

OPTIMUM DESIGN OF REINFORCED CONCRETE PLANE FRAMES
USING HARMONY SEARCH ALGORITHM

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USING HARMONY SEARCH ALGORITHM**

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ABSTRACT

OPTIMUM DESIGN OF REINFORCED CONCRETE PLANE FRAMES USING HARMONY SEARCH ALGORITHM

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In this thesis, the optimum design algorithm is presented for reinforced concrete special moment frames. The objective function is considered as the total cost of reinforced concrete frame which includes the cost of concrete, formwork and reinforcing steel bars. The cost of any component is inclusive of material, fabrication and labor. The design variables in beams are selected as the width and the depth of beams in each span, the diameter and the number of longitudinal reinforcement bars along the span and supports. In columns the width and the depth of the column section, the number and the diameter of bars in x and y directions are selected as design variables. The column section database is prepared which includes the width and height of column section, the diameter and the number of reinforcing bars in the column section is constructed. This database is used by the design algorithm to select appropriate sections for the columns of the frame under consideration. The design constraints are implemented from ACI 318-05 which covers the flexural and shear strength, serviceability, the minimum

and maximum steel percentage for flexural and shear reinforcement, the spacing requirements for the reinforcing bars and the upper and lower bound requirements for the concrete sections. The optimum design problem formulated according to ACI 318-05 provisions with the design variables mentioned above turns out to be a combinatorial optimization problem. The solution of the design problem is obtained by using the harmony search algorithm (HS) which is one of the recent additions to meta-heuristic optimization techniques which are widely used in obtaining the solution of combinatorial optimization problems. The HS algorithm is quite simple and has few parameters to initialize and consists of simple steps which make it easy to implement. Number of design examples is presented to demonstrate the efficiency and robustness of the optimum design algorithm developed.

Keywords: structural optimization, combinatorial optimization, harmony search algorithm, special moment frames, reinforced concrete, cost optimization, discrete variables,

ÖZ

BETONARME DÜZLEM ÇERÇEVELERİN ARMONİ TARAMA YÖNTEMİ İLE OPTİMUM TASARIMI

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Doktora, Mühendislik Bilimleri Bölümü

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Bu tezde süneklik düzeyi yüksek betonarme çerçeveler için optimum tasarım algoritması sunulmuştur. Amaç fonksiyonu olarak betonarme çerçevenin beton, demir donatı ve kalıp maliyetlerini içeren toplam maliyeti seçilmiştir. Çerçeveyi oluşturan her elemanın maliyeti malzeme, üretim ve işçilik masraflarını içermektedir. Kiriş elemanların kesit boyutları, kiriş açıklıklarındaki ve mesnetlerdeki donatıların çap ve sayıları tasarım değişkeni olarak alınmıştır. Kolonlarda ise kolon kesitinin eni ve derinliği ile x ve y doğrultularındaki donatıların çapları ve toplam sayıları tasarım değişkeni olarak seçilmiştir. Kolonlar için kolonların en, boy, donatı çapı ve kesitteki toplam sayısını içeren bir kolon veri tabanı oluşturulmuştur. Bu veri tabanı geliştirilen optimum tasarım algoritması tarafından göz önüne alınan betonarme çerçevenin kolonları için gerekli kesitlerin seçiminde kullanılmıştır. Tasarım sınırlayıcıları ACI 318-05 de verilen eğilme momenti dayanımı, kesme kuvveti dayanımı, kullanılabilirlik, minimum ve maximum donatı oranları, donatı çubuklar arasındaki boşluklar ve

betonarme kesit boyutları ile ilgili sınırlayıcılar olarak uygulanmıştır. Yukarıda bahsedilen tasarım değişkenleri ile ACI 318-05 de verilen kurallara göre formüle edilen optimum tasarım problemi kombinasyonel optimizasyon problemine dönüşmektedir. Tasarım probleminin çözümü yeni bir kombinasyonel optimizasyon problemlerinin çözümünde çokça kullanılan sezgi ötesi bir yöntem olan harmoni arama algoritması uygulanarak elde edilmiştir. Harmoni arama algoritması oldukça basit, başlangıçta az karakteristik özelliğe sahip olan ve uygulama açısından basit adımlar içeren bir yöntemdir. Sunulan algortimanın gücünü ve etkinliğini sergilmek için çeşitli sayıda tasarım örneği sunulmuştur.

Anahtar kelimeler: Optimum yapı tasarımı, kombinasyonel optimizasyon, armoni tarama yöntemi, süneklik düzeyi yüksek çerçeveler, betonarme, maliyet optimizasyonu, ayrık değişkenler

To My Family,

For your endless support and love

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TABLE OF CONTENTS

ABSTRACT	iv
ÖZ	vi
ACKNOWLEDGEMENTS	ix
TABLE OF CONTENTS	x
LIST OF FIGURES	xiv
LIST OF TABLES	xviii
LIST OF ABBREVIATIONS	xx
CHAPTERS	
1.INTRODUCTION	1
1.1 General	1
1.2 Structural Optimization	2
1.3 Elements of Structural Optimization.....	6
1.3.1 Design Variables	7
1.3.2 Objective Function	8
1.3.3 Constraints	8
1.4 Literature Review on Optimum Design of Reinforced Concrete Structures	9
1.5 The Scope of the Thesis	13
2.HARMONY SEARCH ALGORITHM	17
2.1 Meta-heuristic Methods	17
2.1.1 The Genetic algorithm	18

2.1.2 Evolutionary Strategies	19
2.1.3 Simulated Annealing	20
2.1.4 Ant Colony Optimization.....	20
2.1.5 Particle Swarm Optimization	21
2.1.6 Bee Colony Optimization.....	22
2.2 Harmony Search Method	23
2.2.1 Case Study 1: Constrained Function Optimization.....	31
2.2.2 Case Study 2: Pressure Vessel Design	32
2.2.3 Case Study 3: Welded Beam Design	34
2.3 Applications of Harmony Search Algorithm	37
3.MATHEMATICAL FORMULATION OF OPTIMUM DETAILING DESIGN PROBLEM OF REINFORCED CONCRETE PLANE FRAMES	43
3.1 Introduction	43
3.2 Objective Function	44
3.3 Design Variables	46
3.3.1 Beam Design variables.....	47
3.3.2 Column Design Variables	51
3.4 Constraints.....	62
3.4.1 Constraints for Column Groups	63
3.4.2 Constraints for Beam Groups.....	69
3.4.3 Constraints for Joints	75
4.OPTIMUM DESIGN ALGORITHM FOR REINFORCED CONCRETE FRAMES.....	78

4.1 Introduction	78
4.2 Optimum Design Process with Harmony Search Algorithm	78
5.DESIGN EXAMPLES	82
5.1 Introduction	82
5.2 Two-Bay Seven-Story RC Frame	83
5.3 Two-Bay Five-Story RC Frame	92
5.4 Two-Bay Six-Story RC Frame	111
5.5 Five-Bay Five-Story RC Frame	119
6.DISCUSSION	128
6.1 Discussion	128
7.SUMMARY AND CONCLUSIONS	134
7.1 General	134
7.2 This Study	135
7.3 Recommendations for Future Studies	137
References	138
APPENDIX A. ANALYSIS OF PLANE FRAMES WITH MATRIX DISPLACEMENT METHOD	148
A.1 Degree of Freedoms	148
A.2 Member Stiffness Matrix	149
A.3 Displacement and Force Transformation Matrices	152
A.3.1 Displacement transformation Matrix	152
A.3.2 Force Transformation Matrix	154
A.3.3 Overall Stiffness Matrix	155

A.4 General steps for plane frame analysis.....	155
APPENDIX B. CONCRETE DESIGN of SPECIAL SEISMIC MOMENT FRAMES (ACI 318-05 / IBC 2006)	157
B.1 Design Load Combinations	157
B.2 Column Design.....	158
B.3 Beam Design	168
APPENDIX C. SEISMIC LOADS (IBC 2006 / ASCE 7-05)	175
CURRICULUM VITAE	181

LIST OF FIGURES

FIGURES

Figure 1.1	The conventional design process	3
Figure 1.2	The optimum design process	4
Figure 2.1	The main model of Genetic Algorithm	19
Figure 2.2	The pseudo-code for the Bees Algorithm	23
Figure 2.3	Analogy between musical improvisation and optimization	26
Figure 2.4	Harmony search algorithm optimization procedure	30
Figure 2.5	Schematic of pressure vessel	32
Figure 2.6	Schematic of welded beam	34
Figure 2.7	BakRyun water distribution network	38
Figure 2.8	Cross section of slope	39
Figure 2.9	20-city travelling salesman problem	39
Figure 2.10	Two-hundred-bar planar truss	40
Figure 2.11	Seventy-two-bar space truss	41
Figure 2.12	Optimum geometry for geodesic dome with four rings	41
Figure 2.13	60-member grillage system	42
Figure 2.14	Three-bay twenty four-storey frame example	42
Figure 3.1	The reinforcement bar layout and the design variables for i^{th} beam group	49
Figure 3.2	The design variables for construction of column section database	53
Figure 4.1	Optimum design procedure of RC frames with Harmony search algorithm	81
Figure 5.1	Two-bay seven-storey RC plane frame example	84
Figure 5.2	The design history for two-bay seven-storey RC plane frame	87

Figure 5.3	The detailing reinforcement of beam group-1 in the optimum two-bay seven-storey RC plane frame	88
Figure 5.4	The detailing reinforcement of beam group-2 in the optimum two-bay seven-storey RC plane frame	89
Figure 5.5	The section details and the interaction diagrams for column design groups in two-bay seven-storey RC plane frame, (a) for column design group 1 and (b) for column design group 2	90
Figure 5.6	Two-bay five-storey RC plane frame	94
Figure 5.7	Design history for two-bay five-storey RC plane frame (<i>for I=1.00</i>)	96
Figure 5.8	The detailing reinforcement of beam group-1 in the optimum design of two-bay five-storey RC plane frame (<i>for I=1.00</i>)	97
Figure 5.9	The detailing reinforcement of beam group-2 in the optimum design of two-bay five-storey RC plane frame (<i>for I=1.00</i>)	98
Figure 5.10	The section detailing and interaction diagrams for column design groups in two-bay five-storey RC plane frame (<i>for I=1.00</i>), (a) for column group 1 and (b) for column group 2	99
Figure 5.11	Design history for two-bay five-storey RC plane frame (<i>for I=1.25</i>)	101
Figure 5.12	The detailing reinforcement of beam group-1 in the optimum design of two-bay five-storey RC plane frame (<i>for I=1.25</i>)	102
Figure 5.13	The detailing reinforcement of beam group-2 in the optimum design of two-bay five-storey RC plane frame (<i>for I=1.25</i>).	103
Figure 5.14	The section detailing and interaction diagrams for column design groups designed in two-bay five-storey RC plane frame (<i>for I=1.25</i>), (a) for column design group 1 and (b) for column design group 2	104
Figure 5.15	Design history for two-bay five-storey RC plane frame (<i>for I=1.50</i>)	106
Figure 5.16	The detailing reinforcement of beam group-1 in the optimum design of two-bay five-storey RC plane frame (<i>for I=1.50</i>).	107

Figure 5.17	The detailing reinforcement of beam group-2 in the optimum design of two-bay five-storey RC plane frame (for $I=1.50$)	108
Figure 5.18	The section detailing and interaction diagrams for column design groups designed in two-bay five-storey RC plane frame (for $I=1.50$), (a) for column design group 1 and (b) for column design group 2	109
Figure 5.19	Two-bay six-storey RC plane frame	113
Figure 5.20	The design history for two-bay six-storey RC plane frame (load combinations are not considered)	115
Figure 5.21	The detailing reinforcement of beam group-1 in the optimum design of two-bay six-storey RC plane frame	116
Figure 5.22	The detailing reinforcement of beam group-2 in the optimum design of two-bay six-storey RC plane frame	117
Figure 5.23	The section detailing and interaction diagrams for column design groups in two-bay six-storey RC plane frame, (a) for column design group 1 and 3, (b) for column design group 2	118
Figure 5.24	Five-bay five-storey RC plane frame	120
Figure 5.25	Gravity loading for five-bay five-storey frame (a) for beams at first and last spans, (b) for beams at second and fourth spans, (c) for beams at third span	121
Figure 5.26	The design memory history for the design of five-bay five-storey RC plane frame (for Case 1)	124
Figure 5.27	The design history for five-bay five-storey RC plane frame (for Case 2)	126
Figure 6.1	The requirement reinforcement steel areas (cm^2) for optimum sections of two-bay seven-storey RC plane frame example ($I=1.00$) (SAP2000 results).	120
Figure 6.2	The placed reinforcement steel areas (cm^2) for the optimum design of two-bay seven-storey RC plane frame example ($I=1.00$).	121
Figure A.1	Plane frame	149

Figure A.2	Joint end displacements, end forces and moments in frame members	149
Figure A.3	Member forces and moments for degree of freedoms	150
Figure B.1	Typical Axial Load-Moment interaction diagram	159
Figure B.2	Idealized strain distribution for generation of interaction surface	159
Figure B.3	Idealization of stress and strain distribution in a column section	160
Figure B.4	Shear stress area A_{cv} for rectangular column section	165
Figure B.5	Confinement requirements at column ends	167
Figure B.6	Idealization of stress and strain distribution in a beam section	169
Figure B.7	Reinforcement requirements for flexural members of Special Seismic Moment Frames	170
Figure B.8	Transverse reinforcement requirements for flexural members of Special Seismic Moment Frames.	173

LIST OF TABLES

TABLES

Table 2.1	Optimal results of the constrained function problem	32
Table 2.2	Optimal results for pressure vessel design	34
Table 2.3	Input values and bounds for design variables	37
Table 2.4	Optimal results of welded beam design	37
Table 3.1	Cross-sectional and reinforcement design variables for beam groups	48
Table 3.2	Variable poll for beam design variables	50
Table 3.3	Design variable bounds for RC column design examples	53
Table 3.4	Design variables database for column sections	55
Table 5.1	Input data for two-bay seven story RC frame example	85
Table 5.2	The optimum values of design variables for two-bay seven story RC frame	86
Table 5.3	The material quantities and costs in the optimum two-bay seven story RC frame	91
Table 5.4	The obtained optimum results for different harmony search parameters for two-bay seven-story RC frame example	92
Table 5.5	Input data for two-bay, five-story RC frame	93
Table 5.6	The optimum values of design variables for two-bay five-story RC frame (<i>for I=1.00</i>)	95
Table 5.7	The optimum values of design variables for two-bay five-story RC frame (<i>for I=1.25</i>)	100
Table 5.8	The optimum values of design variables for two-bay five-story RC frame (<i>for I=1.50</i>)	105
Table 5.9	The details of material quantities and costs in the optimum two-bay five-story RC frame	110

Table 5.10	Input data for two-bay six-story RC frame example	112
Table 5.11	The optimum values of design variables for two-bay six-story RC frame	114
Table 5.12	Input data for five-bay five-story RC frame	122
Table 5.13	The optimum values of design variables of five-bay five-story RC frame example (for Case 1)	123
Table 5.14	The optimum values of design variables for five-bay five-story RC frame (Case 2)	125
Table 5.15	The details of material quantities and costs of the optimum designs for five-bay five-story RC frame	127
Table C.1	Importance factors (Table 11.5-1 of ASCE 7-05)	176
Table C.2	Site coefficient, F_a (Table 11.4-1 of ASCE 7-05)	177
Table C.3	Site coefficient, F_v (Table 11.4-2 of ASCE 7-05)	177
Table C.4	Allowable story drift (Table 12.12-1 ASCE 7-05)	180

LIST OF ABBREVIATIONS

ACI	American Concrete Institute
ACO	Ant Colony Optimization
ASCE	American Society of Civil Engineers
BA	Bees Algorithm
D	Dead load
E	Earthquake load
ES	Evolutionary Strategies
FA	Firefly Algorithms
GAs	Genetic Algorithms
HM	Harmony Memory Matrix
HMCR	Harmony Memory Considering Rate
HMS	Harmony Memory Size
HS	Harmony Search Algorithm
HSM	Harmony Search Memory
IP	Integer Programming
L	Live load
LP	Linear Programming
MCE	Mapped Maximum Considered Earthquake
MP	Mathematical Programming
NLP	Non-linear Programming
PAR	Pitch Adjusting Rate
PSO	Particle Swarm Optimization
RC	Reinforced Concrete
SA	Simulated Annealing
TS	Tabu Search Method

CHAPTER 1

INTRODUCTION

1.1 General

Optimization is a mathematical discipline that concerns the finding of minima or maxima of a function which is subjected to what is called constraints. The main purpose of the optimization process is to find the optimal values of decision variables within the variety of possible ranges defined in the programming problem. Currently, optimization is widely applied in the fields of operational research, artificial intelligence and computer science, and is used to improve processes in the field of management, engineering, and various other disciplines.

Optimization can be defined as solving a decision making problem to find out the best decision for a given operation while satisfying certain restrictions. Similarly, the structural optimization problems can be considered as finding the optimum weight or cost while satisfying strength, ductility, serviceability, detailing, and architectural constraints derived from specifications and engineering experiences. Structural optimization problems are characterized as optimum design for minimum weight or optimum design for minimum cost. In general, the minimum weight design is mainly used in the optimum design of steel structures. However, the minimum cost design is more appropriate for optimum design of reinforced concrete structures. In civil engineering, optimization is also used to find the minimum factor of safety of soil slopes, the shortest route of vehicles, the minimum cost of water distribution networks, and the time tabling in the construction management.

1.2 Structural Optimization

Every structural designer tries to design structural systems for the best behavior and performance. In conventional structural design, the designers use their experiences and observations to develop the structural systems. These developed systems are tested several times until the desired performances are achieved. Although the conventional design process gives some advantages such as the freedom to design structural systems and to apply additional specifications, this approach only uses trial and error not concept and not powerful enough to obtain economical designs. In structural design, structural engineers should not only consider behavior and performance of systems but also consider the cost or weight of systems. Especially, for large-scale structural design problems, the time spent for the design process is high; however, there is no guarantee that the design constraints (such as strengths and displacements of structural members) would be satisfied. Moreover, the designs created by the conventional design methods are far away being the economical solutions under the design constraints considered.

As mentioned above, the conventional design is closely related to the capabilities of the design engineers and may not provide appropriate solutions for complex structural systems. The steps of the conventional design process are summarized in Figure 1.1. The optimum design process leads to create economical design while satisfying the required constraints. At the beginning of optimum design process, the design variables, the objective function (cost or weight), and the constraints are defined by design engineer. Similar to the conventional design process, the structural system is described and analyzed. After checking the performance of constraints, the convergence of the process is checked. The convergence check is required in order to terminate the design process within the minimum number of cycles, and in case where the convergence is not attained, the design is changed (updated) according to the logic of the optimization method used in the process. The optimum design procedure is summarized in Figure 1.2.

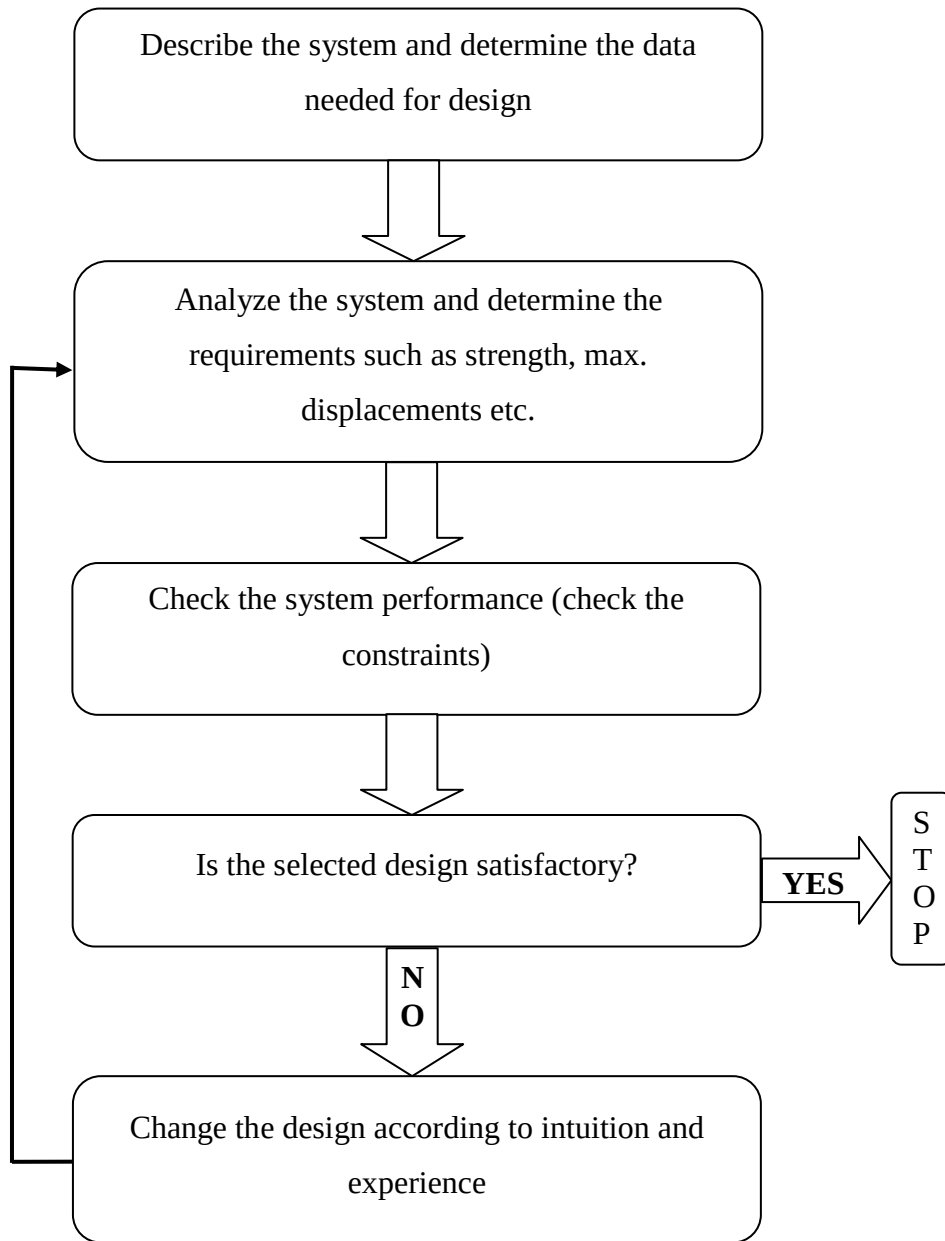


Figure 1.1 The conventional design process.

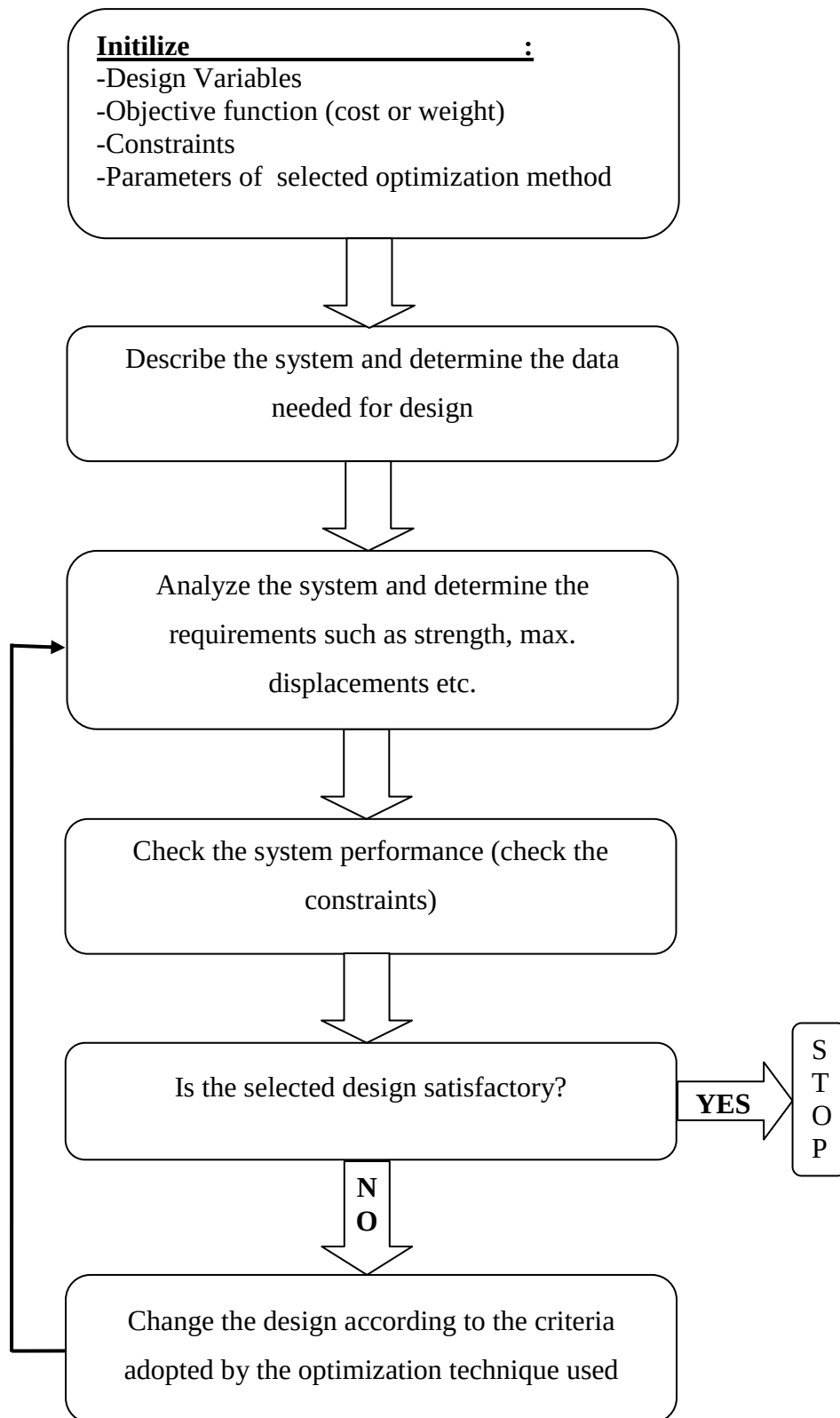


Figure 1.2 The optimum design process.

In many practical engineering design problems, design variables may consist of continuous or discrete variables. In structural optimization problems, the design variables are functions of the cross sections of the members and they are often chosen from a limited set of available values. For example, steel structural members are chosen from standard steel profiles in the market, structural timber members are provided in certain sizes, and reinforced concrete members are usually designed and constructed with discrete dimensional increments. Due to practical considerations, the design variables in the structural optimization generally are discrete variables, and the structural optimization problem formulated is to be solved using one of the solution techniques of discrete (linear or non-linear) programming.

First practical application of optimization techniques in the field of structural optimization were carried out by Gerard [1], Livesley [2], and Shanlay [3] in the 1940's and 1950's. In these studies, the non-linear programming was applied to structural design. By the early 1970's, the developments in the computer technology provided the capability of solving large scale problems in the field of structural optimization. Many mathematical programming methods were developed to solve the different types of structural optimization problems of different degrees of complexity, which may range from linear problems to complicated non-linear large scale problems. The most widespread used mathematical programming (MP) methods are linear programming (LP), integer programming (IP) and non-linear programming (NLP). LP is a technique for the solution of optimization problems having a linear objective function, subject to linear equality and linear inequality constraints. LP problems arise in many fields of engineering such as system engineering, the management of resources, civil engineering, and electrical engineering. The solution of the problem in LP is always a global minimum, because the local optimum solutions of the problem are not encountered in LP. Also, the solutions of non-linear programming problems may be obtained by transforming them to a sequence of linear programs. The only difference between integer programming (IP) and LP is that the values of the

design variables are restricted to be integers for IP. The IP problem is solved like an LP problem but the design variables are checked to determine whether they are all integers or not. If any variable is not an integer, a constraint is added so the optimal point would be infeasible and then the new LP problem is solved until all design variables are integers. If some or all functions of the optimization problem are non-linear functions, then the problem is called a non-linear programming problem (NLP). Many numerical methods have been developed for non-linear optimization problems. Some of them are Sequential linear programs, Lagrange multipliers method, penalty function method, gradient methods, feasible directions method [4,5].

However, the applicability of classical optimization techniques is limited to the optimization problems with a small search space. In this respect, artificial intelligence and meta-heuristic techniques have recently become very popular. During the last few decades, the research in the field of structural optimization have led to the development of the new optimization technique. These new meta-heuristic techniques will be introduced in Chapter 2.

1.3 Elements of Structural Optimization

The main target of the optimization process is to find appropriate combination of values for decision variables from a specified value set such that with these values the objective function is the minimum and the constraints are all satisfied. In structural optimization, the problem is defined by various objectives and constraints. These are generally the functions of the design variables. Each set of objectives and constraints define a different optimization problem. The formulation of an optimum design problem consists of identification of design variables, and statement of objective function and constraints. The general formulation of an optimum design problem may be expressed mathematically as;

$$\text{minimize} \quad f(\mathbf{x}) \quad (1.1)$$

$$\text{subject to} \quad g_i(\mathbf{x}) \leq 0 \quad i=1, \dots, n \quad (x^l \leq x \leq x^u) \quad (1.2)$$

where, $\mathbf{x}$ = vector of design variables

$f(\mathbf{x})$ = Objective function

$g_i(\mathbf{x})$ = Set of constraints

x^l and x^u ; vector of lower and upper bounds of design variables.

In structural optimization problems the structure should be modeled correctly so that it represents the behaviour of the structure properly. The design variables, objective function and constraints in the problem must be clearly selected and defined. The structural analysis tool is needed in structural optimization problems in order to find out the response of the structure when cross sectional properties of its members change during the design process. The structural analysis routine must give correct results and carry out the analysis as fast as possible. The selected optimization technique should be consistent with the nature of structural optimization problem. The elements of structural optimization are explained in the following sections.

1.3.1 Design Variables

Design variables are the parameters used in the formulation of the objective function. Design variables can be cross-sectional dimensions, material properties, or the parameters representing the geometry of the structure. For example, in shape optimization problems, the nodal coordinates of the system are selected as design variables. Design variables may take continuous or discrete values. Continuous design variables have a range of variation, and may take any value within this range. Discrete design variables may take only pre-determined values

from a list of permissible values. In structural optimization, design variables are usually discrete variables to represent the nature of the design. For instance, if the structural system is optimized to minimize the cost, then we need to limit ourselves to commercially available cross-sections or materials. But, solving an optimization problem with discrete design variables is usually much more difficult than solving a similar problem with a continuous design problem. The selection of design variables may be crucial to the success of the optimization process. Therefore, the selection of the design variables must be consistent with the structural model and optimization algorithm used.

1.3.2 Objective Function

The objective function is a function or functions which determine how good a solution is and is used as a measure of effectiveness of the design. Optimization with more than one objective function is called Multi-criteria Optimization. In structural optimization problems, the cost of the structure, the weight of the structure, the nodal displacements, the member stresses, etc. or any combination of these can be used as the objective function.

1.3.3 Constraints

The constraint is a condition that a solution to an optimization problem must satisfy. Structural optimization problems cannot be realistic without constraints. There are two types of constraints; equality constraints and inequality constraints. The set of solutions that satisfy all constraints is called the feasible set. The constraints in the structural optimization problem can be size constraint, strength constraint, stress constraint, detailing constraint, architectural constraint, geometric constraint, displacement constraint, behavioral constraint, etc. These constraints are generally derived from design specifications and engineering experiences.

1.4 Literature Review on Optimum Design of Reinforced Concrete Structures

Design optimization of reinforced concrete (RC) structures is more challenging than that of steel structures because of the complexity associated with reinforcement design. Also, only one material is considered for the optimization problems of steel structures and the structural cost is directly proportional to its weight in general. But in the case of the optimum design of concrete structures, three different cost components due to concrete, steel and formwork are to be considered and the overall cost of the structure is affected from any slight variation in the quantity of these components. Therefore, the optimization problem of concrete structures becomes the selection of a combination of appropriate values of design variables to make the total cost component is minimal.

Several other researchers have used mathematical and meta-heuristic optimization techniques for the optimum design of RC column sections. Zielinski et al. [6] used the internal penalty function (non-linear programming) for minimum cost design of reinforced concrete short tied rectangular columns based on Canadian Standard specifications. Genetic algorithm is used by Govindaraj and Ramasamy [7] to optimize RC columns and the results are compared with obtained in [6]. It is concluded that the genetic algorithm has achieved a reduction of 34% in the minimum cost of RC column design. In these optimization problems, the width and depth of column section, number of intermediate bars along extreme rows parallel to width and depth, diameters of these bars and the diameters of corner bars are selected as the design variables. The strength, serviceability, ductility and side limitations are considered as the design constraints and they are implemented from Indian code of practice, IS: 5525-69 [7]. The total cost of the RC column (per meter) which includes the cost of concrete, formwork and reinforcing bars is taken as the objective function in this study.

