optimization method has been utilized.

According to Moharramiand Grierson (1993), for elastic optimization of reinforced concrete plane frame a method have been proposed. Cross-sectional width and height with the amount of reinforcement which have been selected as design variables in this study, only tension constraints has been taken into consideration and Optimality criteria method has been conducted as optimization technique.

Yang (1982) developed a discrete optimization method for reinforced concrete structures. Cross section dimension and amount of reinforcement are chosen as design variables. Designproceduresatisfies ACI 318-77 code requirement and total structure cost is chosen as objective function.

Guerra andKiousis (2006) developed a novel formulation targets to gain optimal design of multi-bay and multi-story reinforcement concrete plane frame. Design procedure confirms to ACI-2005 requirement code. A nonlinear programming algorithm method developed to achieve optimum cost of the reinforced concrete frames and for numerical solution, a MATLAB program has been proposed.

Chung and Sun (1994) proposed a method based on nonlinear theory for weight optimization of reinforcement concrete beams with nonlinear materials. In this study, the thickness of beams and amount of reinforcemethas been selected as design variables, constraints are chosen as stress, and displacement limits. The objective function are chose as weight of the beams.

Afonso, SienzandBelblidia (2005) have searched about structural optimization of free-shaped shells and variable thickness plates. They proposed a method in which they applied simultaneously the topology optimization and shape optimization procedure that result fully integrated design optimization tool to obtain optimum designs.

Lin and Frangopol (1996) developed a method for optimization highway bridge beams that satisfied AASHTO requirement codes for constraints and design method. Proposed method involves two optimization formulation, first formulation includes load resistance factor design (LRFD), and second one is based on reliability approachesmethod. Analysis has been conducted by reliable nonlinear computer program and the beams with T cross section are used in numerical examples.

Ling, Haukaas and Royset (2007) proposed a functional method to engineers for balancing construction cost and safety. They utilized OpenSees software in which they added variety of modifications to software algorithm by using direct displacement method. Numerical methods includes tree bay six-story reinforcement concrete plane frame.

AouesandChateauneuf (2008) using reliability-based design optimization (RBDO) method have done a study to equilibrate (balance) safety-cost relationship in the structures. The optimization aimed not the element but the system itself. The chosen aim looks for an optimal solution that provides security. Proposed methods were tested on the concrete plane frames.

According to Perea, Alcala, Yepes, VidosaandHospitaler (2008), to optimize the used reinforced concrete box-section bridge beams of the highway bridges in the proposed method, four separate formulations thatare called heuristic algorithm have been used to solve a problem with fifty design variables. In all formulations, using the same sample with 13 meters horizontal openings, the economy have been considered by gaining sections varying between % 1.4 - % 7.5.

According to Lagaros andPapadrakis (2007), in order to evaluate Eurocode-8 earthquake regulations, a multi-objective optimization method attempts to compare the three-dimensional structures confirmed by Eurocode-8 regulations. In multi-objective optimization method by considering the initial construction cost and Eurocode-8 earthquake regulations for seismic level, the method tries to optimize the maximum lateral displacement.

Chan andZou (2004) have proposed an optimization procedure, which consists of two, pages defining elastic design optimization and inelastic design optimization. Using the suggested optimization technique in a system sample, behavioral spectrum analysis and push over analysis have been performed. In the scope of the study, considering elastic displacement obtained from behavioral spectrum analysis and inelastic displacement obtained from non-linear static analysis (push over analysis), an assumption-using element resizing to debate design variable has been formed. As a method of optimization, optimality criteria approach was utilized. Applying the proposed method numerically, two ten-story reinforced concrete plane frames have been used.

Govindaraj and Ramasamy (2007) developed a method implementing generic algorithm to optimize reinforcement concrete plane frames. Constraints confirm to indian building code requirements. Total cost of building has chosen as objective function which involves concrete cost, reinforcement cost and mold cost altogether.

Orakdogen(2002) proposed a method to minimum weight design of plane frame that is based on matrix displacement method. In this study, bending moments of cross section in yield constraintsare stated as unit hinge rotations and the external loads. Conventional Simplex algorithm with some modification is used for the solution of linear programming problem. Because formulation is based on matrix displacement method, it may be simply adopted to the weight optimization of frames with displacement and rotation constraints.

2. MINIMUM WEIGHT DESIGN OF THE R/C FRAMES WITH PLASTIC ROTATION CONSTRAINTS

In this chapter, the method thatis proposed for minimum weight design of the reinforced concrete plane frames, which based on first-order limit load is given. In this study, a modified method is proposed for minimum weight design of reinforced concrete frames.

The method is basically similar to the previous studies according to Camp, Pezeshk and Hansson (2003) and Özer (1975). The basic differences from the previous methods are non-linear objective function, whichis constructed for r/c frames, the yield conditions that are constructed by SAP2000, and excel macro, which is used for the solution of linear programming problem and plastic rotation limitation.

In the optimization method that is proposed in this study, weigh of structure is stated as objective function that relates to plastic bending moment of the cross-sections. Therefore, detailed correlations between characteristics of dimension of the reinforced concrete beams and columnsare expressed, and dimension characteristics such as width, height, cross-sectional area, and moment of inertia will be stated as parameter related to plastic bending moment of cross-sections.

Relations between the dimensions of the reinforced concrete structure that are related to the selected reinforcement percentage are derived, and will be stated in upcoming chapters.

In the following sections, assumptions and formulation of the method are given in details.

2.1 Assumptions

- Materials are ideal elasto-plastic and plastic hinge hypothesis is valid. Nonlinear deformations are ghadered in plastic sections and except these areas, material behavior is linear elastic.

- Direction of the external loads does not vary during the system deformations.

- Bernoulli-Navier hypothesis is valid. Plane sections are accepted to remain plane after deformations.

- In order to make the yield constraints linear functions, yield surface is idealized to be composed of plane surface.

- Structural elements are assumed to have symmetric reinforcement sections and amount of reinforcement are constant along the length of elements.

- Impact of shear deformations to analysis results are abandoned.

- Non-linearity conditions depend on geometric variations are taken into account in equilibrium equations and are abandoned in geometrical compatibility conditions.

- Impact of second-order deformation to analysis results are abandoned.

- In order to obtain the plastic hinges formation only at the ends of the members and under concentrated loads, only concentrated loads are applied to systems and distributed loads are converted to equivalent concentrated loads.

- The frame members are straight and prismatic.

2.2 Successive Approximation Method

The frames, which are designed by first order limit load, should satisfy the following conditions:

- Equilibrium and the geometric compatibility conditions
- The yield conditions in the whole structure when the first-order limit loadis reached.
- Structural constraints related to cross-sectional dimensions and plastic hinge rotations

The proposed method is successive approximation method and it is comprised of optimum design and pushover analysis phases, since the objective function is non-linear and plastic hinge pattern is unknown in the beginning of the procedure. In the following sections, these phases are explained in detail.

2.2.1 Design and analysis phases

Any step of this iterative process is composed of minimum weight design and push-over analysis phases. A flowchart for the approximation is given in Figure 2.1. As seen in the flowchart, in every step, cross sections and plastic hinge patterns are taken from the previous step.

Successive approximation method that each step consists of design and pushover analysis phases are proposed for optimum design of the reinforced concrete plane frame structures of this thesis. In the proposed method, plastic hinge rotation limitations and some cross-section dimension limitations may considered together with the yield condition constraints, which include equilibrium and geometric compatibility conditions.

In order to optimize reinforced concrete frame based on the first-order limit load and selected reinforcement percentage in the proposed successive approximationmethod, structure weight which is chosen as objective function and expressed linearly depends on section plastic moment will be optimized.

In every step, optimally designed system at the previous step, will be analyzed by static pushover method to obtain plastic hinge pattern. At the optimum design phase, the cross-sectional properties and the linearized objective function from previous step and plastic hinge pattern from the analysis phase are used. At each step, non-linear real F-$M_P$relationship is linearized by Newton-Raphson method according to the plastic moments obtained in previous design phase.

Iterative approach as shown in the flowchart is used for obtaining the new cross-section dimension. When the design variables and objective function are similar at the two successive steps, the optimal solution will be obtained.

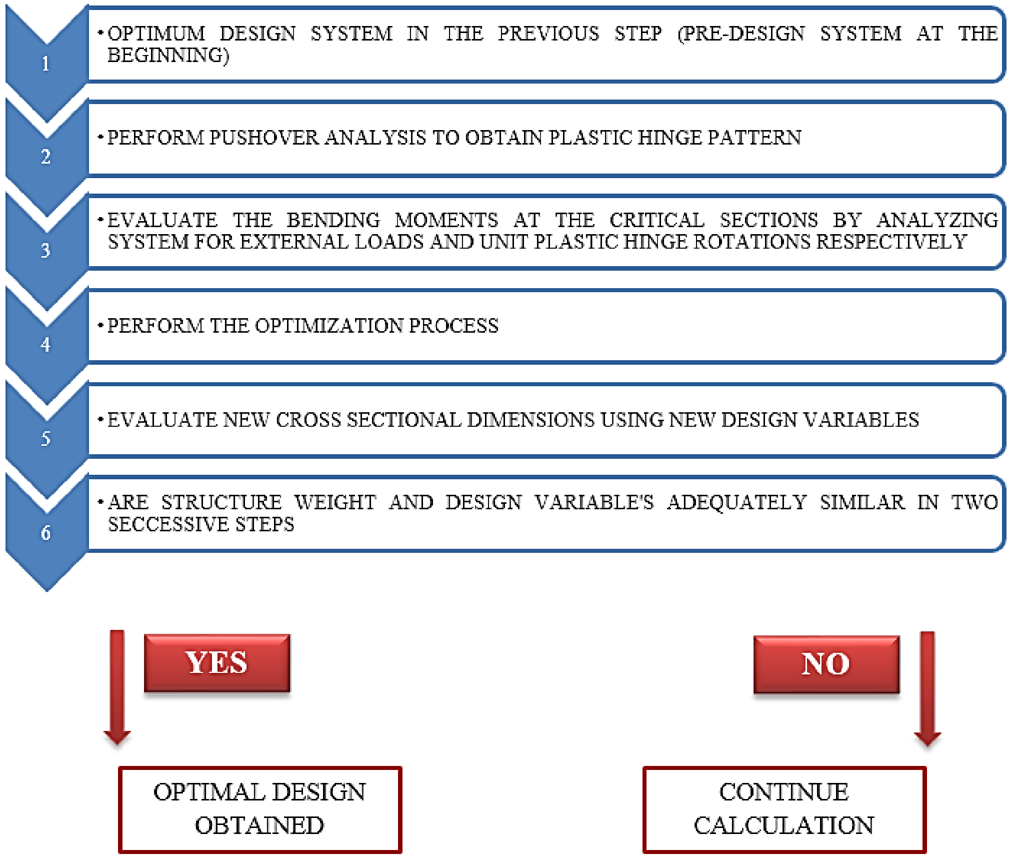


Figure 2.1: Optimum design and analyze steps.

2.2.2 Optimum design problems: yield constraints by superposition rule and additional constraints

This part is contained of three parts: Yield constraints, bending moments at critical sections and additional constraints.

2.2.2.1 Yield constraints

Yield condition constraints are inequalities that define the limits of the bending moment at a potential plastic hinge points, these limits depend on member plastic bending moment. Yield constraints in proposed optimum design method are related to members bending moment in one direction,

$$K\ (M_X) \leq 0 \qquad \textbf{(2.1)}$$

Constraints for a critical section on a plane frame memberwhen the axial force effects on yield condition is neglected can be stated as

$$|M| - M_P \leq 0 \tag{2.2}$$

Where, M is the bending moment of a critical section and $M_P$is the plastic bending moment of a critical section.

Inequality Eq. (2.2) results two conditions for a critical section because, the sign of bending moment are not known at the beginning of the optimization procedure. ThereforeEq. (2.2) can be rewritten as

$$M - M_P \leq 0 \tag{2.3}$$

$$-M - M_P \leq 0 \tag{2.4}$$

2.2.2.2 Bending moments at critical sections

According to the elementary theory of plasticity, a structure is statically determinate at incipient collapse state and that bending moment may be evaluated depending just on the equilibrium equations.

In the proposed method, values at the critical sections and bending moment value may be determined by analyzing the plane frame system respectively for external loads and unit rotation of the plastic sections.

The bending moment at critical section of the frame which is transformed into statically determinate or indeterminate system by introducing hinges, may be stated in terms of those introduced hinge rotations and external loads by superposition as

$$m_i = \sum_{j=1}^{n} (m_x)_{i(\Phi\, j=1)} . \Phi_j + (m_x)_i \tag{2.5}$$

In this expression,

N is the number of introduced hinges or degree of redundancy,

C is the number of critical sections,

$\Phi_j$is the rotation of *j*th introduced hinge,

$(m)_{i(\Phi j=1)}$is the bending moment at the *i*th critical section due to the unit value of $\Phi_j$introduced hinge rotation, and

$m_{0i}$is the bending moment at the *i*th critical section due to the external loads while the all introduced hinge rotations are equal to zero.

If the bending moment expression in (2.5) is written for all critical sections in matrix form, it results

$$[m] = [m_\Phi][\Phi] + [m_0] \qquad \textbf{(2.6a)}$$

$[m_\Phi]$ is a c×n matrix of bending moments due to the introduced hinge rotations. Any $m_{\Phi ij}$element of the matrix is equal to the bending moment at the *i*th critical section due to the Φ_j=1 rotation, while the other introduced hinge rotations and external loads are equal to zero,

$[\Phi]$ is a n×1 vector of introduced hinge rotations,

$[m_0]$ is a c×1 vector of bending moments due to the external loads while all introduced hinge rotations are zero.

If the equation, which is stated in (2.6), is replaced in (2.2), and if the group number of members, which consist of the same cross section, is stated *g,* the yield constraints represent in matrix form, are as follows;

$$[m_\Phi][\Phi] + [m_0] +[m_p]+ \leq 0 \qquad \textbf{(2.6b)}$$

In this expression;

$[m_p]$ is a g×1 vector of plastic bending moments of the group members.

In order to obtain yield constraints with applying superposition rule, the statically determinate or indeterminate frames analysis by the conventional matrix displacement method for (n+1) loading vector that is due to external loads and due to unit introduced hinge rotation is done. For this analysis the load vector is due to external loads can be done easily, only the fixed end forces due to the unit rotation of an introduced hinge located on any section of the prismatic frame member is given in Figure 2.2.

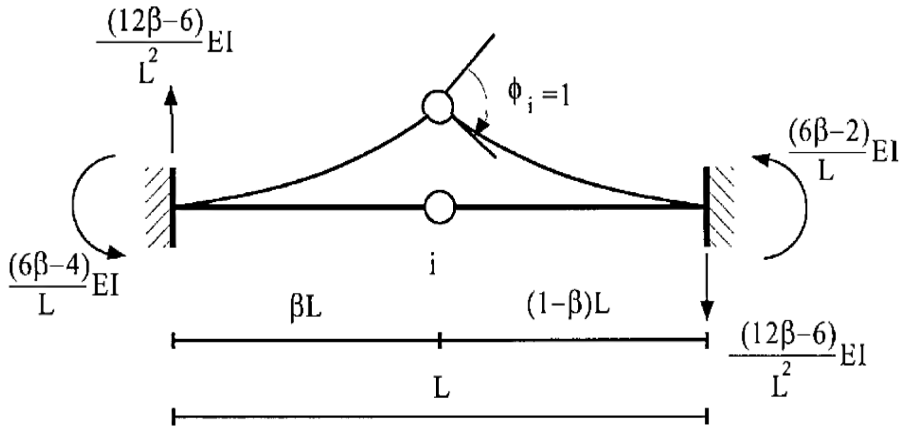


Figure 2.2: Fixed end forces of a prismatic member due to the unit introduced hinge rotation.

2.2.2.3 Additional constraints

In practical structural engineering problems, the minimum weight design procedure targets to minimize the weight function while satisfying the yield constrains. In some cases however, some constructive additional conditions such as plastic bending moment of the bottom story columns or beams are larger than those of upper story should be satisfied.

The limitation of nodal displacement or plastic hinge rotations are very important for the optimization of the frames which collapse before the mechanism load due to excessive displacement or plastic deformationof hinges.

The constructive constraints related to the displacements or plastic hinge rotations may be expressed as two inequalities for each of constraints.

$$\pm\delta - \delta_s \leq 0 \qquad \textbf{(2.7)}$$

Where,

δ is the plastic hinge rotation or displacement parameter that should be restricted,

δ_s is limit value for the stated constraintsand for the constructive constraints related to cross sectional characteristics following inequalities are stated;

$$-M_{pi} + (M_{pi})_s \leq 0 \qquad \textbf{(2.8)}$$

$$M_{pi} \leq M_{pk} \qquad \textbf{(2.9)}$$

In these expressions, $M_{pi}$and M_{pk} are plastic moment of member*i*and *k*, respectively $(M_{pi})_s$ is the limiting value for plastic moment M_{pi}.

2.3 Optimization of Objective Function

2.3.1 Weight function

This study targets to achieve optimal design of the reinforced concrete frames. As already mentioned in previous chapters, weight of structure is chosen as objective function. It will be described in detail how the weight of structure will be expressed as parameter that related to cross sectional characteristics and the relation between cross sectional characteristics of different cross section types will be stated.

If structure is consist of members with n different cross sections, and weight per unit volume of concrete assumed to be 25 kN/m^3, the weight of structure as objective function can be written as;

$$G = 25 \cdot \sum_{i=1}^{n} L_i \cdot F_i \qquad \textbf{(2.10a)}$$

In this expression,

G is the weight of structure (“objective function”),

Fi cross sectional area of the member with different cross sections,

Li is the total length of the members with Fi cross section areas.

Because the weight of structures are stated as a parameter related to plastic moment of cross sections, above expression can be revised as;

$$G = 25 \cdot \sum_{i=1}^{n} L_i \cdot F_i(m_{pi}) \qquad \textbf{(2.10b)}$$

Weight of structure in the expression (2.10b) can be expressed also as a matrix form that related to the design variables.

2.4 Relations Between the Cross-Sectional Dimensions and Plastic Moments

Targeting optimal design of reinforced concrete structures based on first-order limit load, in this study, structure weight is expressed by section plastic moments (M_P). Regarding this target, in plane frame systems, cross section areas of r/c columns and beams should be expressed in terms of member plastic moments.

Since steel members behave as homogeneous material, these equations may be obtained using analytical methods depending on plastic modulus of the section. For the r/c members, member plastic moments depend on the reinforcement together with the cross-sectional dimensions. In this thesis, for the constant steel ratios, F-$M_P$relationships are obtained. In the relationships, b/d ratios is taken as constant also.

2.4.1 Assumptions

Based on the internal force- deformation relationship in reinforced concrete frame members, these basic assumptions and principles are assumed.

a. After deformation, plane section remains the same

b. There is a full adherence between concrete and reinforcement

c. Tensile strength of the cracked concrete may be neglected.

For the σ-ε diagram, as shown in Figure 2.3, parabola and rectangular model are used.

For reinforced steelideal elasto-plastic, material is assumed as Figure 2.4 in the next page.

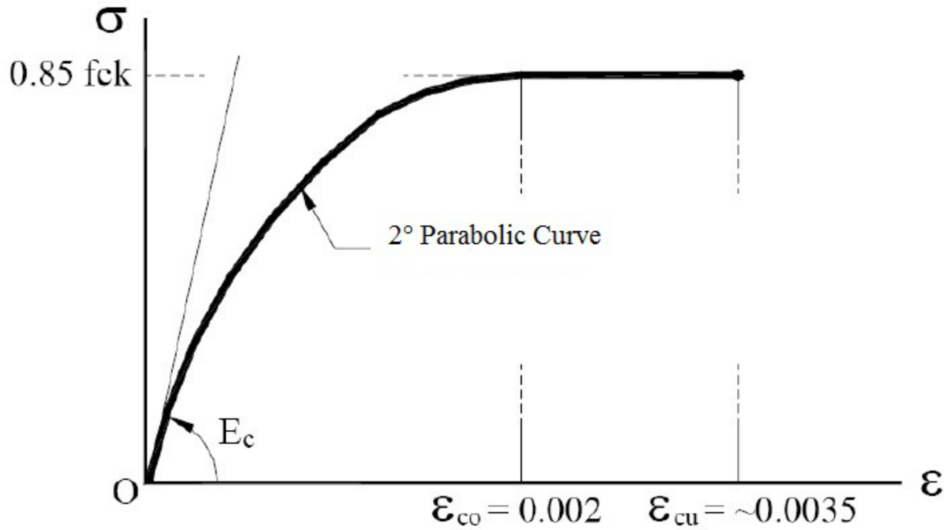


Figure 2.3: σ-ε diagram for reinforced concrete members' (flexural deformation).

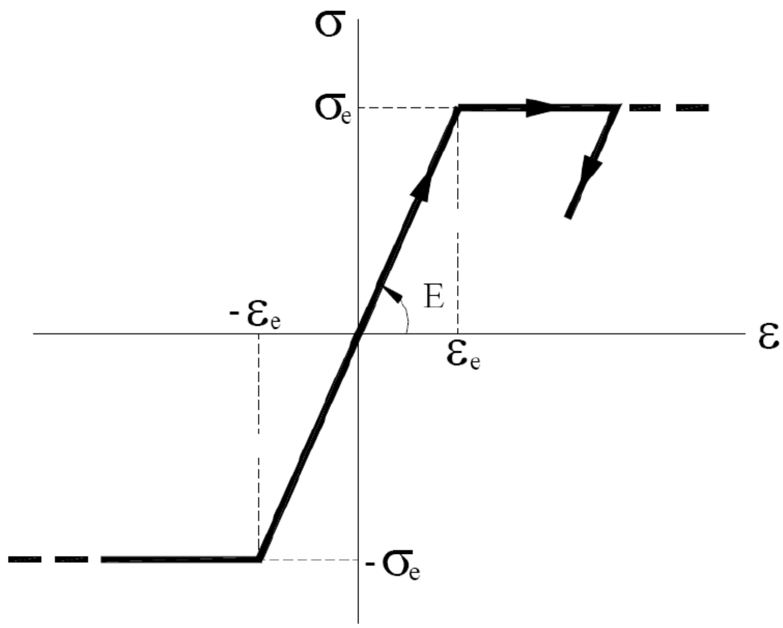


Figure 2.4: σ-ε diagram for reinforcement.

2.4.2 Reinforced concrete members under bending moment

M-χdiagram of reinforced concrete member affected by increasing bending moment is composed ofthree regions Figure 2.5.Conditions that explaining these regions (L_0, L_1 and L_2 points) are described below:

L_0: Beginning of the cracks at the extreme fiber of the tension zone of the concrete section. While the normal stress in bending of the extreme fiber in tension zone in concrete gets equal to tensile strength of concrete, appearance of the cracks in the concrete is acceptable. Tensile strength under bending moment of the concreteis calculated by the equation written below:

$$f'_{ctk} = .70x\sqrt{f_{ck}} \ \text{(N/ mm}^2\text{)} \qquad \textbf{(2.11)}$$

When M_{Lo}moments is against cracking point of L_o, concrete section is assumed homogenous and concrete σ - ε equation is considered linear-elastic.

$L_1$is a condition that plastic deformations begin in the extreme compression fiberorin tension reinforcements. Plastic deformation in concrete is considered as ε_{co}= 0.002 for every unit strain, while in steel $\varepsilon_e$is noticed as yield limitation begins.

Calculating M_{L1}bending moment, tensile strengthis not considered.

L_2: as bending moment increases and getting equal to the bending moment capacity of the section M_{L2}= M_P, concrete under the pressure gets crushed and cracked or its tension reinforcement losses the strength. Cracking of the concrete in compression zone happens when strain reaches to the value of ε_{cu} limit. For the sake of short-term loads in unconfined concrete, the limit value of $\varepsilon_{cu} \approx 0.0035$ considering the confinement, this valueis beingincreased.

In designing reinforced concretesections, usuallystrain of the steel is limited with the value of ε_{su}= 0.01.

Since steel members behave as homogeneous material, these equations may be obtained using analytical methods depending on plastic modulus of the section. For the r/c members, member plastic moments depend on the reinforcement together with the cross-sectional dimensions. In this thesis, for the constant steel ratios, F-M_P relationships are obtained. In the relationships, b/d ratios is taken as constant also.

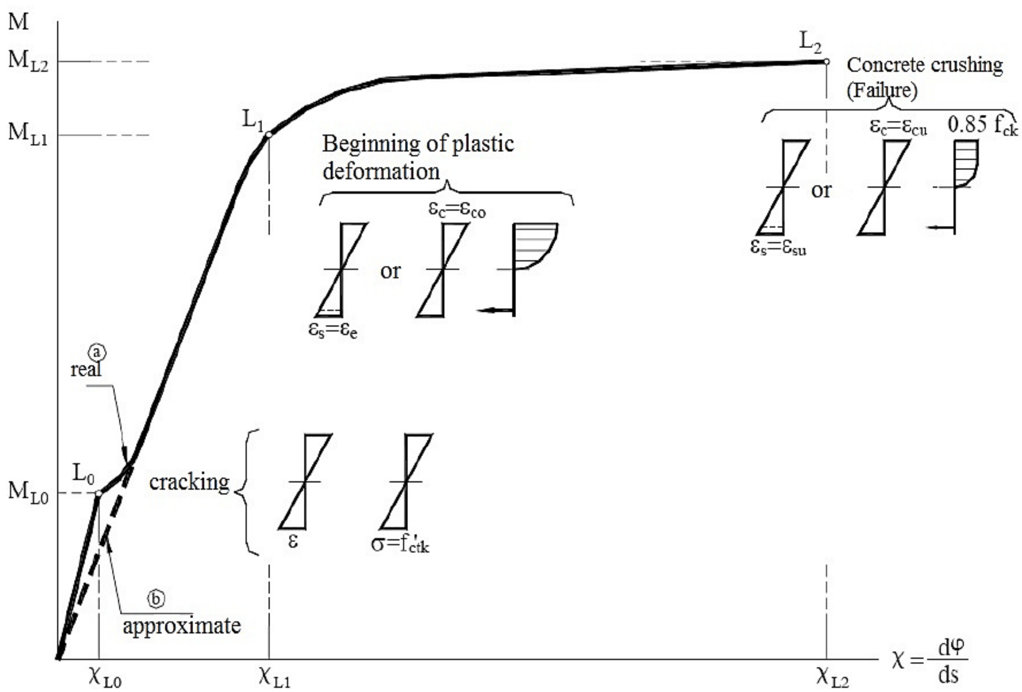


Figure 2.5: M-χ diagram of reinforced concrete members.

As neglecting concrete tensile strength, curve (b) approximately represents the relation between M–χ in the reinforced concrete section before the cracking occurs.

Designing reinforced concrete sectionsby means of load carrying capacities, concrete and reinforcement strength characteristics are decreased by dividingthese value by material safety coefficients. In contrast, in analyzing the behavior of reinforced concrete systems for limit loads, there is no need for consuming material safety coefficients and restricting steel strainconsidering ε_{su}= 0.01 value (Orakdöğen**,** 2002).

2.4.3 Idealization of reinforcement concrete member behavior

Two proposed models are explained below to idealize bending moment–curvature relation in reinforced concrete sections. The first type idealization is shown in figure 2.6, the relationship between M – χ which units O - L_1 - L_2 points is assumed to be composed of two segments. Generally, this method is used in systems which nonlinear deformations are assumed to spread constantly in the system members. In the second type idealization, the line comprised of two segment with coordinates of O as starting point L_1 with (χ_{L1}, M_{L2}) coordinate and L_2 with (χ_{L2}, M_{L2}) coordinate approximately produces M – χ relationship (Figure 2.7). This idealization is used in systems which nonlinear deformations are assumed to gather in plastic sections ("plastic hinges").

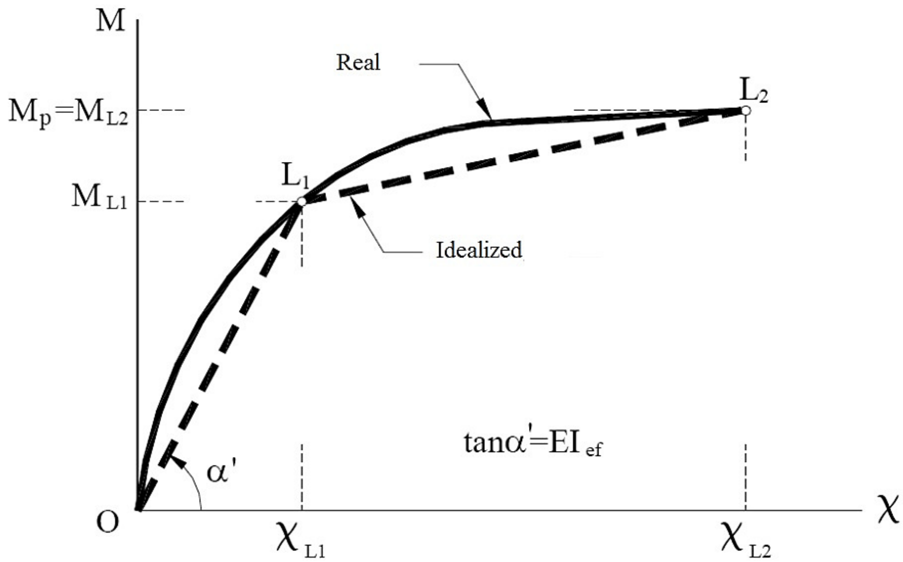


Figure 2.6: Idealized M-χ diagram of reinforced concrete members (type 1).

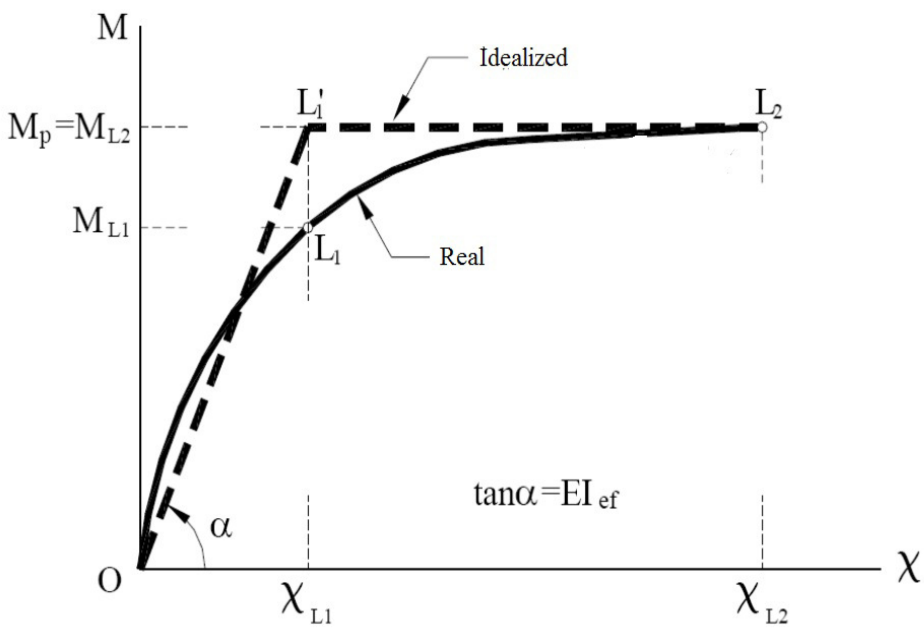


Figure 2.7: Idealized M-χ diagram of reinforced concrete members (type 2).

2.4.4 Relations between cross sectional characteristics and plastic moments for r/c members

In this study, based on the assumption that nonlinear deformations are gathered at certain points called plastic section (plastichinge), section plastic moments are taken as $M_P = M_{L2}$which proposed in the past study (Çakıroğlu and Özer**,** 1980). In this case, Fig 2.7 is valid which is the idealized of $M - \chi$ diagram.

The relationship between cross sectional characteristics is evaluatedbyconsidering some assumptions that cross section has symmetric reinforcement and plastic hinge hypothesis is valid. In such a ductile system, reinforcement in compression and tension zone reach yield limits while the strain reachedε_{cu} = 0.0035 in compression zone and before the concrete crushing.

Therefore, plastic moment (M_P) which expressed cross sectional capacitymay be stated with help of Fig 2.8;

$$M_p = M_{l2} = f_{cd} \cdot b \cdot \bar{x} \cdot \frac{d - \bar{x}}{2} + f_{yd} \cdot \rho_t \cdot b \cdot d \cdot \left(\frac{d}{2} - h' \right) \qquad \textbf{(2.12)}$$

In (2.16) expression ρtcan be displaced by;

$$\rho_t = \frac{A_s + A'_s}{b \cdot d} \qquad \textbf{(2.13)}$$

The ratio between dimension of reinforced concrete beams and column are selected based on the dimension that are commonly used in practice. Cross sections that are utilized in this study are shown in Figure 2.9, and relations between cross sectional characteristics will be obtained for practical percentage of reinforcement for these cross sections.

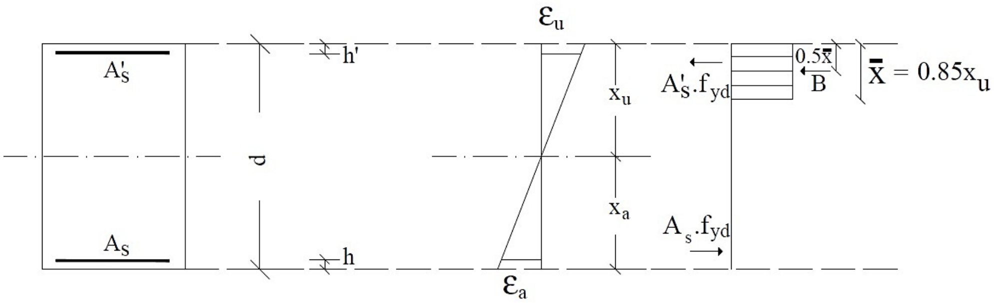


Figure 2.8: Reinforced concrete member with symmetric reinforcement.

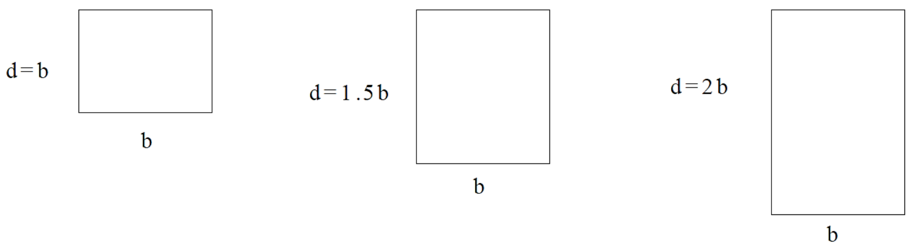


Figure 2.9: Relation between dimension of reinforced concrete beams and columns.

Relation between cross sectional characteristics of the reinforcement concrete beams and columns, cross sections are evaluated for the constant percentage of reinforcements, which arecommonly used in the practice. For reinforced concrete columns these percentages is used; 0.01, 0.015 and for beams; 0.006 is used. With the assumption that concrete cover in cross sections is h = d/20, the analytic relations between cross sectional characteristics are obtained for the given cross section types in Fig.2.9 with selected reinforcement percentages.

C30/S420 materialtypes, which are used in most of the reinforcement concrete frames, are chosen in this study for calculating the relation between cross-sectional characteristics in plane frame systems.

Table 2.1: Characteristics of loading members.