The optimum design of RC beams has been investigated by many researchers. For a singly reinforced beam sections, Leroy Friel [8] presented an equation to obtain the optimum steel percentage and used only moment strength constraints in his study. The Lagrange multiplier method was used by Chou [9] to obtain minimum cost design of singly reinforced T-beam sections based on ACI specifications. Three-level iterative optimization procedure was developed by Kirsch [10] for multi-span RC continuous beams. In this study, the amount of reinforcement at first level, the concrete dimensions at the second level and the design moments at the third level were optimized for RC continuous beams with rectangular cross-sections. Lakshmanan and Parameswaran [11] tried to determine the ratios between span to effective depth. This approach helps to avoid the trial and error approaches. For this purpose, they derived a formula for the direct determination of span to effective depth ratios for the optimum flexural design of RC continuous beam sections according to Indian Code. The Lagrangian and simplex methods were used by Prakash et al. [12] to obtain minimum cost designs for singly and doubly reinforced rectangular and T-shape RC beams based on Indian code specifications. Kanagasundaram and Karihaloo [13, 14] presented minimum cost design of simply supported and multi-span beams with rectangular and T-sections using sequential linear programming and sequential convex programming techniques. They consider the crushing strength of concrete, cross-sectional dimensions and steel ratio as a design variable in their study. Chakrabarty [15, 16] used the geometric programming and Newton-Rapson methods to minimize the cost of RC rectangular beams. Al-Salloum and Siddiqi [17] derived a closed form solution for the steel area and depth of beam section to minimize the cost of singly reinforced rectangular concrete beams as per the ACI code. The steel area and depth of beam section were expressed in terms of the cost and strength parameters by taking the derivatives of the augmented Lagrangian function with respect to the area of steel reinforcement.

One of the pioneering works on optimum design of RC structures using meta-heuristic methods belongs to Coello Coello et al. [18]. They used genetic

algorithms for the cost optimum design of singly reinforced rectangular beams. The dimensions of beam sections and the area of tensile reinforcement are considered as design variables in their optimum design model. Another application of genetic algorithms for the optimum detailed design of RC beams was presented by Koumousis and Arsenis [19]. Their study was based on the multi-criterion objective that represents a compromise between the minimum weight design, the maximum uniformity and the minimum number of bars for a group of members. The most striking work on optimum design of RC continuous beams was presented by Govindaraj and Ramasamy [20]. They presented the application of Genetic Algorithms for the optimum detailed design of reinforced concrete continuous beams based on Indian Standard specifications. The distinctive feature of this study is that the cross-sectional dimensions of the beam alone are considered as variables, while most of the approaches reported in the literature consider the steel reinforcement as a design variable. The areas of longitudinal steel obtained from the design are converted into the least weight detailing of steel reinforcements. In their study, the produced optimum design satisfies the strength, serviceability, ductility, durability and other constraints related to good design and detailing practice.

The traditional optimization techniques have been applied by many researchers to the design of reinforced concrete frames. The linear programming, sequential linear programming and sequential convex programming techniques were used to optimize the cost of RC frames based on the design specifications [14, 21]. Krishnamoorthy and Munro [21] used a simplex algorithm to solve linear programming problems for the optimum design of RC frames which includes considerations such as limited ductility and serviceability constraints in the optimum design formulation. Fadaee and Grier [22] applied the optimality criteria method to optimize three-dimensional reinforced concrete structures using continuous design variables.

Practical applications of traditional optimization methods are not suitable for

optimum design of RC frames and these algorithms require additional modifications to fit the discrete nature of structural design variables. Choi and Kwak [23] have suggested more practical discrete optimization techniques. Choi and Kwak used direct search method to select appropriate design sections from some pre-determined discrete member sections for the cost optimization of rectangular beams and columns of RC frames based on the ACI and Korean codes. In their study, the discrete optimum sections for design variables are directly searched based on the relationship between the section identification numbers and the resistant capacities of member sections, which is established by regression. The candidate sections in the section database are selected from widely used sections in practical design applications. More recently, meta-heuristic optimization algorithms have been used for structural optimization problems. Balling and Yao used the simulated annealing method to optimize three-dimensional reinforced concrete frames [24]. Discrete variables as well as limits on the number of reinforcing bars and their topological arrangements are considered in their study. Rajeev and Krishnamoorthy [25] applied a simple genetic algorithm to the cost optimization of two-dimensional RC frames. The detailing of reinforcement is considered as a design variable in addition to cross-section dimensions. The allowable combinations of reinforcement bars for columns and beams were tabulated. Camp et al. [26] used genetic algorithms (GAs) by constructing a database for beams and columns which contains the sectional dimensions and the reinforcement data in the practical range to optimize for optimum design of plane frames. Lee and Ahn [27] used the genetic algorithms to optimize reinforced concrete plane frames subject to gravity loads and lateral loads. In their study, the constructing data sets, which contain a finite number of sectional properties of beams and columns in a practical range removed the difficulties in finding optimum sections from a semi-infinite set of member sizes and reinforcement arrangements. Kwak and Kim [28] studied on optimum design of RC plane frames based on pre-determined section database. In their study, pre-determined section databases of RC columns and beams are constructed and arranged in order of resisting capacity and optimum solutions are

obtained using direct search method. They also used genetic algorithms for similar optimization problems [29]. Govindaraj and Ramasamy [7] used genetic algorithms for optimum detailed design for RC frames based on Indian Standard specifications. The dimensions and reinforcement arrangement of column, and the dimensions of beam members alone are considered as a design variables and the detailing of reinforcements in the beam members is carried out as a sub-level optimization problem. The modular sizes of members, available standard reinforcement bar diameters, spacing requirements of reinforcing bars, architectural requirements on member sizes and other practical requirements are arranged in order to obtain for the optimum designs to be directly constructible without any further modifications.

1.5 The Scope of the Thesis

This study deals with the problem of optimizing *Special Seismic Moment* reinforced concrete (RC) frames subject to ACI 318-05 [30] and ASCE 7-05 [31]. In the literature, there are a number of studies on optimization of RC members and frames. In these studies, there are some gaps in regards to obtaining constructible design. In some of them, the values selected for dimensions of the members and the reinforcement steel areas are not available in the market. In these studies, the shear design calculations of concrete members are not considered and the cost of shear reinforcement (ties) is not taken into consideration in the total cost of the frame. Only the simple constraints such as capacity and regulations for flexural reinforcement are derived in the optimization problems of these studies. The detailing of the reinforcement bars is oversimplified and the development length of bars is not considered in cost calculations. In some of these studies, the lateral loading on the frame is considered; however, the values of the lateral loads are taken as a constant even though the value of lateral loads change with the weight of the structure subject to seismic specifications. In addition to the aforementioned design flaws, there is a

lack of application of modern meta-heuristic optimization methods to the optimization of concrete structures.

In this thesis, harmony search method is used to obtain the optimum detailed design for reinforced concrete special seismic moment frames. Reinforced concrete special moment frames are used as part of seismic force-resisting systems in buildings that are designed to resist earthquakes. Beams, columns, and joints in moment frames are proportioned and detailed to resist flexural, axial, and shearing actions during an earthquake. RC frames with special proportioning and detailing requirements are capable of resisting strong earthquake shaking without significant loss of stiffness or strength. These moment-resisting frames are called “Special Moment Frames” because of these additional requirements. A new optimum design algorithm is developed for the RC special seismic moment frames and the variable pool constructed to obtain buildable optimum designs. In the design formulation, the objective function is selected as the cost of the RC structure which includes the cost of concrete, reinforcement and formwork. To satisfy uniformity of the structure and to obtain constructible designs, the beam and column members are separated to design groups. The design variables are categorized into two groups and arranged for the beam and column members. For the columns, the section database which includes the dimensions and the reinforcement detailing of column sections is constructed with the most commonly used sections. The design constraints are implemented according to ACI 318-05. This study not only considers the flexural design constraints, but also the shear design constraints and the seismic design constraints. The development lengths of the reinforcement steel bars are calculated according to the given regulations in the design specifications. The cost of the shear reinforcement and the impact of the development length on the cost are considered. In the design of frames, the matrix displacement method is used for the structural analysis and the load combinations are taken from the ACI code. The self weight of RC beams is included in the structural analysis and it is updated in each iteration of the optimization process. The lateral seismic forces are calculated according to ASCE

7-05 and it is updated in each iteration according to the selected design weight.

The solution of the design problem formulated in this study is obtained using the harmony search method which is one of the recently developed meta-heuristic optimization methods. The constraints derived from the code are checked to obtain feasible designs. Optimum designs are produced using harmony search method. Number of design examples is considered to demonstrate the efficiency of the optimum design algorithm developed.

This study is unique as it is the first application of the harmony search method to the optimization of RC frames. Additionally, the detailing of the reinforcement in the concrete members, the consideration of the shear design of members, and the derivation of the constraints are handled in more detail in this study. The lateral seismic forces affecting the RC frame are obtained for the site properties according to ASCE 7-05 even though other existing studies do not consider the lateral seismic forces in their design. Also, this study is the sole study about the optimization of *Special Seismic Moment RC Frames* to date.

In chapter 2, the harmony search algorithm, which is used as a new meta-heuristic optimization method in this study, is introduced and the available applications on civil engineering are summarized.

In chapter 3, the modelling of the detailed optimum design problem is explained; the objective function, the design variables and the constraints derived from design specifications are described.

In chapter 4, the optimum design process of RC frames with the harmony search algorithm is presented.

In chapter 5, illustrative examples are provided to demonstrate the efficiency and

the performance of the algorithm presented in this study.

In chapter 6, the summary and the main conclusions of this work are provided.

CHAPTER 2

HARMONY SEARCH ALGORITHM

2.1 Meta-heuristic Methods

Meta-heuristic techniques are based on certain strategies and they are non-deterministic algorithms. These strategies guide the search process in the design domain. In other words, meta-heuristic can be described as a general algorithmic framework. However; the common agreed definition for meta-heuristic cannot be accepted due to a variety of techniques and concepts comprised by meta-heuristic. In recent years, the word '*meta-heuristics*' refers to all modern higher level algorithms. The meta-heuristic methods make random moves rather than in the descending direction of the gradient vector of objective function. Recently, meta-heuristic algorithms are widely applied in solving combinatorial optimization engineering problems. Furthermore the literature survey also indicates that the meta-heuristic techniques are widely preferred in solving engineering optimization problems compared to traditional mathematical programming techniques. There are a number of meta-heuristic techniques, such as genetic algorithm (GA), tabu search method (TS), simulated annealing (SA), evolutionary strategies (ES), ant colony optimization (ACO), particle swarm optimization (PSO), bees algorithm (BA), firefly algorithms (FA), and certainly harmony search algorithm (HS). Some of these algorithms are discussed in the following sections. The harmony search algorithm is one of the recent additions to the meta-heuristic optimization methods. This new meta-heuristic method imitates the music improvisation of a musician into a numerical approach for general optimization problems. The harmony search algorithm is presented by Geem, Kim

and Loganathan [32]. The harmony search algorithm is used in the engineering optimization problems effectively, because this algorithm not only needs adjustment of a few parameters and also consists of simple steps and has a fast convergence rate. For these reasons, the harmony search algorithm is adopted in the solution of the optimum detailed reinforced concrete design problem in this thesis.

2.1.1 The Genetic algorithm

The genetic algorithm (GA) method is developed by Holland [33] in 1975. The GA is a search and optimization method based on the natural selection and the principles of genetics. These principles are derived from biology and rely on Darwin's theory of survival of the fittest. The specific mechanics of this algorithm use the language of microbiology, and mimics genetic operations. The basic idea of a genetic algorithm is to generate a new set of designs (population) from the current set such that the average fitness of the population is improved. The process is continued until the number of iterations exceeds a specified limit. Three genetic operators: reproduction, crossover and mutation, are used to accomplish this task. The main model of GA is given in Figure 2.1. Genetic algorithms have been studied, experimented and applied in many fields in engineering worlds, such as optimal control problems, job scheduling, transportation problems, pattern recognition, machine-learning [34, 35], etc. Genetic algorithms are extremely successful in solving unconstrained and constrained optimization problems. GAs do not only provide alternative methods to solve a problem, but it also consistently outperforms other traditional methods in most of the problems. In future, we would witness some developments of variants of GAs to tailor for some very specific tasks [36].

Main GA algorithm

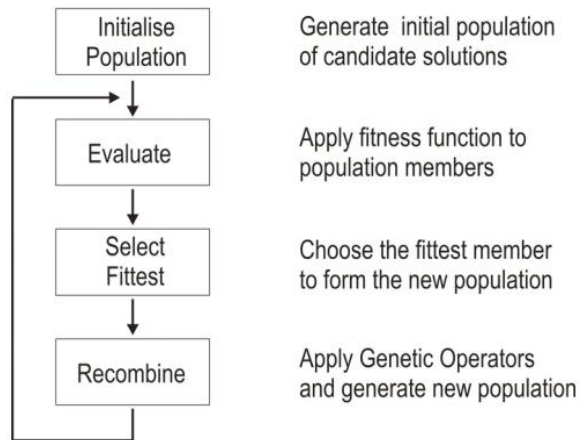


Figure 2.1 The main model of Genetic Algorithm.

2.1.2 Evolutionary Strategies

Evolution Strategies (ES) were developed by Rechenberg [37] and Schwefel [38] at the Technical University of Berlin in Germany and have been extensively studied in Europe. It was created in the early 1960s and developed further along the 1970s and later. Evolutionary Strategy is an optimization technique based on ideas of adaptation and evolution. Evolution Strategies are in many ways similar to Genetic Algorithms (GA). They mainly use mutation, selection and element-wise average for intermediate recombination as the genetic operators. The important working mechanisms are mutation and adaptation of mutation rates in the ESs. In evolutionary strategies, the representation used is a fixed-length real-valued vector. As with the bit-strings of genetic algorithms, each position in the vector corresponds to a feature of the individual. However; the features are considered to be behavioral rather than structural. Consequently, arbitrary non-linear interactions between features during evaluation are expected which forces a more holistic approach to evolving solutions [39]. ESs have been applied to a wide range of optimization problems [40, 41].

2.1.3 Simulated Annealing

Simulated annealing (SA) is the one of modern meta-heuristic algorithms. SA was first developed by Kirkpatrick et al. [42] in 1983. The current state of the thermodynamic system is analogous to the current solution to the combinatorial problem, the energy equation for the thermodynamic system is analogous to the objective function, and ground state is analogous to the global minimum. The concept is based on the manner in which liquids freeze or metals recrystallize in the process of annealing. In an annealing process a melt, initially at high temperature and disordered, is slowly cooled so that the system at any time is approximately in thermodynamic equilibrium. As cooling proceeds, the system becomes more ordered and approaches a "frozen" ground state at $T=0$. Hence the process can be thought of as an adiabatic approach to the lowest energy state.

The optimization process typically starts from an initial guess with higher energy. It then moves to another location randomly with slightly reduced energy. The move is accepted if the new state has lower energy and the solution improves with a better objective or lower value of the objective function for minimization.

2.1.4 Ant Colony Optimization

The ant colony optimization (ACO) formulated by Dorigo et al. [43, 44] is another population-based meta-heuristic algorithm. This algorithm is based upon the characteristics of behaviours of social ants. Ants can find the shortest path to food by laying a pheromone (chemical) trail as they walk and other ants follow the pheromone trail to food. Ants that happen to pick the shorter path will create a strong trail of pheromone faster than the ones choosing a longer path. Since stronger pheromone attracts ants better, more and more ants choose the shorter path until eventually all ants have found the shortest path. Consequently, new ants will select that path first and further reinforce the pheromone level on that path. Eventually all the ants will follow the shortest path to food [45].

The ACO is used for discrete routing and scheduling optimization problems, because this problem closely resembles the shortest path to food source. This algorithm is exceptionally successful in combinatorial optimization problems. Ant colony optimization algorithms have been applied to many combinatorial and engineering optimization problems and successful results are obtained [46].

2.1.5 Particle Swarm Optimization

The particle swarm optimization (PSO) algorithm was formulated by Eberhart and Kennedy [47] in 1995, inspired by the social behaviour of animals in nature, such as bird flocking and fish schooling. Unlike genetic algorithms and simulated annealing, PSO has no evolution operators and searches the solution space by adjusting multiple trajectories of individual agents (called particles). PSO begins with a random population matrix and the rows in this matrix are called particles. They contain variables value are not binary coded. Each particle moves about the function surface with a velocity and the particles update their positions and velocities based on local and global best solutions;

$$v_{m,n}^{new} = v_{m,n}^{old} + \Gamma_1 \times r_1 \times (p_{m,n}^{local\ best} - p_{m,n}^{old}) + \Gamma_2 \times r_2 \times (p_{m,n}^{global\ best} - p_{m,n}^{old}) \quad (2.1)$$

$$p_{m,n}^{new} = p_{m,n}^{old} + v_{m,n}^{new} \quad (2.2)$$

where,

$v_{m,n}^{new}$ = particle velocity

$p_{m,n}$ = particle position

r_1, r_2 = independent uniform random numbers

Γ_1, Γ_2 = learning factors

$p_{m,n}^{local\ best}$ = best local solution

$p_{m,n}^{global\ best}$ = best global solution

The PSO algorithm updates the velocity vector for each particle then updates the particle positions or values using updated velocity. Velocity updates are influenced by both the best global and the best local solutions. The particle velocity is reminiscent of local minimisers that use derivative information, because velocity is the derivative of position. The advantages of PSO are that there are few parameters to adjust and PSO algorithm is easy to implement. Particle swarm optimization has been applied to a wide range of engineering optimization problems [48] and there are many variants of PSO in the literature [49].

2.1.6 Bee Colony Optimization

The bees algorithm is a population-based optimization algorithm developed by Pham et al. in 2005 [50]. The bees algorithm is inspired by natural foraging behavior of honey bees to use in optimization problems. A colony of honey bees can extend itself over a large area and in multiple directions to exploit a large number of food sources at the same time [51]. A colony feeds by spreading its foragers to good fields. Practically, a large number of bees visit and collect nectar or pollen from flower patches with plentiful amounts of food, instead of visiting flower patches with less nectar or pollen. This behavior provides feeding of colony with less effort.

At the beginning of the foraging process, the scout bees in a colony are sent to search for rich flower patches. Scout bees move from one patch to another randomly. Bee colony keeps a percentage as scout bees and continues its investigation during the harvesting season. After scout bees return to their hive, they go to the dance floor to perform a dance known as the waggle dance. By performing this dance, successful bees on finding a source share with their hive mates information about the direction to patches of flowers yielding nectar and pollen, the distance of these patches from hive and the quality rating of these patches. Thus the waggle dance helps successful foragers to recruit other bees in

their colony to good locations for collecting various resources. In other words, waggle dance assists the colony to compare relative rate of different source patches according to both the amount of nectar or pollen in the source patch and amount of energy needed to collect them. After the dancer (i.e. the scout bee) finished its waggle dance, this bee turns to the flower patch with follower bees that were waiting inside the hive. For more promising source patches, more follower bees are sent to collect to food quickly and efficiently. The bees decide upon the next waggle dance, they will perform when they return to the hive, by monitoring food level of sources each time. In case these patches are still good enough as a food source, these patches advertised in the waggle dance for more bees will be recruited to these sources. The pseudo-code of Bees Algorithm in its simplest form is given Figure 2.2.

1	Initialize population with random solutions
2	Evaluate fitness of the population
	<i>While (stopping criterion not met)</i>
3	Forming new bee population (select elite bees) and select sites for neighborhood search
4	Recruit bees for selected sites and evaluate their fitness
5	Select the fittest bee from each patch
6	Assign remaining bees to search randomly and evaluate their fitness
	<i>End While</i>

Figure 2.2 The pseudo-code for the Bees Algorithm.

2.2 Harmony Search Method

The Harmony Search (HS) method is an emerging meta-heuristic optimization algorithm, which is proposed by Geem, Kim, and Loganathan [32]. The HS algorithm is inspired by the underlying principles of the musicians' improvisation of the perfect state of harmony. The HS algorithm mimics the improvisation of music players for searching the better harmony. The various possible combinations of the musical pitches stored in the musicians' memory, when

they compose harmony. And the musicians can find the fantastic harmony from their harmony memories.

In the adaptation of this musical improvisation process into the solution of optimization problems, each musician corresponds to a decision variable and possible notes correspond to the possible values of decision variables. The players play any pitch within the possible range, and these sounds make one harmony vector in music improvisation. These harmony vectors are stored in each player's memory if all the pitches make a good harmony, and the possibility to make a better harmony is increased next time. Similarly in optimization problems, each decision variable initially chooses any value within the possible range, and these values of decision variables make one solution vector. If all the values of decision variables make a good solution, these solution vectors are stored in the memory. Thus, the possibility to make a good solution is also increased next time. For example, if the first musician plays {La} while second and third musicians play {Fa} and {Do} from their harmony memories or randomly, a new harmony is created {La, Fa, Do}. And if this new harmony is better than the existing worst harmony in the harmony memory (HM), the new harmony is included in the HM and the worst harmony is excluded from the HM. This procedure is repeated until fantastic harmony is found. In engineering optimization problems, decision variables can be replaced with musicians, and variable's selected values can be replaced with sound pitches of musicians. If first design variable is chosen {100mm} out of {100mm, 200mm, 300mm}, second {500mm} out of {250mm, 500mm, 750mm}, and third {600mm} out of {200mm, 400mm, 600mm, 800mm}, those values {100mm, 500mm, 600mm} make a new solution vector. The new vector is included in the harmony memory (HM) and the worst vector is excluded from the HM, if this new vector is better than existing worst vector in the HM. Until certain termination criterion is satisfied, this iterative procedure is repeated. The details of the explained analogy between music improvisation and engineering optimization can be seen in Figure 2.3.

The optimization procedure of the Harmony Search meta-heuristic algorithm consists of Steps 1-5, as follows;

Step1. Optimization problem and algorithm parameters are initialized.

Step2. Harmony memory matrix (HM) is initialized.

Step3. A new harmony from the HM is improvised.

Step4. Harmony memory matrix is updated.

Step5. Step 3 and step 4 are repeated until the termination criterion is satisfied.

These steps can be summarized in the following:

Step 1- The discrete optimization problem is specified as follows:

$$\textbf{Minimize } f(\mathbf{x}) \text{ s.t. } x_i \in X_i, i=1,2,\dots,N$$

where; X_i is the set of possible candidate values for each decision variable i . A design variable pool is constructed for each design variable. The number of solution vectors in harmony memory matrix (HMS), harmony memory considering rate (HMCR), pitch adjusting rate (PAR), and termination criterion (maximum number of search) are also decided in this step.

Step 2- The “harmony memory” (HM) matrix is filled with randomly generated solution vectors using the values in the design pool and sorted by the values of the objective function, $f(\mathbf{x})$. The Harmony Memory is filled with as many solution vectors as harmony memory size (HMS). The harmony memory matrix has the following form:

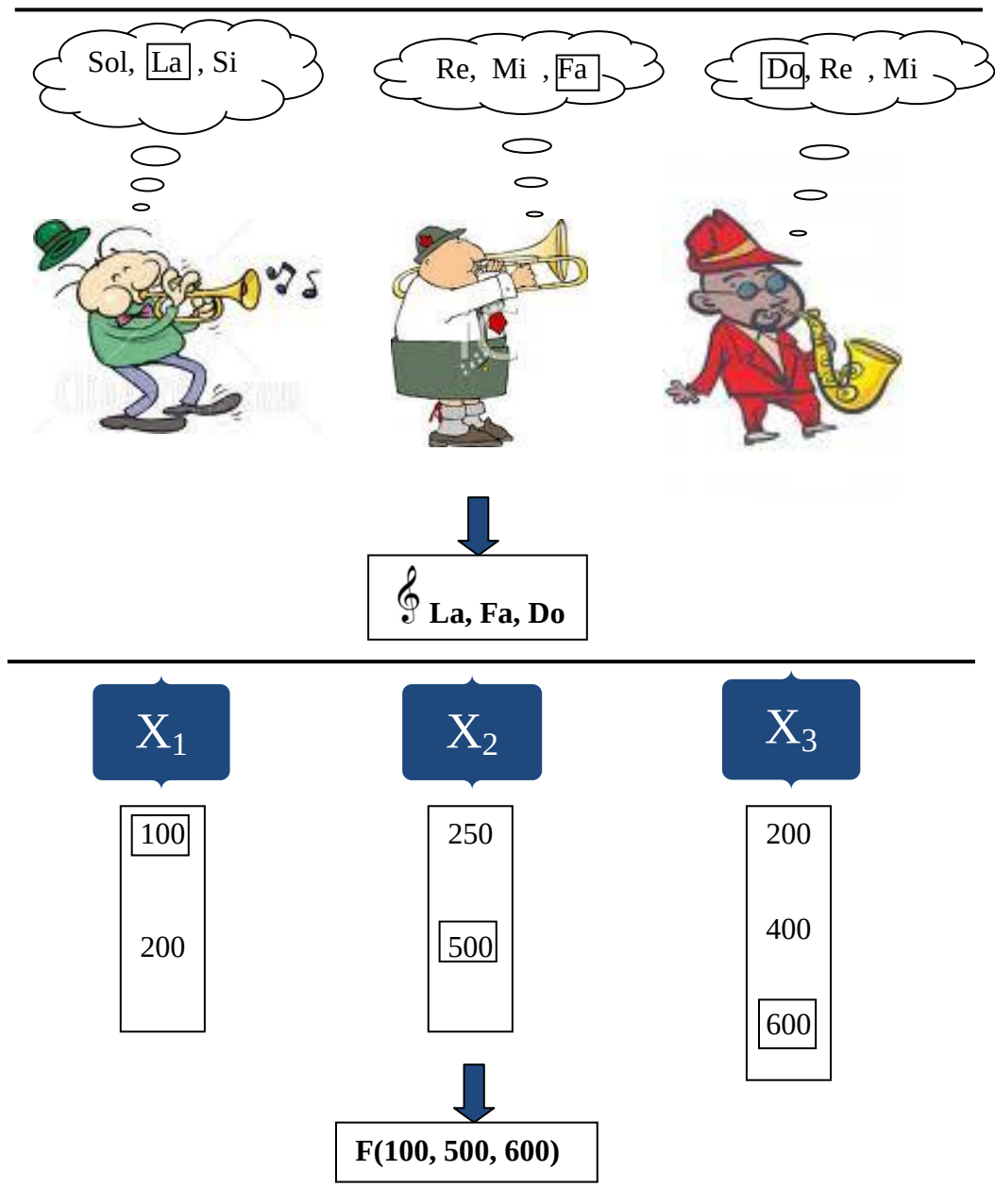


Figure 2.3 Analogy between musical improvisation and optimization.

$$H = \begin{bmatrix} x_1^1 & x_2^1 & \dots & \dots & x_{N-1}^1 & x_N^1 \\ x_1^2 & x_2^2 & \dots & \dots & x_{N-1}^2 & x_N^2 \\ \dots & \dots & \dots & \dots & \dots & \dots \\ \dots & \dots & \dots & \dots & \dots & \dots \\ x_1^{HMS-1} & x_2^{HMS-1} & \dots & \dots & x_{N-1}^{HMS-1} & x_N^{HMS-1} \\ x_1^{HMS} & x_2^{HMS} & \dots & \dots & x_{N-1}^{HMS} & x_N^{HMS} \end{bmatrix}$$

Each row of harmony memory matrix contains the values of design variables of the optimization problem and presents one feasible solution vector. The design variables are randomly selected from the design variable pool for that particular design variable. Hence, this memory matrix has N columns (N ; the total number of design variables) and HMS rows (HMS ; harmony memory size). These candidate solutions are sorted such that the objective function value corresponding to the first solution vector (first row) is the minimum or maximum.

Step 3- A new harmony vector, $\mathbf{x}^i = (x_1^i, x_2^i, x_3^i, \dots, x_N^i)$ is improvised. There are three rules to choose one value for each decision variable; harmony memory consideration rate (HMCR), pitch adjustment rate (PAR), and random selection.

In memory consideration, the value of the first decision variable (x_1^i) can be chosen from any discrete value in the specified HM range $\{x_1^1, x_1^2, x_1^3, \dots, x_1^{HMS-1}, x_1^{HMS}\}$ with the probability of HMCR which varies between 0 and 1. Values of the other decision variables (x_i^i) can be chosen in the same manner. However, there is still a chance where the new value can be randomly chosen from a set of entire possible values in the design variable pools with the probability of (1-HMCR). That is

$$x_i^i \leftarrow \begin{cases} x_i^i \in x_i^1, x_i^2, \dots, x_i^{HMS} & \text{with probability HMCR} \\ x_i^i \in X_i & \text{with probability (1-HMCR)} \end{cases}$$

Any component of the new harmony vector, whose value was chosen from the HM, is then examined to determine whether it should be pitch-adjusted. This operation uses pitch adjusting parameter (PAR) that sets the rate of pitch-adjustment decision as follows:

$$\text{Pitch adjusting decision for } x_i' \leftarrow \begin{cases} \text{Yes with probability PAR} \\ \text{No with probability } (1 - \text{PAR}) \end{cases}$$

If the pitch adjustment decision for x_i' is Yes, x_i' is replaced with $x_i(k)$ (the k^{th} element X_i), and the pitch-adjusted value of $x_i(k)$ becomes:

$$x_i' \leftarrow x_i(k \pm 1)$$

The algorithm chooses -1 or 1 for the neighboring index m with the same probability.

Step 4- After selecting the new values for each design variable the objective function value is calculated for the new feasible harmony vector. If the new Harmony vector is better than the worst harmony in the HM in terms of the objective function value, the new harmony is included in the HM and the existing worst harmony is excluded from the HM. The HM is then sorted by the objective function value.

Step 5- The computations are terminated when the termination criterion is satisfied. If not, Steps 3 and 4 are repeated. The termination criterion is selected large enough such that within this number of cycles no further improvement is observed in the objective function value.

The aforementioned steps of the harmony search method can be applied to structural optimum design problems as described in the following. Firstly, the

harmony search method parameters (HMS, HMCR, PAR, Max. iteration number) are selected. The HS algorithm initiates the design process by selecting random values for the design variables to fill in the harmony memory matrix with feasible design vectors. After the harmony memory matrix is filled in, the new design vectors are created according to the HMCR and PAR values selected earlier. If the created new design vectors satisfy the constraints and its objective function of this design vector is better than the worst harmony in the HM, then HM matrix is updated with the new design vector value. The revised HM matrix is then sorted in ascending order. This process is repeated until the termination criterion is satisfied. At the end of this process, the first row of HM matrix gives the optimum design vector based on the initially selected parameters of the harmony search method. The flowchart of this process is demonstrated in Figure 2.4.

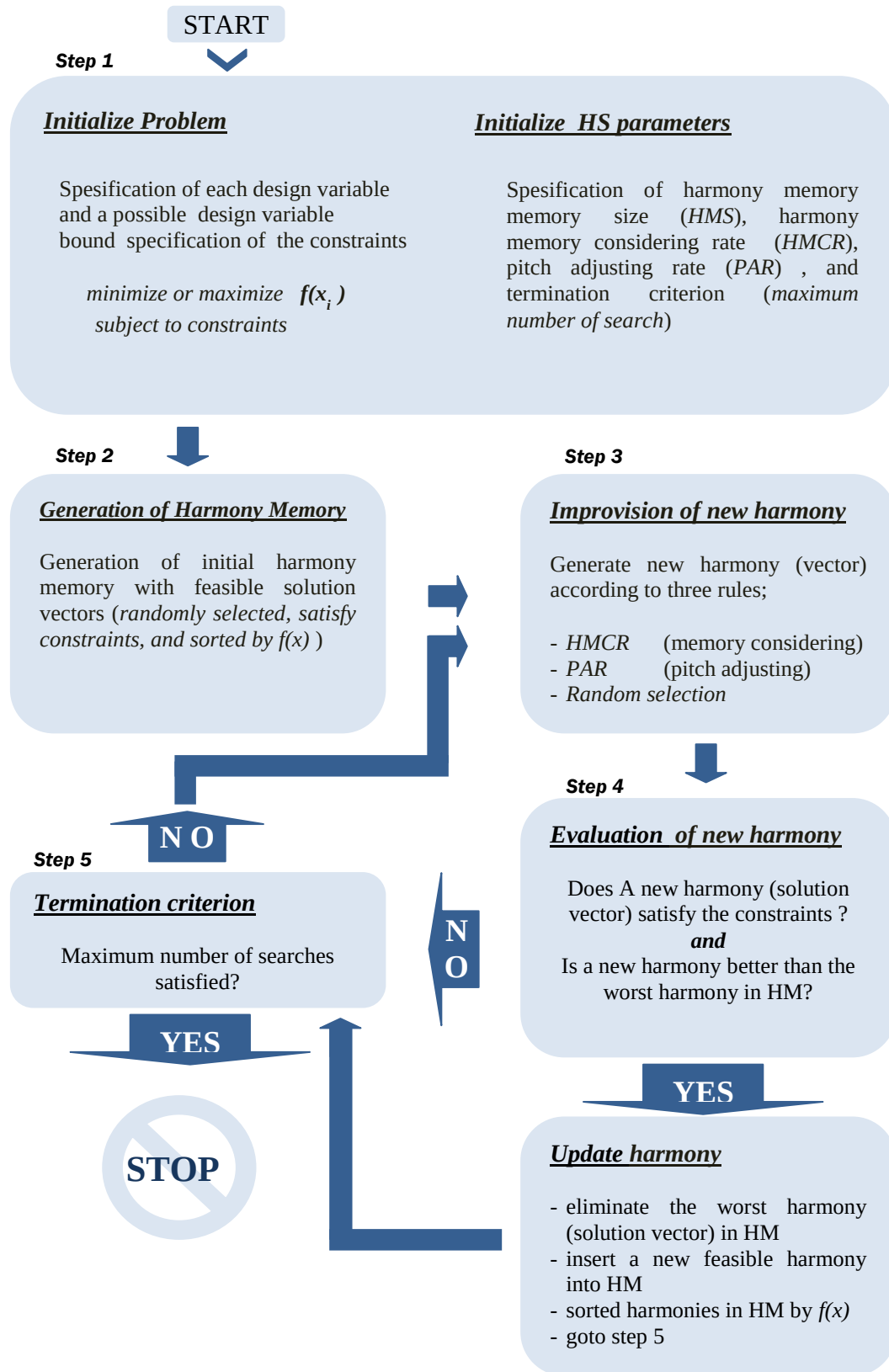


Figure 2.4 Harmony search algorithm optimization procedure.

2.2.1 Case Study 1: Constrained Function Optimization

The function given in Eq.(2.3) with five design variables and six inequality constraints has been solved by several researchers. The optimal values of variables and the function are obtained by the harmony search algorithm by Lee and Geem [52], and these results are compared with earlier results presented by Deb [53], Shi and Eberhart [54], and Coello [55] in Table 2.1.

$$\text{Minimize } f(x) = 5.357847 x_3^2 + 0.8356891 x_1 x_5 + 37.293239 x_1 - 40792141 \quad (2.3)$$

subject to

$$g_1(x) = 85.334407 + 0.0056858 x_2 x_5 + 0.0006262 x_1 x_4 - 0.002205 x_3 x_5 \geq 0 \quad (2.4)$$

$$g_2(x) = 85.334407 + 0.0056858 x_2 x_5 + 0.0006262 x_1 x_4 - 0.002205 x_3 x_5 \leq 92 \quad (2.5)$$

$$g_3(x) = 80.51249 + 0.0071317 x_2 x_5 + 0.0029955 x_1 x_2 + 0.0021813 x_3^2 \geq 90 \quad (2.6)$$

$$g_4(x) = 80.51249 + 0.0071317 x_2 x_5 + 0.0029955 x_1 x_2 + 0.0021813 x_3^2 \leq 110 \quad (2.7)$$

$$g_5(x) = 9.300961 + 0.0047026 x_3 x_5 + 0.0012547 x_1 x_3 + 0.0019085 x_3 x_4 \geq 20 \quad (2.8)$$

$$g_6(x) = 9.300961 + 0.0047026 x_3 x_5 + 0.0012547 x_1 x_3 + 0.0019085 x_3 x_4 \leq 25 \quad (2.9)$$

$$78 \leq x_1 \leq 102 \quad 33 \leq x_2 \leq 45 \quad 27 \leq x_i \leq 45 \quad (i = 3, 4, 5) \quad (2.10)$$

Coello [55] and Shi & Eberhart [54] obtained the best solution function value of $f(x) = -31020.859$ and $f(x) = -31025.561$, respectively, using the GA based method and the modified particle swarm optimization algorithm. Deb [53] solved this problem using an efficient constraint handling method for the GA, and reported an objective function value of $f(x) = -30665.500$, which corresponds to the best known optimal solution to the function is $x^* = \{78.0, 33.0, 29.995, 45.0, 36.776\}$

and the objective function value $f(x) = -30665.500$. The harmony search algorithm based method found the best known solution vector with the best objective function value after 65000 iterations. The best solution obtained using HS algorithm is the same as the optimal solution, and the results are given in Table 2.1.

Table 2.1. Optimal results of the constrained function problem.

						Obj. function value
Unavailable						-30665.5
<i>Lee&Geem</i> [52]	78.049	33.007	27.081	45	44.94	-31020.9
	78	33	27.070	45	44.969	-31025.6
	78	33	29.995	45	36.776	-30665.5
	78	33	29.995	45	36.775	-30665.5

2.2.2 Case Study 2: Pressure Vessel Design

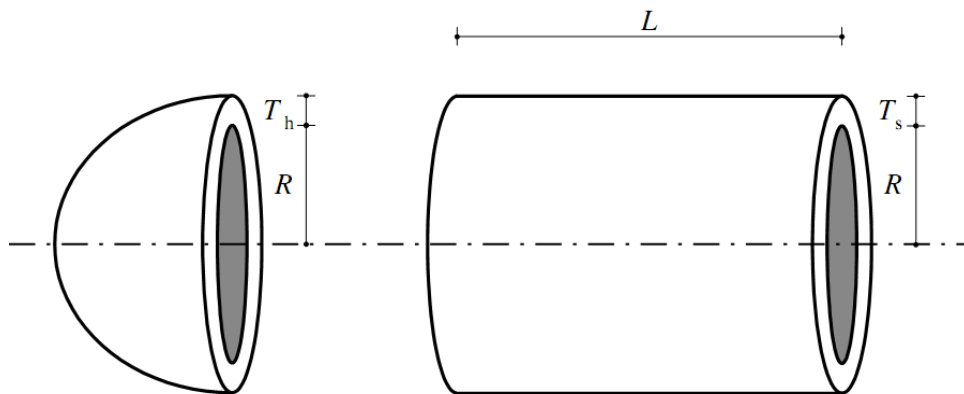


Figure 2.5 Schematic of pressure vessel[52].

The pressure vessel design problem (Figure 2.5) was previously proposed and analyzed by Sandgren [56]. Also, this optimization problem was solved to minimize the total cost of the material, forming and welding of a cylindrical vessel by Wu and Chow [57]. In the pressure vessel optimization problem, there are four design variables; x_1 (T_s , shell thickness), x_2 (T_h , spherical head thickness), x_3 (R , radius of cylindrical shell), and x_4 (L , shell length). The design variables x_1 (T_s) and x_2 (T_h) are integer multipliers of 0.00625 in. in accordance with the available thickness of rolled steel plates, and the other design variables x_3 (R) and x_4 (L) have continuous values of $40 \leq R \leq 80$ inches and $20 \leq L \leq 60$ inches, respectively. This optimization problem is expressed as mathematically,

$$\text{Minimize } f(x) = 0.6224 x_1 x_3 x_4 + 1.7781 x_2 x_3^3 + 3.1611 x_1^2 x_4 + 19.84 x_1^2 x_3 \quad (2.11)$$

subject to

$$g_1(x) = 0.0193 x_3 - x_1 \leq 0 \quad (2.12)$$

$$g_2(x) = 0.00954 x_3 - x_2 \leq 0 \quad (2.13)$$

$$g_3(x) = 750.0 \times 1728.0 - \pi x_3^2 x_4 - \frac{4}{3} \pi x_3^3 \leq 0 \quad (2.14)$$

$$g_4(x) = x_4 - 240.0 \leq 0 \quad (2.15)$$

$$g_5(x) = 1.1 - x_1 \leq 0 \quad (2.16)$$

$$g_6(x) = 0.6 - x_2 \leq 0 \quad (2.17)$$

Sandgren [56] found the optimal values of \$7980.894 using branch and bound method for this optimization problem. Wu and Chow used GA-based approach for this problem and obtained the minimum cost of \$7207.494. Lee and Geem [52] achieves a design with a best solution vector of $x = \{1.125, 0.625, 58.2789, 43.7549\}$ and the minimum cost of \$7207.494 using the harmony search

algorithm without violating any constraint. The results obtained using the HS algorithm are better than earlier results found for this problem, and optimal results are compared in Table 2.2.

Table 2.2 Optimal results for pressure vessel design.

Method	Optimal design variables		Cost (\$)
Sandgren [56]			7980.894
Wu&Chow[57]	1.125	44.2930	7207.494
<i>Lee&Geem [52]</i>		43.7549	7198.433

2.2.3 Case Study 3: Welded Beam Design

The goal in the optimum design problem of the welded beam given in Figure 2.6 is to minimize the fabricating cost of the welded beam subject to constraints on the shear stress, bending stress on the beam, buckling load on the bar, deflection of the beam, and side constraints. The thickness of the weld $h=x_1$, the length of the welded joint $l=x_2$, the width of the beam $t=x_3$, and the thickness of the beam

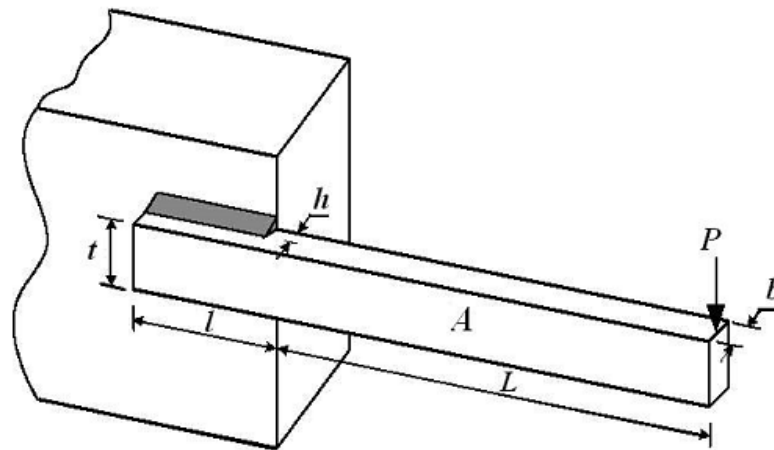


Figure 2.6 Schematic of welded beam[52].

$b=x_4$ are selected as a design variable. The values of x_1 and x_2 are coded with integer multiplies of 0.0065. There are fifteen constrains for the optimization problem of welded beam. This optimization problem is expressed mathematically as;

$$\text{Minimize } f(x) = 1.10471x_1^2 x_2 + 0.04811x_3 x_4 (x_2 + 14) \quad (2.18)$$

subject to

shear stress;

$$g_1(x) = \tau(x) - \tau_{\max} \leq 0 \quad (2.19)$$

bending stress in the beam;

$$g_2(x) = \sigma(x) - \sigma_{\max} \leq 0 \quad (2.20)$$

side constraints;

$$g_3(x) = x_1 - x_4 \leq 0 \quad (2.21)$$

$$g_4(x) = 0.10471x_1^2 + 0.04811x_3 x_4 (x_2 + 14) - 5 \leq 0 \quad (2.22)$$

$$g_5(x) = 0.125 - x_1 \leq 0 \quad (2.23)$$

end deflection of the beam;

$$g_6(x) = \delta(x) - \delta_{\max} \leq 0 \quad (2.24)$$

buckling load on the bar;

$$g_7(x) = P - P_c(x) \leq 0 \quad (2.25)$$

where;

$$\tau(x) = \sqrt{(\tau')^2 + 2\tau'\tau''\frac{x_2}{2R} + (\tau'')^2} \quad (2.26)$$

$$\tau' = \frac{P}{\sqrt{2}x_1x_2} \quad \text{and} \quad \tau'' = \frac{MR}{J} \quad (2.27)$$

$$M = P(L + \frac{x_2}{2}), R = \sqrt{\frac{x_2^2}{4} + (\frac{x_1 + x_3}{2})^2}, J = 2\sqrt{2}x_1x_2 \left[\frac{x_2^2}{12} + (\frac{x_1 + x_3}{2})^2 \right] \quad (2.28)$$

$$\sigma(x) = \frac{6PL}{x_4x_3^2} \quad (2.29)$$

$$\delta(x) = \frac{4PL^3}{Ex_3^3x_4} \quad (2.30)$$

The input data and limit values of design variables are given in Table 2.3 The harmony search algorithm is applied by Geem and Lee [52] to the optimum design problem of the welded beam with the four design variables. Lee and Geem [52] achieves a design with a best solution vector of $x = \{0.2442, 6.2231, 8.2915, 0.2443\}$ and the minimum cost of 2.38 using the harmony search algorithm. The same problem was also solved by Ragsdell and Philips [58] using geometric programming. Deb [59] used a simple Genetic Algorithm (GA). The results are given in Table 2.4.

Table 2.3 Input values and bounds for design variables.

Variable	Lower bound	Upper bound
x_1	0.1	2.0
x_2	0.1	1.0
x_3	0.1	1.0
x_4	0.1	2.0
P=6000 lb	L=14 in	$\delta_{\max}=0.25$ in
$E=30 \cdot 10^6$ psi	$\tau_{\max}=13,600$ psi	
$G=12 \cdot 10^6$ psi	$\sigma_{\max}=30,000$ psi	

Table 2.4 Optimal results of welded beam design.

Method	Optimal design variables				Cost
	H	l	T	b	
Ragsdell and Philips [58]					
Approx	0.2444	6.2189	8.2915	0.2444	2.38
David	0.2434	6.2552	8.2915	0.2444	2.38
GP	0.2455	6.1960	8.2730	0.2455	2.39
Simplex	0.2792	5.6256	7.7512	0.2796	2.53
Random	0.4575	4.7313	5.0853	0.6600	4.12
Deb [53]		unavailable			2.38
Deb [59]	0.2489	6.1730	8.1789	0.2533	2.43
<i>Lee&Geem [52]</i>	<i>0.2442</i>	<i>6.2231</i>	<i>8.2915</i>	<i>0.2443</i>	<i>2.38</i>

2.3 Applications of Harmony Search Algorithm

Since the initial presentation of the harmony search algorithm in the beginning of 2000's, the Harmony Search algorithm has been applied to a widely range of

optimization problems from musical composition to solving Sudoku, from heat exchanger design to timetabling problems, from medical imaging to structural design and environmental problems (The examples below focus only on application of Harmony Search Algorithm to various fields of civil engineering). The harmony search algorithm now has a substantial literature.

Several studies have focused on the design of water distribution networks (Figure 2.7), in particular the selection of optimal pipe size [60, 61, 62]. The design variables are the diameters of pipe segments for this optimization problem. The aim is to select the diameter for each pipe segment that minimizes the total cost of the water distribution network. The problem is constrained by the principles of fluid mechanics and there are some practical and demand requirements. Geem [62] considered the optimal sizing of both the diameters of pipes and the water pump in the water supply system, and minimized the total cost of the system considering the operational cost for the pump. Several water-related studies involve parameter estimation for the nonlinear Muskingum model, which can be used in the prediction of volumetric water flow [63].

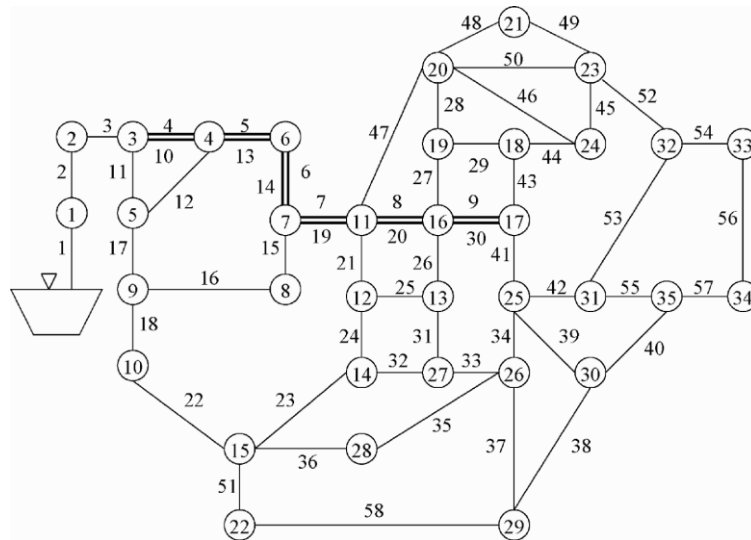


Figure 2.7 BakRyun water distribution network [61].

The harmony search algorithm used also in slope stability analysis to predict the location of the critical slip surface and to estimate the associated factor of safety [64, 65]. Two-dimensional slip surfaces of arbitrary shape and three-dimensional slopes were analyzed using the harmony search algorithm (Figure 2.8). The location of slip surface is found by minimizing the factor of the safety of slope.

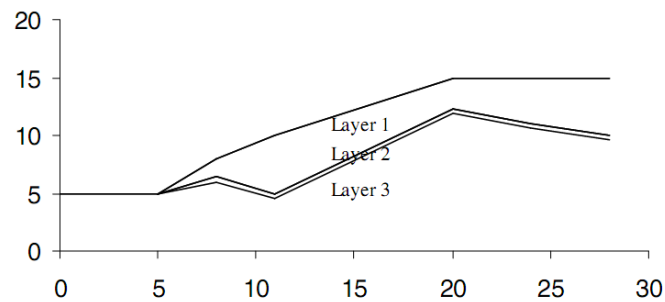


Figure 2.8 Cross section of slope [64].

Transport-related problems are investigated using harmony search algorithm. Travelling salesmen problems with a 20-city and a 51-city are solved using harmony search algorithm [32]. The minimum possible tour routes are found using harmony search algorithm for travelling salesmen problems. Also, a school bus routing problem was investigated by Geem [66]. The required numbers of school buses are found for the best route, and the bus seating capacity and maximum allowable journey time are selected as constraints for this problem.

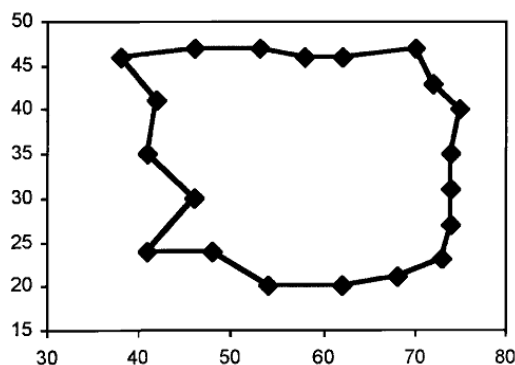


Figure 2.9 20-city travelling salesman problem [32].

The Harmony search algorithm is also used for structural design problems. Lee and Geem [67] present continuous variable applications of harmony search algorithm. Two-dimensional (plane) trusses with 10-200 bars (Figure 2.10), three-dimensional (space) trusses with 22-72 bars, and a 120-bar dome truss problems are investigated in Lee's study. Lee and Geem [68] studied also on discrete variable problems, namely 52-bar plane truss, 25- and 72-bar space trusses, and 47-bar power line tower (Figure 2.11).

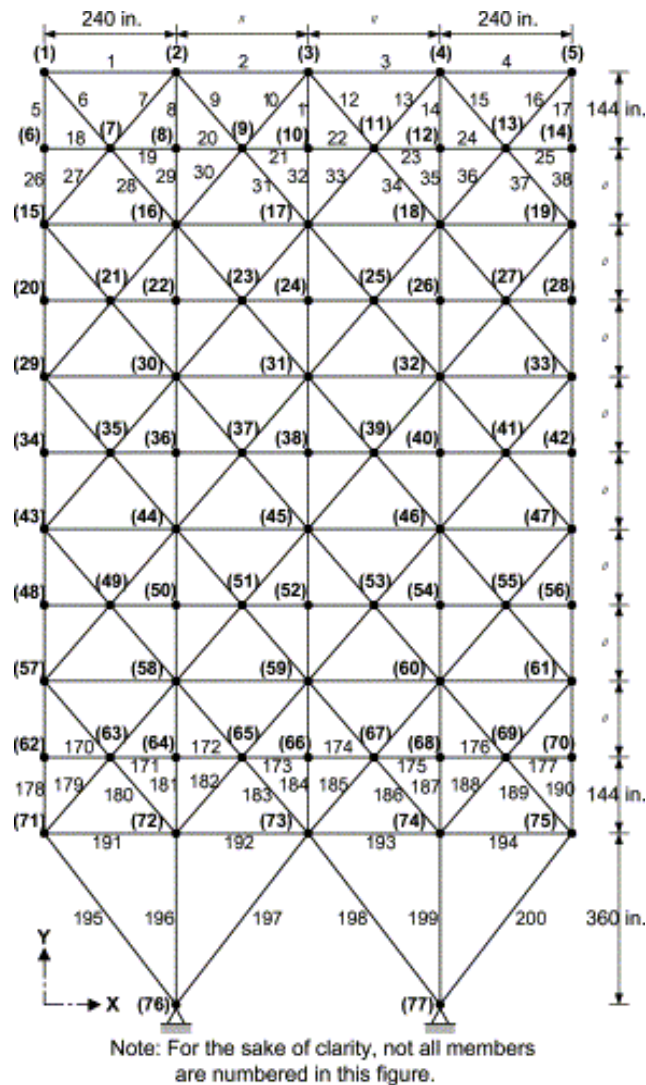


Figure 2.10 Two-hundred-bar planar truss [67].

The optimum designs for geodesic domes having three to six rings (75-285 bars) are presented by Saka [69]. Erdal and Saka [70] present optimization of grillage systems with 24- and 264-member. Değertekin studied on optimization of frame systems, including 1-bay 8-storey plane frame, three-bay twenty four-storey frame and a 4-storey, 84-bar space frame [71, 72].

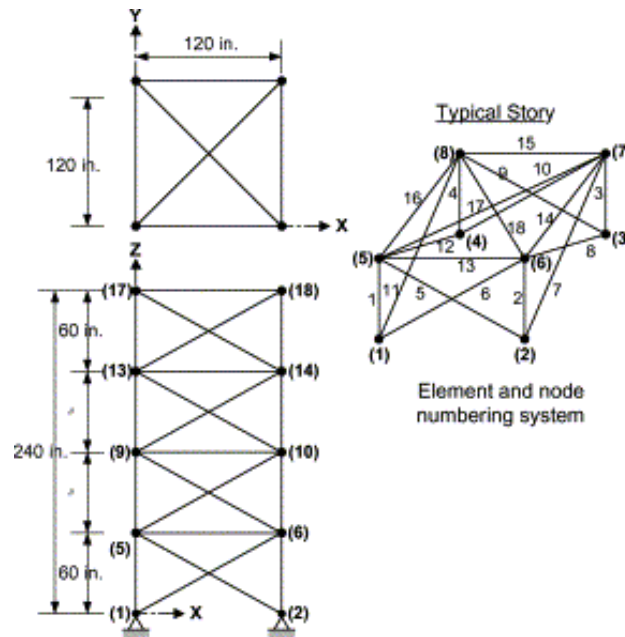


Figure 2.11 Seventy-two-bar space truss [67].

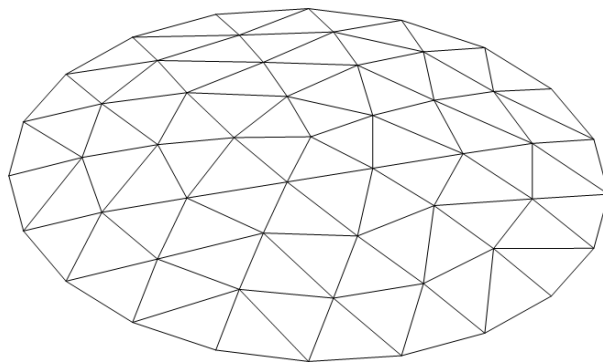


Figure 2.12 Optimum geometry for geodesic dome with four rings [69].

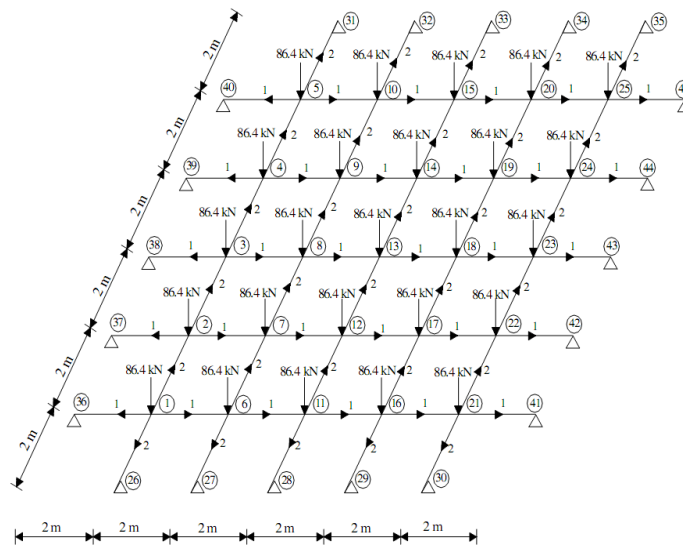


Figure 2.13 60-member grillage system [70].

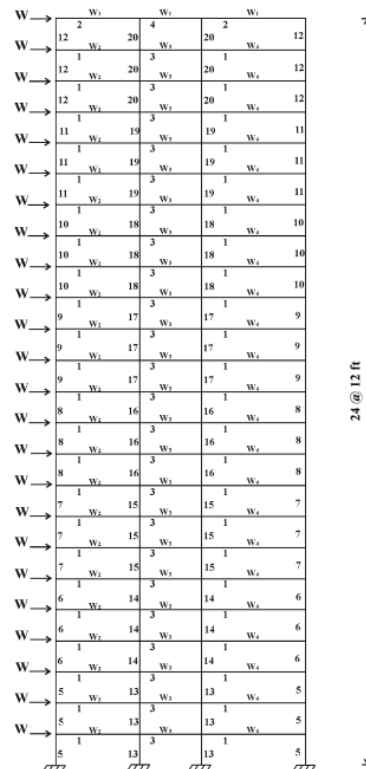


Figure 2.14 Three-bay twenty four-storey frame example [71].

CHAPTER 3

MATHEMATICAL FORMULATION OF OPTIMUM DETAILING DESIGN PROBLEM OF REINFORCED CONCRETE PLANE FRAMES

3.1 Introduction

Formulation of optimum design problem consists of identification of design variables, statement of objective function and constraints to be satisfied. In general, the structural optimization problem may be stated mathematically as,

Minimize

$$f(x) = f(x_1, x_2, x_3, \dots, x_n) \quad (3.1)$$

Subject to

$$g_j(x) = g_j(x_1, x_2, x_3, \dots, x_n) \leq 0 ; j = 1, \dots, r \quad (3.2)$$

where; $f(x)$ is the objective function, $g_j(x)$ are the set of inequality constraints, $x_1, x_2, x_3, \dots, x_n$, are design variables.

3.2 Objective Function

In structural optimization problems, the objective function is generally described as the weight or total cost of structure. Usually, the weight of structure is used for the optimum design of steel structures. For the optimum design of reinforced concrete (RC) structures, the cost of structure is more convenient as an objective function, because concrete structures involve different materials and the nature of concrete design is different. In reality, the minimum weight design may not be the minimum cost design for especially concrete structures. Besides the unit costs of different materials involved concrete construction influence the total cost of the concrete structures. For these reasons, the optimization problem of concrete structures should be formulated in terms of the total cost, which includes the costs of concrete, steel and formwork.

In this study, the objective function is selected as the total cost consisting of individual cost components of concrete, steel and formwork. The cost of any component is inclusive of material, fabrication, and labor. The objective function is expressed mathematically as,

Minimize

$$f_{\text{cost}} = C_c + C_s + C_f \quad (3.3)$$

where, C_c = the cost of concrete, C_s = the cost of reinforcing bars, C_f = the cost of formwork (includes labor and placement) .

The costs of components, C_c , C_s , and C_f , are calculated for each member by the following equations.

For the cost of concrete, C_c ;

$$C_c = V_c \cdot (UC_c) \quad (3.4)$$

where, C_c = the cost of concrete, V_c = the total volume of concrete, (UC_c) = the unit cost of concrete (\$/m³) .

$$V_c = \sum_{i=1}^{N_{col}} b_i d_i L_{column,i} + \sum_{j=1}^{N_{beam}} b_{w,j} h_j L_{clear\ beam,j} \quad (3.5)$$

where, N_{col} = the number of column members, b_i and d_i = the width and depth of i^{th} column member with rectangular cross section, $L_{column,i}$ = the length of i^{th} column member, N_{beam} = the number of beam members, $b_{w,j}$ = the width of j^{th} beam, h_j = the height of j^{th} beam, $L_{clear\ beam,j}$ = the clear span length of j^{th} beam.

For the cost of steel, C_s ;

$$C_s = W_s \cdot (UC_s) \quad (3.6)$$

where, C_s = the cost of steel, W_s = the total weight of steel used as reinforcement bar in the concrete frame, (UC_s) = the unit cost of steel (\$/kg) .

$$W_s = \sum_{i=1}^{N_{col}} \sum_{j=1}^{N_{bar,i}} A_{s,j} l_{bar,j} + \sum_{i=1}^{N_{col}} \sum_{k=1}^{N_{tie,i}} A_{sh,k} l_{tie,k} \quad (3.7)$$

$$+ \sum_{m=1}^{N_{beam}} \sum_{l=1}^{N_{bar,m}} A_{s,l} l_{bar,l} + \sum_{m=1}^{N_{beam}} \sum_{n=1}^{N_{tie,m}} A_{st,n} l_{tie,n}$$

where, $N_{bar,..}$ = the number of longitudinal reinforcement bars placed in the member, $N_{tie,..}$ = the number of ties used in the member, $A_{s,..}$ = the area of reinforcement bars, $l_{bar,..}$ = the length of reinforcement bars, $A_{st,..}$, $A_{sh,..}$ = the area of shear reinforcement bars (ties) $l_{tie,..}$ = the length of ties.

For the cost of formwork, C_f ;

$$C_f = A_f \cdot (UC_f) \quad (3.8)$$

where, C_f = the cost of formwork, A_f = the total formwork area, (UC_f) = the unit cost of formwork (\$/m²) .

$$A_f = \sum_{i=1}^{N_{col}} \{2(b_i + d_i)L_{column,i} - (Area_{beams@ joint,i})\} + \sum_{j=1}^{N_{beam}} \{(b_{w,j} + 2h_j)L_{clear beam,j}\} \quad (3.9)$$

where, $Area_{beams@ joint,i}$ = the cross-section area of beams connected to the i^{th} column at joint.

The detailed evaluation of objective function is defined by Eqs. (3.3-3.9). The unit costs are based on market prices and their values changes from time to time and also from country to country. For these reasons, the unit price data cannot be fixed and needs to be updated. In the previous studies in the literature, researchers used different country design specifications for design and different market prices for unit costs depending to their countries. The different unit material costs are used for the design examples, and these values are given with concerned examples.

3.3 Design Variables

In this study, the column and beams in the structural frame are grouped to satisfy the applicability and efficiency of the introduced optimum design procedure. Also, design variables are divided two groups as columns design variables and beam design variables and to obtain practical designs beams and columns are separated to groups. Total number of design variables is determined according to number of column and beam groups.

3.3.1 Beam Design variables

For beam design groups, the cross-section of beams, the area of reinforcement bars along all beams, and the area of reinforcement bars placed on the top and bottom of beam spans and supports are considered as design variables. The cross-sectional dimensions of the beam are considered as the design variable. It is a general practice to provide the same width and height of beams in all spans. In addition to cross-section dimension of the beam, the areas of longitudinal bars that are placed continuously at the bottom of all the beams and the tensile reinforcements at the spans of beams, and supports for each beam are also considered as the design variable. These design variables and their numbers in the problem are defined in Table 3.1. The design variables relevant with reinforcement bars are not defined as the surface area of reinforcing bars. Instead of this, the reinforcement bar layout is defined and these design variables relevant with reinforcement are expressed in terms of the number of reinforcing bars and the diameter of reinforcing bars. By this way, design process can reach directly constructible optimum designs. In the literature, the similar approaches are used many researchers to obtain rational and practical designs [12,13,15,16,18,19]. The details of the design variables are shown in Figure 3.1.

The total number of the beam design variables, N_{bdv} , for one beam group changes according to number of span (bay) in frame. Total number of beam design variables for one beam group is calculated by Eqn.3.10;

$$N_{bdv} = 5 + 4n_{bay} \quad (3.10)$$

The total number of beam design variables when each beam group considered, Ndv_{beam} , is computed by following equation;

$$Ndv_{beam} = n_{beamgroup} \cdot N_{bdv} \quad (3.11)$$

where, N_{bdv} = the total number of the beam design variables for one beam group, n_{bay} = the number of bays in a frame, $n_{beam\ group}$ = the total number of beam group.

Table 3.1. Cross-sectional and reinforcement design variables for beam groups.