CROSS SECTION TYPE	LOADING MEMBER	DIMENSION RATIO	REINFORCEMENT PERCENTAGE
1	Column	d=b	0,01
2	Column	d=b	0,015
3	Beam	d=1,5b	0,006
4	Beam	d=2b	0,006

For the column with d=b dimension ratio and 0.01 reinforcement percentage;

$$M_p = f_{cd} \cdot b \cdot 0,425b \cdot 0,2875b + f_{yd} \cdot 0,01 \cdot b^2 \cdot 0,01 \cdot b^2 \cdot 0,45b \quad \textbf{(2.14a)}$$

$$M_p = 20000 \cdot 0,425 \cdot 0,2875b^3 + 365217,39 \cdot 0,01 \cdot 0,45b^3 \quad (2.14b)$$

$$M_p = 2443,75b^3 + 1643,4783b^3 = 4087,2283b^3 \quad \textbf{(2.14c)}$$

$$M_p = 2443,75b^3 + 1643,4783b^3 = 4087,2283b^3 \quad \textbf{(2.15)}$$

$$d = b = (2,4466 \cdot 10^{-4} \cdot M_p)^{\frac{1}{3}} \quad \textbf{(2.16)}$$

$$F = b \cdot d = (2,4466 \cdot 10^{-4} \cdot M_p)^{\frac{2}{3}} \quad \textbf{(2.17)}$$

$$I_x = I_y = \frac{b^4}{12} = \frac{(2,4466 \cdot 10^{-4} \cdot M_p)^{\frac{4}{3}}}{12} \quad \textbf{(2.18)}$$

For the column with d=b dimension ratio and 0,015 reinforcement percentage;

$$M_p = f_{cd} \cdot b \cdot 0,425b \cdot 0,2875b + f_{yd} \cdot 0,015 \cdot b^2 \cdot 0,45b \quad \textbf{(2.19a)}$$

$$M_p = 4908,9674b^3 \Rightarrow b = (2,0371.10^{-4} \cdot M_p)^{\frac{1}{3}} \quad \textbf{(2.19b)}$$

$$d = b = (2,0371 \cdot 10^{-4} \cdot M_p)^{\frac{1}{3}} \quad \textbf{(2.20)}$$

$$F = b \cdot d = (2,0371 \cdot 10^{-4} \cdot M_p)^{\frac{2}{3}} \quad \textbf{(2.21)}$$

$$I_x = I_y = \frac{b^4}{12} = \frac{(2,0371 \cdot 10^{-4} \cdot M_p)^{\frac{4}{3}}}{12} \quad \textbf{(2.22)}$$

For the beams with d=1.5b dimension ratio and 0,006 reinforcement percentage;

$$M_p = f_{cd} \cdot b \cdot 0,6375b \cdot 0,43125b + f_{yd} \cdot 0,006 \cdot b^2 \cdot 0,675b \quad \textbf{(2.23a)}$$

$$M_p = 7717,1331b^3 \Rightarrow b = (1,2958.10^{-4} \cdot M_p)^{\frac{1}{3}} \tag{2.23b}$$

$$d = 1,5b = 1,5 \cdot (1,2958 \cdot 10^{-4} \cdot M_p)^{\frac{1}{3}} \tag{2.24}$$

$$F = b \cdot d = 1,5b^2 = 1,5 \cdot (1,2958 \cdot 10^{-4} \cdot M_p)^{\frac{2}{3}} \tag{2.25}$$

$$I = \frac{bd^3}{12} = \frac{b \cdot (1,5b^3)}{12} = 0,28125 \cdot (1,2958 \cdot 10^{-4} \cdot M_p)^{\frac{4}{3}} \tag{2.26}$$

Relation between cross sectional characteristics of the reinforcement concrete beams and columns, cross sections are evaluated for the constant percentage of reinforcements, which arecommonly used in the practice. For reinforced concrete columns these percentages is used; 0.01, 0.015 and for beams; 0.006 is used. C30/S420 material types, which are used in most of the reinforcement concrete frames, are chosen in this study for calculating the relation between cross-sectional characteristics in plane frame systems. With the assumption that concrete cover in cross sections is h = d/20, the analytic relations between cross sectional characteristics are obtained for the given cross section types with selected reinforcement percentages.

3. NUMERICAL EXAMPLES

In this chapter, three numerical examples with different dimension characteristicsare given to illustrate the present formulation and in to compare the result with those obtained in previous studies. All units are KN and m.

*Example 1:*single-bay,single story plane reinforced concrete frame

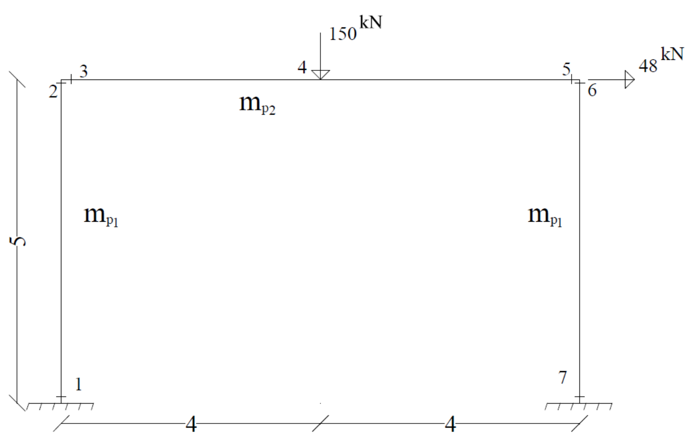


Figure 3.1: System geometry, external loads and critical sections for first example.

The weight optimization of the system with three degree of redundancy whose geometry, external loads and critical section are shown in Figure 3.1 will be done by method, which is proposed in this study.

The system consists of beams and columns whose relations between dimensional characteristics are givenin previous chapters. In this example, columns with square cross section (d=b) and reinforcement percentage of 0.01 are chosen, and cross section of beams are rectangular with theheight two times of the width (d=2b) and 0,006 reinforcement percentage.

As pre-design, for the column d/b is 0,31/0,31 and for the beam d/b is 0,48/0,24 are chosen. The pre-design dimensions are obtained by considering the assumption that the plastic moments are proportion with the member lengths. In this state, the optimal solution may be obtained in one-step (Özer, 1975).

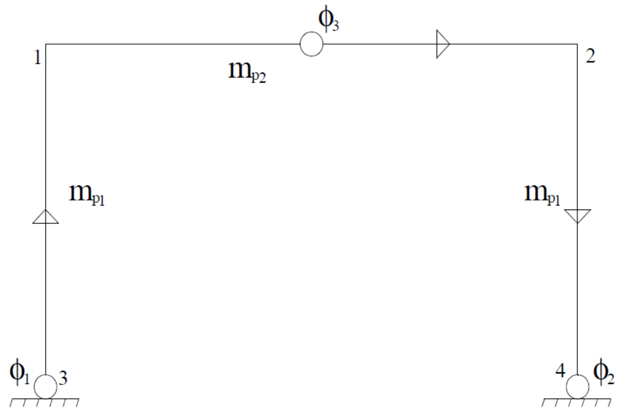
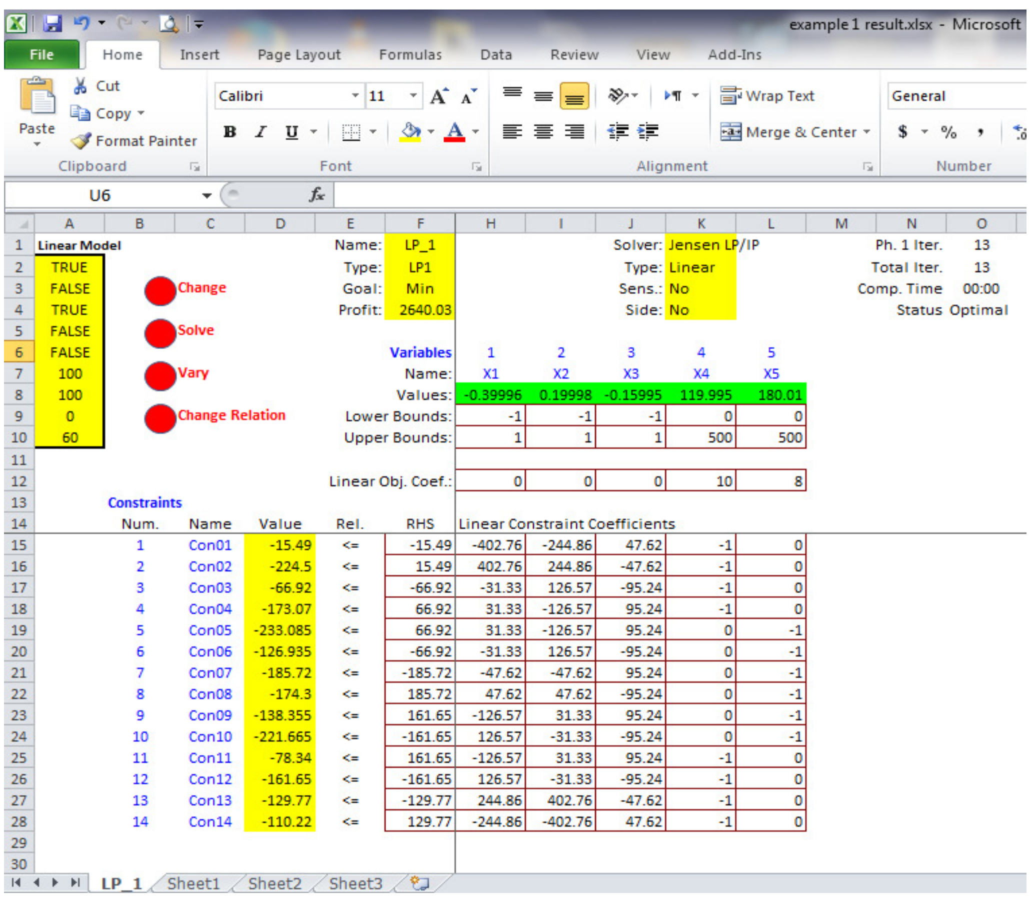


Figure 3.2: Introduced hinge locations for first example first step.

Result of the analyze which are done in the thereseach of Özer (1975), and also evaluated in this thesis by excel macro in which only the length of the member are taken to account to evaluate the optimum weigh of the frame are given inFigure 3.3 and Table 3.1.



	A	E	F	J	K	N	O
1	Linear Model	Name:	LP_1	Solver:	Jensen LP/IP	Ph. 1 Iter.	13
2	TRUE	Type:	LP1	Type:	Linear	Total Iter.	13
3	FALSE	Goal:	Min	Sens.:	No	Comp. Time	00:00
4	TRUE	Profit:	2640.03	Side:	No	Status	Optimal
5	FALSE						
6	FALSE						
7	100						
8	100						
9	0						
10	60						

Change
Solve
Vary
Change Relation

Variables	1	2	3	4	5
Name:	X1	X2	X3	X4	X5
Values:	-0.39996	0.19998	-0.15995	119.995	180.01
Lower Bounds:	-1	-1	-1	0	0
Upper Bounds:	1	1	1	500	500
Linear Obj. Coef.:	0	0	0	10	8

Constraints

Num.	Name	Value	Rel.	RHS	Linear Constraint Coefficients				
1	Con01	-15.49	<=	-15.49	-402.76	-244.86	47.62	-1	0
2	Con02	-224.5	<=	15.49	402.76	244.86	-47.62	-1	0
3	Con03	-66.92	<=	-66.92	-31.33	126.57	-95.24	-1	0
4	Con04	-173.07	<=	66.92	31.33	-126.57	95.24	-1	0
5	Con05	-233.085	<=	66.92	31.33	-126.57	95.24	0	-1
6	Con06	-126.935	<=	-66.92	-31.33	126.57	-95.24	0	-1
7	Con07	-185.72	<=	-185.72	-47.62	-47.62	95.24	0	-1
8	Con08	-174.3	<=	185.72	47.62	47.62	-95.24	0	-1
9	Con09	-138.355	<=	161.65	-126.57	31.33	95.24	0	-1
10	Con10	-221.665	<=	-161.65	126.57	-31.33	-95.24	0	-1
11	Con11	-78.34	<=	161.65	-126.57	31.33	95.24	-1	0
12	Con12	-161.65	<=	-161.65	126.57	-31.33	-95.24	-1	0
13	Con13	-129.77	<=	-129.77	244.86	402.76	-47.62	-1	0
14	Con14	-110.22	<=	129.77	-244.86	-402.76	47.62	-1	0

Figure 3.3: Excel macro sheet and yieldcondition constraints and the weight function for Example 1 taking into account only length of member.

Table 3.1: Design variables and weight function for Example1 with taking into account only length of members.

Design variable	(rad)
Φ1	-0.4
Φ2	0.2
Φ3	-0.16
mp1	120
mp2	180
w	2640

Column type and dimension, beam type and dimension, design variables and minimum weight value of optimum weight design of this type analysis are presented below in Table 3.2.

Table 3.2: Optimum dimension of members for Example 1 taking into account only length of member.

Member No	Member type	M_P	Optimum dimensions(h/b) m/m
3-1	Column	120	0.31/0.31
1-2	Beam	180	0.47/0.24
2-4	Column	120	0.31/0.31

In the beginning of the procedure, the plastic hinge locations are chosen as shown in Figure3.2asin the thereseach of Özer (1975), the analysis is performed for external loads and each of unit plastic hinge rotations separately and respectively, therefore, bending moments will be evaluated in critical sections. At each step of the successive approximation plastic hinge pattern is checked and the new pattern is considered at the next step if there is any changes. Furthermore, at each step objective function is linearized at the vicinity of plastic moment obtained at previous step. Plastic hinge pattern for Example 1 are shown in Figure 3.4.

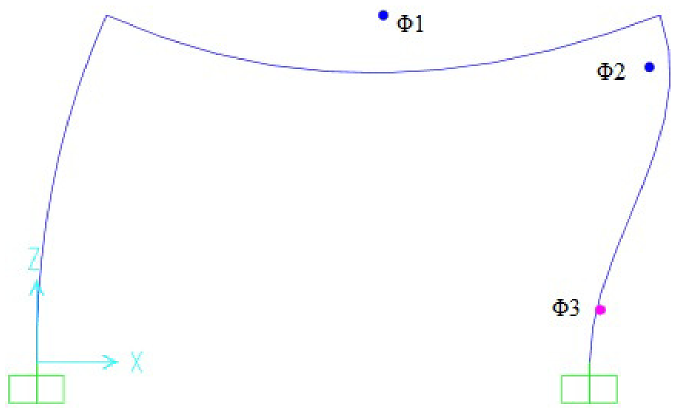
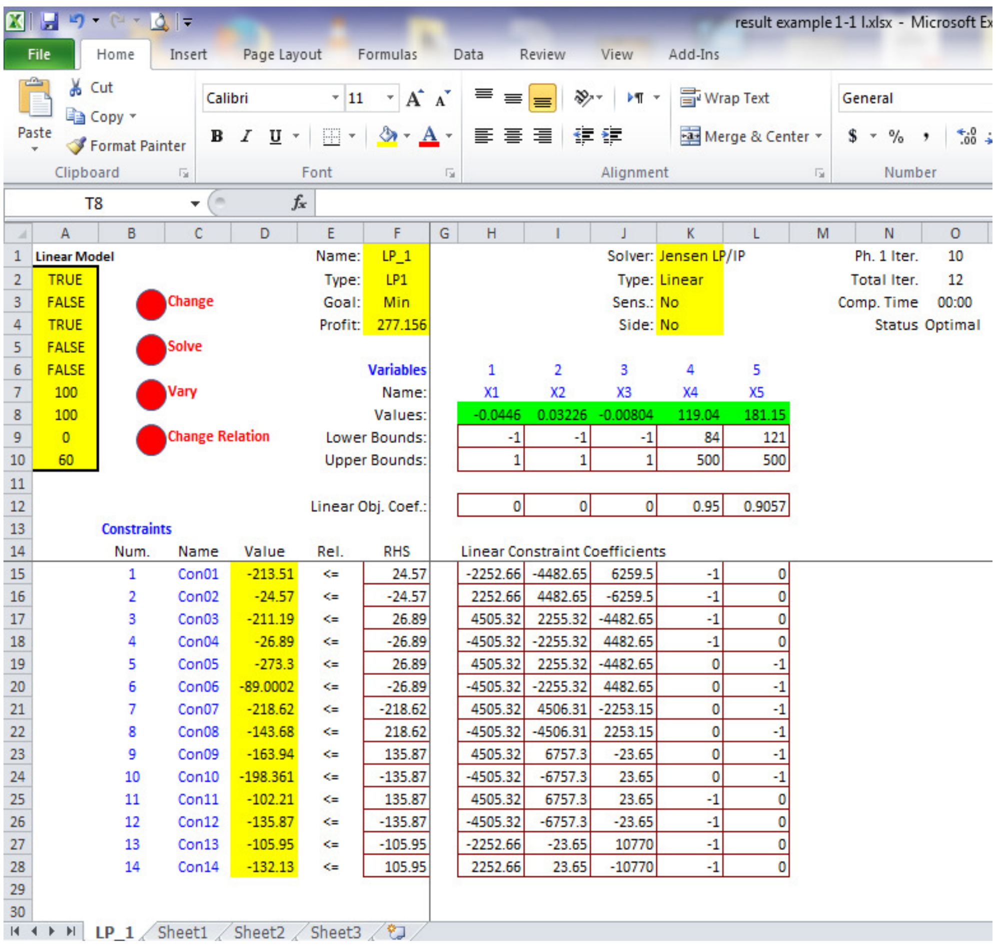


Figure 3.4: Plastic hinge pattern for optimum weight design of Example 1.

The optimal solution is reach after 12 iterations by using Excel macro sheet.Resultsare shownin Figure 3.5 and Table 3.3.