Design Variables		Number
$X_{i,1}$	The width and the height of beam cross-section	1
$X_{i,2}$	The area of the steel reinforcement that continues through the top of all the beams *	1
$X_{i,3}$	The area of the steel reinforcement that continues through the bottom of all the beams *	1
$X_{i,4} - X_{i,k}$	The area of the top steel reinforcement at spans of beams*	n_{bay}
$X_{i,k+1} - X_{i,m}$	The area of the bottom steel reinforcement at spans of beams*	n_{bay}
$X_{i,m+1} - X_{i,n}$	The area of the top steel reinforcement at the supports*	$n_{bay}+1$
$X_{i,n+1} - X_{i,N_{bdv}}$	The area of the bottom steel reinforcement at supports*	$n_{bay}+1$

i = number of beam group (i^{th} beam group), n_{bay} ; number of spans ($k = 3 + n_{bay}$, $m = 3 + 2n_{bay}$, $n = 4 + 3n_{bay}$, $N_{bdv} = 5 + 4n_{bay}$)

* For the reinforcement design variables, the areas of steel reinforcement bars are defined in terms of the number and diameter of the bars ($n\phi d$ n ; number of bars, d ; diameter of bars) to obtain constructable reinforcement areas.

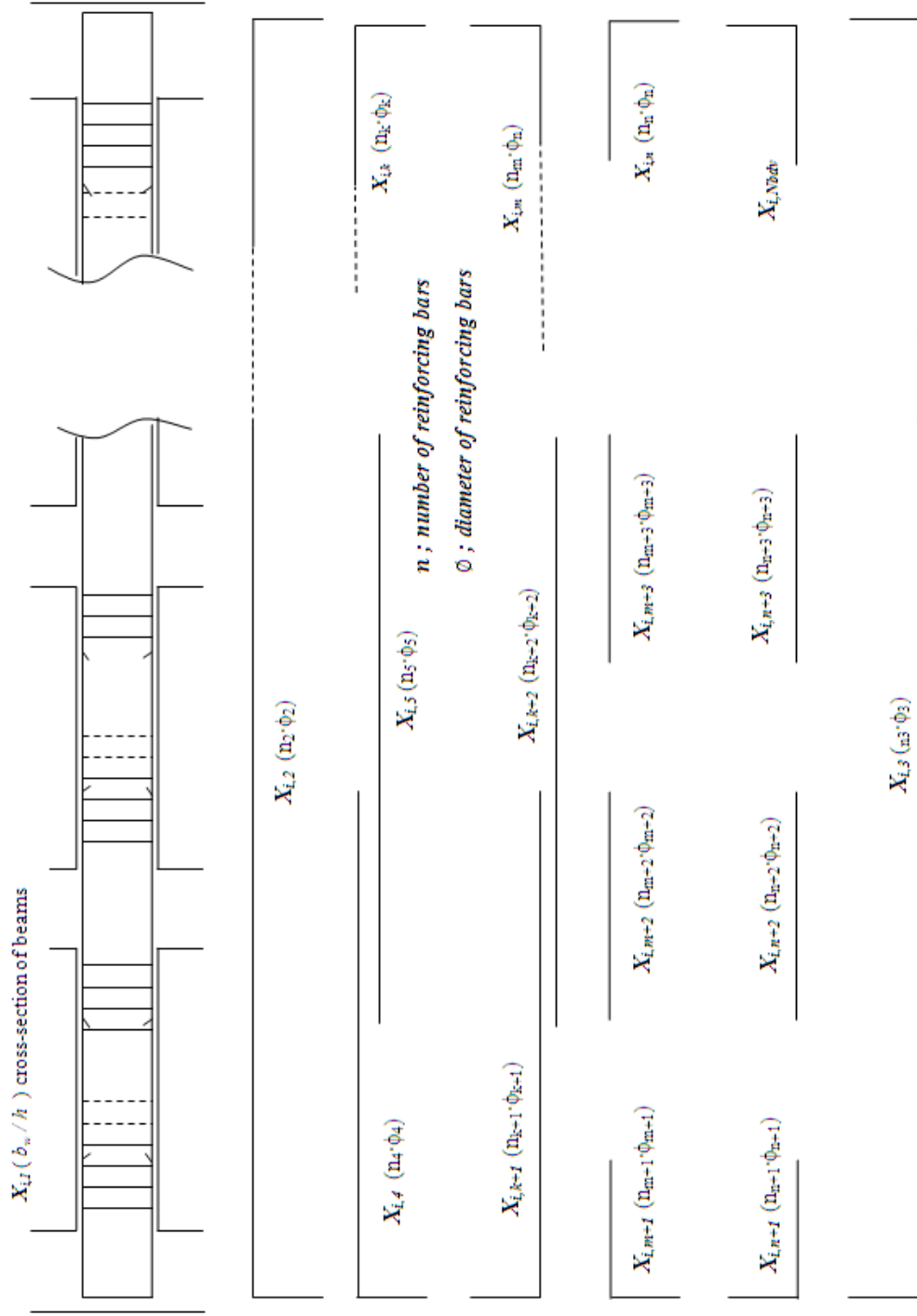


Figure 3.1. The reinforcement bar layout and the design variables for i^{th} beam group.

Table 3.2. Variable poll for beam design variables.

#	X_1 (mm)	#	$X_2 - X_3$	#	$X_4 - X_{Nbdr}$
1	250/400	1	2 ϕ 12	1	NNR
2	250/450	2	2 ϕ 14	2	1 ϕ 12
3	250/500	3	3 ϕ 12	3	1 ϕ 14
4	250/550	4	2 ϕ 16	4	1 ϕ 16
5	250/600	5	3 ϕ 14	5	1 ϕ 18
6	300/400	.6	3 ϕ 16	6	1 ϕ 20
7	300/450			7	1 ϕ 22
8	300/500			8	1 ϕ 24
9	300/550			9	1 ϕ 26
10	300/600			10	1 ϕ 28
11	300/650			11	1 ϕ 30
12	300/700			12	2 ϕ 12
13	350/500			13	2 ϕ 14
14	350/550			14	2 ϕ 16
15	350/600			15	2 ϕ 18
16	350/650			16	2 ϕ 20
17	350/700			17	2 ϕ 22
18	400/600			.	.
19	400/650			.	.
20	400/700			.	.
21	400/750			.	.
22	400/800			.	.
23	450/700			.	.
24	450/750			.	.
25	450/800			45	5 ϕ 18
26	450/850			46	5 ϕ 20
27	450/900			47	5 ϕ 22
28	500/700			48	5 ϕ 24
29	500/750			49	5 ϕ 26
30	500/800			50	5 ϕ 28
31	500/850			51	5 ϕ 30
32	500/900			52	6 ϕ 12
				53	6 ϕ 14
				54	6 ϕ 16
				55	6 ϕ 18
				56	6 ϕ 20
				57	6 ϕ 22
				58	6 ϕ 24
				59	6 ϕ 26
				60	6 ϕ 28
				61	6 ϕ 30

* NNR ; no need reinforcement

For the cross-sectional and reinforcement design variables, the design variable pools are created. For the width of the beam the values in pool starts from 250 mm goes upto 500 mm with the increment of 50mm, similarly for the height of the beam the values starty from 400 mm increases up to 900 mm with the same increment and the combinations of width and height values are is composed in the variable pool table. For the reinforcement design variables, the values are selected such that they give constructable reinforcement areas. In other words, the number and diameter of the bar for a beam can be considered as one design variable ($n\phi d$ n ; number of bars, d ; diameter of bars). For this purpose, the variable pool table is composed of a combination of number and diameter of bars. For the reinforcement design variables, the design variable pool is created by the combination of number and diameter of reinforcement bars. The number of reinforcement bars changes between 2 and 6 and the diameter of bars changes between 12 and 30 with an increment of 2 mm. Then 61 different combination of number and diameter of a bar are arranged for each reinforcement design variables. The 6 reinforcement bar combination are used for the reinforcement continues through the top ($X_{i,2}$) and bottom ($X_{i,3}$) of all the beams. The variable pool composed for reinforcement design variables is given in Table 3.2.

In this optimum design problem, material strengths, unit costs of materials, structural geometry, support conditions, loading conditions, and cover details are pre-assigned as design parameters at the beginning of the optimization process. However, the value of the dead load which includes the self-weight of beam depending on the cross-sectional dimensions is automatically updated during the design cycles.

3.3.2 Column Design Variables

The optimum design problem of RC buildings is more complex than the optimal design of steel structures because only one structural material is considered in steel structures and the cost of the steel structures is only assumed to be

proportional to its weight. Unlike the design of steel structures, there is infinite set of member size and amount of steel reinforcement used in the design of reinforced concrete (RC) buildings. The importance difference between optimum designs of steel and concrete structures is that more combinatorial characteristics affect the determining the cross-sectional dimensions, and the layout and arrangements of reinforcing bars in RC building design. In addition to the discrete and combinatorial nature of the RC sections, the restrictions and reinforcement detailing specified in the design specifications make the optimum design of RC buildings even more complicated.

In the literature, many researchers suggested and used a lot of practical discrete optimization techniques developed for discrete optimum design of RC frames by the construction of a concrete section database [19,23,24,25,26,27]. In these studies, discrete optimum sections are directly searched based on the relationship between the section identification numbers and properties (dimensions and resistant capacities) of sections. The sections widely used in practical design are selected to construct database.

In this thesis, the concrete section database is constructed for the selection of section of column members. Practically, the dimensions of column sections in a RC frames are usually increased by 50 mm a step, and the diameters of reinforcing bars in column members change between 14 and 30 mm most frequently.

Table 3.3 Design variable bounds for RC column design examples.

Design Variables		Lower bound	Upper Bound	Increment (Step size)	Number of possible values in the range
X_{1c}	b (mm)	300	500	50 mm	5
X_{2c}	d (mm)	400	1000	50 mm	13
X_{3c}	ϕ (mm)	{ 14,16,18,20,22,24,26,28,30}			9
X_{4c} and X_{5c}	n_1 and n_2	0	7	1	8

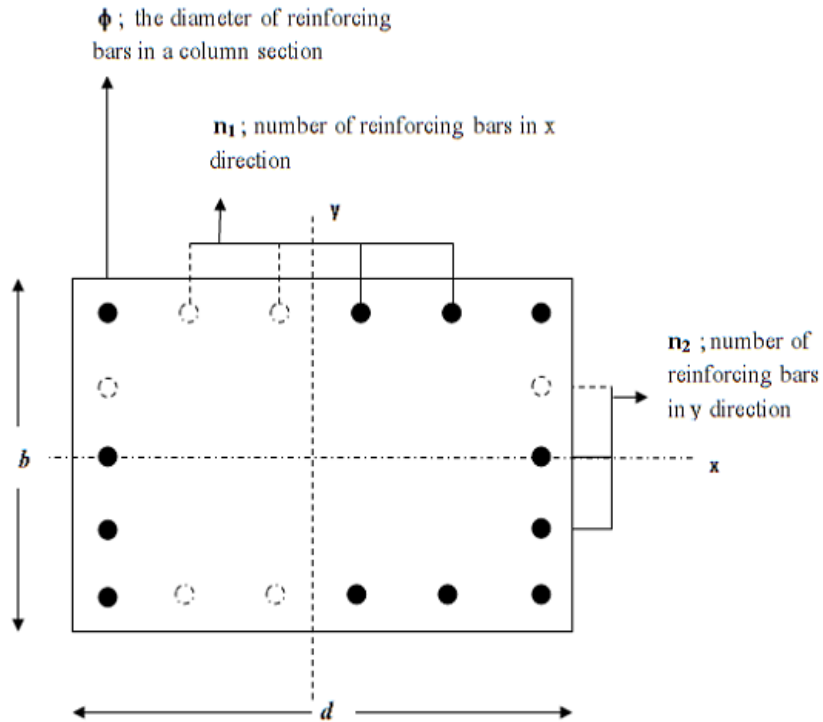


Figure 3.2 The design variables for construction of column section database.

In this study, the design variables for the construction of column section database are selected as the dimensions of columns in x and y directions, the diameter of reinforcement bars at the cross-section of column and numbers of reinforcement bars in both sides of the column as shown in Figure 3.2. The lower-upper bounds and increments of these variables are given in Table 3.3. Assigning these values to variables all possible section combinations are composed, and the combinations satisfy the constraints between 3.14 and 3.22 (given in section 3.3.1) are taken the section pool. However, the number of possible combinations is 37440 and obtained feasible sections number is obtained as 6199. This quantity of feasible column sections is too large to obtain the optimum results using optimization methods. Additionally, some of sections have similar section properties and flexural moment strength. For these reasons, obtained feasible column sections are evaluated to their section properties and flexural strength for balanced case. Finally, the feasible solution number in the column section database is decreased to 219 which is a reasonable number for optimum design process. The selected feasible column sections are given in Table 3.4. The compressive strength of concrete and the yield strength of steel are taken as 30MPa and 400MPa, respectively, and the cover of the concrete section is taken as 50mm to evaluate the strength of sections and to select the sections in the database. In the optimum design process, the interaction diagrams for the assigned sections are computed for the given material strength properties and the detailing of the section.

Table 3.4 Design variables database for column sections.

#	d	b	ϕ	n_1	n_2			(kNm)
1	400	300	14	1	1	2288,78	851	130,97
2	400	300	14	1	2	2363,72	848,46	142,13
3	400	350	14	2	1	2695,22	991,85	149,3
4	400	350	14	1	2	2695,22	991	158,38
5	400	350	22	1	1	3060,77	972,18	193,67
6	400	400	16	2	1	3141,44	1128,26	175,97
7	400	400	18	1	1	3147,55	1125,48	184,34
8	400	400	30	1	2	4372,67	1069,63	322,26
9	450	300	14	2	1	2612,35	978,21	165,86
10	450	300	18	3	1	2980,95	978,22	198,67
11	450	350	18	2	1	3230	1132,58	212,82
12	450	350	20	5	3	4140,05	1134,76	304,82
13	450	400	14	4	3	3658	1305,69	241,09
14	450	400	14	3	4	3658	1302,73	250,54
15	500	300	14	3	2	3010,86	1110,75	221,56
16	500	300	14	5	2	3160,75	1119,95	231,13
17	500	300	22	2	1	3411,59	1093,26	269,78
18	500	300	20	3	2	3556,89	1104,83	289,79
19	500	350	16	2	2	3487,95	1283,73	261,31
20	500	350	18	3	1	3643,95	1294,64	265,44
21	500	350	16	2	3	3585,83	1280,4	280,92
22	500	400	20	2	1	4079,74	1462,73	301,26
23	500	400	26	3	1	4865,9	1466,68	380,36
24	500	400	24	4	2	5076,97	1464,93	410,59
25	500	450	18	2	3	4596,59	1641,79	358,32
26	500	450	18	2	4	4720,48	1637,58	383,01
27	500	450	24	2	2	5050,85	1627,56	410,79
28	550	300	18	2	1	3354,32	1229,6	275,49
29	550	300	18	3	1	3478,2	1242,8	284,23

Table 3.4 Cont. Design variables database for column sections.

#	<i>d</i>	<i>b</i>	ϕ	n_1	n_2			(kNm)
30	550	300	20	2	1	3499,62	1227,39	295,86
31	550	300	20	4	1	3805,51	1248,66	316,54
32	550	300	20	2	2	3652,57	1222,19	330,19
33	550	350	14	5	2	3865,19	1456,02	305,28
34	550	350	14	3	3	3790,25	1441,67	311,33
35	550	350	20	2	1	3955,43	1431,02	326,84
36	550	350	18	2	2	3934,02	1429,43	334,43
37	550	350	20	6	2	4720,17	1468,34	404,48
38	550	400	14	6	2	4395,95	1664,26	341,66
39	550	400	18	3	2	4513,72	1646,67	374,16
40	550	400	16	2	4	4429,6	1630,45	385,69
41	550	400	24	2	1	4747,73	1627,79	404,4
42	550	400	20	5	3	5175,98	1661,33	459,15
43	550	450	16	2	3	4787,52	1838,24	394,51
44	550	500	16	5	2	5439,11	2071,55	424,56
45	550	500	14	2	6	5307,57	2040,23	450,22
46	600	300	14	4	1	3508,11	1374,17	294,71
47	600	300	18	3	2	3850,72	1368,77	362,88
48	600	350	16	3	1	4068,07	1596,43	346,04
49	600	350	16	4	1	4165,96	1601,92	353,42
50	600	350	18	3	1	4224,08	1597,8	368,63
51	600	350	16	6	3	4557,51	1612,9	419,59
52	600	350	16	7	3	4655,39	1625,05	428,66
53	600	350	20	3	2	4551,39	1594,04	432,05
54	600	350	22	3	3	4961,29	1588,18	513,62
55	600	400	16	4	1	4663,21	1827,15	390,38
56	600	400	16	7	1	4956,87	1856,95	416,12
57	600	400	18	3	2	4845,22	1818,41	436,78

Table 3.4 Cont. Design variables database for column sections.

#	d	b	ϕ	n_1	n_2			(kNm)
58	600	400	18	4	2	4969,11	1825,03	446,05
59	600	400	22	3	2	5273,47	1818,46	504,35
60	600	400	18	4	4	5216,88	1816,61	508,45
61	600	450	16	3	2	5160,46	2043,57	444,72
62	600	450	16	4	3	5356,23	2045,72	476,85
63	600	450	16	3	4	5356,23	2036,9	494,23
64	600	450	24	3	2	6016,97	2041,96	579,72
65	600	450	30	3	5	7916,59	2003,81	966,49
66	600	500	18	4	3	6087,49	2270,45	551,15
67	600	500	20	4	2	6196,09	2274,99	554,22
68	600	500	16	6	4	6147,14	2285,27	555,23
69	600	500	20	4	4	6501,98	2264,58	630,92
70	600	500	26	3	5	7557,33	2238,09	850,07
71	650	300	22	3	1	4342,53	1508,58	440,63
72	650	350	16	3	3	4553,91	1738,76	455,28
73	650	350	24	3	1	5092,29	1756,4	517,29
74	650	350	22	6	3	5806,56	1787,03	636
75	650	400	14	5	4	5133,89	1998,02	498,79
76	650	400	18	3	4	5424,49	1980,79	572,46
77	650	450	16	5	5	6022,83	2244,12	614,87
78	650	500	18	5	3	6625,76	2500,91	647,58
79	650	500	20	5	3	6916,36	2505,33	696,33
80	650	500	20	3	6	7069,31	2459,97	795,58
81	650	500	24	3	5	7589,33	2459,47	889,61
82	700	300	14	5	1	4080,31	1641,18	399,37
83	700	300	20	4	2	4704,34	1640,9	527,01
84	700	300	20	5	2	4857,29	1662,59	543,7
85	700	300	26	3	1	5031,65	1649,01	576,21
86	700	300	24	3	2	5022,47	1636,51	603

Table 3.4 Cont. Design variables database for column sections.

#	<i>d</i>	<i>b</i>	ϕ	n_1	n_2			(kNm)
87	700	350	14	4	2	4660,43	1894,71	464,48
88	700	350	14	3	3	4660,43	1886,1	480,4
89	700	350	16	3	2	4746,08	1890,53	488,73
90	700	350	18	6	1	5175,87	1933,86	523,74
91	700	350	16	7	2	5137,63	1934,52	529,61
92	700	350	18	7	1	5299,76	1951,72	537,78
93	700	350	16	7	3	5235,52	1931,19	559,51
94	700	350	24	3	1	5382,35	1909,13	587,06
95	700	350	20	5	2	5437,41	1928,55	594,11
96	700	350	20	4	3	5437,41	1901,65	623,8
97	700	350	28	3	1	5859,55	1918,42	669,01
98	700	400	16	3	4	5521,98	2150,65	598,97
99	700	400	18	5	4	6003,77	2178,17	675,01
100	700	400	24	3	4	6623,21	2151,78	836,42
101	700	450	16	4	3	6102,11	2428,44	628,66
102	700	450	16	3	5	6200	2414,11	679,3
103	700	450	20	5	2	6597,66	2460,47	694,93
104	700	450	24	4	2	6983,09	2447,14	774,25
105	700	450	22	5	4	7256,87	2456,04	856,16
106	700	500	16	6	3	6878,01	2717,23	699,65
107	700	500	18	6	2	7040,13	2728,77	712,71
108	700	500	20	4	6	7636,63	2683,91	914,2
109	700	500	28	3	5	8799,04	2670,53	1178,62
110	750	300	14	4	2	4328,93	1754,73	468,85
111	750	300	16	4	1	4414,58	1762,82	472,77
112	750	300	14	5	2	4403,88	1766,52	477,93
113	750	300	16	5	1	4512,47	1778	484,58
114	750	300	18	5	1	4720,48	1788,23	520,22
115	750	300	18	6	1	4844,37	1798,77	533,7

Table 3.4 Cont. Design variables database for column sections.

#	<i>d</i>	<i>b</i>	ϕ	n_1	n_2			(kNm)
116	750	300	18	4	2	4720,48	1765,06	546,31
117	750	300	20	5	1	4952,96	1799,4	559,81
118	750	300	22	4	2	5209,92	1777,7	642,12
119	750	350	14	4	3	5025,44	2040,18	551,75
120	750	350	18	4	1	5218,15	2056,42	563,28
121	750	350	14	6	3	5175,33	2058,67	569,08
122	750	350	16	5	3	5329,81	2058,9	607,47
123	750	350	18	5	2	5465,93	2071,17	619,12
124	750	350	22	5	1	5831,48	2097,86	661,23
125	750	350	20	6	2	5880,42	2093,61	684,68
126	750	350	20	4	3	5727,47	2052,64	700,28
127	750	350	22	4	3	6016,55	2057,72	760,82
128	750	400	16	7	2	6049,26	2373,07	655,75
129	750	400	18	6	4	6459,16	2360,44	772,46
130	750	400	26	5	1	7040,36	2409,29	817,69
131	750	400	26	4	2	7040,36	2363,13	871,63
132	750	400	22	7	3	7193,31	2414,07	883,36
133	750	450	16	5	2	6475,05	2637,37	690,89
134	750	450	14	5	4	6418,45	2625,39	701,69
135	750	450	14	4	5	6418,45	2611,07	717,55
136	750	450	16	5	5	6768,71	2627,37	788,31
137	750	450	28	7	3	9191,41	2752,48	1193,5
138	750	500	20	4	5	7898,06	2902,44	974,86
139	800	350	14	6	2	5390,45	2209,98	611,59
140	800	350	14	5	3	5390,45	2200,18	629,53
141	800	350	20	4	2	5864,59	2208,09	725,95
142	800	350	20	5	2	6017,54	2232,38	745,8
143	800	350	18	7	3	6127,66	2246,81	767,84
144	800	350	20	7	2	6323,43	2268,88	783,91

Table 3.4 Cont. Design variables database for column sections.

#	d	b	ϕ	n_1	n_2			(kNm)
145	800	350	26	4	1	6450,38	2240,46	812,38
146	800	350	20	6	3	6323,43	2241,05	818,25
147	800	350	28	5	2	7339,01	2285,63	1010,52
148	800	400	14	7	2	6128,39	2529,5	687,36
149	800	400	18	4	1	6171,22	2513,61	698,3
150	800	400	20	4	1	6374,64	2520,81	737,48
151	800	400	18	4	4	6542,88	2500,98	830,96
152	800	400	20	5	4	6986,43	2529,48	920,61
153	800	400	24	5	2	7286,21	2564,06	933,75
154	800	450	18	5	2	7081,99	2837,24	824,64
155	800	450	20	6	2	7496,48	2861,28	895,73
156	800	450	24	4	5	8389,7	2814,16	1205,06
157	800	500	20	5	2	8006,54	3154,92	943,66
158	800	500	24	5	5	9272,95	3154,95	1299,31
159	850	350	14	7	1	5680,51	2371,26	664,1
160	850	350	16	7	1	5909,93	2386,99	707
161	850	350	14	7	3	5830,4	2366,19	721,87
162	850	350	16	6	2	5909,93	2369,95	731,11
163	850	350	18	4	2	5922,17	2350,24	752,5
164	850	350	22	4	2	6411,6	2366	864,17
165	850	350	22	5	2	6596,67	2396,32	889,99
166	850	350	20	6	3	6613,5	2392,78	905,04
167	850	350	24	4	2	6693,03	2374,99	927,95
168	850	350	24	5	2	6913,27	2410,66	958,55
169	850	350	28	5	1	7329,29	2452,34	999,13
170	850	350	24	6	3	7353,77	2423,86	1070,05
171	850	350	28	5	2	7629,07	2442,15	1112,4
172	850	400	16	6	2	6614,37	2699,07	805,64
173	850	400	20	5	1	6859,09	2716,56	843,15

Table 3.4 Cont. Design variables database for column sections.

#	<i>d</i>	<i>b</i>	ϕ	n_1	n_2	(kNm)		
174	850	400	18	7	2	6998,27	2729,54	877,44
175	850	400	22	4	2	7116,04	2693,88	938,66
176	850	450	20	5	3	7869,42	3034,44	1034,54
177	850	450	20	6	3	8022,37	3049,36	1054,05
178	850	450	18	5	5	7826,6	3015,77	1061,38
179	850	450	22	5	5	8560,75	3033,17	1250,53
180	850	500	16	4	6	8219,02	3317,52	1078,72
181	850	500	30	4	2	9797,45	3383,91	1373,69
182	900	400	18	6	2	7205,88	2882,64	951,43
183	900	400	18	5	3	7205,88	2865,38	985,08
184	900	400	18	7	3	7453,66	2895,41	1020,17
185	900	400	22	6	2	7817,68	2914,96	1088,25
186	900	450	16	5	4	7789,63	3201,47	1043,27
187	900	450	20	5	3	8242,36	3226,06	1141,62
188	900	500	20	5	5	9294,13	3564,72	1350,11
189	950	400	20	5	1	7522,09	3057,72	1027,23
190	950	400	18	6	3	7661,27	3048,44	1100,64
191	950	400	20	5	3	7827,98	3047,31	1160,2
192	950	400	28	5	2	8996,51	3119,62	1420,16
193	950	400	26	6	3	9141,81	3119,23	1477,93
194	950	450	18	6	3	8448,58	3418,7	1193,8
195	950	450	20	7	2	8768,25	3459,56	1232,71
196	950	450	24	5	4	9508,52	3437,63	1527,68
197	950	450	26	6	3	9929,12	3487,83	1571,04
198	950	450	28	5	3	10083,6	3477,61	1642,28
199	950	500	20	7	2	9555,56	3829,41	1325,86
200	950	500	28	5	4	11170,7	3835,6	1864,41
201	950	500	30	5	4	11658,6	3851,41	1999,42
202	1000	400	20	5	1	7853,59	3227,6	1125,5

Table 3.4 Cont. Design variables database for column sections.

#	d	b	ϕ	n_1	n_2	(kNm)		
203	1000	400	22	6	1	8295,61	3264,47	1215,93
204	1000	400	22	5	2	8295,61	3238,12	1272,4
205	1000	400	24	5	4	9052,7	3240,31	1557,86
206	1000	400	26	6	3	9473,31	3293,3	1604,32
207	1000	400	30	5	4	10415,5	3291,11	1960,43
208	1000	450	26	6	3	10302,1	3682,68	1707,54
209	1000	450	26	5	4	10302,1	3645,65	1785,85
210	1000	450	26	6	4	10560,5	3673,9	1825,85
211	1000	450	26	5	5	10560,5	3636,86	1904,17
212	1000	450	28	5	5	11056,1	3651,94	2056,94
213	1000	450	30	5	5	11588,4	3667,96	2220,41
214	1000	500	20	5	2	9664,04	4003,65	1402,51
215	1000	500	20	7	2	9969,93	4041,03	1450,92
216	1000	500	20	5	3	9816,98	3998,45	1473,01
217	1000	500	18	5	5	9774,16	3977,65	1504,87
218	1000	500	22	6	4	10508,3	4025,99	1677,74
219	1000	500	28	5	2	10985,5	4071,49	1749,45

3.4 Constraints

Constraints to be considered in the optimum design problem are strength, serviceability, ductility and other side constraints. Constraints can be imposed separately for column groups, beam groups and connection regions. These constraints are taken from ACI 318-05.

3.4.1 Constraints for Column Groups

Two types of design constraints are considered for the column members of the RC frame. The first type includes those constraints on the axial load and moment resistance capacities of the section, clear spacing limits between reinforcing bars, and the minimum and the maximum percentage of steel allowed. The second type consists of those constraints defining architectural requirements and good design and detailing practices. These include the requirement of the minimum and the maximum dimensions of column, the maximum aspect ratio of the section, maximum number of reinforcing bars and other reinforcement requirements. The constraints are explained and expressed in a normalized form as given below.

The maximum axial load capacity of columns, $P_{n,max}$ should be greater than the factored axial design load acting on the column section, P_d ;

$$g_1(x) = \frac{P_{d(i,j)}}{P_{n,max(i,j)}} - 1 \leq 0 \quad i = 1, N_{col} \quad j = 1, 6 \quad (3.12)$$

where, i = number of the column (i^{th} column), N_{col} = total number of columns, j = load combination type.

For a column section with uni-axial bending, the moment carrying capacity of column section, M_n , obtained for each factored axial design load, P_d , should be greater than the applied factored design moment, M_d ;

$$g_2(x) = \frac{M_{d(i,j)}}{M_{n@P_u(i,j)}} - 1 \leq 0 \quad i = 1, N_{col} \quad j = 1, 6 \quad (3.13)$$

where, i = number of the column (i^{th} column), N_{col} = total number of columns, j = load combination type.

The percentage of longitudinal reinforcement steel, ρ , in a column section should be between minimum and maximum limits permitted by design specification ($\rho_{min} = 0.01$ and $\rho_{max} = 0.06$) ;

$$g_3(x) = \frac{\rho_{min}}{\rho_i} - 1 \leq 0 \quad (3.14)$$

and

$$g_4(x) = \frac{\rho_i}{\rho_{max}} - 1 \leq 0 \quad i = 1, N_{col} \quad (3.15)$$

where, i = number of the column (i^{th} column), N_{col} = total number of columns.

The width b and the height d of a column section should not be less than the minimum dimensions limit value given for columns (*min. dimension*, $cd_{min} = 300\text{mm}$);

$$g_5(x) = \frac{cd_{min}}{b_i} - 1 \leq 0 \quad (3.16)$$

and

$$g_6(x) = \frac{cd_{min}}{d_i} - 1 \leq 0 \quad i = 1, N_{col} \quad (3.17)$$

where, i = number of the column (i^{th} column), N_{col} = total number of columns.

The ratio of shorter dimension of column section to longer one should be greater than permitted limit ($cdr_{min} = 0.40$);

$$g_7(x) = \frac{cdr_{\min}}{(b_i / d_i)} - 1 \leq 0 \quad i = 1, N_{col} \quad (3.18)$$

where, i = number of the column (i^{th} column), N_{col} = total number of columns.

The minimum diameter of longitudinal reinforcing bars, $\emptyset$, in a column section should be greater than minimum bar diameter, $\emptyset_{\min}$, specified by design code;

$$g_8(x) = \frac{\phi_{\min}}{\phi_i} - 1 \leq 0 \quad i = 1, N_{col} \quad (3.19)$$

where, i = number of the column (i^{th} column), N_{col} = total number of columns.

The total number of longitudinal reinforcing bars, nrb , in a column section should be smaller than specified maximum number of reinforcing bars, $nrb_{\max}$, for detailing practice ($nrb_{\max} = 24$);

$$g_9(x) = \frac{nrb_i}{nrb_{\max}} - 1 \leq 0 \quad i = 1, N_{col} \quad (3.20)$$

where, i = number of the column (i^{th} column), N_{col} = total number of columns.

The minimum and maximum clear spacings between longitudinal bars, a , in a column section should be between minimum and maximum limits, $a_{\min}$ and $a_{\max}$, specified for detailing practice ($a_{\min} = 50 \text{ mm}$ and $a_{\max} = 150 \text{ mm}$);

$$g_{10}(x) = \frac{a_{\min}}{a_i} - 1 \leq 0 \quad (3.21)$$

and

$$g_{11}(x) = \frac{a_i}{a_{\max}} - 1 \leq 0 \quad i = 1, N_{col} \quad (3.22)$$

where, i = number of the column (i^{th} column), N_{col} = total number of columns.

The shear force capacity of column section, ϕV_n , should be greater than applied factored design shear force, V_d ;

$$g_{12}(x) = \frac{V_{d(i,j)}}{\phi V_{n(i,j)}} - 1 \leq 0 \quad i = 1, N_{col} \quad j = 1, 6 \quad (3.23)$$

where, i = number of the column (i^{th} column), N_{col} = total number of columns, j = load combination type.

Also, the shear force capacity of column section, ϕV_n , should be greater than the minimum capacity shear forces, $\min\{V_e^c, V_e^b\}$, based on probable maximum flexural strength of column, V_e^c , and based on probable maximum flexural strengths at the ends of beams framed into the top joint of column, V_e^b ;

$$g_{13}(x) = \frac{\min V_{e(k)}^c, V_{e(k)}^b}{\phi V_{n(k)}} - 1 \leq 0 \quad k = 1, N_{column \text{ group}} \quad (3.24)$$

where, k = number of the column group (k^{th} column group), N_{col} = total number of column groups.

The factored design shear force acting on column section, V_d , should be less than allowed maximum shear force capacity, ϕV_{max} ;

$$g_{14}(x) = \frac{V_{d(i,j)}}{\phi V_{\max(i,j)}} - 1 \leq 0 \quad i = 1, N_{col} \quad j = 1, 6 \quad (3.25)$$

where, i = number of the column (i^{th} column), N_{col} = total number of columns, j = load combination type.

The area of shear reinforcement (ties), A_{sh} , should be greater than limitations on the minimum area of shear reinforcement, $A_{sh,min}$;

$$g_{15}(x) = \frac{A_{sh,min}}{A_{sh(i,j)}} - 1 \leq 0 \quad k = 1, N_{column \in group} \quad j = 1, 3 \quad (3.26)$$

$$A_{sh,min} \rightarrow \max \left\{ \begin{array}{l} 0.062 \sqrt{f_c} b d \frac{s}{f_y} \geq 0.35 d \frac{s}{f_y} \\ 0.3 s b_c \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_y} \\ 0.09 s b_c \frac{f_c}{f_y} \end{array} \right. \quad (3.27)$$

where, k = number of the column group (k^{th} column group), $N_{column group}$ = total number of column groups, j = three (top, middle and bottom parts) shear design region of column, f_c = the compressive strength of concrete, f_y = the yielding strength of reinforcing steel, b and d = the width and height of column section, s = the spacing between stirrups (ties), b_c = the cross-sectional dimension of column core measured center-to-center of outer legs of the transverse reinforcement comprising area A_{sh} , A_g = the gross area of section, A_{ch} = the cross-sectional area of member measured out-to-out of transverse reinforcement.

The spacing between stirrups, s , in the column should be greater than minimum spacing, s_{min} ($s_{min} = 50 \text{ mm}$) for constructional requirements;

$$g_{16}(x) = \frac{s_{min}}{s_{(i,j)}} - 1 \leq 0 \quad k = 1, N_{column group} \quad j = 1, 3 \quad (3.28)$$

where, k = number of the column group (k^{th} column group), $N_{column group}$ = total

number of column groups, $j = \text{three (top, middle and bottom parts) shear design region of the column.}$

At the top and bottom ends of column members, the spacing between stirrups, s , should be less than maximum spacing of shear reinforcement for end regions of column, $s_{max,end}$;

$$g_{17}(x) = \frac{s_{(i,j)}}{s_{max,end}} - 1 \leq 0 \quad k=1, N_{column\ group} \quad j=1,2 \quad (3.29)$$

$$s_{max,end} \rightarrow \min \begin{cases} \frac{d}{4} \\ \frac{b}{4} \\ 6\phi_b \end{cases} \quad (3.30)$$

where, $k = \text{number of the column group (} k^{\text{th}} \text{ column group), } N_{column\ group} = \text{total number of column groups, } j = \text{two (top and middle parts) shear design region of the column, } b \text{ and } d = \text{the width and height of column section, } s = \text{the spacing between stirrups (ties), } \phi_b = \text{the diameter of longitudinal reinforcing bars.}$

For the middle parts column members, the spacing between stirrups, s , should be less than maximum spacing of shear reinforcement, $s_{max,middle}$;

$$g_{18}(x) = \frac{s_{(i,middle)}}{s_{max,middle}} - 1 \leq 0 \quad k=1, N_{column\ group} \quad (3.31)$$

$$s_{max,middle} \rightarrow \min \begin{cases} 6\phi_b \\ 15\ cm \end{cases} \quad (3.32)$$

where, k = number of the column group (k^{th} column group), $N_{\text{column group}}$ = total number of column groups, ϕ_b = the diameter of longitudinal reinforcing bars.

The length of top and bottom shear regions, l_o , should be greater than allowable design length, $l_{o,\min}$;

$$g_{19}(x) = \frac{l_{o,\min}}{l_{o(i,j)}} - 1 \leq 0 \quad k = 1, N_{\text{column group}} \quad j = 1, 2 \quad (3.33)$$

$$l_{o,\min} \rightarrow \max \begin{cases} \text{larger of } b \text{ and } d \\ 1/6 \text{ clear span of column} \\ 45 \text{ cm} \end{cases} \quad (3.34)$$

where, k = number of the column group (k^{th} column group), $N_{\text{column group}}$ = total number of column groups, j = two (top and middle parts) shear design region of column, b and d = the width and height of column section.

The detailed explanation of design philosophy of columns subject to ACI 318-05 specifications and the derivation of the design constraints accordingly are given in Appendix B.

3.4.2 Constraints for Beam Groups

Constraints to be imposed for each beam group are based on strength, serviceability, ductility and other side constraints. The normalized forms of all constraints considered in optimum design problem are given below.

For three critical sections (left end, middle part and right end) of each beam, the negative (with top steel in tension) and positive (with bottom steel in tension) moment carrying capacities of section for M_n , should be greater than the applied

factored design moments M_d ;

$$g_{20}(x) = \frac{M_{d(i,j,k,l)}}{M_{n(i,j,k,l)}} - 1 \leq 0 \quad i = 1, N_{beam} \quad j = 1, \dots, 6 \quad k = 1, 3 \quad l = 1, 2 \quad (3.35)$$

where, i = number of the beam (i^{th} beam), N_{beam} = total number of beams, j = load combination type, k = three critical sections for flexural design of beam, l = the negative moment and positive moment situations (top steel in tension or bottom steel in tension).

The tension area of longitudinal reinforcement steel bars in tension A_s , for three critical sections in a beam should satisfy the minimum and the maximum requirements permitted by design specification;

$$g_{21}(x) = \frac{A_{s,\min(i,k,l)}}{A_{s(i,k,l)}} - 1 \leq 0 \quad i = 1, N_{beam} \quad k = 1, 3 \quad l = 1, 2 \quad (3.36)$$

and

$$g_{22}(x) = \frac{A_{s(i,k,l)}}{A_{s,\max(i,k,l)}} - 1 \leq 0 \quad i = 1, N_{beam} \quad k = 1, 3 \quad l = 1, 2 \quad (3.37)$$

$$A_{s,\min} \rightarrow \max \left\{ \begin{array}{l} 0.25 \frac{f'_c}{f_y} b_w h \\ 1.4 \frac{b_w h}{f_y} \end{array} \right. \quad (3.38)$$

$$A_{s,\max} = 0.025 b_w h \quad (\text{for the total area of top and bottom flexural bars})$$

where, i = number of the beam (i^{th} beam), N_{beam} = total number of beams, k = three critical sections for flexural design of beam, l = the negative moment and

positive moment situations (checking for top and bottom reinforcement steel area), b_w and h = the width and height of beam section, f_c = the compressive strength of concrete, f_y = the yielding strength of reinforcing steel.

At any end (support) of the beams, the positive moment capacity M_n^- , (i.e., associated with the bottom steel) should be greater than 1/2 of the beam negative moment capacity M_n^+ , (i.e., associated with the top steel) at that end;

$$g_{23}(x) = \frac{\frac{1}{2} M_{n,(i,k)}^-}{M_{n,(i,k)}^+} - 1 \leq 0 \quad i = 1, N_{beam} \quad k = 1, 2 \quad (3.39)$$

where, i = number of the beam (i .th beam), N_{beam} = total number of beams, k = the left and right ends of beam.

The positive flexural moment strength, $M_{n,middle}^+$, at span of beams should be greater than a quarter of negative and positive flexural moment strengths at the ends of beams, M_n^- and M_n^+ ;

$$g_{24}(x) = \frac{\frac{1}{4} M_{n,(i,k)}^-}{M_{n,(i,middle)}^+} - 1 \leq 0 \quad (3.40)$$

and

$$g_{25}(x) = \frac{\frac{1}{4} M_{n,(i,k)}^+}{M_{n,(i,middle)}^-} - 1 \leq 0 \quad i = 1, N_{beam} \quad k = 1, 2 \quad (3.41)$$

where, i = number of the beam (i .th beam), N_{beam} = total number of beams, k = the left and right ends of beam.

The shear force capacity of three regions (left and right ends, and span) of beam, ϕV_n , should be greater than applied factored design shear force, V_d ;

$$g_{26}(x) = \frac{V_{d(i,j,k)}}{\phi V_{n(i,j,k)}} - 1 \leq 0 \quad i = 1, N_{beam} \quad j = 1, 6 \quad k = 1, 3 \quad (3.42)$$

where, i = number of the beam (i .th beam), N_{beam} = total number of beams, j = load combination type, k = three critical region (left and right ends, and span) for shear design of beam.

Also, the shear force capacity of design regions in a beam, ϕV_n , should be greater than the probable shear forces based on probable maximum flexural strengths with the factored gravity loads at beam ends of beam, $\max\{V_{e1}, V_{e2}\}$;

$$g_{27}(x) = \frac{\max V_{e1(i)} V_{e1(i)}}{\phi V_{n(i,k)}} - 1 \leq 0 \quad i = 1, N_{beam} \quad k = 1, 3 \quad (3.43)$$

where, where, i = number of the beam (i .th beam), N_{beam} = total number of beams, k = three critical region (left and right ends, and span) for shear design of beam.

The factored design shear forces at the middle and ends of beam, V_d , should be less than allowed maximum shear force capacity, ϕV_{max} ;

$$g_{28}(x) = \frac{V_{d(i,k)}}{\phi V_{max(i,k)}} - 1 \leq 0 \quad i = 1, N_{beam} \quad k = 1, 3 \quad (3.44)$$

where, where, i = number of the beam (i .th beam), N_{beam} = total number of beams, k = three critical region (left and right ends, and span) for shear design of beam.

The spacing between stirrups s , in the middle and the ends of the beam should be greater than the minimum spacing, s_{min} ($s_{min} = 50 \text{ mm}$) for constructional requirements;

$$g_{29}(x) = \frac{s_{min}}{s_{(i,k)}} - 1 \leq 0 \quad k = 1, N_{beam} \quad j = 1, 3 \quad (3.45)$$

where, where, i = number of the beam (i .th beam), N_{beam} = total number of beams, k = three critical region (left and right ends, and span) for shear design of beam.

At the left and right ends of beam members, the spacing between stirrups, s , should be less than maximum spacing limit of shear reinforcement for end regions of beam $s_{max,end}$;

$$g_{30}(x) = \frac{s_{(i,j)}}{s_{max,end}} - 1 \leq 0 \quad i = 1, N_{beam} \quad k = 1, 2 \quad (3.46)$$

$$s_{max,end} \rightarrow \min \begin{cases} \frac{h}{4} \\ 30 \text{ cm} \\ 24\phi_b \end{cases} \quad (3.47)$$

where, where, i = number of the beam (i^{th} beam), N_{beam} = total number of beams, k = three critical regions (left and right ends and the span) for shear design of beam, h = the height of beam section, ϕ_b = the diameter of shear reinforcing bars (ties) .

Along the span of a beam member, the spacing between stirrups s should be less than maximum spacing of the shear reinforcement, $s_{max,middle}$;

$$g_{31}(x) = \frac{S_{(i,middle)}}{S_{max,middle}} - 1 \leq 0 \quad i = 1, N_{beam} \quad (3.48)$$

$$S_{max,middle} \rightarrow \min \begin{cases} \frac{h}{2} \\ 60 \text{ cm} \\ 30 \frac{A_{sv}}{f_y} b_w \end{cases} \quad (3.49)$$

where, where, i = number of the beam (i^{th} beam), N_{beam} = total number of beams, h = the height of beam section, b_w and h = the width and height of column section, A_{sv} = the area of shear reinforcement (tie), f_y = the yielding strength of reinforcing steel.

The area of shear reinforcement (ties) in beam sections (span and ends of beam), A_{sv} , should be greater than limitations on the minimum area of shear reinforcement, $A_{sv,min}$;

$$g_{32}(x) = \frac{A_{sv,min}}{A_{sv(i,k)}} - 1 \leq 0 \quad i = 1, N_{beam} \quad k = 1, 3 \quad (3.50)$$

$$A_{sv,min} = 0.062 \sqrt{f_c} b_w h \frac{s}{f_y} \geq 0.35 h \frac{s}{f_y} \quad (3.51)$$

where, where, i = number of the beam (i^{th} beam), N_{beam} = total number of beams, k = three critical regions (left and right ends, and along the span) for shear design of a beam, f_c = the compressive strength of concrete, f_y = the yielding strength of reinforcing steel, b_w and h = the width and height of beam section, s = the spacing between stirrups (ties) .

The width of beams, b_w , should be greater than allowable minimum width for beams, $b_{w,min}$;

$$g_{33}(x) = \frac{b_{w,\min}}{b_{w(i)}} - 1 \leq 0 \quad i = 1, N_{beam} \quad (3.52)$$

$$b_{w,\min} \rightarrow \max \begin{cases} 25 \text{ cm} \\ \frac{L_{beam(i)}}{50} \end{cases} \quad (3.53)$$

where, where, i = number of the beam (i^{th} beam), N_{beam} = total number of beams, L_{beam} = the length of beam.

The height of beams, h , should be greater than allowable minimum height for beams, $h_{\min}$;

$$g_{34}(x) = \frac{h_{\min}}{h_{(i)}} - 1 \leq 0 \quad i = 1, N_{beam} \quad (3.54)$$

$$h_{\min} \rightarrow \begin{cases} \frac{L_{beam(i)}}{18.5} & \text{for one end continuous} \\ \frac{L_{beam(i)}}{21} & \text{for both end continuous} \end{cases} \quad (3.55)$$

where, where, i = number of the beam (i^{th} beam), N_{beam} = total number of beams, L_{beam} = length of the beam.

The detailed explanation of design philosophy of a beam member subject to ACI 318-05 specifications and the relevant constraints derived from these provisions are given in Appendix B.

3.4.3 Constraints for Joints

At frame joints, the width of beams, b_w , should be smaller than the width of column, b , framed into the ends joint of beam ;

$$g_{35}(x) = \frac{b_{w(i)}}{b_{(i)}} - 1 \leq 0 \quad i = 1, N_{joint} \quad (3.56)$$

where, i = number of the joint (i^{th} joint), N_{joint} = total number of joints.

The width of the top column, b , should be equal or smaller than the width of the bottom column at the column joints;

$$g_{36}(x) = \frac{b_{top\ column(i)}}{b_{bottom\ column(i)}} - 1 \leq 0 \quad i = 1, N_{joint} \quad (3.57)$$

where, i = number of the joint (i^{th} joint), N_{joint} = total number of joints.

The height of the top column, h , should be equal or smaller than the width of the bottom column at the column joints;

$$g_{37}(x) = \frac{h_{top\ column(i)}}{h_{bottom\ column(i)}} - 1 \leq 0 \quad i = 1, N_{joint} \quad (3.58)$$

where, i = number of the joint (i^{th} joint), N_{joint} = total number of joints.

The sum of nominal flexural strengths of columns framing into the joint, ΣM_{nc} , should be 1.2 times greater than the sum of nominal flexural strengths of beams framing into same joint, ΣM_{nb} ,

$$g_{38}(x) = \frac{\frac{6}{5} \sum M_{nb(i)}}{\sum M_{nc(i)}} - 1 \leq 0 \quad i = 1, N_{joint} \quad (3.59)$$

where, i = number of the joint (i^{th} joint), N_{joint} = total number of joints.

The relative story displacements, Δ_i , should be smaller than the allowable story drift, Δ_a , given in the design specification;

$$g_{39}(x) = \frac{\left(\frac{\Delta_i}{h_i}\right)}{\Delta_a} - 1 \leq 0 \quad i = 1, N_{story} \quad (3.60)$$

where, i = number of the story (i^{th} story), N_{story} = total number of stories, h_i = the height of story from base level.

CHAPTER 4

OPTIMUM DESIGN ALGORITHM FOR REINFORCED CONCRETE FRAMES

4.1 Introduction

Meta-heuristic techniques are based on certain strategies that guide the search process in the design domain and they are non-deterministic algorithms. The literature survey indicates that the meta-heuristic techniques are widely used in solving engineering optimization problems compared to traditional mathematical programming techniques. There are a number of meta-heuristic techniques, such as genetic algorithm, simulating annealing, tabu search method, evolutionary strategies, ant colony optimization, particle swarm optimization, and harmony search algorithm that are used in the optimization problems effectively [73, 74, 75, 76]. The harmony search algorithm is one of the recent additions to the meta-heuristic optimization methods. The harmony search algorithm only needs adjustment of a few parameters and consists of simple steps. Furthermore, it also has a fast convergence rate [69, 78]. For these reasons, the harmony search algorithm is adopted in the solution of the optimum detailed design problem in this study.

4.2 Optimum Design Process with Harmony Search Algorithm

The optimum design problem of RC frames necessitates finding the appropriate values of design variables described in Section 3.3 such that the design

constraints given by the inequalities from (3.12) to (3.60) are satisfied and the objective function shown in equation (3.3) has the minimum value. The constraints used in this optimization problem are derived from ACI 318-05 [30] for *Special Seismic Moment Frames*. Also, the design calculations are performed for regulations of *Special Seismic Moment Frames* in ACI 318-05. The design variables are considered as two groups which are beam and column design variables. The values of beam design variables are to be selected from a design pool (Table 3.2) that contains discrete values for beam design variables due to practical reasons. The values of column design variables are to be selected from a design pool (Table 3.4) that contains pre-determined column sections. For each column group, only one variable is assigned and this assigned value represents sequence number of the column section. This number changes between 1 and 219. The section properties of the column section (dimensions of column, diameter of reinforcing bars, and the number of reinforcing bars etc.) are taken from the assigned row number. Hence, the solution technique to be adopted for the optimum design problem should be capable of obtaining the solutions of discrete programming problems efficiently.

Firstly, beams and columns are grouped to satisfy uniformity of members subject to close design forces and have similar behaviors according to their place in the frame and loading conditions. Then the total number of design variables is calculated by using the numbers of beam groups, column groups and bays in a frame and the harmony search method parameters (HMS, HMCR, PAR, Max. iteration number) are selected. The randomly selected values are assigned for the design variables (beam and column) and the flexural and shear strength of beams are calculated for assigned section properties. Also, the column interaction diagrams are obtained for the assigned column section and material strengths and the flexural capacities of the column sections are determined for each load combination using the interaction diagram obtained for assigned column section. The constraints are checked to determine feasible design vectors and the cost (the objective function) of the frame is calculated. The Harmony Search Memory

(HSM) is filled by these feasible design vectors randomly. After the HSM is filled, the new design vectors are obtained according to the logic of the Harmony Search Algorithm. In this phase, harmony memory consideration rate (HMCR), pitch adjustment rate (PAR), and random selection are used to determine the new values of design variables for each iteration. If the new design vector obtained in each iteration satisfies the constraints and the objective function value (the cost of frame) of the new design vector is better (smaller) than the worst objective function value in the Harmony Memory Matrix (HM), then the new design vector (design variables) is included in the HM and the existing worst harmony is excluded from the HM. The new HM with new design vector is then sorted by the objective function value (the cost of design vector). This process is repeated until the termination criterion is satisfied. At the end of this process, the first row of HM matrix gives the optimum design vector which includes the optimum values of design variables. The flowchart of this process is demonstrated in Figure 4.1. The computer program in Fortran programming language was written to realize the explained procedure above. The results of the program were verified by considering solved examples from the literature.

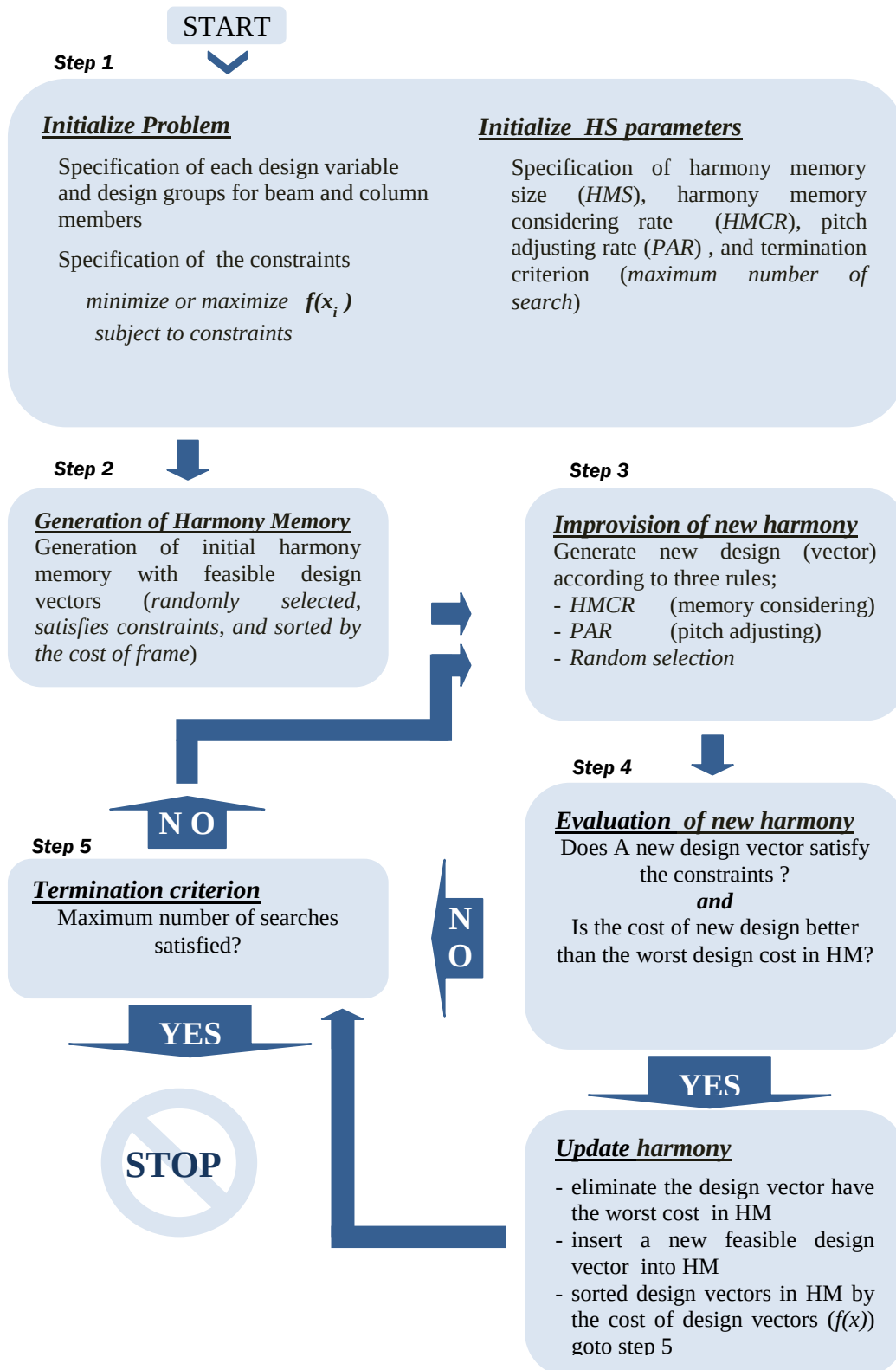


Figure 4.1 Optimum design procedure of RC frames with Harmony search algorithm.

CHAPTER 5

DESIGN EXAMPLES

5.1 Introduction

To verify the effectiveness of the present algorithm in this study, four RC plane frames are selected and solved by the design method developed in this thesis. These are two-bay seven-storey, two-bay five-story, two-bay six-story and five-bay five-story frames. Design examples with different design variable numbers are selected to show the efficiency and the performance of the harmony search algorithm. The performance of the harmony search algorithm is investigated for reinforced concrete plane frames having different number of stories. Uniform gravity loads are assumed for dead load (D) and live load (L) and for seismic loading the lateral equivalent static earthquake loads which are calculated with respect to ASCE 7-05 are applied as joint loads. Six different factored load combinations are considered as suggested in ACI 318-05 design code. Different unit costs of concrete, steel and formwork are used in some design examples in order to investigate its effect on the optimum design of RC frames. The design examples considered are solved several times using with different set of harmony search parameters and among the optimum frames obtained for each set, the best one is taken as the optimum design.

5.2 Two-Bay Seven-Story RC Frame

Two-bay, seven-storey RC frame shown in Figure 5.1, is selected as the first design example. This frame is composed of 14 beams and 21 columns, which are collected in four design groups: two groups for beams and two groups for columns. The compressive strength of concrete and yield strength of steel are taken as 30 MPa and 400 MPa, respectively. The unfactored uniformly distributed dead load of 35 kN/m and live load of 25 kN/m are applied on the beams of the left span, and distributed dead load of 35 kN/m and live load of 20 kN/m are applied on the beams of the right span. The load combinations that are given in App. B (ACI 318-05) are considered in the optimum design. The unit costs of concrete, steel and formwork are 100 TL/m³, 1 TL/kg and 25 TL/m², respectively. The unit weights of concrete and steel are taken as 25kN/m³ and 78.5kN/m³, respectively. The seismic lateral forces are calculated as specified in ASCE 7-05 for given site and seismic design properties. The input data of frame is given in Table 5.1. In this particular example, there are 28 design variables in the optimum design problem. Harmony search method parameters HMS, HMCR, PAR are selected as 40, 0.75, and 0.15, respectively. The minimum cost of optimum design obtained with harmony search method is 10023.42 TL. The unit quantities of concrete, formwork and steel in the optimum RC frame are 17.70 m³, 173.16 m² and 3924.27 kg, respectively. The optimum values of design variables obtained by the harmony search algorithm are given in Table 5.2. The design history is shown in Figure 5.2 and the design detailing for beams and the interaction diagrams for column groups are given Figure 5.3-5.5. The cost components obtained are tabulated in Table 5.3.

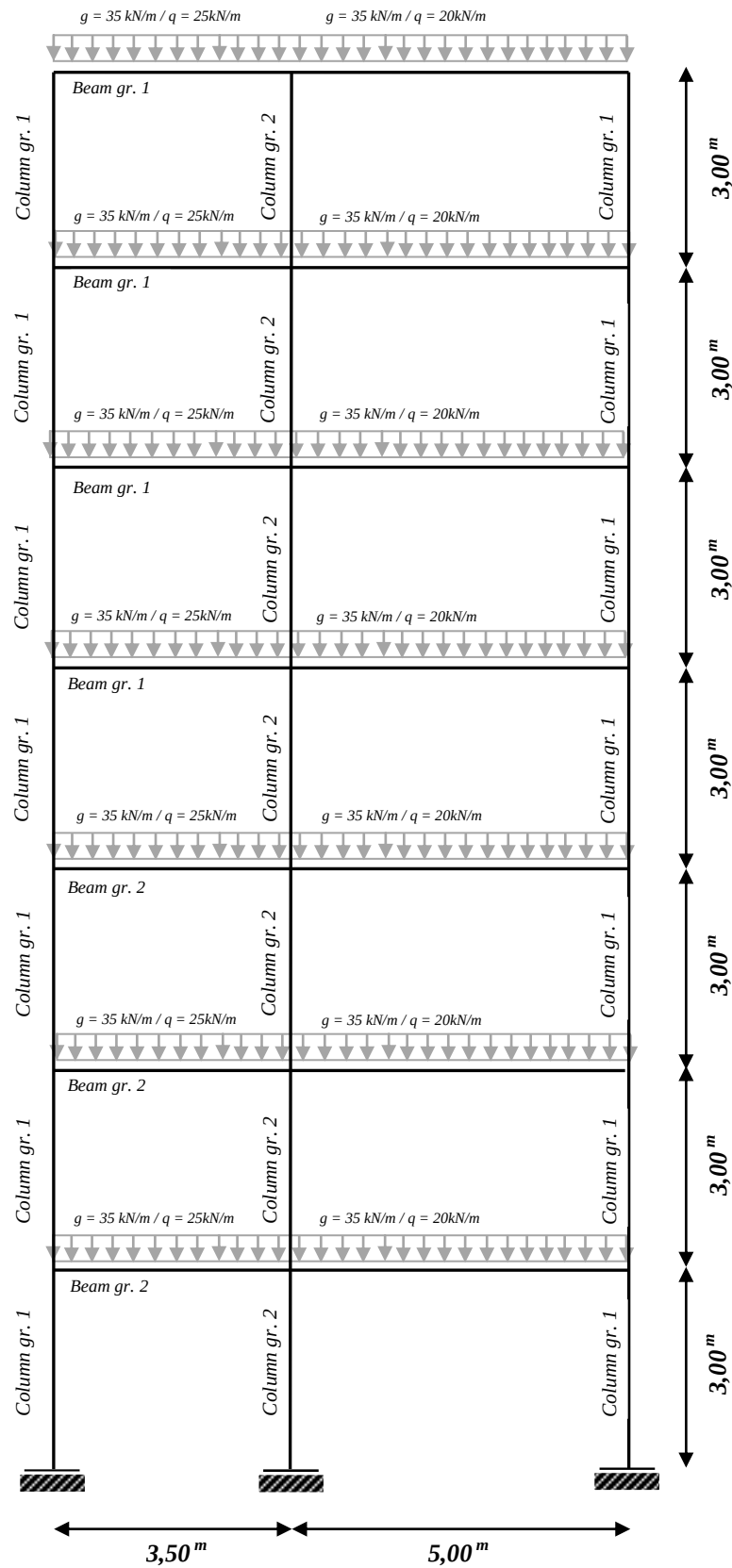


Figure 5.1 Two-bay seven-storey RC plane frame example.

Table 5.1 Input data for two-bay seven story RC frame example.

<i>Input datas for two-bay seven-storey frame</i>	
Compressive strength of concrete, f_c (N/mm ²)	30
Yielding stress of steel, f_y (N/mm ²)	400
Cover for beams, mm	40
Cover for columns, mm	50
Total number of beam groups	2
Total number of beams	14
Total number of column groups	2
Total number of columns	21
<i>Seismic Parameters</i>	
Site Class	C
S_s	0.50
S_1	0.20
F_a	1.20
F_v	1.60
R	8
I	1.0
T_L	6.0
I	1
S_{DS}	0.533
S_{D1}	0.2133
<i>Harmony Search Algorithm Parameters</i>	
HMS	40
HMCR	0.75
PAR	0.15
Max. Iteration	100,000

Table 5.2 The optimum values of design variables for two-bay seven story RC frame.

<i>Optimum design results for two-bay seven-storey frame</i>		
<i>Optimum column design results</i>		
<i>Design Variables</i>	<i>Column Gr. 1</i>	<i>Column Gr. 2</i>
Section number in design pool	(28)	(110)
The height of section, h (mm)	550	750
The height of section, b (mm)	300	300
The diameter of reinforcement bars, ϕ (mm)	18	14
The number of bars in x direction, n_1	2	4
The number of bars in y direction, n_2	1	2
Total reinforcement	10 ϕ 18	18 ϕ 14
<i>Optimum beam design results</i>		
<i>Design Variables</i>	<i>Beam Gr. 1</i>	<i>Beam Gr. 2</i>
x_1	25/50 (5)	25/45 (3)
x_2	2 ϕ 12 (1)	3 ϕ 12 (3)
x_3	3 ϕ 12 (3)	2 ϕ 14 (2)
x_4	4 ϕ 12 (12)	2 ϕ 12 (5)
x_5	1 ϕ 12 (2)	1 ϕ 12 (2)
x_6	1 ϕ 22 (10)	4 ϕ 14 (19)
x_7	1 ϕ 18 (6)	2 ϕ 14 (7)
x_8	3 ϕ 12 (9)	4 ϕ 14 (19)
x_9	1 ϕ 12 (2)	1 ϕ 14 (3)
x_{10}	1 ϕ 14 (3)	2 ϕ 14 (7)
x_{11}	NNR* (1)	NNR* (1)
x_{12}	5 ϕ 14 (26)	5 ϕ 12 (17)
x_{13}	1 ϕ 12 (2)	1 ϕ 12 (2)

NNR*, no need reinforcement, (n_i), the sequence number of i^{th} design variable in the design variable pool.