	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	Linear Model				Name:	LP_1				Solver:	Jensen LP/IP			Ph. 1 Iter.	10
2	TRUE				Type:	LP1				Type:	Linear			Total Iter.	12
3	FALSE		Change		Goal:	Min				Sens.:	No			Comp. Time	00:00
4	TRUE				Profit:	277.156				Side:	No			Status	Optimal
5	FALSE		Solve												
6	FALSE					Variables		1	2	3	4	5			
7	100		Vary			Name:		X1	X2	X3	X4	X5			
8	100					Values:		-0.0446	0.03226	-0.00804	119.04	181.15			
9	0		Change Relation			Lower Bounds:		-1	-1	-1	84	121			
10	60					Upper Bounds:		1	1	1	500	500			
11															
12						Linear Obj. Coef.:		0	0	0	0.95	0.9057			
13		Constraints													
14		Num.	Name	Value	Rel.	RHS		Linear Constraint Coefficients							
15		1	Con01	-213.51	<=	24.57		-2252.66	-4482.65	6259.5	-1	0			
16		2	Con02	-24.57	<=	-24.57		2252.66	4482.65	-6259.5	-1	0			
17		3	Con03	-211.19	<=	26.89		4505.32	2255.32	-4482.65	-1	0			
18		4	Con04	-26.89	<=	-26.89		-4505.32	-2255.32	4482.65	-1	0			
19		5	Con05	-273.3	<=	26.89		4505.32	2255.32	-4482.65	0	-1			
20		6	Con06	-89.0002	<=	-26.89		-4505.32	-2255.32	4482.65	0	-1			
21		7	Con07	-218.62	<=	-218.62		4505.32	4506.31	-2253.15	0	-1			
22		8	Con08	-143.68	<=	218.62		-4505.32	-4506.31	2253.15	0	-1			
23		9	Con09	-163.94	<=	135.87		4505.32	6757.3	-23.65	0	-1			
24		10	Con10	-198.361	<=	-135.87		-4505.32	-6757.3	23.65	0	-1			
25		11	Con11	-102.21	<=	135.87		4505.32	6757.3	23.65	-1	0			
26		12	Con12	-135.87	<=	-135.87		-4505.32	-6757.3	-23.65	-1	0			
27		13	Con13	-105.95	<=	-105.95		-2252.66	-23.65	10770	-1	0			
28		14	Con14	-132.13	<=	105.95		2252.66	23.65	-10770	-1	0			
29															
30															

Figure 3.5: Excel macro sheet for the Example 1 without rotation constraints.

Table 3.3: Design variables and the weight of frame for Example 1 without rotation constraints.

Design variables	(rad)
Φ_1	-0,0446
Φ_2	0,0322
Φ_3	-0,008
m_{p1}	119,04
m_{p2}	181,15
w	277,16

In order to evaluate optimum dimension of loading members, relation between cross sectional characteristics, which are expressed in related chapter, will be used by considering values in Table 3.3.

Table 3.4: Optimum dimension of members for Example 1 without rotation constraints.

Member No	Member type	MP	Optimum dimensions(h/b) m/m
3-1	column	119.04	0.308/0.308
1-2	beam	181,15	0.476/0.245
2-4	column	181,15	0.308/0.308

Optimum weight design of the frame is also obtainedwith considering plastic hinge rotation constraints in beams and columns. By implementing these constraints to the problem, optimal design of the system is performed once again. After 8 step of iteration optimal solution is reached.

Plastic rotation limits for beams and columns, whichare stated in ATC-40 code for different performance levels and were used in this study, are shown in Table 3.5.

Table 3.5: Plastic rotation capacity limits for columns and beams.

Plastic cross-section	Collapse safety (rad)
Plastic cross-sections in beams	0.025
Plastic cross-sections in columns	0.02

The optimum weight design of the Example.1 frames with rotation constrains is obtained and is shown in Figure 3.6.

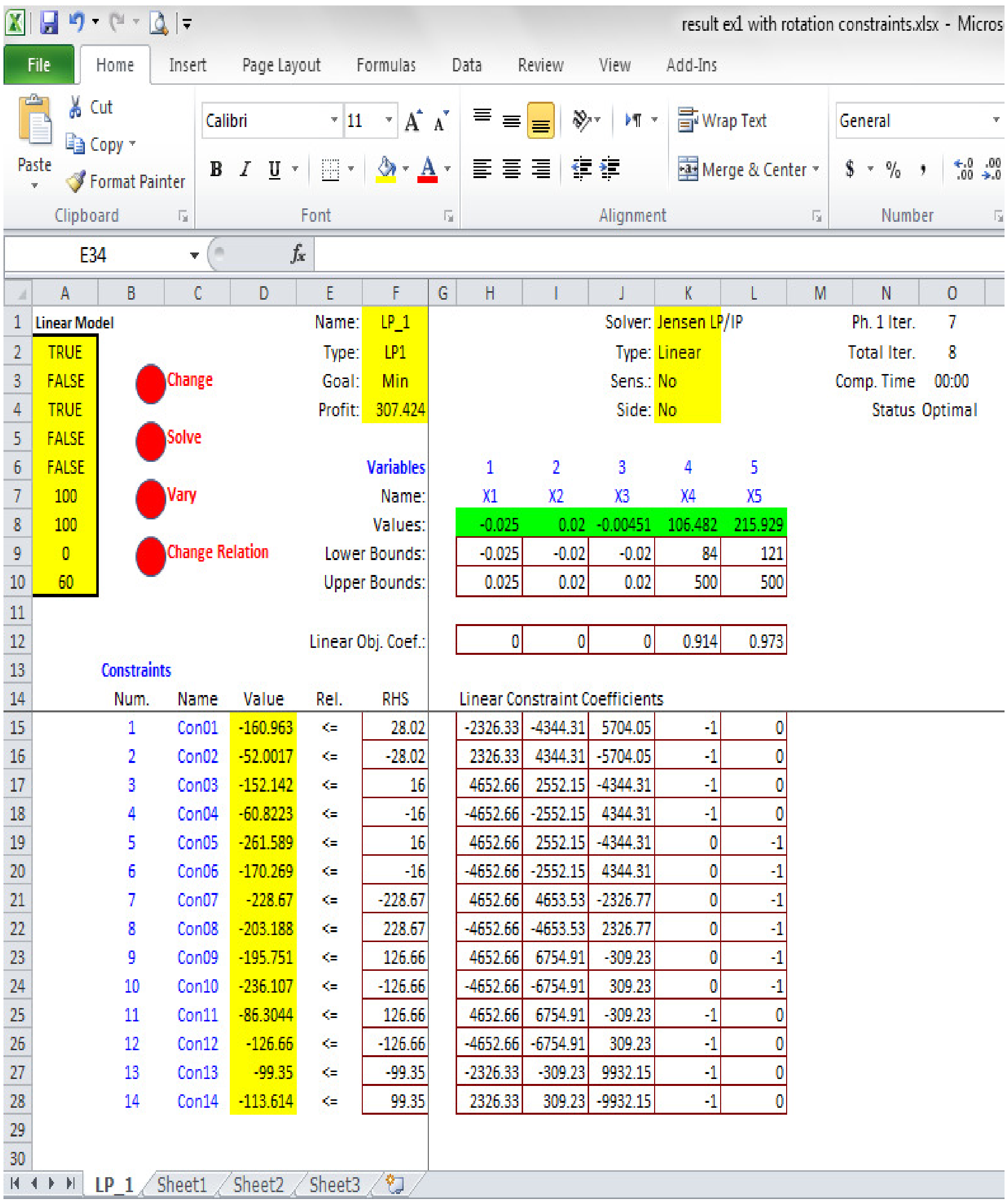


Figure 3.6: Excel macro sheet for the Example 1 with rotation constraints.

Column type and dimension, beam type and dimension, design variables and minimum weight value of optimum weight design with rotation constraints are presented in below.

Table 3.6: Design variables and the weight of frame for Example 1 with rotation constraints.

Design variables(rad)	
Φ_1	-0,025
Φ_2	0,02
Φ_3	-0,0045
m_{p1}	107
m_{p2}	215
w	307

Optimum dimension of beams and column for Example.1 with rotation constraints are shown in Table 3.7.

Table 3.7: Optimum dimension of members for ex.1 with rotation constraints.

Member No	Loading Member	M_P	Optimum dimensions(h/b) m/m
3-1	column	107	0.297/0,297
1-2	beam	215	0,5/0,25
2-4	column	107	0,297/0,297

One additional constraint, which state that the plastic bending moment of columns should be larger than beams,are added to the problem and the optimal design procedure repeated. The optimal solution is obtainedafter 14 iterations. Last step of this procedure is shown in Figure 3.7.

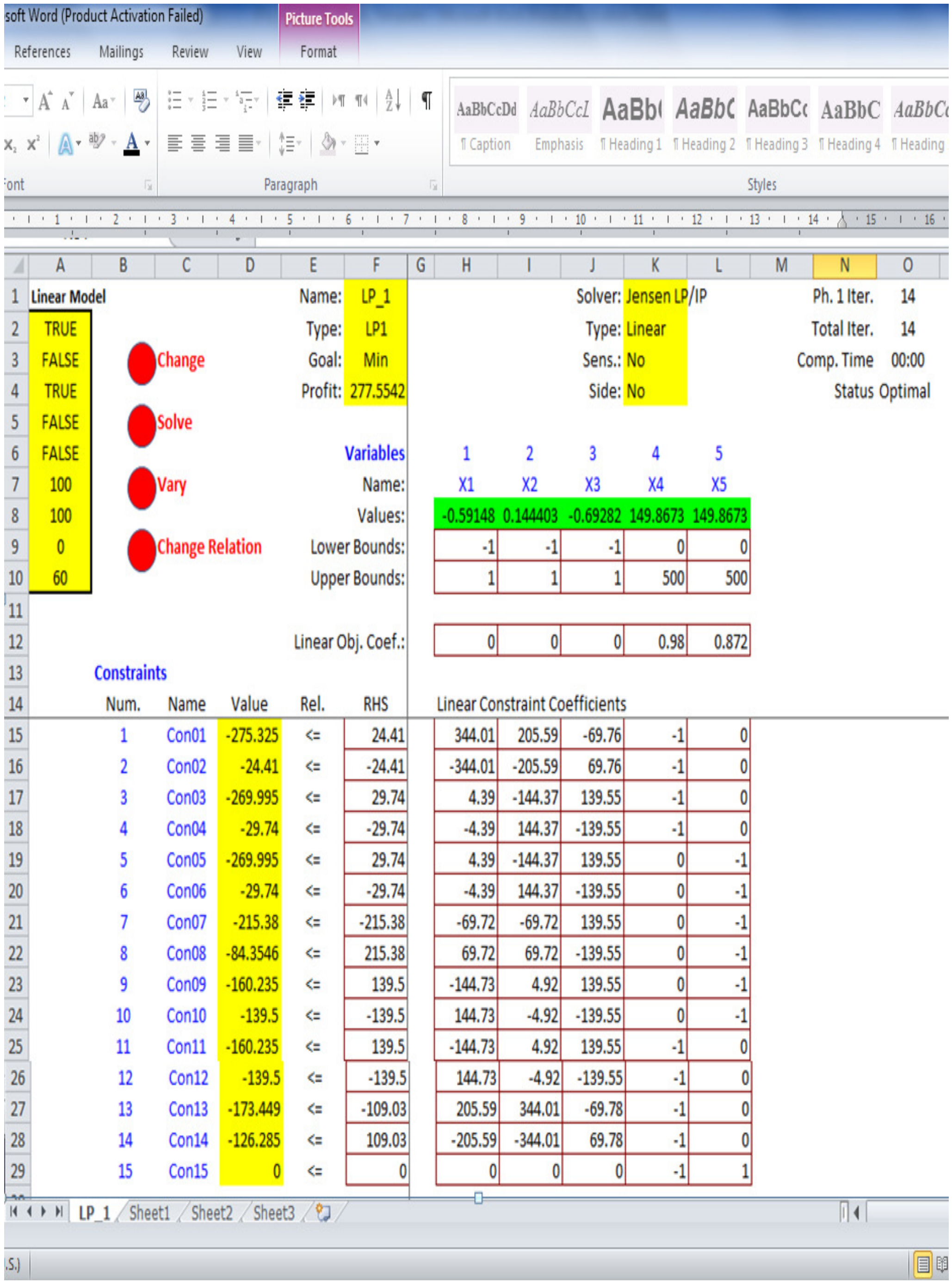


Figure 3.7: Excel macro sheet for the Example 1 with additional constraints.

Design variables and minimum weight value of optimum weight design with additional constraints and optimum dimension of beams and column for Example.1 with additional constraints are shown in Table 3.8 and Table 3.9 respectively.

Table 3.8: Design variables and the weight of frame for Example 1 with additional constraints.

Design variables	(rad)
Φ_1	-0,591
Φ_2	0,144
Φ_3	-0,693
m_{p1}	149,87
m_{p2}	149,87
w	277.55

Table 3.9: Optimum dimension of members for Example.1 with additional constraints.

Member No	Member type	M_P	Optimum dimensions(h/b) m/m
3-1	column	133,02	0.319/0.319
1-2	beam	218,87	0.502/0.251
2-4	column	133,02	0.319/0.319

Example 2: The two-bay, two-story plane reinforcement concrete frame.

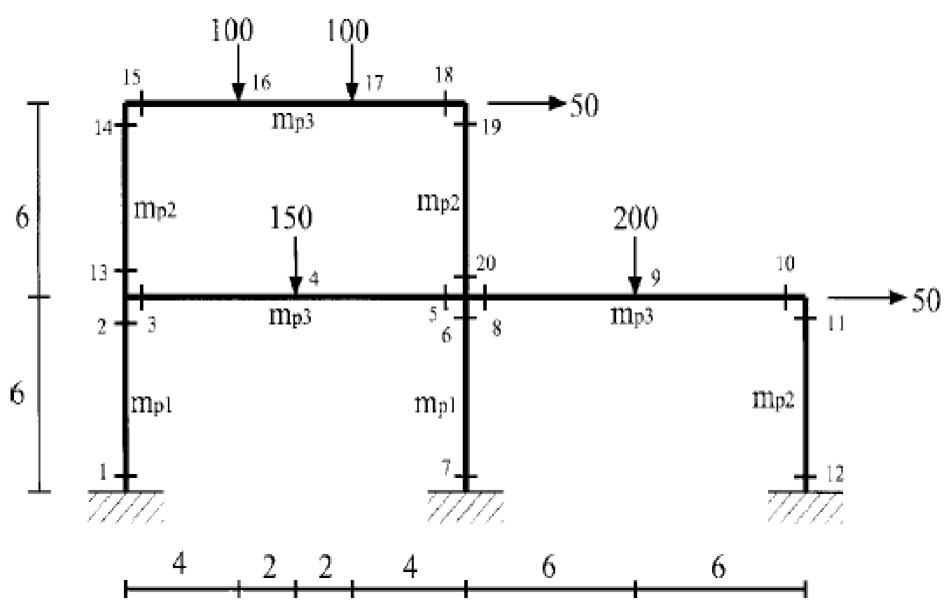


Figure 3.8: System geometry, external loads and critical sections for second example.

The weight optimization of the system with nine degree of redundancy whose geometry, external loads and critical section are shown in Figure 3.8 will be done by method, which is proposed in this study.

The system consists of beams and columns whose relations between dimensional characteristics are given in previous chapters. In this example, columns with square cross section (d=b) and reinforcement percentage of 0.01 are chosen, and cross section of beams are rectangular with the height two times of the width (d=1.5b) and 0,006 reinforcement percentage.

As pre-design, for the column d/b is 0,274/0,274 and 0,395/0,395 and for the beam d/b is 0,517/0,345 are chosen. The pre-design dimensions are obtained by considering the assumption that the plastic moments are proportion with the member lengths. In this state, the optimal solution may be obtained in one-step (Özer, 1975).