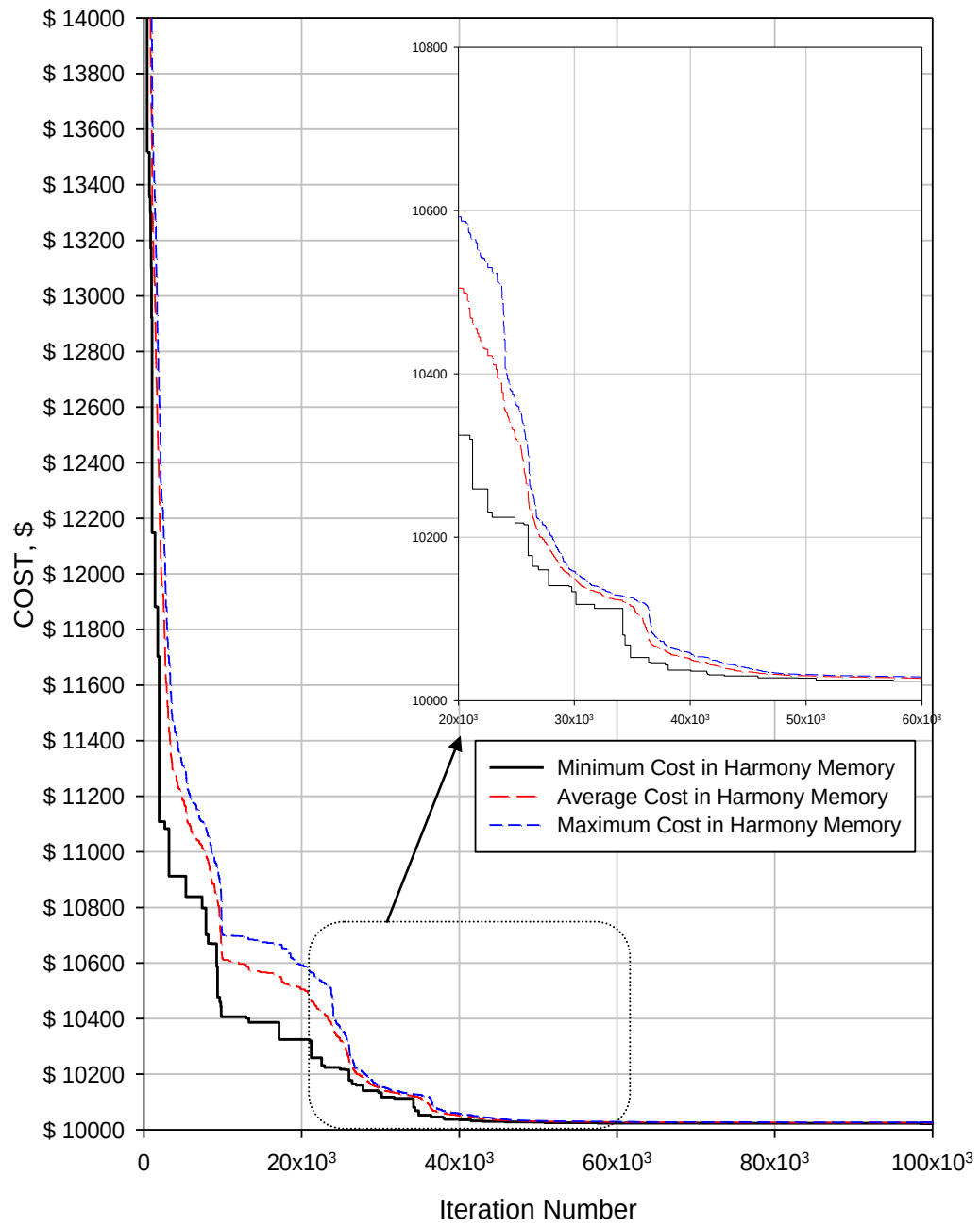
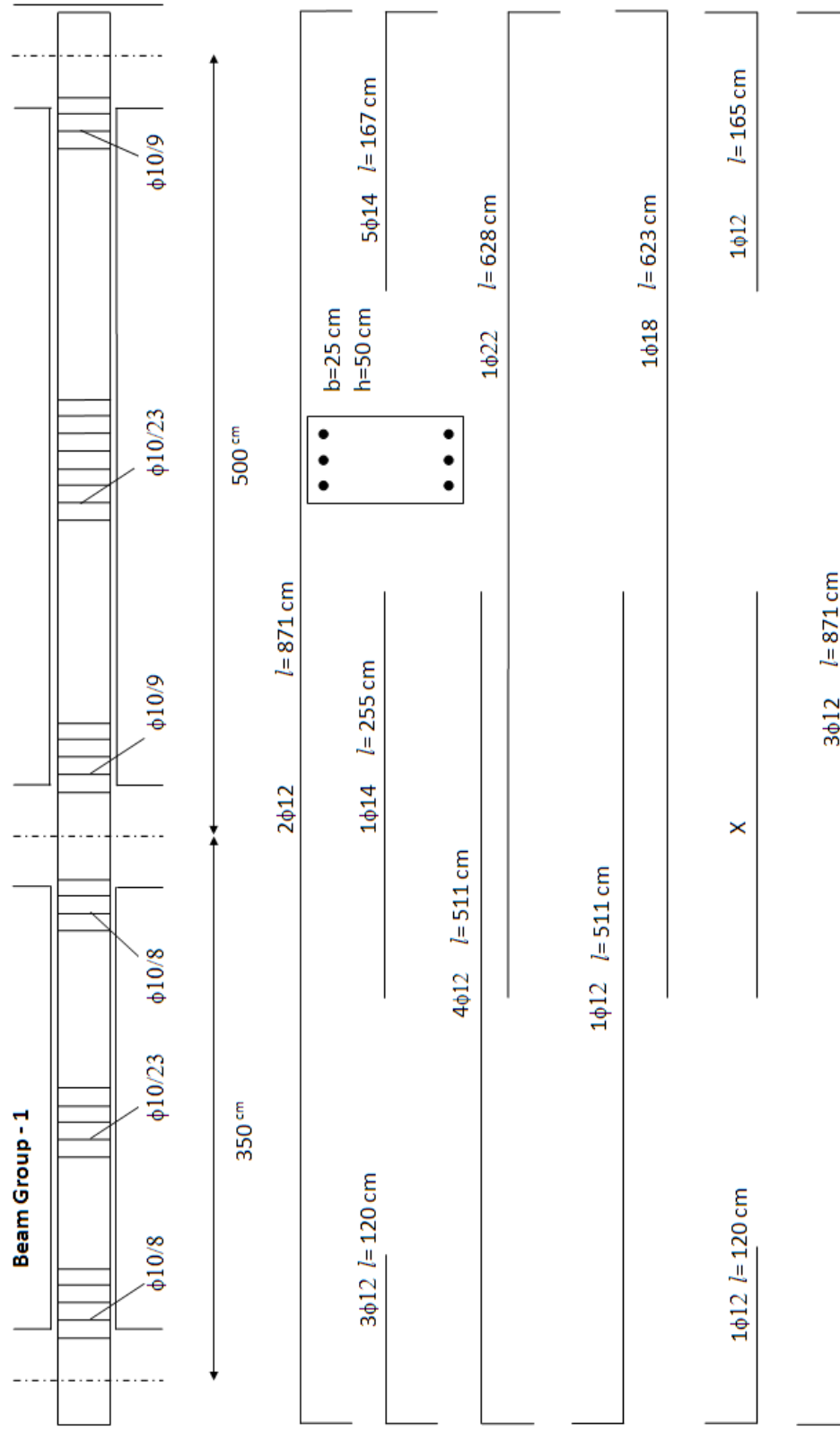


Figure 5.2 The design history for two-bay seven-storey RC plane frame.



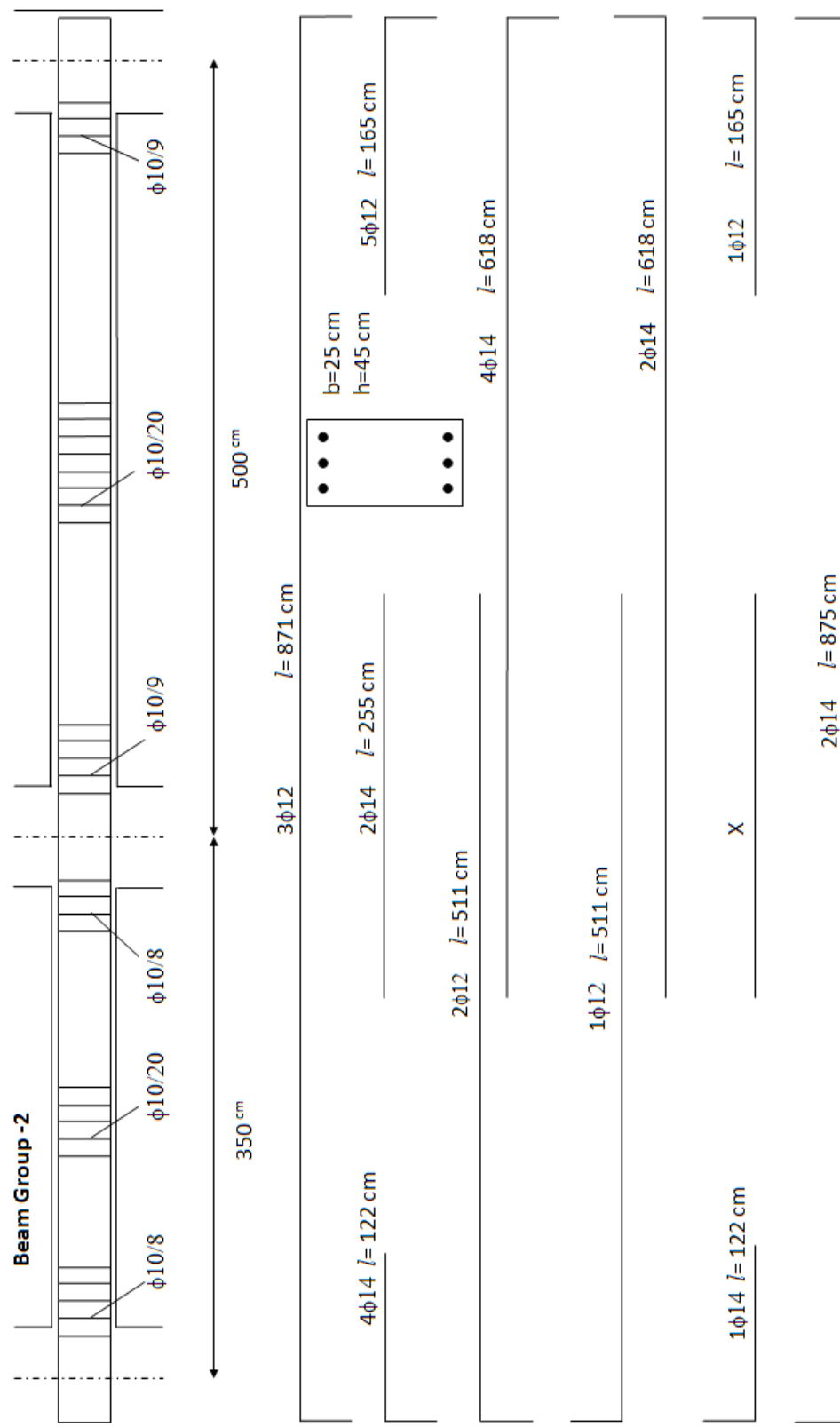
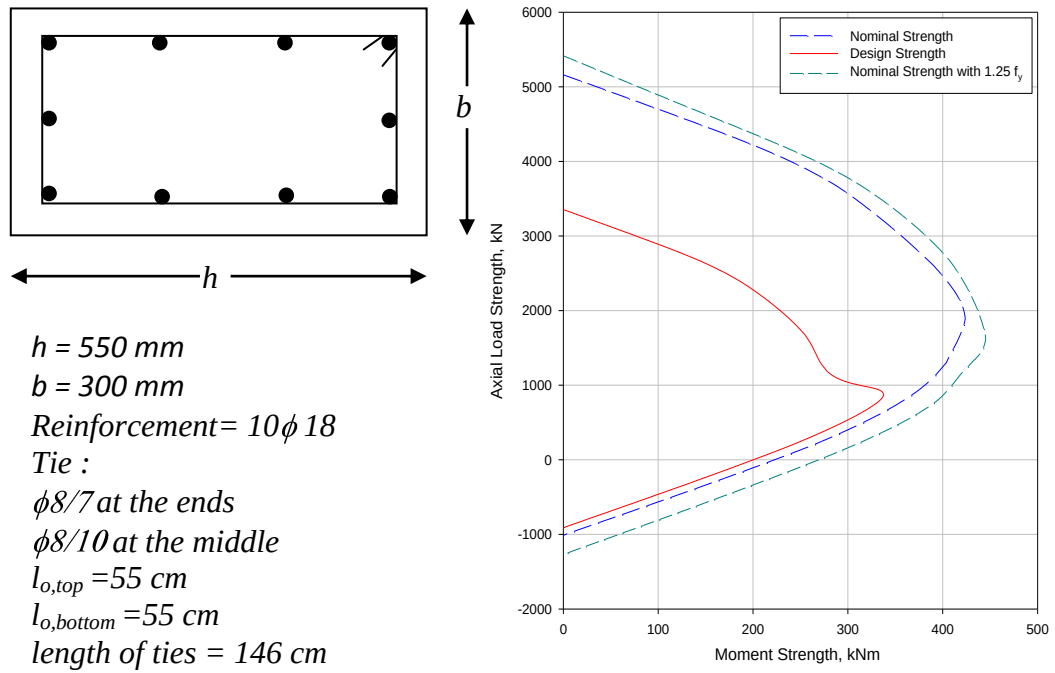
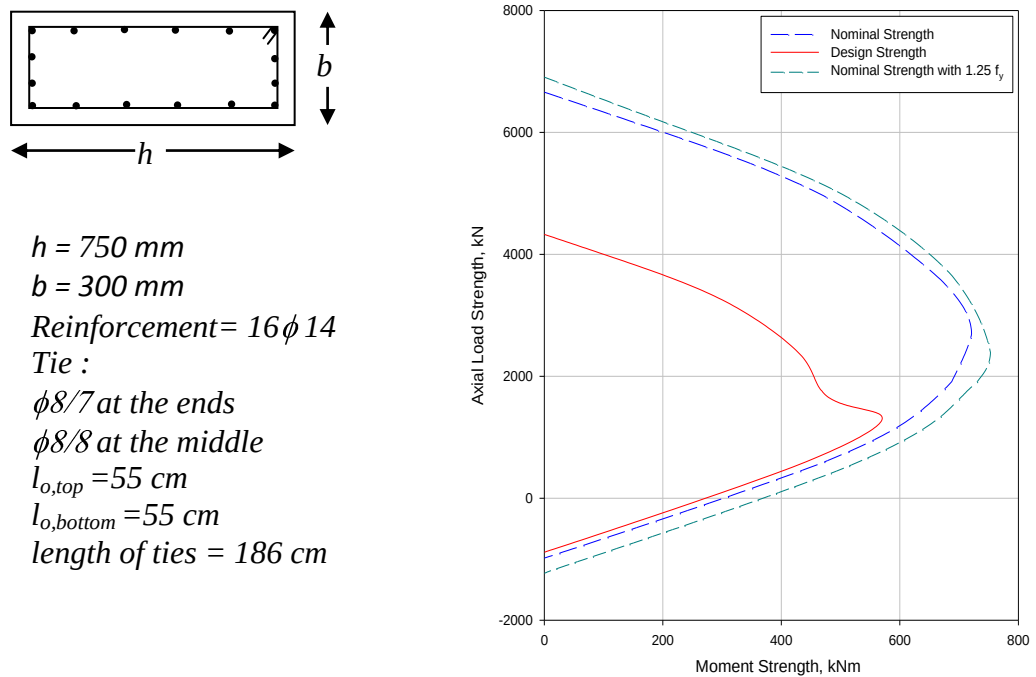


Figure 5.4 The detailing reinforcement of beam group-2 in the optimum two-bay seven-storey RC plane frame.



(a) Column design group 1



(b) Column design group 2

Figure 5.5 The section details and the interaction diagrams for column design groups in two-bay seven-storey RC plane frame, (a) for column design group 1 and (b) for column design group 2.

Table 5.3. The material quantities and costs in the optimum two-bay seven story RC frame.

Components	Quantity	Cost (TL)
Concrete	17.702 m³	1770.20
<i>Beam</i>	6.047 m ³	604.70
<i>Column</i>	11.655 m ³	1165.50
Steel	3.924,266 kg	3924.26
<i>Beam</i>	1480.972 kg	1538.35
<i>Column</i>	2443.294 kg	4130.48
Formwork	173,159 m²	4328.97
<i>Beam</i>	61.009 m ²	1525.22
<i>Column</i>	112.150 m ²	2803.75
Total Cost (\$) :		10023.42

(1\$ =1.51 TL , \$; American Dollar, TL; Turkish Liras)

The optimum two-bay, seven-storey RC frame has the minimum cost of 10023.42 TL which is obtained at 83660th iteration of the optimum design algorithm presented in this study. The same RC frame is also designed several times using different harmony search algorithm parameters. The optimum costs of the RC frames obtained with each set of parameters are listed in Table 5.4. It is apparent from the table that the values of the harmony search algorithm parameters selected have an effect on the optimum cost values. The optimum design is sensitive to values of parameters. Hence, it should be pointed out that the design algorithm is required to be run several times with different set of values of these parameters, before the optimum result decided. The average time of each run takes about 25-30 minutes of computer time for 100000 iterations.

Table 5.4. The obtained optimum results for different harmony search parameters for two-bay seven-story RC frame example.

	HMCR	PAR	Optimum Cost (TL)
50	0.75	0.15	10094.71
35	0.65	0.20	10391.83
70	0.85	0.30	10461.03
45	0.90	0.20	
80	0.75	0.20	10517.15
55	0.80	0.15	10667.64
30	0.85	0.20	10202.98

5.3 Two-Bay Five-Story RC Frame

Symmetrical two-bay five-story RC frame shown in Figure 5.6 is optimized with harmony search algorithm based optimum design method as the second example. This frame has 10 beams and 15 columns, which are collected in four design groups: two groups for beams and two groups for columns. The compressive strength of concrete and yield strength of steel are 25 MPa and 400 MPa, respectively. The frame is loaded symmetrically and the unfactored uniformly distributed dead load of 35 kN/m and live load of 20 kN/m are applied on all beams. The six different load combinations given in App.B (Eqn. B.1- B.4) are considered in the optimum design. The unit costs of concrete, steel and formwork are taken as 100 TL/m³, 1 TL/kg and 25 TL/m², respectively. The unit weight of concrete and steel is taken as 25kN/m³ and 78.5kN/m³, respectively. The seismic lateral forces are calculated according to ASCE 7-05. The input data of the frame are given in Table 5.5. For this example, the frame is considered to be symmetrical and there are 20 design variables in the design problem. In this example, the frame is analyzed for three cases considering different building importance factors, *I* in each case (*I*=1,1.25 and 1.50 respectively). The Harmony

search method parameters HMS, HMCR, PAR are selected as 50, 0.85, and 0.20 respectively in the first case, 40, 0.85, and 0.15 in the second case, and 35, 0.75, and 0.15 in the third case. The minimum costs of optimum designs obtained for these three different cases are found to be as 6838.94 TL, 7070.07 TL, and 7565.18 TL, respectively. The optimum values of design variables obtained in three cases are given in Table 5.6-Table 5.8. For three different optimization problems, the harmony search algorithm design histories and the detailing design of beam groups and the interaction diagrams of column groups are shown in Figures 5.7-5.18.

Table 5.5 Input data for two-bay, five-story RC frame.

<i>Input datas for two-bay five-storey frame</i>	
Compressive strength of concrete, f_c (N/mm ²)	25
Yielding stress of steel, f_y (N/mm ²)	400
Cover for beams, mm	40
Cover for columns, mm	50
Total number of beam groups	2
Total number of beams	10
Total number of column groups	2
Total number of columns	15
<i>Seismic Parameters</i>	
Site Class	C
S_s	0.50
S_1	0.20
F_a	1.20
F_v	1.60
R	8
I	1.0
T_L	6.0
I (variously)	1, 1.25, 1.50
S_{DS}	0.533
S_{D1}	0.2133
<i>Harmony Search Algorithm Parameters</i>	
Case 1 – HMS =50, HMCR=0.85, PAR =0.20	
Case 2 – HMS =40, HMCR=0.85, PAR =0.15	
Case 3 – HMS =35, HMCR=0.75, PAR =0.15	

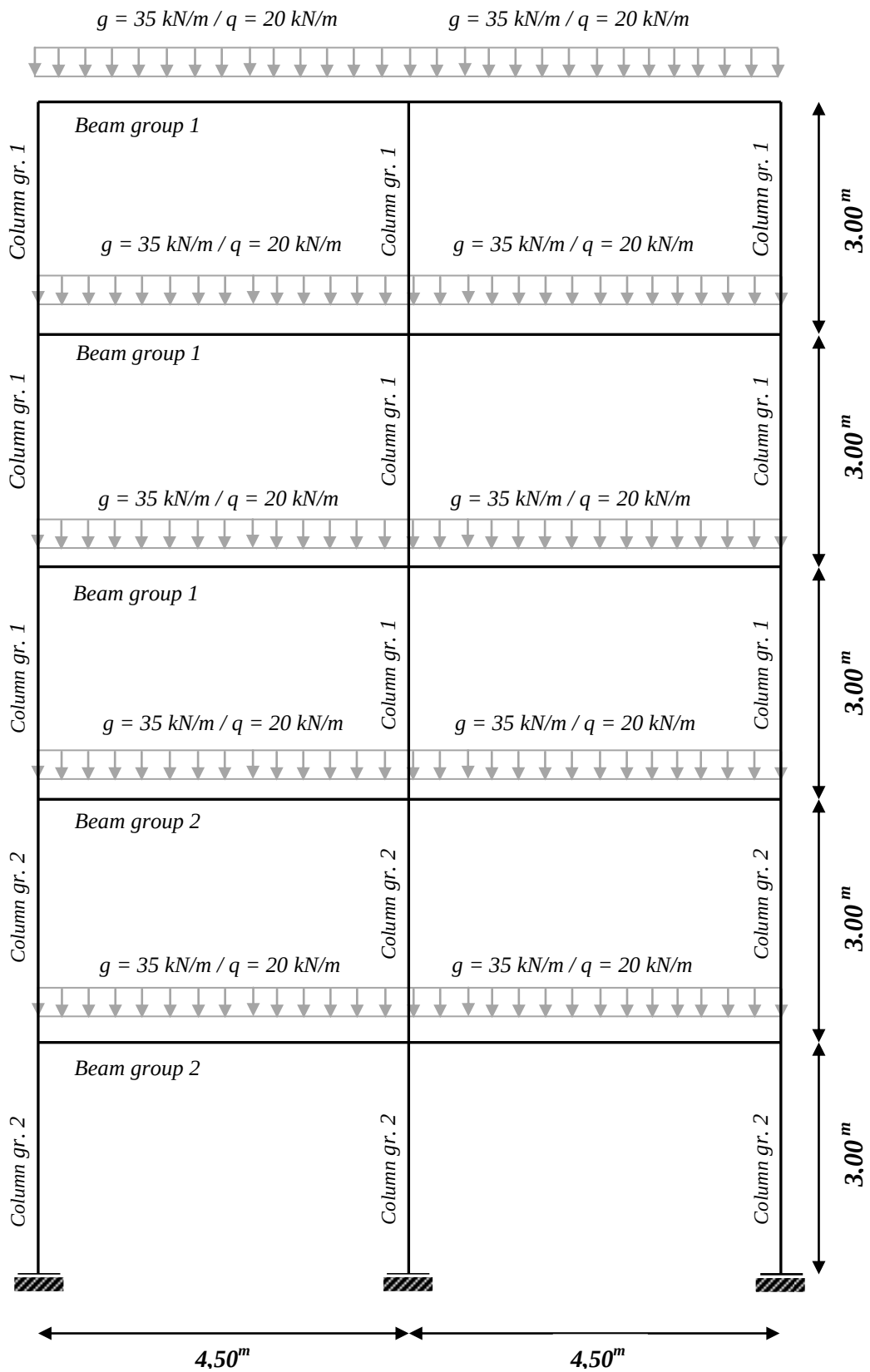


Figure 5.6 Two-bay five-storey RC plane frame.

Table 5.6 The optimum values of design variables for two-bay five-story RC frame (for $I=1.00$).

<i>Optimum design results for two-bay five-storey frame</i>		
<i>Optimum column design results</i>		
<i>Design Variables</i>	<i>Column Gr. 1</i>	<i>Column Gr. 2</i>
Section number in design pool	(28)	(28)
The height of section, h (mm)	550	550
The height of section, b (mm)	300	300
The diameter of reinforcement bars, ϕ (mm)	18	18
The number of bars in x direction, n_1	2	2
The number of bars in y direction, n_2	1	1
Total reinforcement	10 ϕ 18	10 ϕ 18
<i>Optimum beam design results</i>		
<i>Design Variables</i>	<i>Beam Gr. 1</i>	<i>Beam Gr. 2</i>
x_1	25/45 (3)	25/45 (3)
x_2	3 ϕ 12 (3)	2 ϕ 14 (2)
x_3	2 ϕ 16 (4)	3 ϕ 12 (3)
x_4	1 ϕ 16 (4)	1 ϕ 16 (4)
x_5	1 ϕ 12 (2)	1 ϕ 14 (3)
x_6	1 ϕ 16 (4)	1 ϕ 16 (4)
x_7	1 ϕ 12 (2)	1 ϕ 14 (3)
x_8	6 ϕ 12 (22)	5 ϕ 14 (26)
x_9	1 ϕ 12 (2)	1 ϕ 12 (2)
x_{10}	3 ϕ 14 (9)	2 ϕ 18 (15)
x_{11}	NNR* (1)	NNR* (1)
x_{12}	6 ϕ 12 (22)	5 ϕ 14 (26)
x_{13}	1 ϕ 12 (2)	1 ϕ 12 (2)

NNR*, no need reinforcement, (n_i), the sequence number of i^{th} design variable in the design variable pool.

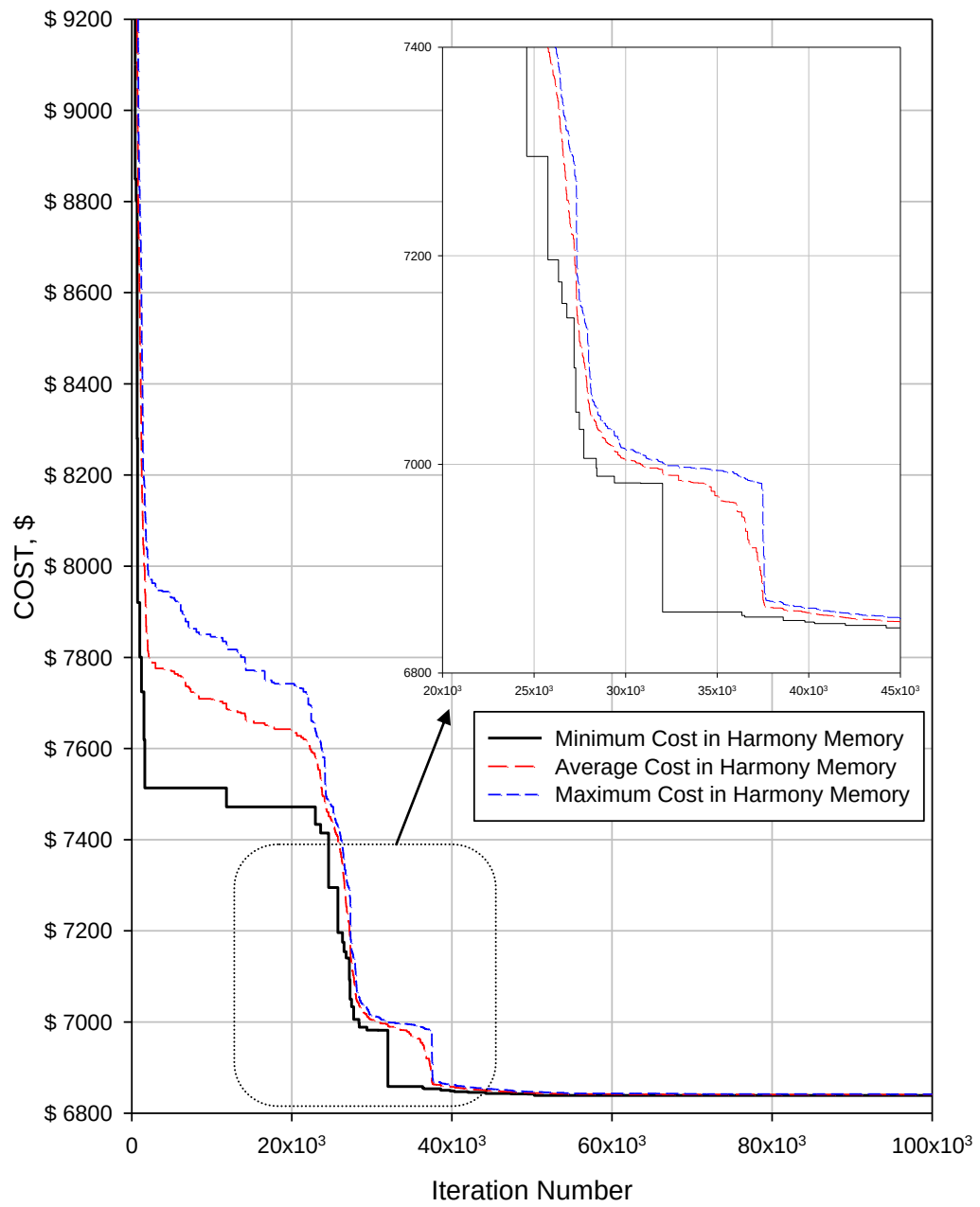


Figure 5.7 Design history for two-bay five-storey RC plane frame (for $I=1.00$).

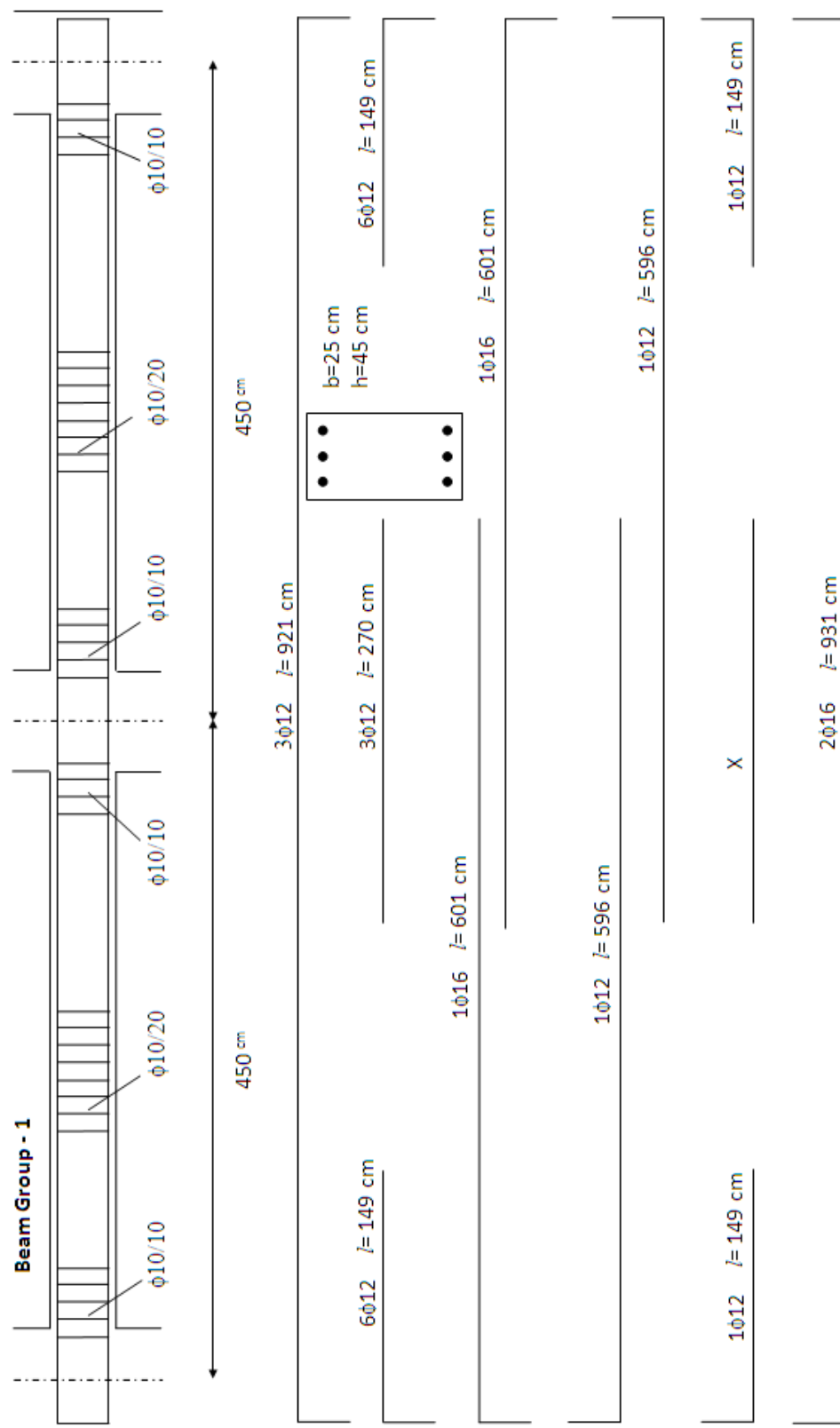


Figure 5.8 The detailing reinforcement of beam group-1 in the optimum design of two-bay five-storey RC plane frame (for I=1.00).