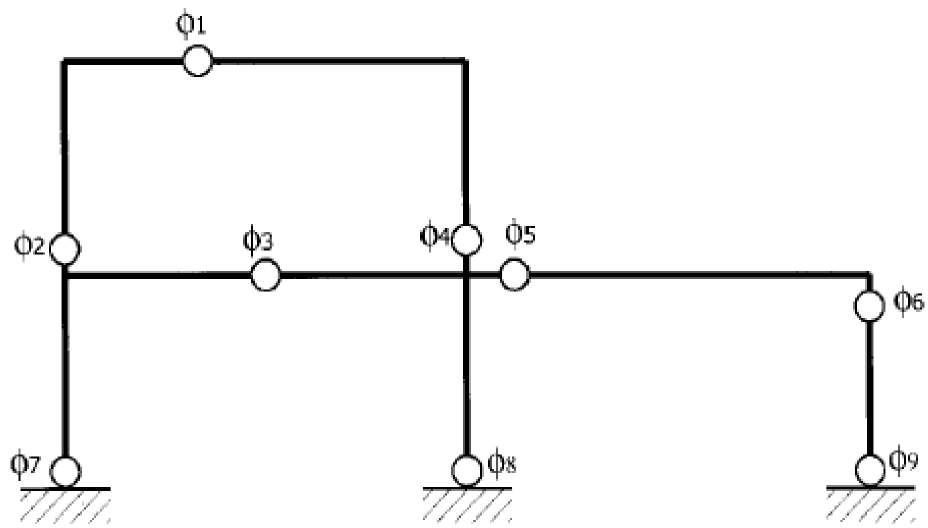
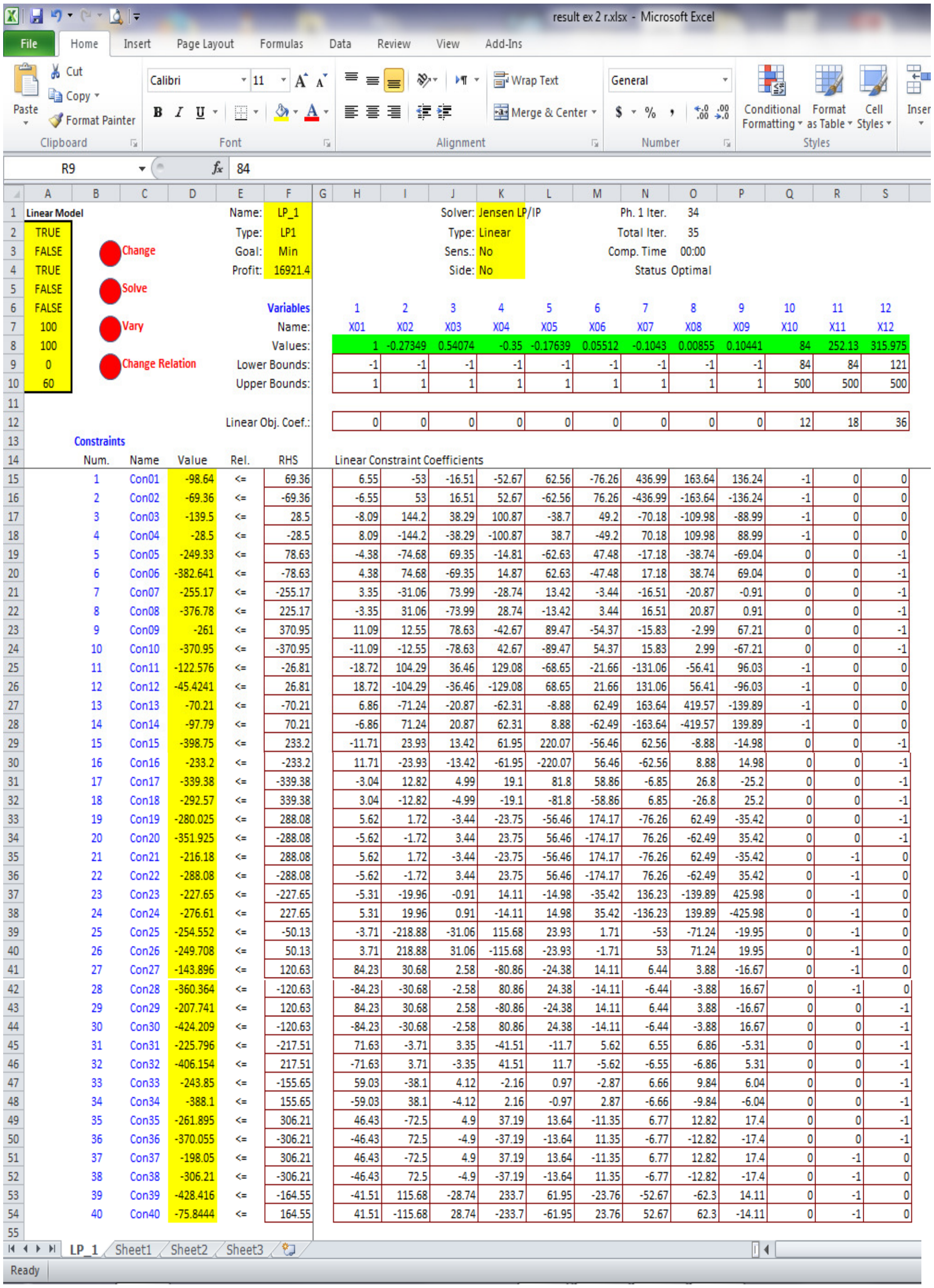


Figure 3.9: Introduced hinge locations for second example first step.

Result of the analyze which are done in the studies of Özer (1975), and also evaluated in this thesis by excel macro in which only the length of the member are taken to account to evaluate the optimum weigh of the frame are given in Figure 3.10 and Table 3.10.



Linear Model			Name:	LP_1	Solver:	Jensen LP/IP	Ph. 1 Iter.	34
TRUE			Type:	LP1	Type:	Linear	Total Iter.	35
FALSE	Change		Goal:	Min	Sens.:	No	Comp. Time	00:00
TRUE			Profit:	16921.4	Side:	No	Status	Optimal
FALSE	Solve							
FALSE								
100	Vary							
100								
0	Change Relation							
60								

Variables	1	2	3	4	5	6	7	8	9	10	11	12
Name:	X01	X02	X03	X04	X05	X06	X07	X08	X09	X10	X11	X12
Values:	1	-0.27349	0.54074	-0.35	-0.17639	0.05512	-0.1043	0.00855	0.10441	84	252.13	315.975
Lower Bounds:	-1	-1	-1	-1	-1	-1	-1	-1	-1	84	84	121
Upper Bounds:	1	1	1	1	1	1	1	1	1	500	500	500
Linear Obj. Coef.:	0	0	0	0	0	0	0	0	0	12	18	36

Constraints

Linear Constraint Coefficients

Num.	Name	Value	Rel.	RHS	X01	X02	X03	X04	X05	X06	X07	X08	X09	X10	X11	X12
1	Con01	-98.64	<=	69.36	6.55	-53	-16.51	-52.67	62.56	-76.26	436.99	163.64	136.24	-1	0	0
2	Con02	-69.36	<=	-69.36	-6.55	53	16.51	52.67	-62.56	76.26	-436.99	-163.64	-136.24	-1	0	0
3	Con03	-139.5	<=	28.5	-8.09	144.2	38.29	100.87	-38.7	49.2	-70.18	-109.98	-88.99	-1	0	0
4	Con04	-28.5	<=	-28.5	8.09	-144.2	-38.29	-100.87	38.7	-49.2	70.18	109.98	88.99	-1	0	0
5	Con05	-249.33	<=	78.63	-4.38	-74.68	69.35	-14.81	-62.63	47.48	-17.18	-38.74	-69.04	0	0	-1
6	Con06	-382.641	<=	-78.63	4.38	74.68	-69.35	14.87	62.63	-47.48	17.18	38.74	69.04	0	0	-1
7	Con07	-255.17	<=	-255.17	3.35	-31.06	73.99	-28.74	13.42	-3.44	-16.51	-20.87	-0.91	0	0	-1
8	Con08	-376.78	<=	225.17	-3.35	31.06	-73.99	28.74	-13.42	3.44	16.51	20.87	0.91	0	0	-1
9	Con09	-261	<=	370.95	11.09	12.55	78.63	-42.67	89.47	-54.37	-15.83	-2.99	67.21	0	0	-1
10	Con10	-370.95	<=	-370.95	-11.09	-12.55	-78.63	42.67	-89.47	54.37	15.83	2.99	-67.21	0	0	-1
11	Con11	-122.576	<=	-26.81	-18.72	104.29	36.46	129.08	-68.65	-21.66	-131.06	-56.41	96.03	-1	0	0
12	Con12	-45.4241	<=	26.81	18.72	-104.29	-36.46	-129.08	68.65	21.66	131.06	56.41	-96.03	-1	0	0
13	Con13	-70.21	<=	-70.21	6.86	-71.24	-20.87	-62.31	-8.88	62.49	163.64	419.57	-139.89	-1	0	0
14	Con14	-97.79	<=	70.21	-6.86	71.24	20.87	62.31	8.88	-62.49	-163.64	-419.57	139.89	-1	0	0
15	Con15	-398.75	<=	233.2	-11.71	23.93	13.42	61.95	220.07	-56.46	62.56	-8.88	-14.98	0	0	-1
16	Con16	-233.2	<=	-233.2	11.71	-23.93	-13.42	-61.95	-220.07	56.46	-62.56	8.88	14.98	0	0	-1
17	Con17	-339.38	<=	-339.38	-3.04	12.82	4.99	19.1	81.8	58.86	-6.85	26.8	-25.2	0	0	-1
18	Con18	-292.57	<=	339.38	3.04	-12.82	-4.99	-19.1	-81.8	-58.86	6.85	-26.8	25.2	0	0	-1
19	Con19	-280.025	<=	288.08	5.62	1.72	-3.44	-23.75	-56.46	174.17	-76.26	62.49	-35.42	0	0	-1
20	Con20	-351.925	<=	-288.08	-5.62	-1.72	3.44	23.75	56.46	-174.17	76.26	-62.49	35.42	0	0	-1
21	Con21	-216.18	<=	288.08	5.62	1.72	-3.44	-23.75	-56.46	174.17	-76.26	62.49	-35.42	0	-1	0
22	Con22	-288.08	<=	-288.08	-5.62	-1.72	3.44	23.75	56.46	-174.17	76.26	-62.49	35.42	0	-1	0
23	Con23	-227.65	<=	-227.65	-5.31	-19.96	-0.91	14.11	-14.98	-35.42	136.23	-139.89	425.98	0	-1	0
24	Con24	-276.61	<=	227.65	5.31	19.96	0.91	-14.11	14.98	35.42	-136.23	139.89	-425.98	0	-1	0
25	Con25	-254.552	<=	-50.13	-3.71	-218.88	-31.06	115.68	23.93	1.71	-53	-71.24	-19.95	0	-1	0
26	Con26	-249.708	<=	50.13	3.71	218.88	31.06	-115.68	-23.93	-1.71	53	71.24	19.95	0	-1	0
27	Con27	-143.896	<=	120.63	84.23	30.68	2.58	-80.86	-24.38	14.11	6.44	3.88	-16.67	0	-1	0
28	Con28	-360.364	<=	-120.63	-84.23	-30.68	-2.58	80.86	24.38	-14.11	-6.44	-3.88	16.67	0	-1	0
29	Con29	-207.741	<=	120.63	84.23	30.68	2.58	-80.86	-24.38	14.11	6.44	3.88	-16.67	0	0	-1
30	Con30	-424.209	<=	-120.63	-84.23	-30.68	-2.58	80.86	24.38	-14.11	-6.44	-3.88	16.67	0	0	-1
31	Con31	-225.796	<=	-217.51	71.63	-3.71	3.35	-41.51	-11.7	5.62	6.55	6.86	-5.31	0	0	-1
32	Con32	-406.154	<=	217.51	-71.63	3.71	-3.35	41.51	11.7	-5.62	-6.55	-6.86	5.31	0	0	-1
33	Con33	-243.85	<=	-155.65	59.03	-38.1	4.12	-2.16	0.97	-2.87	6.66	9.84	6.04	0	0	-1
34	Con34	-388.1	<=	155.65	-59.03	38.1	-4.12	2.16	-0.97	2.87	-6.66	-9.84	-6.04	0	0	-1
35	Con35	-261.895	<=	306.21	46.43	-72.5	4.9	37.19	13.64	-11.35	6.77	12.82	17.4	0	0	-1
36	Con36	-370.055	<=	-306.21	-46.43	72.5	-4.9	-37.19	-13.64	11.35	-6.77	-12.82	-17.4	0	0	-1
37	Con37	-198.05	<=	306.21	46.43	-72.5	4.9	37.19	13.64	-11.35	6.77	12.82	17.4	0	-1	0
38	Con38	-306.21	<=	-306.21	-46.43	72.5	-4.9	-37.19	-13.64	11.35	-6.77	-12.82	-17.4	0	-1	0
39	Con39	-428.416	<=	-164.55	-41.51	115.68	-28.74	233.7	61.95	-23.76	-52.67	-62.3	14.11	0	-1	0
40	Con40	-75.8444	<=	164.55	41.51	-115.68	28.74	-233.7	-61.95	23.76	52.67	62.3	-14.11	0	-1	0

Figure 3.10: Excel macro sheet and yield condition constraints and the weight function for Example 2 taking into account only length of member.

.

Table 3.10: Design variables and weight function for Example2 with taking into account only length of members.

Design variables	(rad)
Φ_1	1
Φ_2	-0,273
Φ_3	0,54
Φ_4	-0,35
Φ_5	-0,176
Φ_6	0,055
Φ_7	-0,104
Φ_8	0,008
Φ_9	0,104
m_{p1}	84
m_{p2}	252
m_{p3}	316
w	16921

Column type and dimension, beam type and dimension, design variables and minimum weight value of optimum weight design of this type analysisare presented in below.

Table 3.11: optimum dimension of members for Example 1 with taking into account only length of members.

Member No	Member type	M_P	Optimum dimensions (d/b) m/m
1-2	Beam	316	0,517/0,345
3-4	Beam	316	0,517/0,345
4-5	Beam	316	0,517/0,345
3-1	Column	252	0,395/0,395
2-4	Column	252	0,395/0,395
5-8	Column	252	0,395/0,395
6-3	Column	84	0,274/0,274
4-7	Column	84	0,274/0,274

In the beginning of the procedure, the plastic hinge locations are chosen as shown in Figure3.9 as in the past study (Özer, 1975). The analysis is performed for external loads and each of unit plastic hinge rotations separately and respectively, therefore, bending moments will be evaluated in critical sections. At each step of the successive approximation plastic hinge pattern is checked and the new pattern is considered at the next step if there is any changes. Furthermore, at each step objective function is linearized at the vicinity of plastic moment obtained at previous step. Plastic hinge pattern for Example 1 are shown in Figure 3.11.

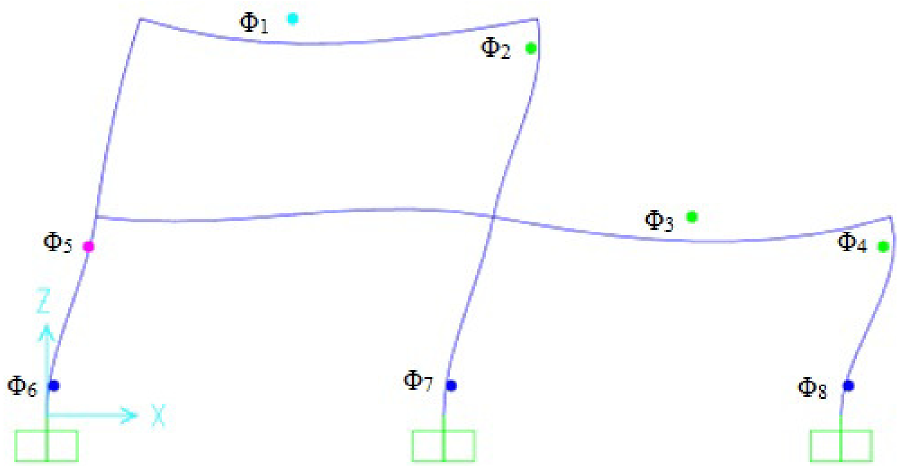
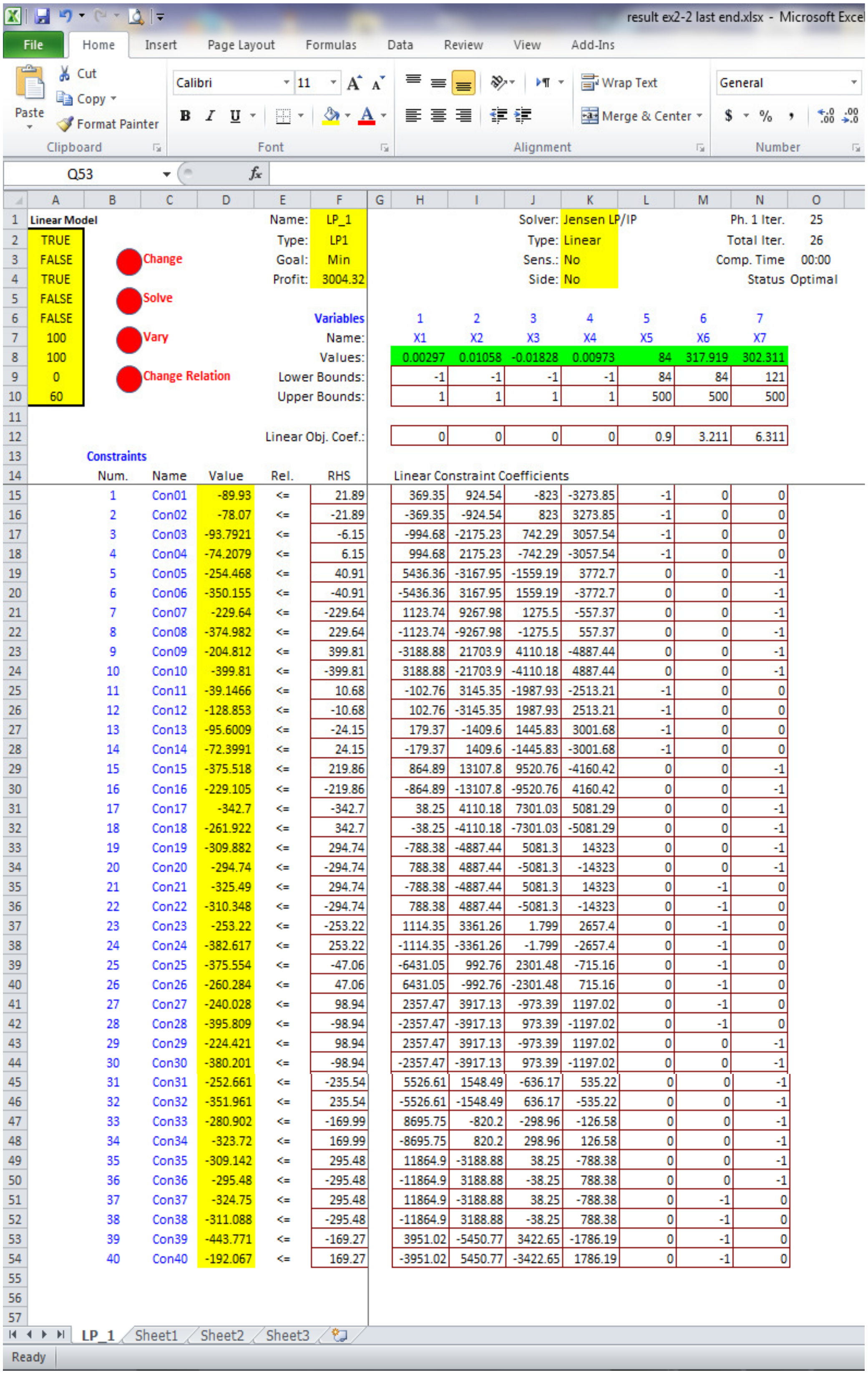


Figure 3.1: Plastic hinge pattern for optimum weight design of Example 2.

The optimal solution is reach after 26 iterations by using Excel macro sheet.Results will be shown in Figure 3.12 and Table 3.12.



	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	Linear Model				Name:	LP_1				Solver:	Jensen LP/IP			Ph. 1 Iter.	25
2	TRUE				Type:	LP1				Type:	Linear			Total Iter.	26
3	FALSE		Change		Goal:	Min				Sens.:	No			Comp. Time	00:00
4	TRUE				Profit:	3004.32				Side:	No			Status	Optimal
5	FALSE		Solve												
6	FALSE					Variables		1	2	3	4	5	6	7	
7	100		Vary			Name:		X1	X2	X3	X4	X5	X6	X7	
8	100					Values:		0.00297	0.01058	-0.01828	0.00973	84	317.919	302.311	
9	0		Change Relation			Lower Bounds:		-1	-1	-1	-1	84	84	121	
10	60					Upper Bounds:		1	1	1	1	500	500	500	
11															
12						Linear Obj. Coef.:		0	0	0	0	0.9	3.211	6.311	
13		Constraints													
14		Num.	Name	Value	Rel.	RHS		Linear Constraint Coefficients							
15		1	Con01	-89.93	<=	21.89		369.35	924.54	-823	-3273.85	-1	0	0	
16		2	Con02	-78.07	<=	-21.89		-369.35	-924.54	823	3273.85	-1	0	0	
17		3	Con03	-93.7921	<=	-6.15		-994.68	-2175.23	742.29	3057.54	-1	0	0	
18		4	Con04	-74.2079	<=	6.15		994.68	2175.23	-742.29	-3057.54	-1	0	0	
19		5	Con05	-254.468	<=	40.91		5436.36	-3167.95	-1559.19	3772.7	0	0	-1	
20		6	Con06	-350.155	<=	-40.91		-5436.36	3167.95	1559.19	-3772.7	0	0	-1	
21		7	Con07	-229.64	<=	-229.64		1123.74	9267.98	1275.5	-557.37	0	0	-1	
22		8	Con08	-374.982	<=	229.64		-1123.74	-9267.98	-1275.5	557.37	0	0	-1	
23		9	Con09	-204.812	<=	399.81		-3188.88	21703.9	4110.18	-4887.44	0	0	-1	
24		10	Con10	-399.81	<=	-399.81		3188.88	-21703.9	-4110.18	4887.44	0	0	-1	
25		11	Con11	-39.1466	<=	10.68		-102.76	3145.35	-1987.93	-2513.21	-1	0	0	
26		12	Con12	-128.853	<=	-10.68		102.76	-3145.35	1987.93	2513.21	-1	0	0	
27		13	Con13	-95.6009	<=	-24.15		179.37	-1409.6	1445.83	3001.68	-1	0	0	
28		14	Con14	-72.3991	<=	24.15		-179.37	1409.6	-1445.83	-3001.68	-1	0	0	
29		15	Con15	-375.518	<=	219.86		864.89	13107.8	9520.76	-4160.42	0	0	-1	
30		16	Con16	-229.105	<=	-219.86		-864.89	-13107.8	-9520.76	4160.42	0	0	-1	
31		17	Con17	-342.7	<=	-342.7		38.25	4110.18	7301.03	5081.29	0	0	-1	
32		18	Con18	-261.922	<=	342.7		-38.25	-4110.18	-7301.03	-5081.29	0	0	-1	
33		19	Con19	-309.882	<=	294.74		-788.38	-4887.44	5081.3	14323	0	0	-1	
34		20	Con20	-294.74	<=	-294.74		788.38	4887.44	-5081.3	-14323	0	0	-1	
35		21	Con21	-325.49	<=	294.74		-788.38	-4887.44	5081.3	14323	0	-1	0	
36		22	Con22	-310.348	<=	-294.74		788.38	4887.44	-5081.3	-14323	0	-1	0	
37		23	Con23	-253.22	<=	-253.22		1114.35	3361.26	1.799	2657.4	0	-1	0	
38		24	Con24	-382.617	<=	253.22		-1114.35	-3361.26	-1.799	-2657.4	0	-1	0	
39		25	Con25	-375.554	<=	-47.06		-6431.05	992.76	2301.48	-715.16	0	-1	0	
40		26	Con26	-260.284	<=	47.06		6431.05	-992.76	-2301.48	715.16	0	-1	0	
41		27	Con27	-240.028	<=	98.94		2357.47	3917.13	-973.39	1197.02	0	-1	0	
42		28	Con28	-395.809	<=	-98.94		-2357.47	-3917.13	973.39	-1197.02	0	-1	0	
43		29	Con29	-224.421	<=	98.94		2357.47	3917.13	-973.39	1197.02	0	0	-1	
44		30	Con30	-380.201	<=	-98.94		-2357.47	-3917.13	973.39	-1197.02	0	0	-1	
45		31	Con31	-252.661	<=	-235.54		5526.61	1548.49	-636.17	535.22	0	0	-1	
46		32	Con32	-351.961	<=	235.54		-5526.61	-1548.49	636.17	-535.22	0	0	-1	
47		33	Con33	-280.902	<=	-169.99		8695.75	-820.2	-298.96	-126.58	0	0	-1	
48		34	Con34	-323.72	<=	169.99		-8695.75	820.2	298.96	126.58	0	0	-1	
49		35	Con35	-309.142	<=	295.48		11864.9	-3188.88	38.25	-788.38	0	0	-1	
50		36	Con36	-295.48	<=	-295.48		-11864.9	3188.88	-38.25	788.38	0	0	-1	
51		37	Con37	-324.75	<=	295.48		11864.9	-3188.88	38.25	-788.38	0	-1	0	
52		38	Con38	-311.088	<=	-295.48		-11864.9	3188.88	-38.25	788.38	0	-1	0	
53		39	Con39	-443.771	<=	-169.27		3951.02	-5450.77	3422.65	-1786.19	0	-1	0	
54		40	Con40	-192.067	<=	169.27		-3951.02	5450.77	-3422.65	1786.19	0	-1	0	

Figure 3.12: Excel macro sheet for the Example 2 without rotation constraints.

Table 3.12: Design variable and weight of frame for Example 2 without rotation constraints.

Design variables	(rad)
Φ1	0,003
Φ2	0,01
Φ3	-0,018
Φ4	0,0097
Φ5	-0,176
Φ6	0,055
Φ7	-0,011
Φ8	0,008
mp1	84
mp2	318
mp3	303
w	3004

In order to evaluate optimum dimension of loading members, relation between cross sectional characteristics, which are expressed in related chapter, will be used by considering values in Table 3.12.

Table 3.13: Optimum dimension of members for Example 2 without rotation constraints.

Member No	Loading member	M_P	Optimum dimensions (d/b) m/m
1-2	Beam	303	0,510/0,34
3-4	Beam	303	0,510/0,34
4-5	Beam	303	0,510/0,34
3-1	Column	318	0,426/0,426
2-4	Column	318	0,426/0,426
5-8	Column	318	0,426/0,426
6-3	Column	84	0,274/0,274
4-7	Column	84	0,274/0,274

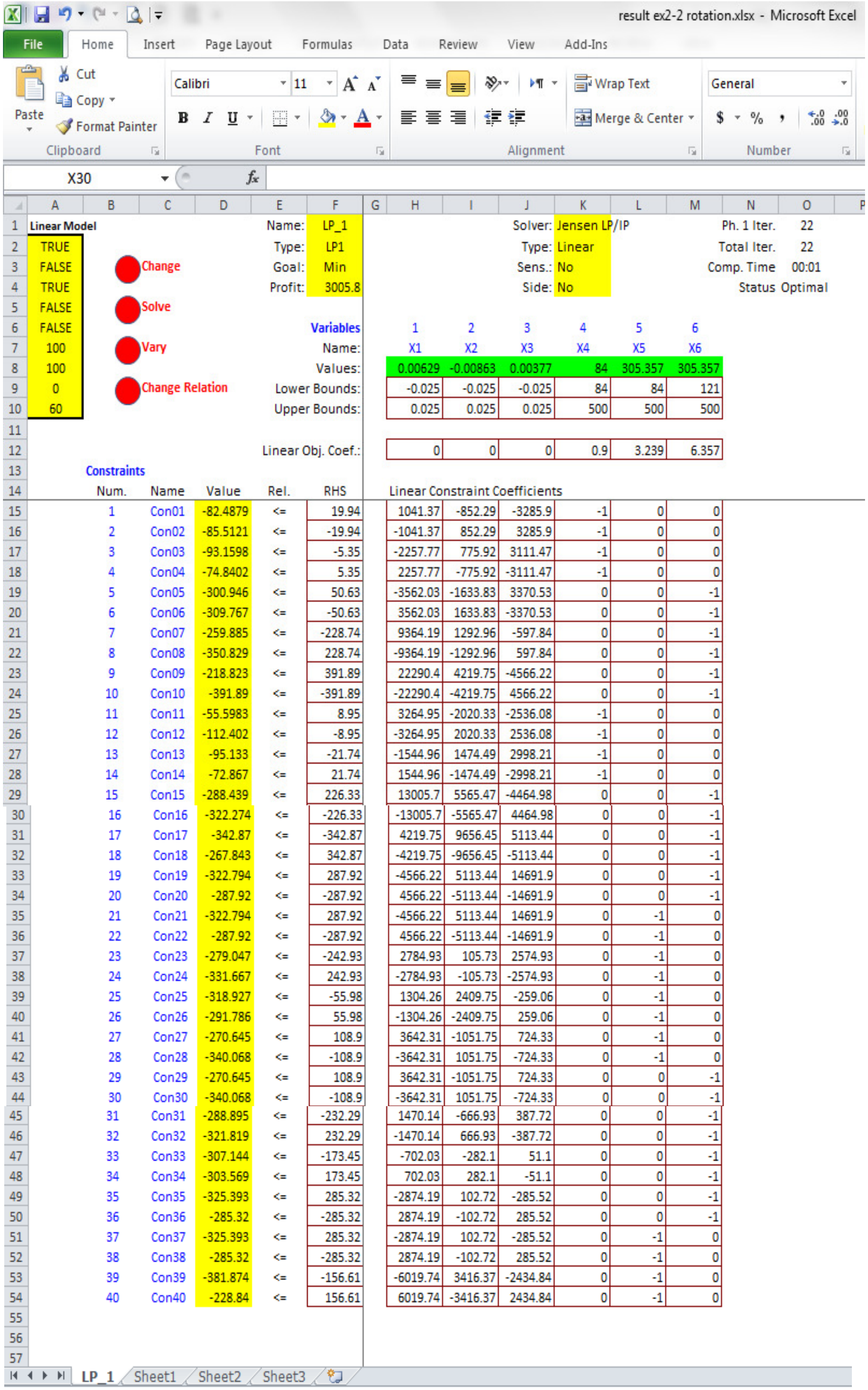
Optimum weight design of the frame is also obtained with considering plastic hinge rotation constraints in beams and columns. By implementing these constraints to the problem, optimal design of the system is performed once again. After 22 step of iteration, optimal solution is reached.

Plastic rotation limits for beams and columns, which are stated in ATC-40 code for different performance levels and were used in this study, are shown in Table 3.15.

In this two-bay, two-story reinforcement concrete frame, considering plastic hinges rotation limits according to the mentioned codes can verify and secure the frames stability and also prevent excessive rotation and displacement which can cause collapse of structure.

As pre-design, for the column d/b is 0,274/0,274 and 0,395/0,395 and for the beam d/b is 0,517/0,345 are chosen. The pre-design dimensions are obtained by considering the assumption that the plastic moments are proportion with the member lengths. In this state, the optimal solution may be obtained in one-step(Özer, 1975).

In the beginning of the procedure, the plastic hinge locations are chosen as shown in Figure 3.9 as in the past study (Özer, 1975). The analysis is performed for external loads and each of unit plastic hinge rotations separately and respectively, therefore, bending moments will be evaluated in critical sections. At each step of the successive approximation plastic hinge pattern is checked and the new pattern is considered at the next step if there is any changes. Furthermore, at each step objective function is linearized at the vicinity of plastic moment obtained at previous step.



Linear Model		Name:	LP_1	Solver:	Jensen LP/IP	Ph. 1 Iter.	22
TRUE		Type:	LP1	Type:	Linear	Total Iter.	22
FALSE	Change	Goal:	Min	Sens.:	No	Comp. Time	00:01
TRUE		Profit:	3005.8	Side:	No	Status	Optimal
FALSE	Solve						
FALSE							
100	Vary						
100							
0	Change Relation						
60							

Variables	1	2	3	4	5	6
Name:	X1	X2	X3	X4	X5	X6
Values:	0.00629	-0.00863	0.00377	84	305.357	305.357
Lower Bounds:	-0.025	-0.025	-0.025	84	84	121
Upper Bounds:	0.025	0.025	0.025	500	500	500
Linear Obj. Coef.:	0	0	0	0.9	3.239	6.357

Constraints

Num.	Name	Value	Rel.	RHS	Linear Constraint Coefficients					
1	Con01	-82.4879	<=	19.94	1041.37	-852.29	-3285.9	-1	0	0
2	Con02	-85.5121	<=	-19.94	-1041.37	852.29	3285.9	-1	0	0
3	Con03	-93.1598	<=	-5.35	-2257.77	775.92	3111.47	-1	0	0
4	Con04	-74.8402	<=	5.35	2257.77	-775.92	-3111.47	-1	0	0
5	Con05	-300.946	<=	50.63	-3562.03	-1633.83	3370.53	0	0	-1
6	Con06	-309.767	<=	-50.63	3562.03	1633.83	-3370.53	0	0	-1
7	Con07	-259.885	<=	-228.74	9364.19	1292.96	-597.84	0	0	-1
8	Con08	-350.829	<=	228.74	-9364.19	-1292.96	597.84	0	0	-1
9	Con09	-218.823	<=	391.89	22290.4	4219.75	-4566.22	0	0	-1
10	Con10	-391.89	<=	-391.89	-22290.4	-4219.75	4566.22	0	0	-1
11	Con11	-55.5983	<=	8.95	3264.95	-2020.33	-2536.08	-1	0	0
12	Con12	-112.402	<=	-8.95	-3264.95	2020.33	2536.08	-1	0	0
13	Con13	-95.133	<=	-21.74	-1544.96	1474.49	2998.21	-1	0	0
14	Con14	-72.867	<=	21.74	1544.96	-1474.49	-2998.21	-1	0	0
15	Con15	-288.439	<=	226.33	13005.7	5565.47	-4464.98	0	0	-1
16	Con16	-322.274	<=	-226.33	-13005.7	-5565.47	4464.98	0	0	-1
17	Con17	-342.87	<=	-342.87	4219.75	9656.45	5113.44	0	0	-1
18	Con18	-267.843	<=	342.87	-4219.75	-9656.45	-5113.44	0	0	-1
19	Con19	-322.794	<=	287.92	-4566.22	5113.44	14691.9	0	0	-1
20	Con20	-287.92	<=	-287.92	4566.22	-5113.44	-14691.9	0	0	-1
21	Con21	-322.794	<=	287.92	-4566.22	5113.44	14691.9	0	-1	0
22	Con22	-287.92	<=	-287.92	4566.22	-5113.44	-14691.9	0	-1	0
23	Con23	-279.047	<=	-242.93	2784.93	105.73	2574.93	0	-1	0
24	Con24	-331.667	<=	242.93	-2784.93	-105.73	-2574.93	0	-1	0
25	Con25	-318.927	<=	-55.98	1304.26	2409.75	-259.06	0	-1	0
26	Con26	-291.786	<=	55.98	-1304.26	-2409.75	259.06	0	-1	0
27	Con27	-270.645	<=	108.9	3642.31	-1051.75	724.33	0	-1	0
28	Con28	-340.068	<=	-108.9	-3642.31	1051.75	-724.33	0	-1	0
29	Con29	-270.645	<=	108.9	3642.31	-1051.75	724.33	0	0	-1
30	Con30	-340.068	<=	-108.9	-3642.31	1051.75	-724.33	0	0	-1
31	Con31	-288.895	<=	-232.29	1470.14	-666.93	387.72	0	0	-1
32	Con32	-321.819	<=	232.29	-1470.14	666.93	-387.72	0	0	-1
33	Con33	-307.144	<=	-173.45	-702.03	-282.1	51.1	0	0	-1
34	Con34	-303.569	<=	173.45	702.03	282.1	-51.1	0	0	-1
35	Con35	-325.393	<=	285.32	-2874.19	102.72	-285.52	0	0	-1
36	Con36	-285.32	<=	-285.32	2874.19	-102.72	285.52	0	0	-1
37	Con37	-325.393	<=	285.32	-2874.19	102.72	-285.52	0	-1	0
38	Con38	-285.32	<=	-285.32	2874.19	-102.72	285.52	0	-1	0
39	Con39	-381.874	<=	-156.61	-6019.74	3416.37	-2434.84	0	-1	0
40	Con40	-228.84	<=	156.61	6019.74	-3416.37	2434.84	0	-1	0

Figure 3.13: Excel macro sheet for the Example 2 with rotation constraint.

Column type and dimension, beam type and dimension, design variables and minimum weight value of optimum weight design with rotation constraints are presented in below.

Table 3.14: Design variables and the weight of frame for Example 2 with rotation constraints.

Design variables	(rad)
Φ1	0,0063
Φ2	-0,0086
Φ3	0,0038
Φ4	0,0053
Φ5	0,0077
Φ6	0,02
Φ7	-0,011
Φ8	0,008
mp1	84
mp2	306
mp3	306
w	3006

Optimum dimension of beams and column for Example.2 with rotation constraints are shown in Table 3.15.

Table 3.15: Optimum dimension of members for Example.2 with rotation constraints.