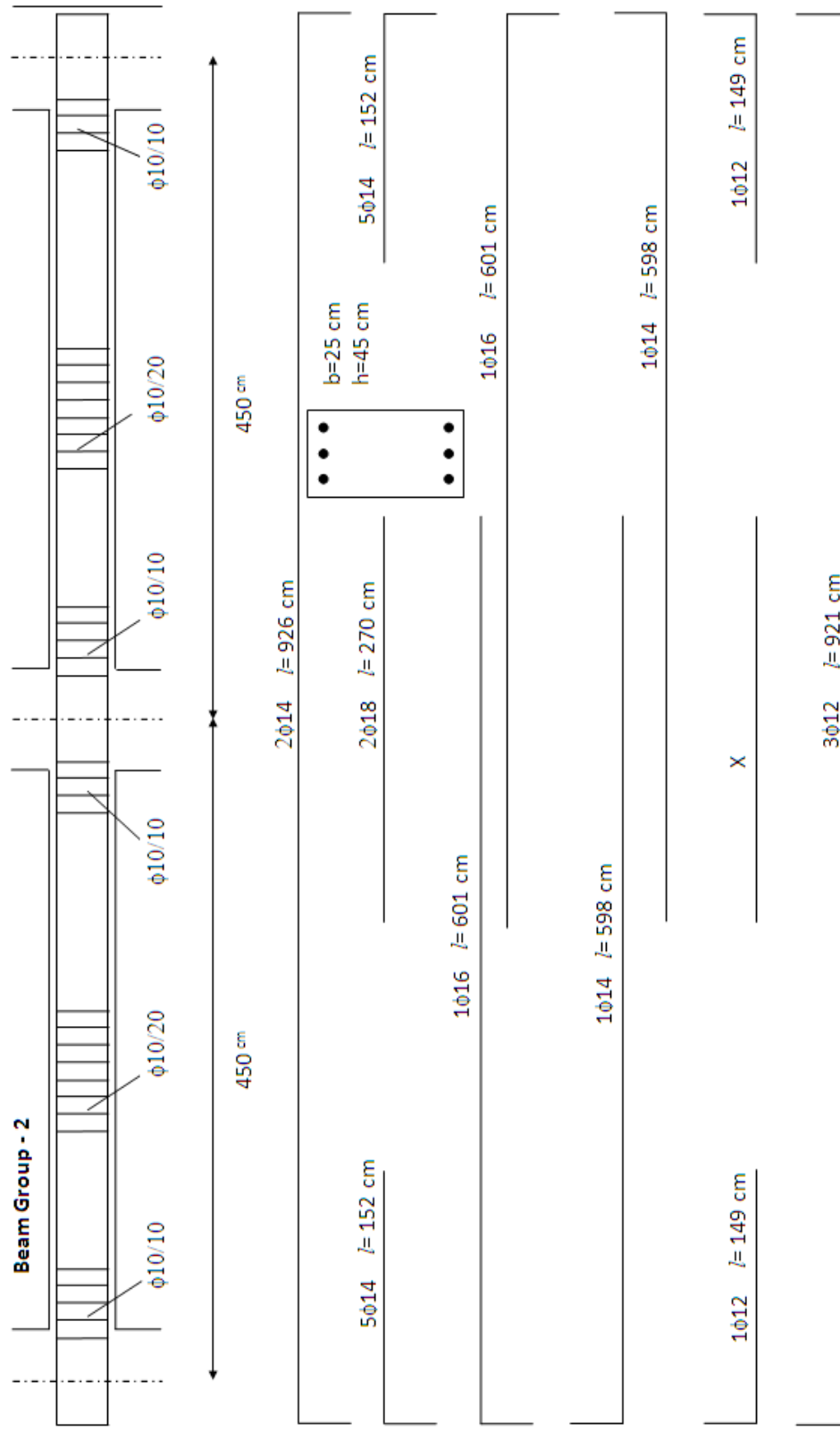
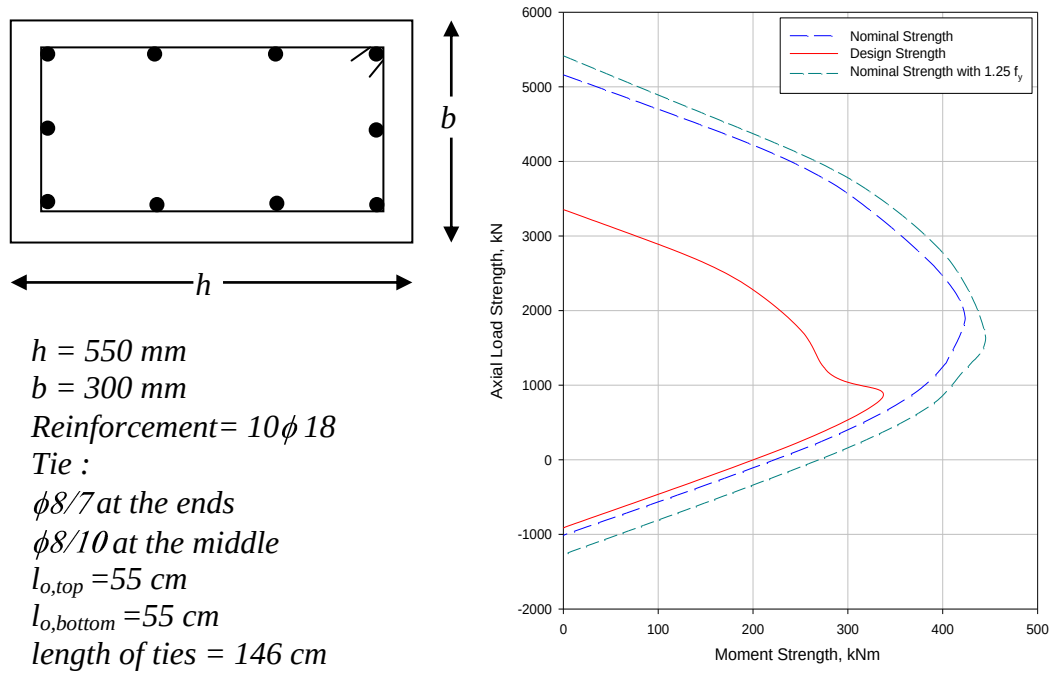
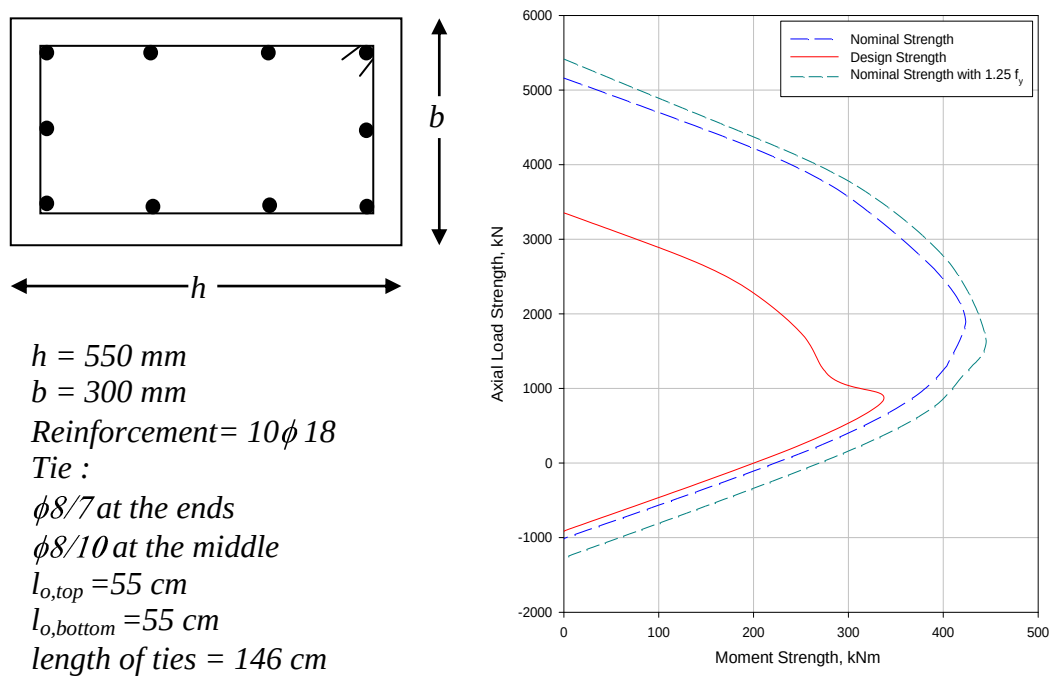


Figure 5.9 The detailing reinforcement of beam group-2 in the optimum design of two-bay five-storey RC plane frame (for I=1.00).



(a) Column design group 1



(b) Column design group 2

Figure 5.10 The section detailing and interaction diagrams for column design groups in two-bay five-storey RC plane frame (for $I=1.00$), (a) for column group 1 and (b) for column group 2.

Table 5.7 The optimum values of design variables for two-bay five-story RC frame (for $I=1.25$).

<i>Optimum design results for two-bay five-storey frame</i>		
<i>Optimum column design results</i>		
<i>Design Variables</i>	<i>Column Gr. 1</i>	<i>Column Gr. 2</i>
Section number in design pool	(17)	(19)
The height of section, h (mm)	500	500
The height of section, b (mm)	300	350
The diameter of reinforcement bars, ϕ (mm)	22	16
The number of bars in x direction, n_1	2	2
The number of bars in y direction, n_2	1	2
Total reinforcement	10 ϕ 22	12 ϕ 16
<i>Optimum beam design results</i>		
<i>Design Variables</i>	<i>Beam Gr. 1</i>	<i>Beam Gr. 2</i>
x_1	25/45 (3)	25/45 (3)
x_2	2 ϕ 14 (2)	2 ϕ 12 (1)
x_3	2 ϕ 16 (4)	3 ϕ 12 (3)
x_4	2 ϕ 12 (5)	1 ϕ 16 (4)
x_5	1 ϕ 12 (2)	1 ϕ 14 (3)
x_6	2 ϕ 12 (5)	1 ϕ 16 (4)
x_7	1 ϕ 12 (2)	1 ϕ 14 (3)
x_8	4 ϕ 16 (27)	5 ϕ 16 (31)
x_9	1 ϕ 12 (2)	1 ϕ 16 (4)
x_{10}	1 ϕ 24 (13)	1 ϕ 30 (23)
x_{11}	NNR* (1)	NNR* (1)
x_{12}	4 ϕ 16 (27)	5 ϕ 16 (31)
x_{13}	1 ϕ 12 (2)	1 ϕ 16 (4)

NNR*, no need reinforcement, (n_i), the sequence number of i^{th} design variable in the design variable pool.

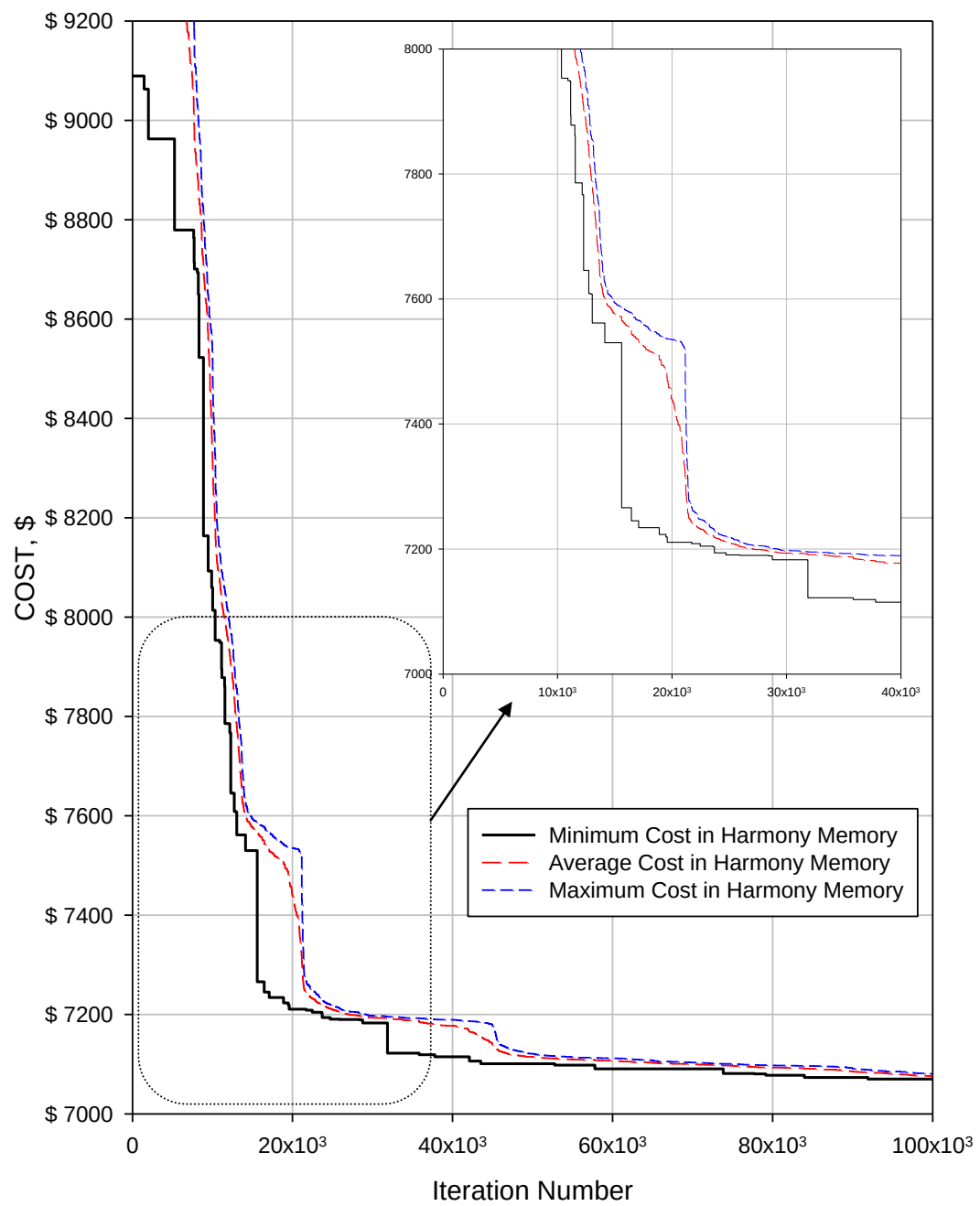


Figure 5.11 Design history for two-bay five-storey RC plane frame (for $I=1.25$).

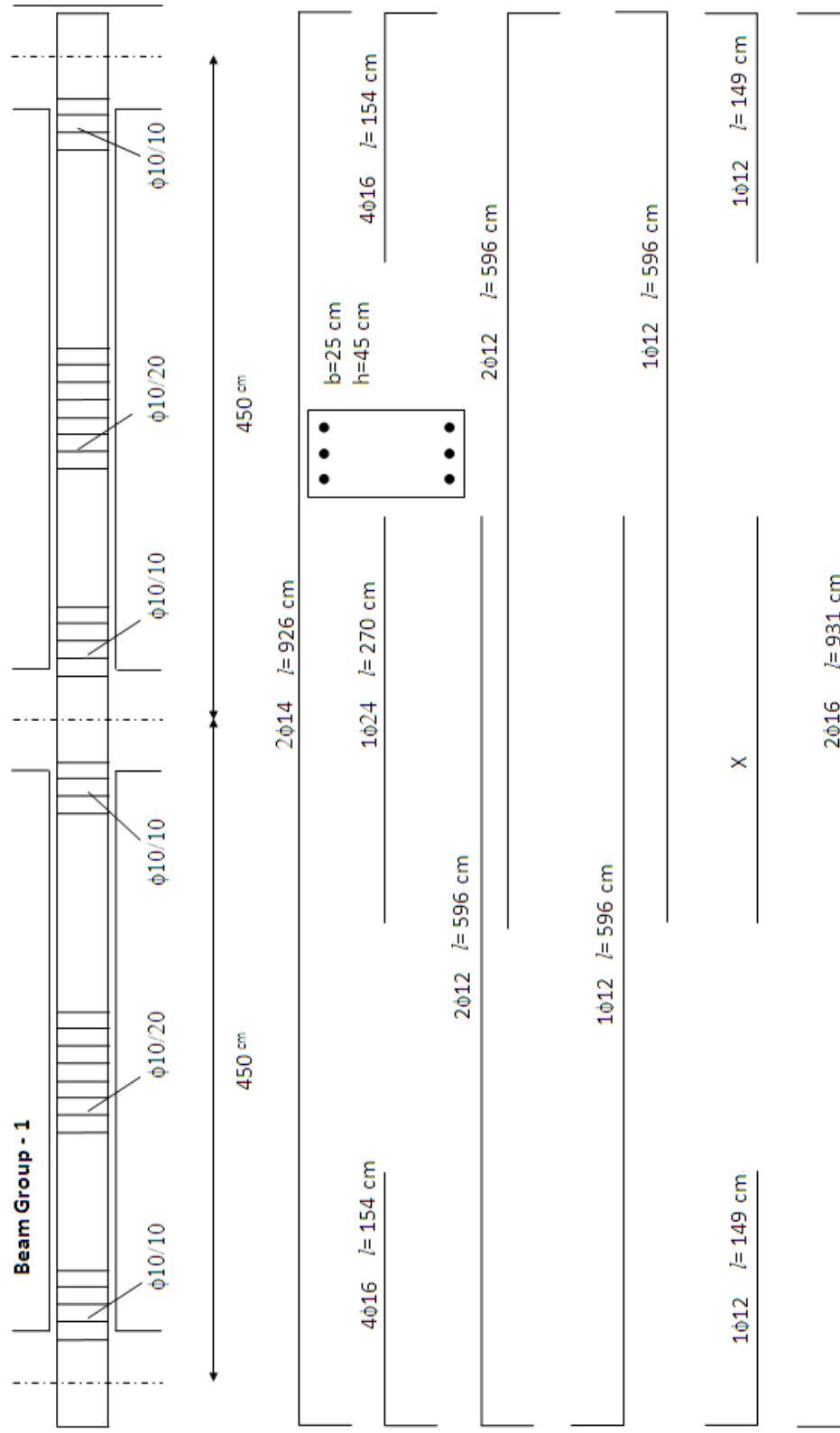
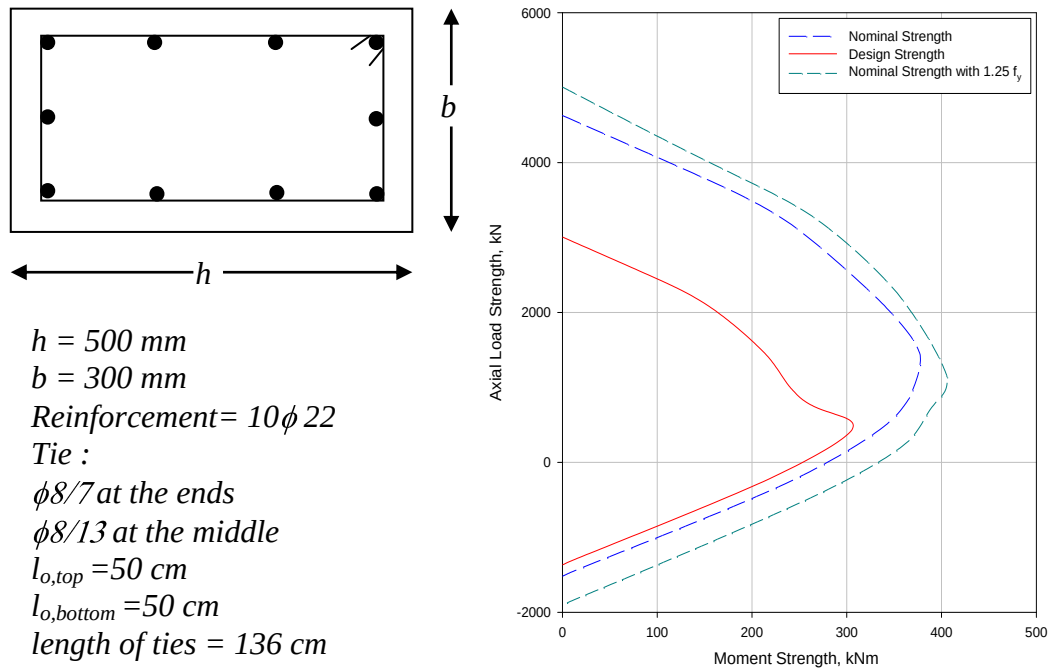
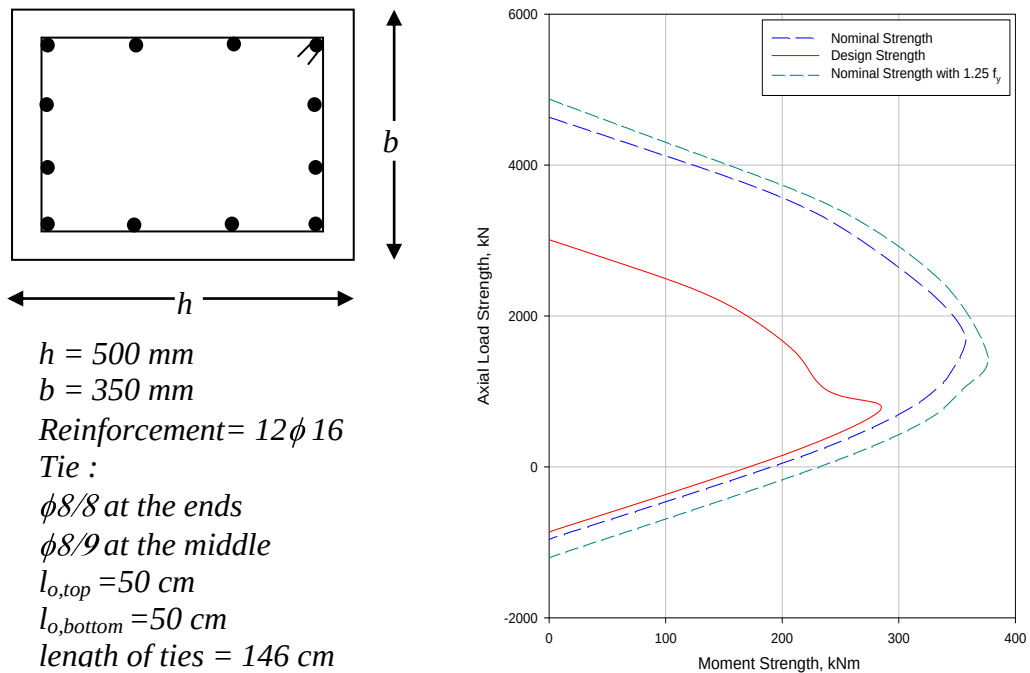


Figure 5.12 The detailing reinforcement of beam group-1 in the optimum design of two-bay five-storey RC plane frame (for I=1.25).



(a) Column design groups 1



(b) Column design groups 2

Figure 5.14 The section detailing and interaction diagrams for column design groups designed in two-bay five-storey RC plane frame (for $I=1.25$), (a) for column design group 1 and (b) for column design group 2.

Table 5.8 The optimum values of design variables for two-bay five-story RC frame (for $I=1.50$).

<i>Optimum design results for two-bay five-storey frame</i>		
<i>Optimum column design results</i>		
<i>Design Variables</i>	<i>Column Gr. 1</i>	<i>Column Gr. 2</i>
Section number in design pool	(47)	(82)
The height of section, h (mm)	600	700
The height of section, b (mm)	300	300
The diameter of reinforcement bars, ϕ (mm)	18	14
The number of bars in x direction, n_1	3	5
The number of bars in y direction, n_2	2	1
Total reinforcement	14 ϕ 18	16 ϕ 14
<i>Optimum beam design results</i>		
<i>Design Variables</i>	<i>Beam Gr. 1</i>	<i>Beam Gr. 2</i>
x_1	25/50 (5)	25/50 (5)
x_2	2 ϕ 14 (2)	3 ϕ 12 (3)
x_3	3 ϕ 12 (3)	3 ϕ 12 (3)
x_4	1 ϕ 16 (4)	1 ϕ 16 (4)
x_5	1 ϕ 12 (2)	1 ϕ 12 (2)
x_6	1 ϕ 16 (4)	1 ϕ 16 (4)
x_7	1 ϕ 12 (2)	1 ϕ 12 (2)
x_8	4 ϕ 16 (27)	6 ϕ 14 (29)
x_9	1 ϕ 16 (4)	2 ϕ 12 (5)
x_{10}	1 ϕ 24 (13)	2 ϕ 20 (20)
x_{11}	NNR* (1)	1 ϕ 12 (2)
x_{12}	4 ϕ 16 (27)	6 ϕ 14 (29)
x_{13}	1 ϕ 16 (4)	2 ϕ 12 (5)

NNR*, no need reinforcement, (n_i), the sequence number of i^{th} design variable in the design variable pool.

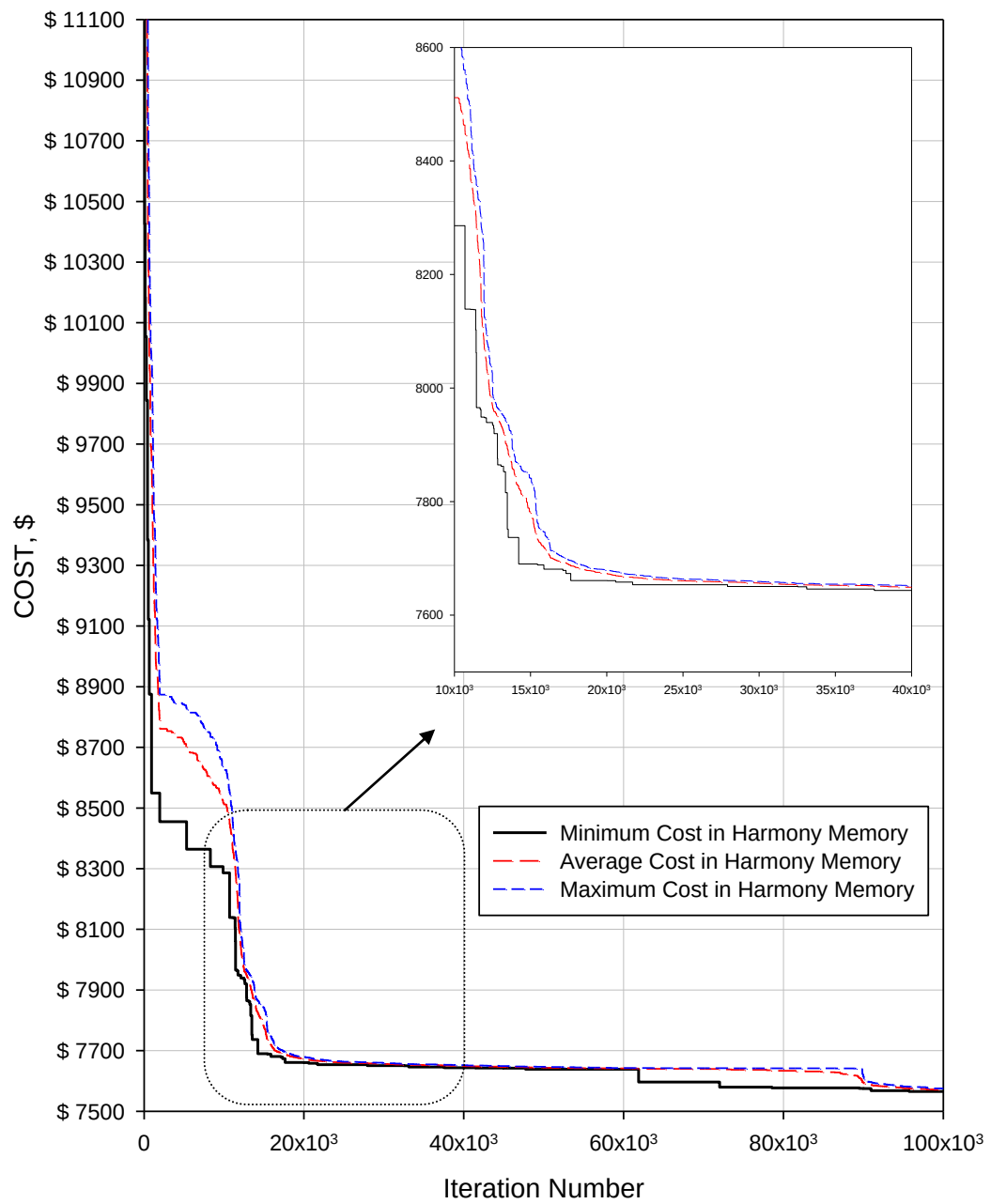


Figure 5.15 Design history for two-bay five-storey RC plane frame (*for $I=1.50$*).

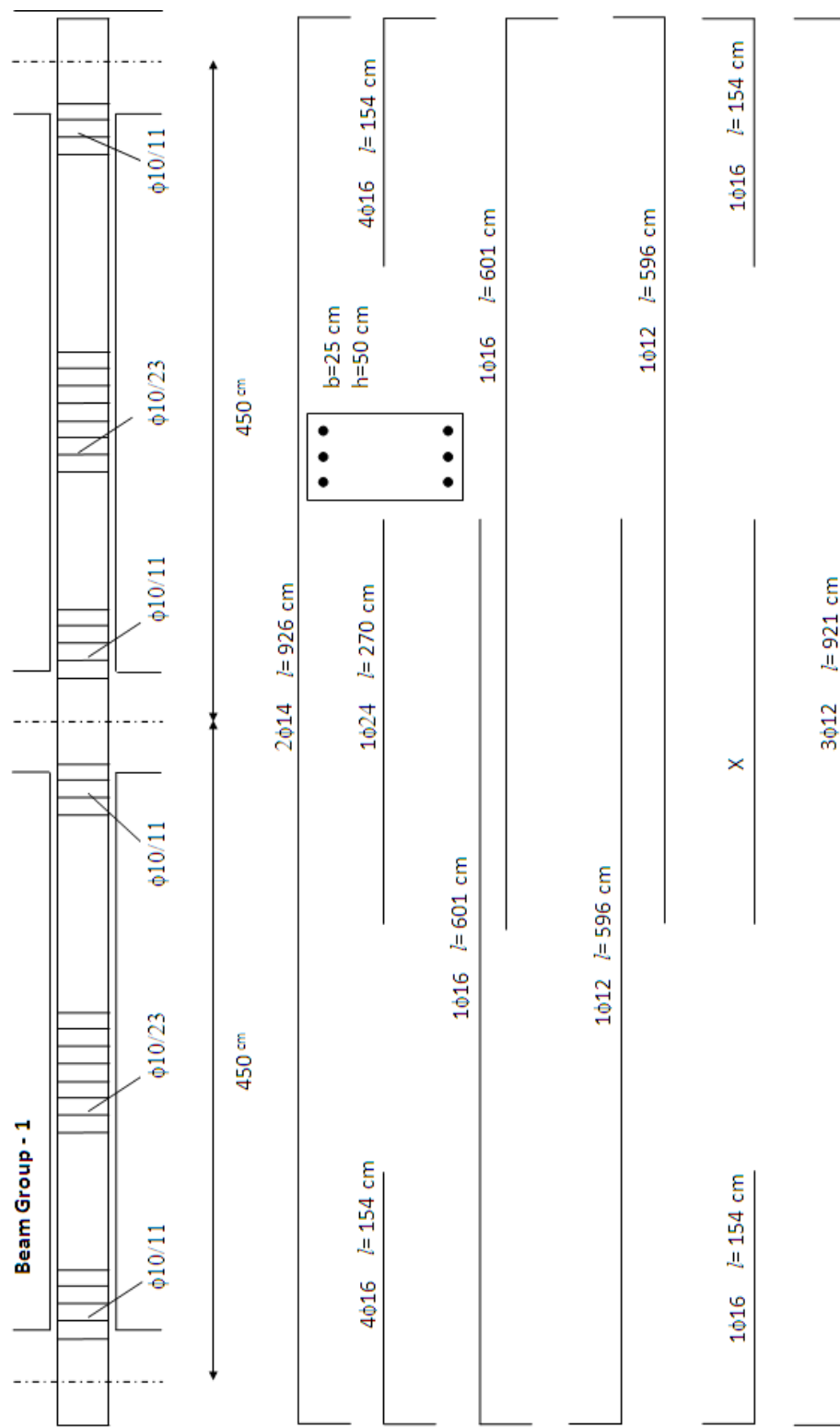


Figure 5.16 The detailing reinforcement of beam group-1 in the optimum design of two-bay five-storey RC plane frame (for I=1.50).