Member No	Member type	M_P	Optimum dimensions (h/b)m/m
1-2	Beam	306	0,511/0,341
3-4	Beam	306	0,511/0,341
4-5	Beam	306	0,511/0,341
3-1	Column	306	0,426/0,426
2-4	Column	306	0,426/0,426
5-8	Column	306	0,426/0,426
6-3	Column	84	0,274/0,274
4-7	Column	84	0,274/0,274

Example 3: The single bay, two-story plane reinforcement concrete frame

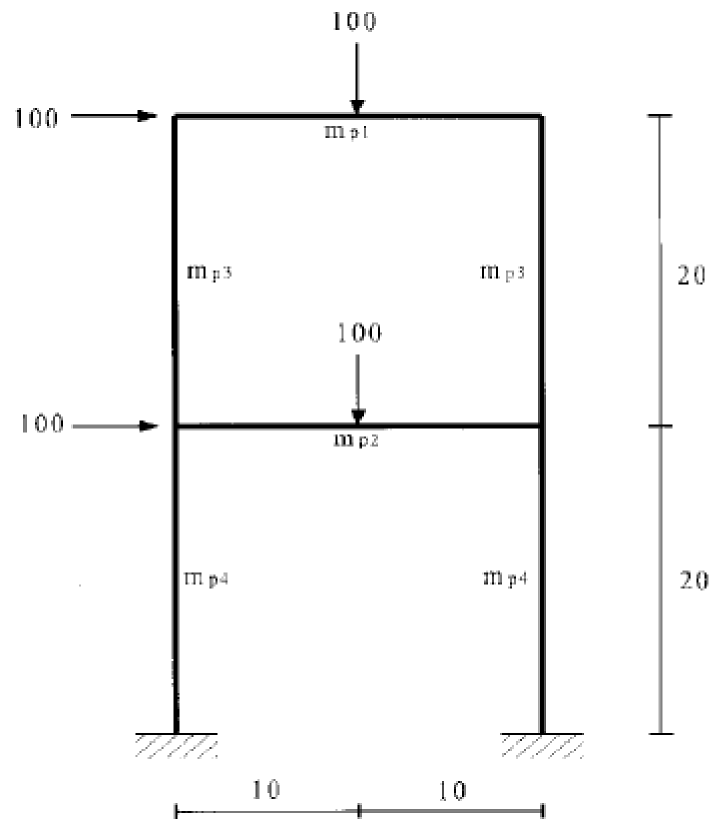


Figure 3.14: System geometry, external loads and critical sections for third example.

The weight optimization of the system with six degree of redundancy whose geometry, external loads and critical section are shown in Figure 3.14 will be done by method, which is proposed in this study.

The system consists of beams and columns whose relations between dimensional characteristics are given in previous chapters. In this example, columns with square cross section (d=b) and reinforcement percentage of 0.01 are chosen, and cross section of beams are rectangular with the height two times of the width (d=1.5b) and 0,006 reinforcement percentage.

As pre-design, for the column d/b is 0,496/0,496 and 0,625/0,625 and for the beam d/b is 0,603 /0,402 and 0,869/0,579 are chosen. The pre-design dimensions are obtained by considering the assumption that the plastic moments are proportion with the member lengths. In this state, the optimal solution may be obtained in one-step (Özer, 1975).

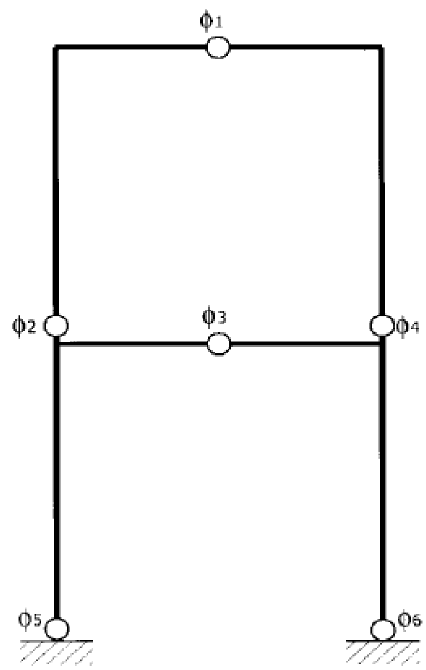
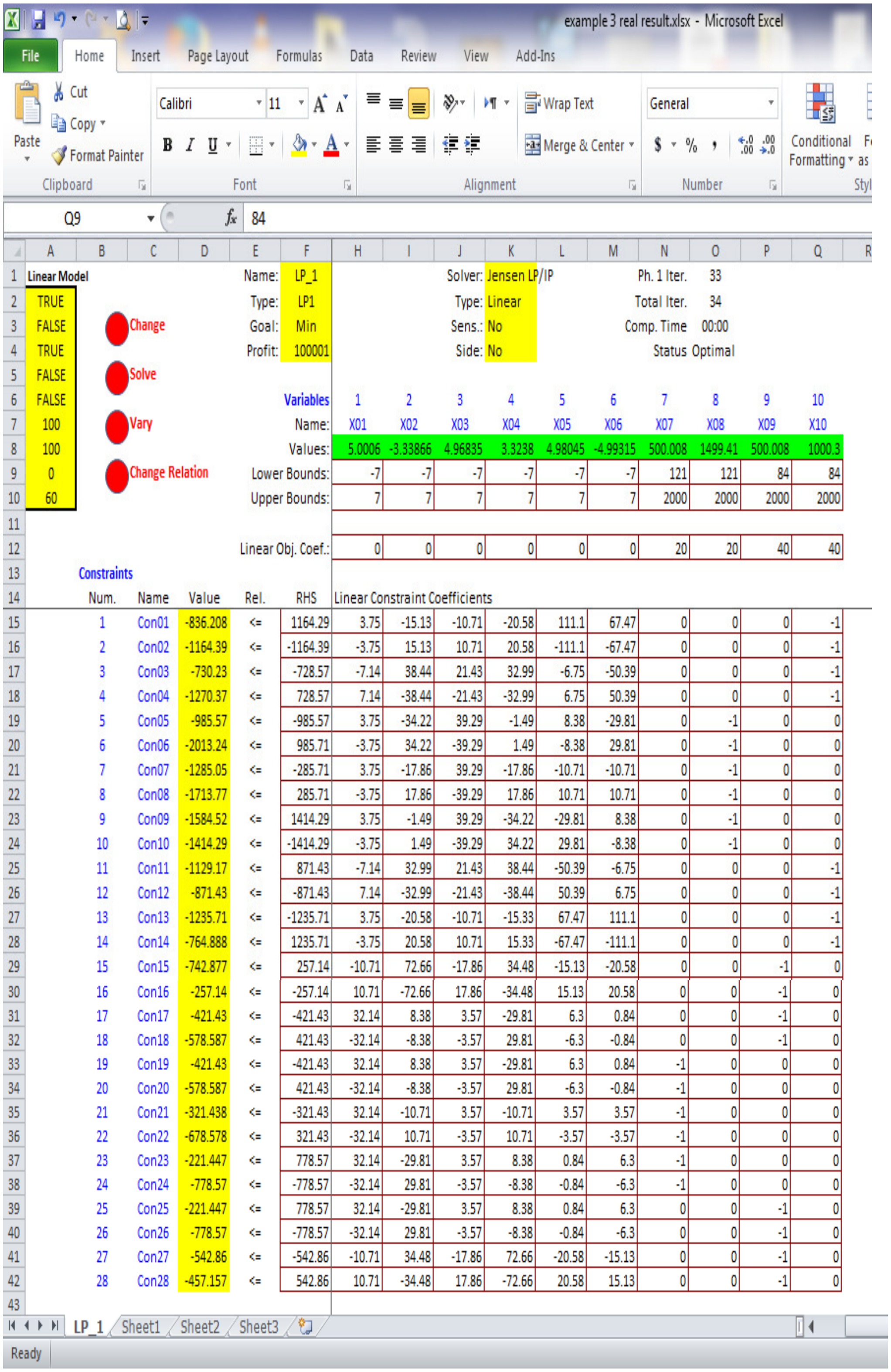


Figure 3.15: Introduced hinge locations for third example first step.

Result of the analysis, which is done, in the study of Özer (1975). And also evaluated in this thesis by excel macro in which only the length of the member are taken into account to evaluate the optimum weigh of the frame are given in Figure 3.16 and Table 3.16.

In the beginning of the procedure, the plastic hinge locations are chosen as shown in Figure 3.15 as in the past study (Özer, 1975). The analysis is performed for external loads and each of unit plastic hinge rotations separately and respectively, therefore, bending moments will be evaluated in critical sections. At each step of the successive approximation plastic hinge pattern is checked and the new pattern is considered at the next step if there is any changes. Furthermore, at each step objective function is linearized at the vicinity of plastic moment obtained at previous step.



	1	2	3	4	5	6	7	8	9	10
Name:	X01	X02	X03	X04	X05	X06	X07	X08	X09	X10
Values:	5.0006	-3.33866	4.96835	3.3238	4.98045	-4.99315	500.008	1499.41	500.008	1000.3
Lower Bounds:	-7	-7	-7	-7	-7	-7	121	121	84	84
Upper Bounds:	7	7	7	7	7	7	2000	2000	2000	2000
Linear Obj. Coef.:	0	0	0	0	0	0	20	20	40	40

Constraints

Num.	Name	Value	Rel.	RHS	Linear Constraint Coefficients									
1	Con01	-836.208	<=	1164.29	3.75	-15.13	-10.71	-20.58	111.1	67.47	0	0	0	-1
2	Con02	-1164.39	<=	-1164.39	-3.75	15.13	10.71	20.58	-111.1	-67.47	0	0	0	-1
3	Con03	-730.23	<=	-728.57	-7.14	38.44	21.43	32.99	-6.75	-50.39	0	0	0	-1
4	Con04	-1270.37	<=	728.57	7.14	-38.44	-21.43	-32.99	6.75	50.39	0	0	0	-1
5	Con05	-985.57	<=	-985.57	3.75	-34.22	39.29	-1.49	8.38	-29.81	0	-1	0	0
6	Con06	-2013.24	<=	985.71	-3.75	34.22	-39.29	1.49	-8.38	29.81	0	-1	0	0
7	Con07	-1285.05	<=	-285.71	3.75	-17.86	39.29	-17.86	-10.71	-10.71	0	-1	0	0
8	Con08	-1713.77	<=	285.71	-3.75	17.86	-39.29	17.86	10.71	10.71	0	-1	0	0
9	Con09	-1584.52	<=	1414.29	3.75	-1.49	39.29	-34.22	-29.81	8.38	0	-1	0	0
10	Con10	-1414.29	<=	-1414.29	-3.75	1.49	-39.29	34.22	29.81	-8.38	0	-1	0	0
11	Con11	-1129.17	<=	871.43	-7.14	32.99	21.43	38.44	-50.39	-6.75	0	0	0	-1
12	Con12	-871.43	<=	-871.43	7.14	-32.99	-21.43	-38.44	50.39	6.75	0	0	0	-1
13	Con13	-1235.71	<=	-1235.71	3.75	-20.58	-10.71	-15.33	67.47	111.1	0	0	0	-1
14	Con14	-764.888	<=	1235.71	-3.75	20.58	10.71	15.33	-67.47	-111.1	0	0	0	-1
15	Con15	-742.877	<=	257.14	-10.71	72.66	-17.86	34.48	-15.13	-20.58	0	0	-1	0
16	Con16	-257.14	<=	-257.14	10.71	-72.66	17.86	-34.48	15.13	20.58	0	0	-1	0
17	Con17	-421.43	<=	-421.43	32.14	8.38	3.57	-29.81	6.3	0.84	0	0	-1	0
18	Con18	-578.587	<=	421.43	-32.14	-8.38	-3.57	29.81	-6.3	-0.84	0	0	-1	0
19	Con19	-421.43	<=	-421.43	32.14	8.38	3.57	-29.81	6.3	0.84	-1	0	0	0
20	Con20	-578.587	<=	421.43	-32.14	-8.38	-3.57	29.81	-6.3	-0.84	-1	0	0	0
21	Con21	-321.438	<=	-321.43	32.14	-10.71	3.57	-10.71	3.57	3.57	-1	0	0	0
22	Con22	-678.578	<=	321.43	-32.14	10.71	-3.57	10.71	-3.57	-3.57	-1	0	0	0
23	Con23	-221.447	<=	778.57	32.14	-29.81	3.57	8.38	0.84	6.3	-1	0	0	0
24	Con24	-778.57	<=	-778.57	-32.14	29.81	-3.57	-8.38	-0.84	-6.3	-1	0	0	0
25	Con25	-221.447	<=	778.57	32.14	-29.81	3.57	8.38	0.84	6.3	0	0	-1	0
26	Con26	-778.57	<=	-778.57	-32.14	29.81	-3.57	-8.38	-0.84	-6.3	0	0	-1	0
27	Con27	-542.86	<=	-542.86	-10.71	34.48	-17.86	72.66	-20.58	-15.13	0	0	-1	0
28	Con28	-457.157	<=	542.86	10.71	-34.48	17.86	-72.66	20.58	15.13	0	0	-1	0

Figure 3.16: Excel macro sheet and yield condition constraints and the weight function for Example 3 taking into account only length of member.

Table 3.16: Design variables and weight function for Example 3 taking into account only length of member.

Design variables	(rad)
Φ1	5
Φ2	-3,339
Φ3	4,968
Φ4	3,324
Φ5	4,98
Φ6	-4,993
mp1	500
mp2	1499
mp3	500
mp4	1000
w	100001

Column type and dimension, beam type and dimension, design variables and minimum weight value of optimum weight design are presented in below.

Table 3.17: Optimum dimension of members for Example 3 with taking into account only length of member.

Member No	Member type	M_P	Optimum dimensions (d/b) m/m
1-2	Beam	500	0,603/0,402
3-4	Beam	1499	0,869/0,579
3-1	Column	500	0,496/0,496
2-4	Column	500	0,496/0,496
5-3	Column	1000	0,625/0,625
4-6	Column	1000	0,625/0,625

In the beginning of the procedure, the plastic hinge locations are chosen as shown in Figure 3.15 as in the past study (Özer, 1975). The analysis is performed for external loads and each of unit plastic hinge rotations separately and respectively, therefore, bending moments will be evaluated in critical sections. At each step of the successive approximation plastic hinge pattern is checked and the new pattern is considered at the next step if there is any changes. Furthermore, at each step objective function is linearized at the vicinity of plastic moment obtained at previous step. Plastic hinge pattern for Example 3 are shown in Figure 3.17.

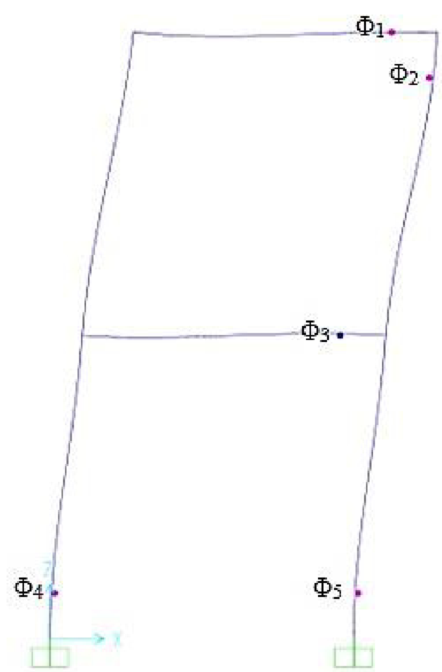
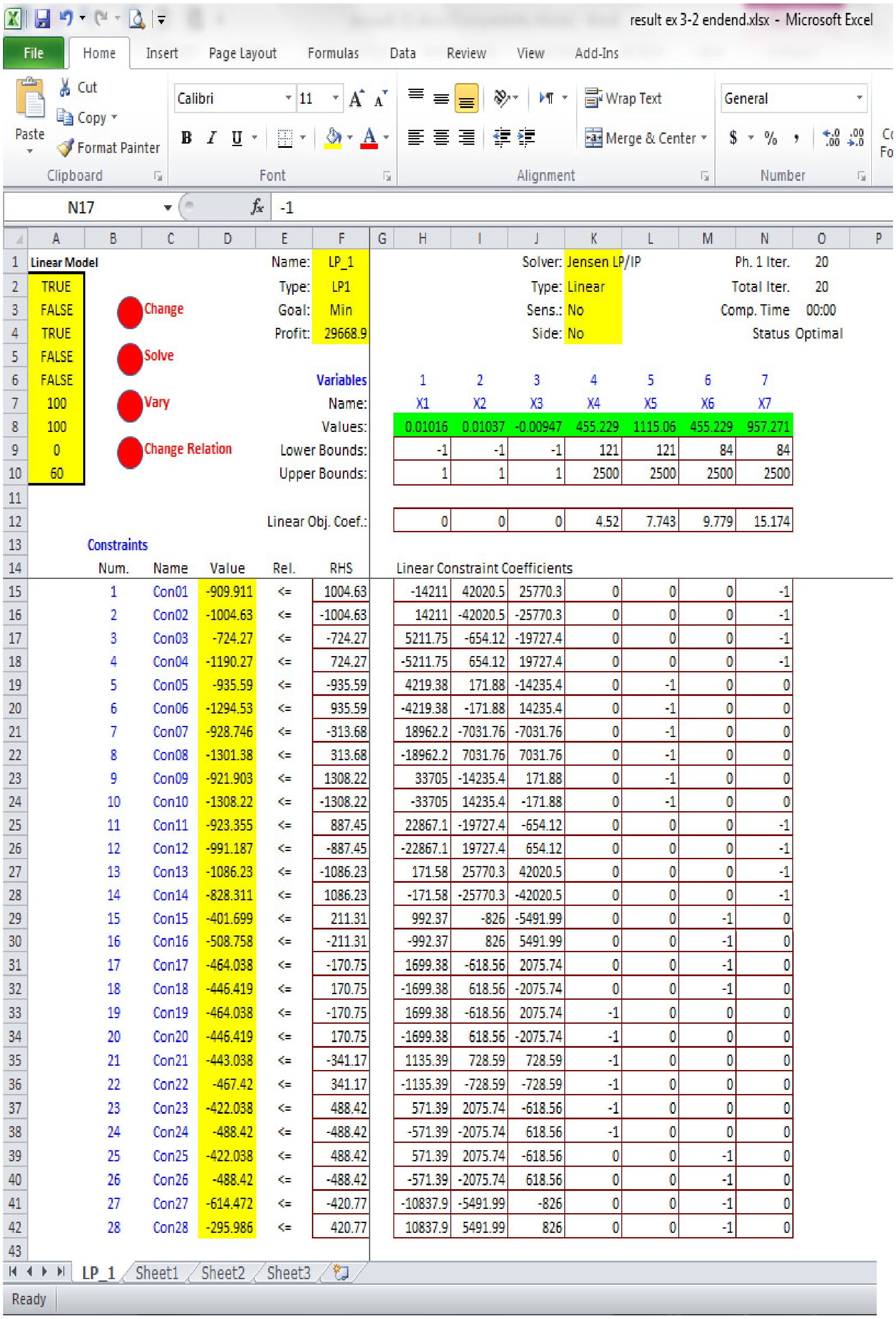


Figure 3.17: Hinges location for optimum weight design of Example 3.

The optimal solution is reach after 20 iterations by using Excel macro sheet.Results will be shown in Figure 3.18 and Table 3.18.



result ex 3-2 endend.xlsx - Microsoft Excel

N17 | f_x -1

Linear Model		Name:	LP_1	Solver:	Jensen LP/IP
TRUE		Type:	LP1	Type:	Linear
FALSE	Change	Goal:	Min	Sens.:	No
TRUE		Profit:	29668.9	Side:	No
FALSE	Solve				
FALSE					
100	Vary				
100					
0	Change Relation				
60					

Ph. 1 Iter.	20
Total Iter.	20
Comp. Time	00:00
Status	Optimal

Variables	1	2	3	4	5	6	7
Name:	X1	X2	X3	X4	X5	X6	X7
Values:	0.01016	0.01037	-0.00947	455.229	1115.06	455.229	957.271
Lower Bounds:	-1	-1	-1	121	121	84	84
Upper Bounds:	1	1	1	2500	2500	2500	2500
Linear Obj. Coef.:	0	0	0	4.52	7.743	9.779	15.174

Constraints

Num.	Name	Value	Rel.	RHS	Linear Constraint Coefficients						
1	Con01	-909.911	<=	1004.63	-14211	42020.5	25770.3	0	0	0	-1
2	Con02	-1004.63	<=	-1004.63	14211	-42020.5	-25770.3	0	0	0	-1
3	Con03	-724.27	<=	-724.27	5211.75	-654.12	-19727.4	0	0	0	-1
4	Con04	-1190.27	<=	724.27	-5211.75	654.12	19727.4	0	0	0	-1
5	Con05	-935.59	<=	-935.59	4219.38	171.88	-14235.4	0	-1	0	0
6	Con06	-1294.53	<=	935.59	-4219.38	-171.88	14235.4	0	-1	0	0
7	Con07	-928.746	<=	-313.68	18962.2	-7031.76	-7031.76	0	-1	0	0
8	Con08	-1301.38	<=	313.68	-18962.2	7031.76	7031.76	0	-1	0	0
9	Con09	-921.903	<=	1308.22	33705	-14235.4	171.88	0	-1	0	0
10	Con10	-1308.22	<=	-1308.22	-33705	14235.4	-171.88	0	-1	0	0
11	Con11	-923.355	<=	887.45	22867.1	-19727.4	-654.12	0	0	0	-1
12	Con12	-991.187	<=	-887.45	-22867.1	19727.4	654.12	0	0	0	-1
13	Con13	-1086.23	<=	-1086.23	171.58	25770.3	42020.5	0	0	0	-1
14	Con14	-828.311	<=	1086.23	-171.58	-25770.3	-42020.5	0	0	0	-1
15	Con15	-401.699	<=	211.31	992.37	-826	-5491.99	0	0	-1	0
16	Con16	-508.758	<=	-211.31	-992.37	826	5491.99	0	0	-1	0
17	Con17	-464.038	<=	-170.75	1699.38	-618.56	2075.74	0	0	-1	0
18	Con18	-446.419	<=	170.75	-1699.38	618.56	-2075.74	0	0	-1	0
19	Con19	-464.038	<=	-170.75	1699.38	-618.56	2075.74	-1	0	0	0
20	Con20	-446.419	<=	170.75	-1699.38	618.56	-2075.74	-1	0	0	0
21	Con21	-443.038	<=	-341.17	1135.39	728.59	728.59	-1	0	0	0
22	Con22	-467.42	<=	341.17	-1135.39	-728.59	-728.59	-1	0	0	0
23	Con23	-422.038	<=	488.42	571.39	2075.74	-618.56	-1	0	0	0
24	Con24	-488.42	<=	-488.42	-571.39	-2075.74	618.56	-1	0	0	0
25	Con25	-422.038	<=	488.42	571.39	2075.74	-618.56	0	0	-1	0
26	Con26	-488.42	<=	-488.42	-571.39	-2075.74	618.56	0	0	-1	0
27	Con27	-614.472	<=	-420.77	-10837.9	-5491.99	-826	0	0	-1	0
28	Con28	-295.986	<=	420.77	10837.9	5491.99	826	0	0	-1	0

LP_1 | Sheet1 | Sheet2 | Sheet3

Ready

Figure 3.18: Excel macro sheet for the Example 3 without rotation constraints.

Table 3.18: Design variable and weight of frame for Example 3 without rotation constraints.

Design variables	(rad)
Φ1	0,01
Φ2	0,01
Φ3	-0,0095
Φ4	-0,046
Φ5	0,047
mp1	452
mp2	1115
mp3	455
mp4	957
w	29668

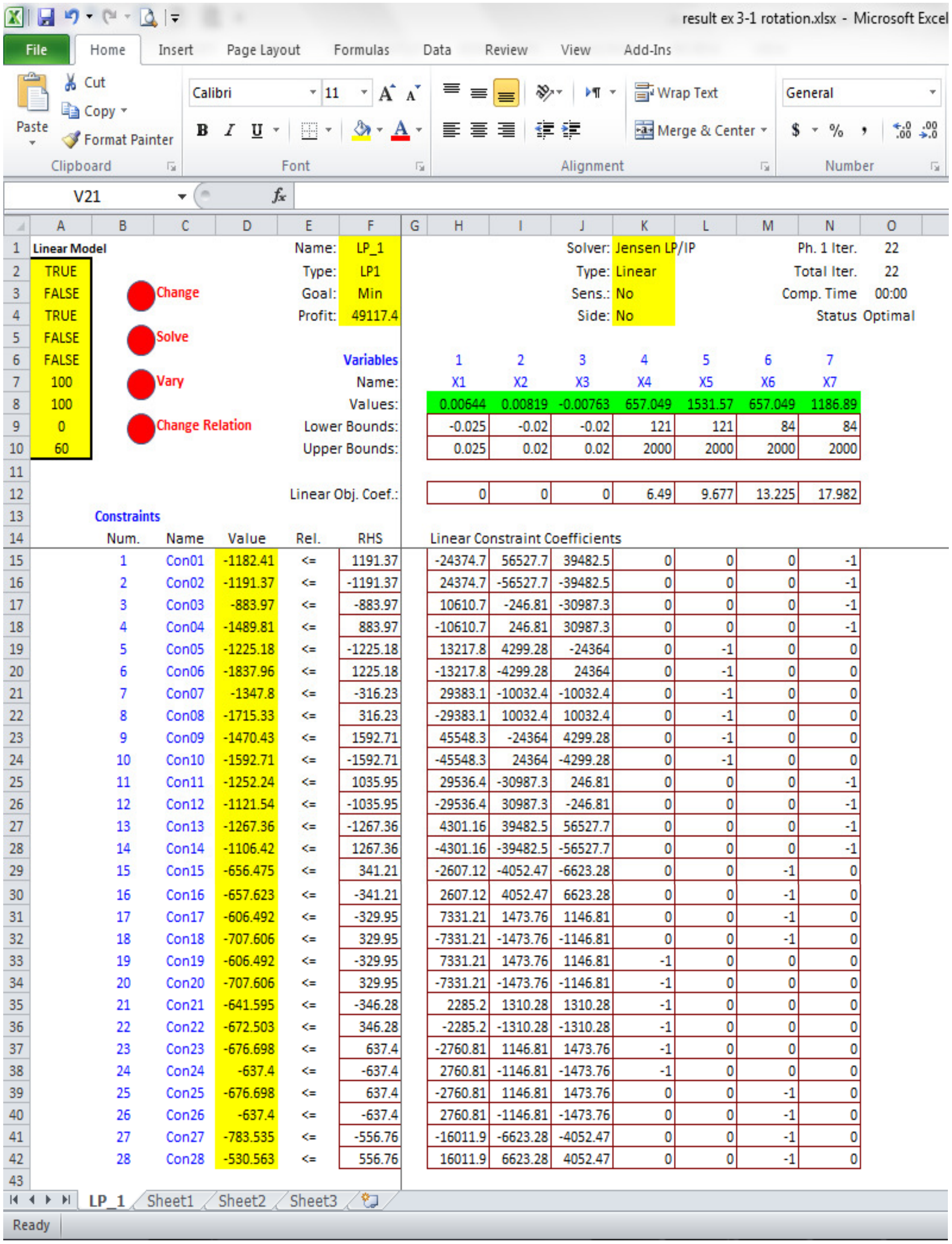
In order to evaluate optimum dimension of loading members, relation between cross sectional characteristics, which are expressed in related chapter, will be used by considering values in Table 3.18.

Table 3.19: Optimum dimension of members for Example 2 without rotation constraints.

Member No	Member type	M_P	Optimum dimensions (d/b) m/m
1-2	Beam	455	0,584/0,389
3-4	Beam	1115	0,787/0,525
3-1	Column	455	0,481/0,481
2-4	Column	455	0,481/0,481
5-3	Column	957	0,616/0,616
4-6	Column	957	0,616/0,616

Optimum weight design of the frame is also obtained with considering plastic hinge rotation constraints in beams and columns. By implementing these constraints to the problem, optimal design of the system is performed once again. After 22 step of iteration, optimal solution is reached.

Plastic rotation limits for beams and columns, which are stated in ATC-40 code for different performance levels and were used in this study, are shown in Table 3.5.In this two-bay, two-story reinforcement concrete frame, considering plastic hinges rotation limits according to the mentioned codes can verify and secure the frames stability and also prevent excessive rotation and displacement which can cause collapse of structure.



	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	Linear Model				Name:	LP_1				Solver:	Jensen LP/IP			Ph. 1 Iter.	22
2	TRUE				Type:	LP1				Type:	Linear			Total Iter.	22
3	FALSE	Change			Goal:	Min				Sens.:	No			Comp. Time	00:00
4	TRUE				Profit:	49117.4				Side:	No			Status	Optimal
5	FALSE	Solve													
6	FALSE					Variables		1	2	3	4	5	6	7	
7	100	Vary				Name:		X1	X2	X3	X4	X5	X6	X7	
8	100					Values:		0.00644	0.00819	-0.00763	657.049	1531.57	657.049	1186.89	
9	0	Change Relation				Lower Bounds:		-0.025	-0.02	-0.02	121	121	84	84	
10	60					Upper Bounds:		0.025	0.02	0.02	2000	2000	2000	2000	
11															
12						Linear Obj. Coef.:		0	0	0	6.49	9.677	13.225	17.982	
13		Constraints													
14		Num.	Name	Value	Rel.	RHS		Linear Constraint Coefficients							
15		1	Con01	-1182.41	<=	1191.37		-24374.7	56527.7	39482.5	0	0	0	-1	
16		2	Con02	-1191.37	<=	-1191.37		24374.7	-56527.7	-39482.5	0	0	0	-1	
17		3	Con03	-883.97	<=	-883.97		10610.7	-246.81	-30987.3	0	0	0	-1	
18		4	Con04	-1489.81	<=	883.97		-10610.7	246.81	30987.3	0	0	0	-1	
19		5	Con05	-1225.18	<=	-1225.18		13217.8	4299.28	-24364	0	-1	0	0	
20		6	Con06	-1837.96	<=	1225.18		-13217.8	-4299.28	24364	0	-1	0	0	
21		7	Con07	-1347.8	<=	-316.23		29383.1	-10032.4	-10032.4	0	-1	0	0	
22		8	Con08	-1715.33	<=	316.23		-29383.1	10032.4	10032.4	0	-1	0	0	
23		9	Con09	-1470.43	<=	1592.71		45548.3	-24364	4299.28	0	-1	0	0	
24		10	Con10	-1592.71	<=	-1592.71		-45548.3	24364	-4299.28	0	-1	0	0	
25		11	Con11	-1252.24	<=	1035.95		29536.4	-30987.3	246.81	0	0	0	-1	
26		12	Con12	-1121.54	<=	-1035.95		-29536.4	30987.3	-246.81	0	0	0	-1	
27		13	Con13	-1267.36	<=	-1267.36		4301.16	39482.5	56527.7	0	0	0	-1	
28		14	Con14	-1106.42	<=	1267.36		-4301.16	-39482.5	-56527.7	0	0	0	-1	
29		15	Con15	-656.475	<=	341.21		-2607.12	-4052.47	-6623.28	0	0	-1	0	
30		16	Con16	-657.623	<=	-341.21		2607.12	4052.47	6623.28	0	0	-1	0	
31		17	Con17	-606.492	<=	-329.95		7331.21	1473.76	1146.81	0	0	-1	0	
32		18	Con18	-707.606	<=	329.95		-7331.21	-1473.76	-1146.81	0	0	-1	0	
33		19	Con19	-606.492	<=	-329.95		7331.21	1473.76	1146.81	-1	0	0	0	
34		20	Con20	-707.606	<=	329.95		-7331.21	-1473.76	-1146.81	-1	0	0	0	
35		21	Con21	-641.595	<=	-346.28		2285.2	1310.28	1310.28	-1	0	0	0	
36		22	Con22	-672.503	<=	346.28		-2285.2	-1310.28	-1310.28	-1	0	0	0	
37		23	Con23	-676.698	<=	637.4		-2760.81	1146.81	1473.76	-1	0	0	0	
38		24	Con24	-637.4	<=	-637.4		2760.81	-1146.81	-1473.76	-1	0	0	0	
39		25	Con25	-676.698	<=	637.4		-2760.81	1146.81	1473.76	0	0	-1	0	
40		26	Con26	-637.4	<=	-637.4		2760.81	-1146.81	-1473.76	0	0	-1	0	
41		27	Con27	-783.535	<=	-556.76		-16011.9	-6623.28	-4052.47	0	0	-1	0	
42		28	Con28	-530.563	<=	556.76		16011.9	6623.28	4052.47	0	0	-1	0	
43															

Figure 3.19: Excel macro sheet for the Example 3 with rotation constraints.

Column type and dimension, beam type and dimension, design variables and minimum weight value of optimum weight design with rotation constraints are presented in below.

Table 3.20: Design variables and the weight of frame for Example 3 with rotation constraints.

Design variables	(rad)
Φ_1	0,0064
Φ_2	0,0082
Φ_3	-0,0076
Φ_4	-0,02
Φ_5	0,015
m_{p1}	657
m_{p2}	1532
m_{p3}	657
m_{p4}	1187
w	49117

Optimum dimension of beams and column for Example.3 with rotation constraints are shown in Table 3.21.

Table3.21: Optimum dimension of members for Example 3 with rotation constraints.

Member No	Member type	M_P	Optimum dimensions (d/b) m/m
1-2	Beam	657	0,66/0,44
3-4	Beam	1532	0,875/0,583
3-1	Column	657	0,544/0,544
2-4	Column	657	0,544/0,544
5-3	Column	1187	0,662/0,662
4-6	Column	1187	0,662/0,662

4.CONCLUSION

In this thesis, a minimum weight design formulation for R/C plane frames is given considering the first order limit load. The formulation is based on matrix displacement method. As the equilibrium equations and yield conditions at the plastic hinges are linear constraints, minimum weight design problem is transformed into a linear programming problem and the problem is solved by using an Excel macro based on Simplex Method. In the minimum design formulation, the relationships between plastic moments and reinforcement ratio or between plastic moments and cross section heights are linearized by a Newton-Raphson algorithm. As the design variables of the problem displacements and plastic hinge rotations, the constraints on these variables may also easily be added to the problem. Therefore, the limitations on story drifts and plastic hinge rotation given in earthquake codes can be considered in the minimum weight design. As the plastic hinge locations change by the cross sectional characteristics obtained previous design step, a pushover analysis is necessary before the next design step. Numerical examples show that the minimum weight design without considering the story drift and plastic hinge limitations is unsafe and the constraints on themmust be considered together with the yield conditions at the plastic hinges in the minimum weight design of R/C frames. By considering the limitations on the plastic hinge rotations and the story drifts in the minimum weight design problem, designed frames fulfill the anticipated collapse safety against earthquake loads.

Real weight of frames are evaluated by considering reinforcement weight per unit volume 78.5 KN/m3 and concrete weight per unit volume 25 KN/m3.Resultsare shown in Table 4.1.

Table 4.1: Analysis results for thethree examples.

Example No	Real weight considering only length of member (KN/m^3)	Real weight with rotation constraints (KN/m^3)	Real weight without rotation constraints (KN/m^3)
1	47,3887	47,4332	47,8451
2	257,3077	264,4805	265,4167
3	1025,571	945,6869	1152,4393

As shown in Table 4.1, in all three types of frames, the real weights of optimum design with rotation constraints are bigger than the weight without rotation constraints. In Example 1 and Example 2 since only one plastic hinge rotation is bigger than the rotation constraints limit in code, the difference between two weights are not so much. However, in Example 3, two of the hinge rotations do not conform code requirement; therefore, the difference between results is considrable.

Furthermore, it can be cocluded that the weight of frames and cross section types of the members change depending on the type of the optimum weight design. In Example 1 and Example 2 weight of the frames increase when optimization are done by considering the nonlinearity of objective function in camparison to the optimization in which the material are assumed homogeneous. Nevertheless, in Example 3 the result is reverse; therefore, the result of optimization in this situation depends on the type of the frames.

Moreover, in Example 1 , an additional constraints are stated in which the plastic moment of the column should be bigger than beam and in this situation the optimum weight of the frame is evaluated 51,5086 KN/m^3 . By comparing this weight with previously obtained weight, it is clear that the optimum weight of frame increases when the additional constraints are implemented.

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