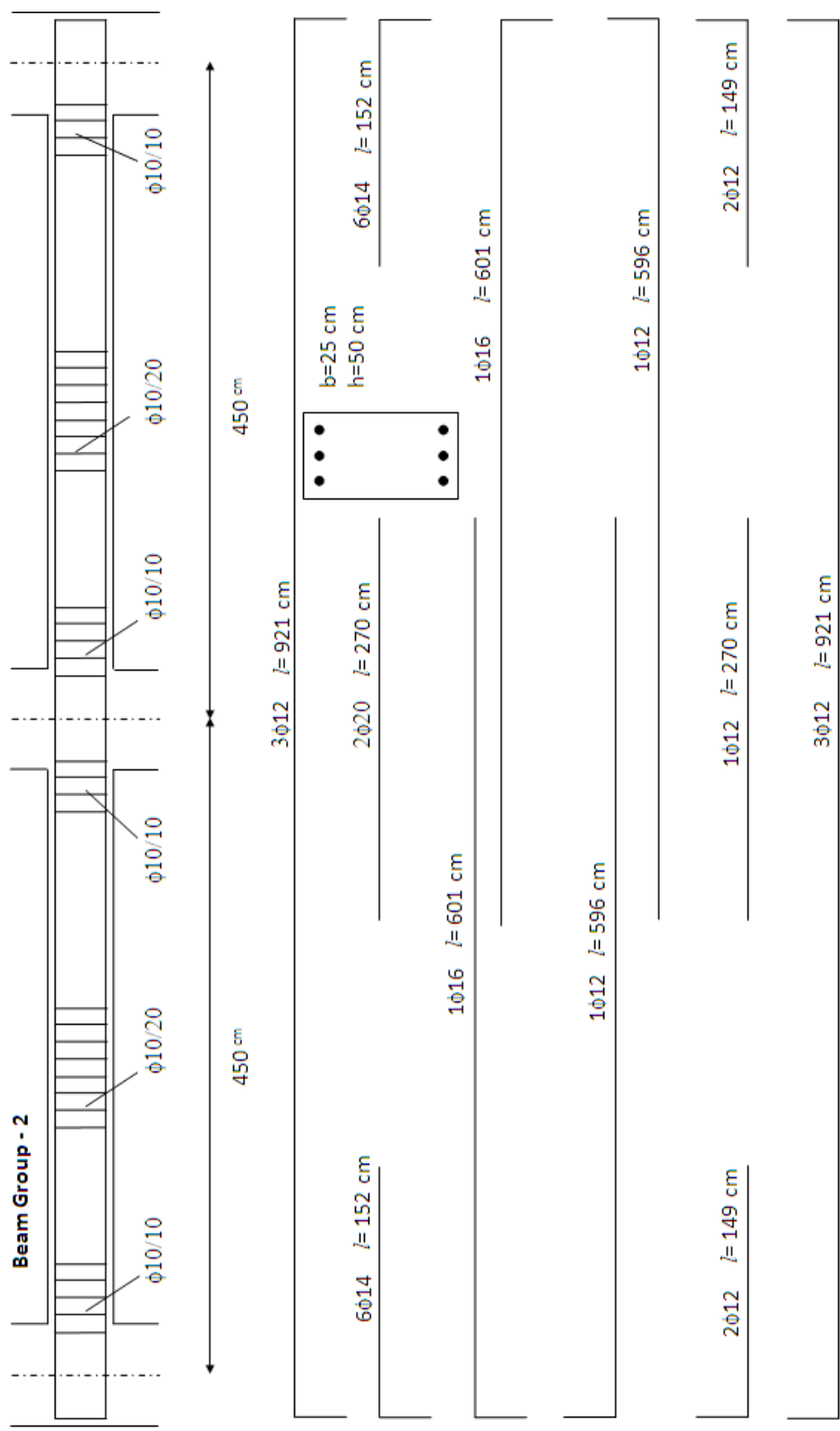
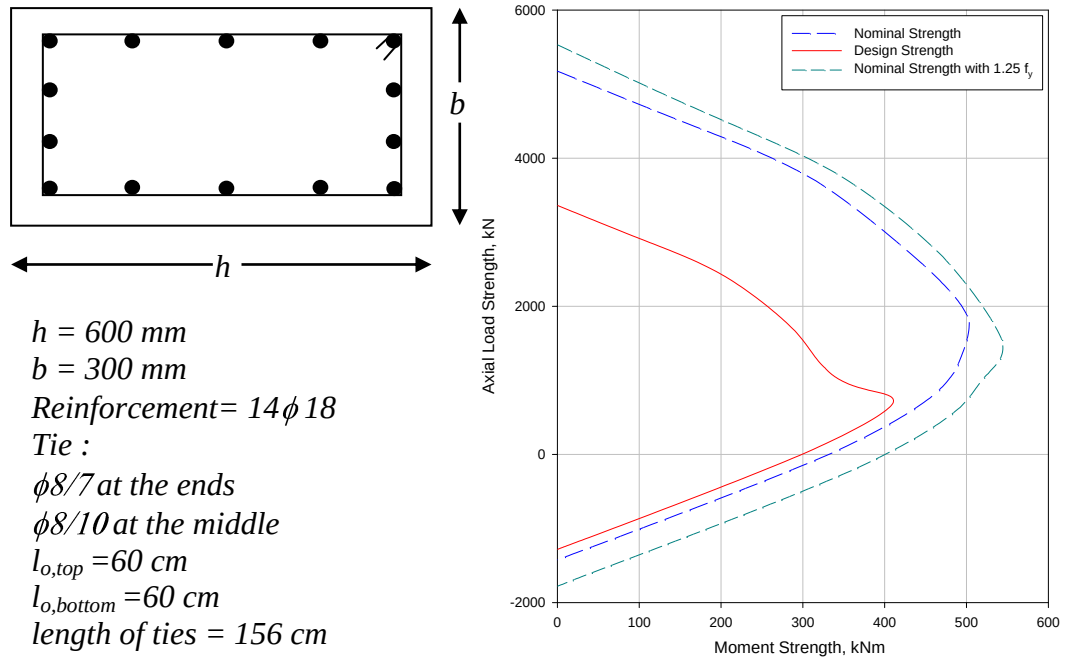
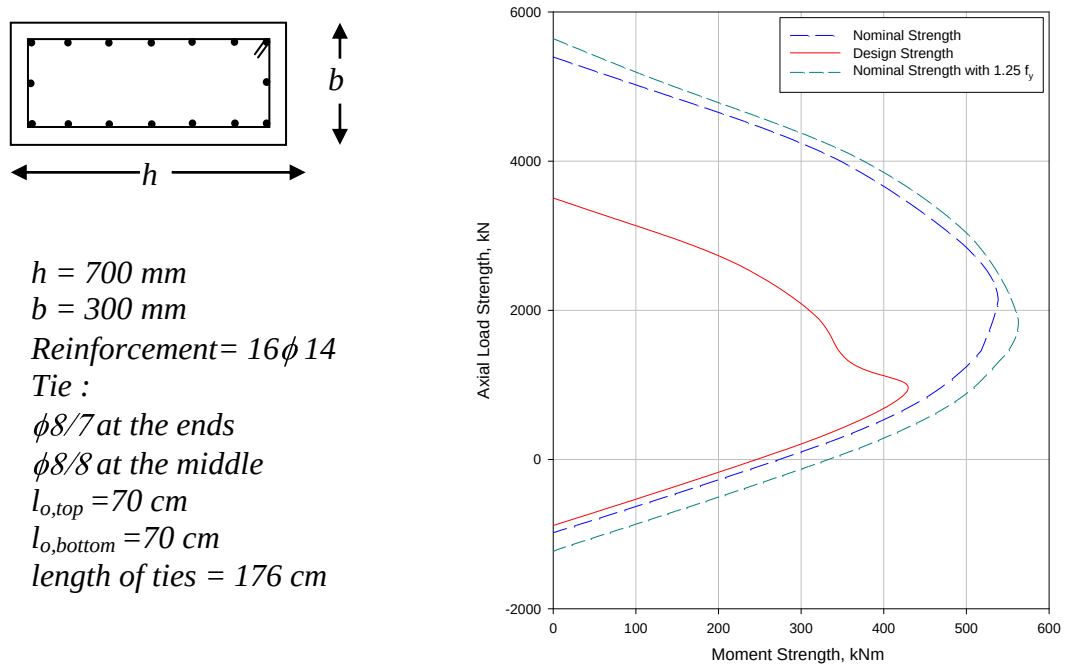


Figure 5.17 The detailing reinforcement of beam group-2 in the optimum design of two-bay five-storey RC plane frame (for $I=1.50$).



(a) Column design group 1



(b) Column design group 2

Figure 5.18 The section detailing and interaction diagrams for column design groups designed in two-bay five-storey RC plane frame (for $I=1.50$), (a) for column design group 1 and (b) for column design group 2.

The cost of the frame increases as expected with the increased building importance coefficient. The optimum sections that determine the optimum quantity of the concrete, steel, and formwork are selected based on the design forces acting the concrete members and based on the unit cost of the concrete, steel, and formwork. The unit quantities of concrete, formwork and steel obtained for three cases are given in Table 5.9.

Table 5.9 The details of material quantities and costs in the optimum two-bay five-story RC frame.

Components	Case 1	Case 2	Case 3
Concrete (m³)			8
<i>Beam</i>			
<i>Column</i>			
Steel (kg)			3014.43
<i>Beam</i>			
<i>Column</i>			
Formwork (m²)			
<i>Beam</i>			
<i>Column</i>			
Total Cost (\$)		4	

(1\$ =1.51 TL , \$; American Dollar, TL; Turkish Liras)

5.4 Two-Bay Six-Story RC Frame

A two-bay six-story RC frame given in Figure 5.19 is designed by Rajeev and Krishnamoorthy [25], Camp and Pezeshk [26], Govindaraj and Ramasamy [7]. This frame has 12 beams and 18 columns, which are collected in five design groups: two groups for beams and three groups for columns. The compressive strength of concrete and yield strength of steel are 20 MPa and 415 MPa, respectively. The frame is loaded with the unfactored uniformly distributed load of 30 kN/m and the lateral force of 10 kN is applied at each story level. The load combinations are not considered in this practical optimum design problem. The unit costs of concrete, steel and formwork are 735 Rs./m³, 7.1 Rs./kg and 54 Rs./m², respectively as given in [25]. The unit weight of concrete and steel is taken as 25kN/m³ and 78.5kN/m³, respectively. The input data of the frame is given in Table 5.10. In this design example, there are 29 design variables, 26 of which are beams (13 for each beam design group), the remaining 3 is for columns. The Harmony search method parameters HMS, HMCR, PAR are selected as 45, 0.80, and 0.15, respectively. The cost of optimum design obtained as 43586.19 Rs. The optimum values for design variables obtained are given in Table 5.11. The harmony search algorithm design history and the detailing design of beam design groups and the interaction diagrams for column groups are given in Figures 5.20-5.23. For this optimization problem, the minimum costs of optimum designs are obtained as 26052 Rs., 24959 Rs., and 22966 Rs. by Rajeev [25], Camp [26], Govindaraj, respectively (1Rs =0.032 TL , Rs; Indian Rupees, TL; Turkish Liras). In these studies, shear design of concrete members are not considered. The frame is not considered as Special Seismic Moment Frames and only simple flexural constraints are used to obtain the optimum design. The development lengths and hook lengths of reinforcement steel bars in the concrete members and the other detailing issues are not considered in the calculation of the frame cost. Therefore, the differences between obtained the optimum costs is based on different of design philosophies which exist between this study and the ones

available literature.

Table 5.10 Input data for two-bay six-story RC frame example.

<i>Input datas for two-bay six-storey frame</i>	
Compressive strength of concrete, f_c (N/mm ²)	20
Yielding stress of steel, f_y (N/mm ²)	415
Cover for beams, mm	25
Cover for columns, mm	25
Total number of beam groups	2
Total number of beams	12
Total number of column groups	3
Total number of columns	18
Lateral force for each story (kN)	10
Factored uniformly distributed load on beams (kN/m)	30
<i>Harmony Search Algorithm Parameters</i>	
HMS	45
HMCR	0.80
PAR	0.15
Max. Iteration	100,000

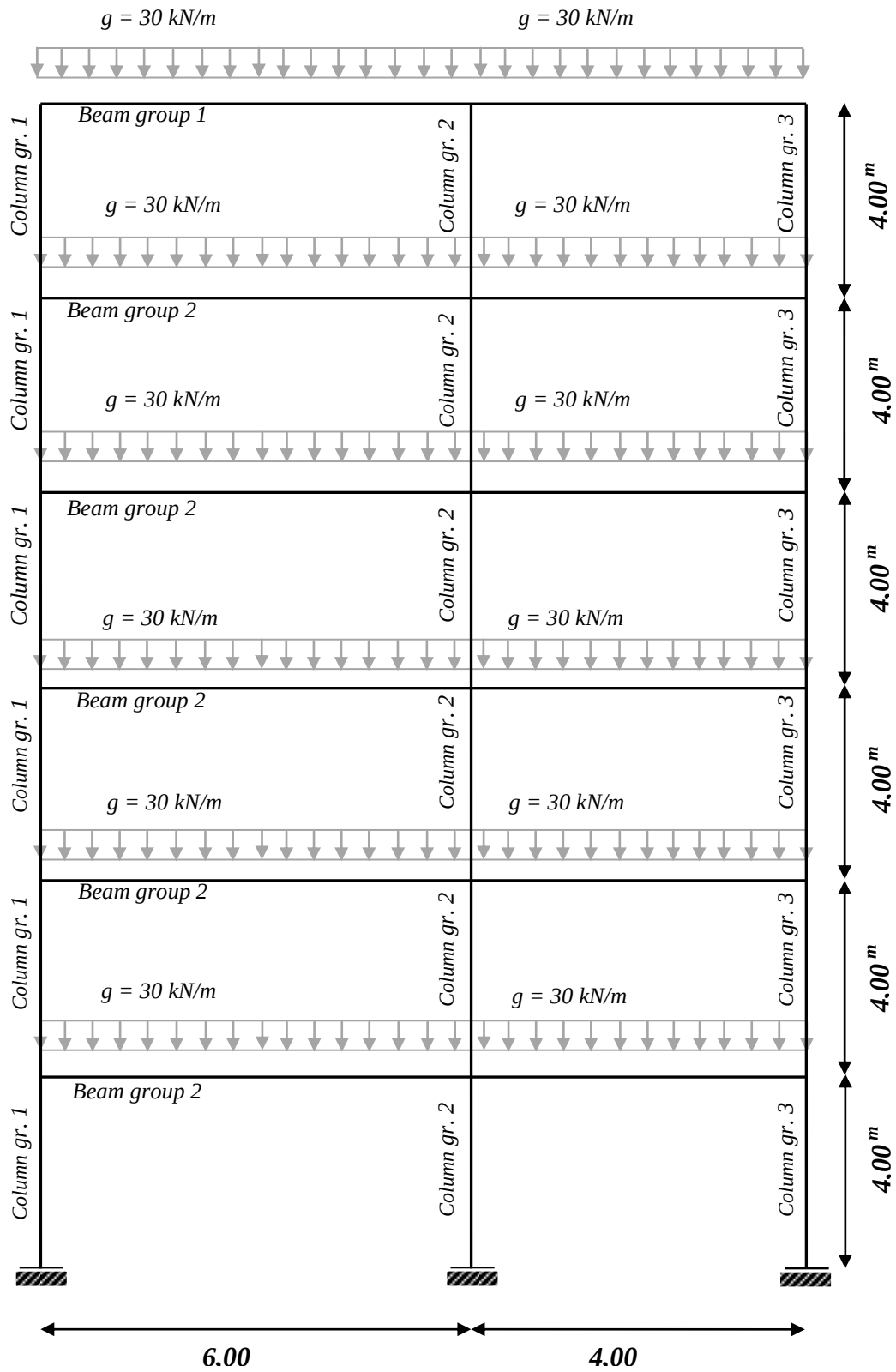


Figure 5.19 Two-bay six-storey RC plane frame.

Table 5.11 The optimum values of design variables for two-bay six-story RC frame.

<i>Optimum design results for two-bay six-storey frame</i>			
<i>Optimum column design results</i>			
<i>Design Variables</i>	<i>Column Gr. 1</i>	<i>Column Gr. 2</i>	<i>Column Gr. 3</i>
Section number in design pool	(3)	(82)	(3)
The height of section, h (mm)	400	500	400
The height of section, b (mm)	350	300	350
The diameter of reinforcement bars, ϕ (mm)	14	14	14
The number of bars in x direction, n_1	2	3	2
The number of bars in y direction, n_2	1	2	1
Total reinforcement	10 ϕ 14	14 ϕ 14	10 ϕ 14
<i>Optimum beam design results</i>			
<i>Design Variables</i>	<i>Beam Gr. 1</i>	<i>Beam Gr. 2</i>	
x_1	25/40 (1)	25/40 (1)	
x_2	2 ϕ 12 (1)	2 ϕ 12 (1)	
x_3	2 ϕ 12 (1)	2 ϕ 12 (1)	
x_4	1 ϕ 16 (4)	1 ϕ 16 (4)	
x_5	1 ϕ 16 (4)	1 ϕ 14 (3)	
x_6	1 ϕ 12 (2)	1 ϕ 12 (2)	
x_7	1 ϕ 12 (2)	1 ϕ 12 (2)	
x_8	1 ϕ 14 (3)	2 ϕ 14 (7)	
x_9	NNR* (1)	NNR* (1)	
x_{10}	1 ϕ 14 (3)	1 ϕ 16 (4)	
x_{11}	NNR* (1)	NNR* (1)	
x_{12}	NNR* (1)	NNR* (1)	
x_{13}	NNR* (1)	NNR* (1)	

NNR*, no need reinforcement, (n_i), the sequence number of i^{th} design variable in the design variable pool.

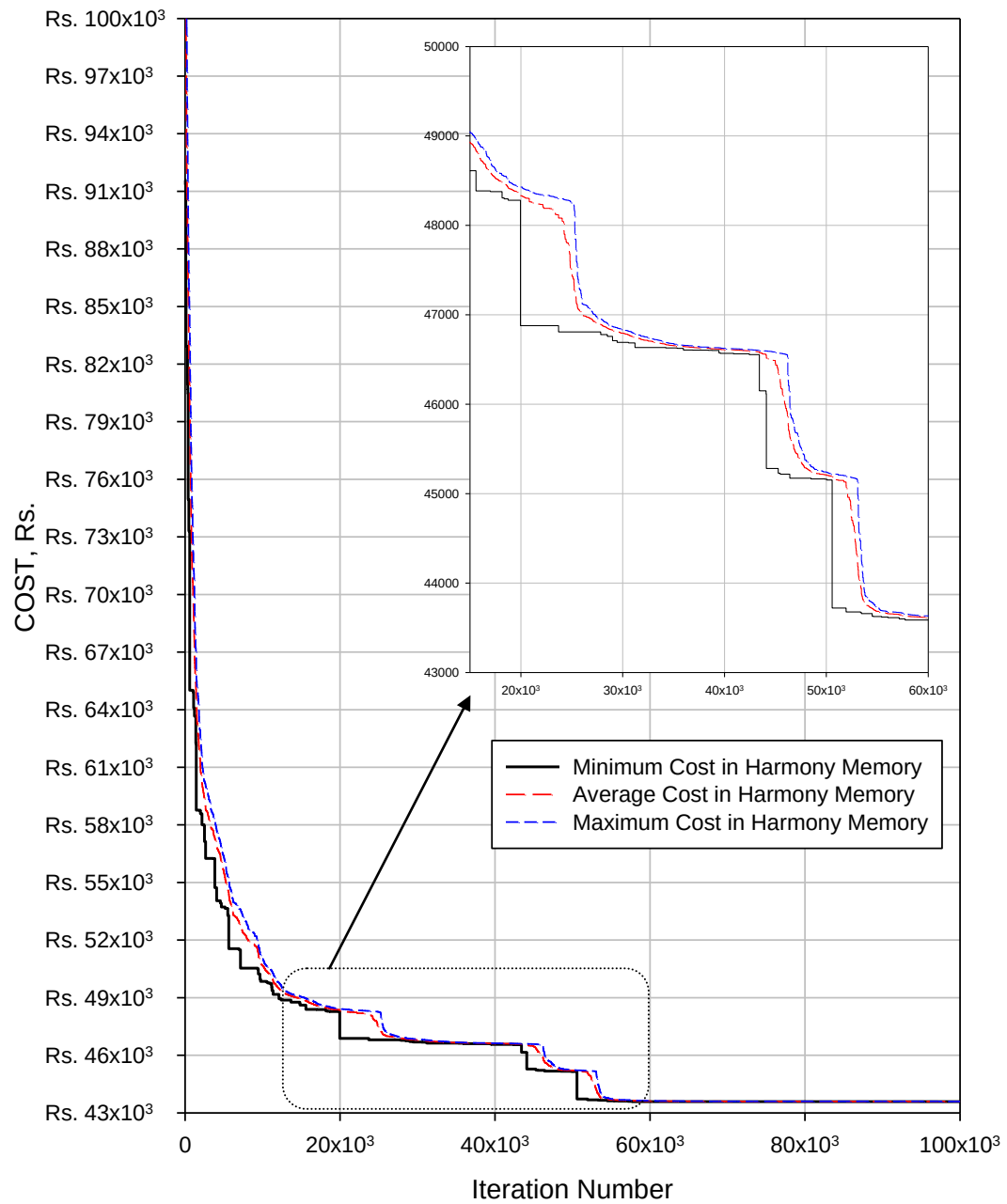


Figure 5.20 The design history for two-bay six-storey RC plane frame (load combinations are not considered) .

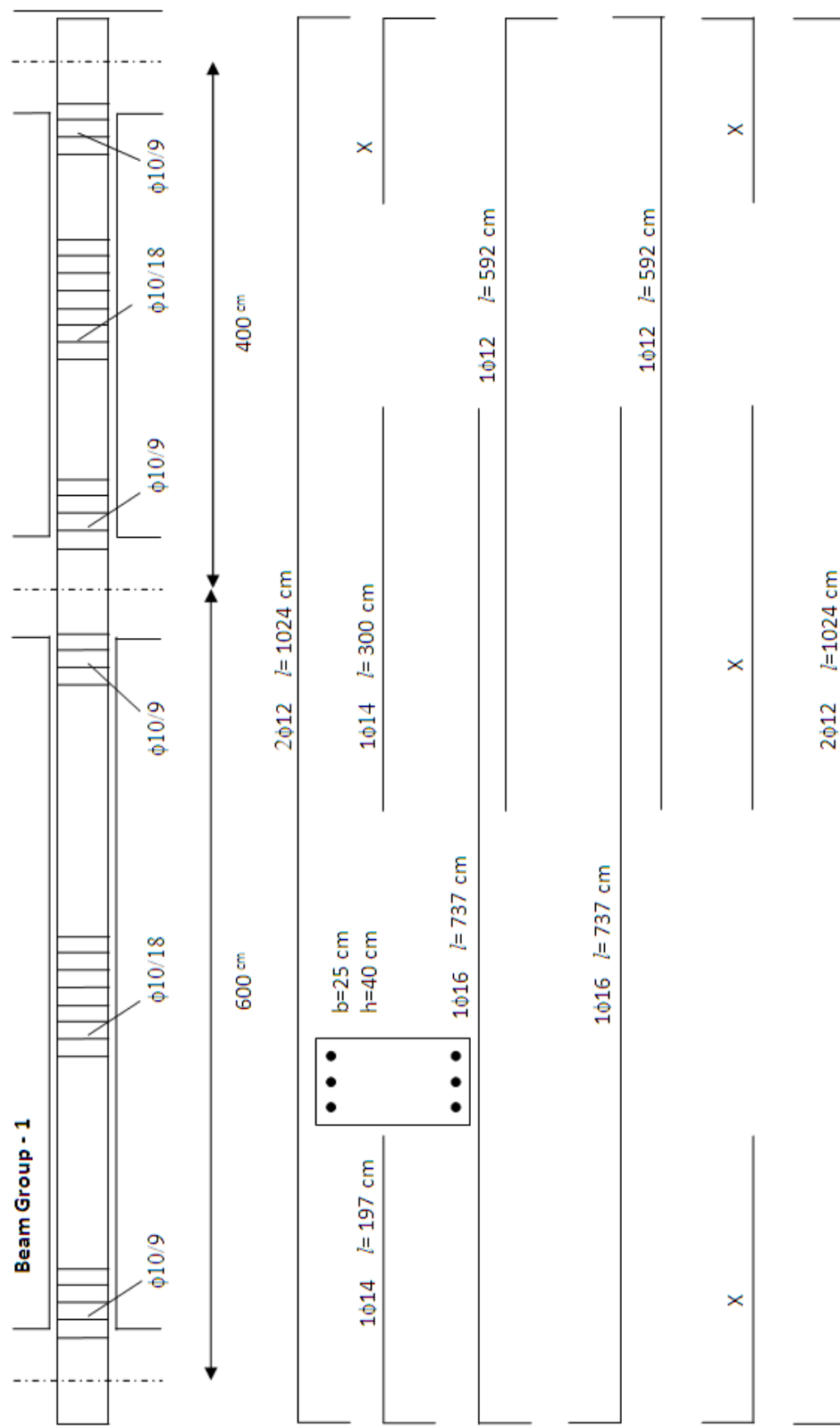


Figure 5.21 The detailing reinforcement of beam group-1 in the optimum design of two-bay six-storey RC plane frame.

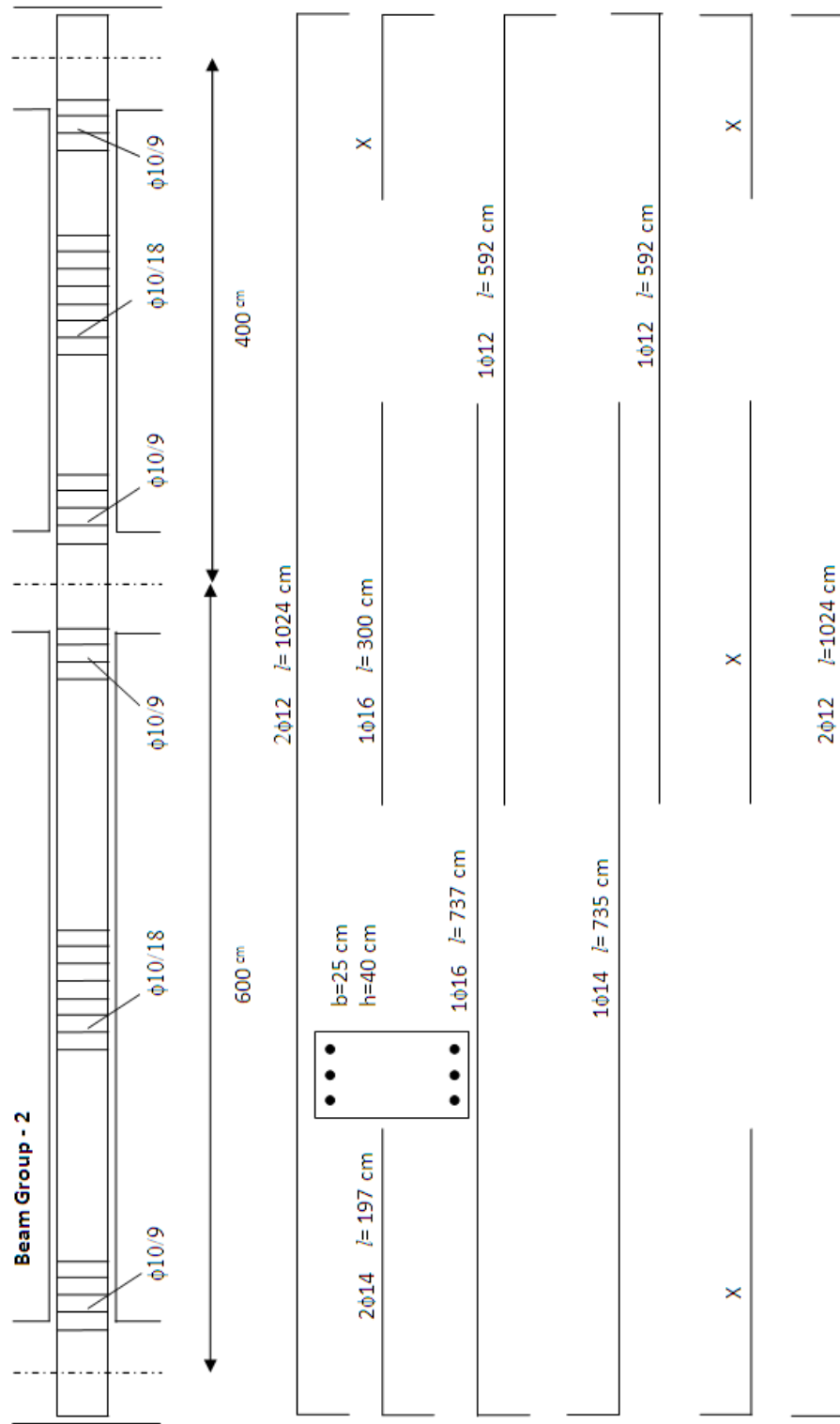
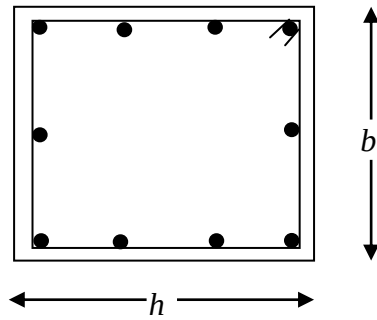
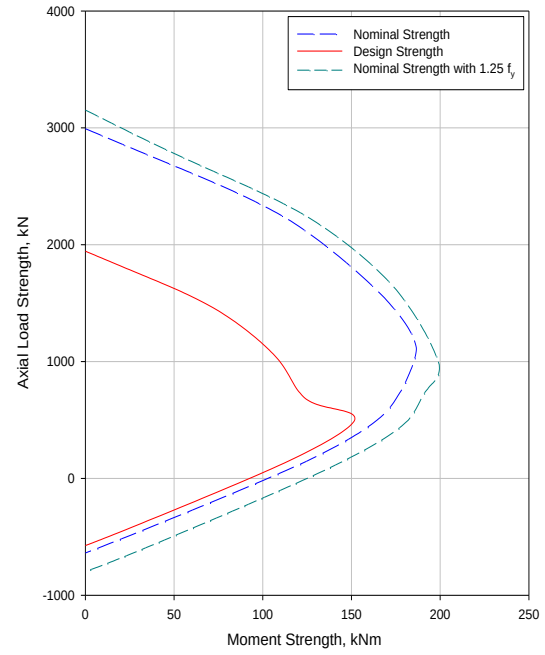


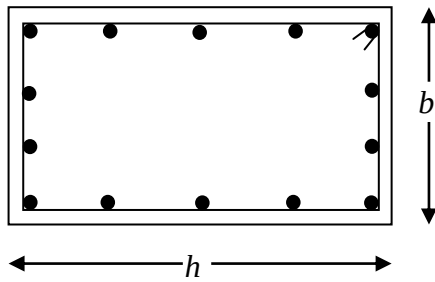
Figure 5.22 The detailing reinforcement of beam group-2 in the optimum design of two-bay six-storey RC plane frame.



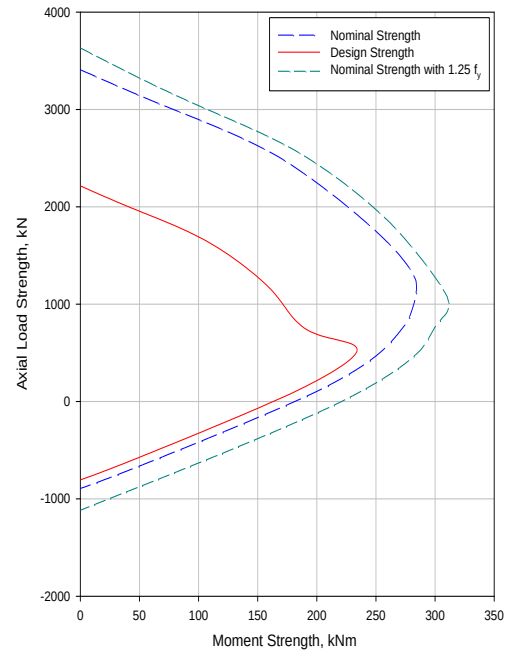
$h = 400 \text{ mm}$
 $b = 350 \text{ mm}$
 Reinforcement = $10\phi 14$
 Tie :
 $\phi 8/8$ at the ends
 $\phi 8/8$ at the middle
 $l_{o,top} = 60 \text{ cm}$
 $l_{o,bottom} = 60 \text{ cm}$
 length of ties = 146 cm



(a) Column design group 1



$h = 500 \text{ mm}$
 $b = 300 \text{ mm}$
 Reinforcement = $14\phi 14$
 Tie :
 $\phi 8/7$ at the ends
 $\phi 8/8$ at the middle
 $l_{o,top} = 60 \text{ cm}$
 $l_{o,bottom} = 60 \text{ cm}$
 length of ties = 156 cm



(b) Column design group 2

Figure 5.23 The section detailing and interaction diagrams for column design groups in two-bay six-storey RC plane frame, (a) for column design group 1 and 3, (b) for column design group 2.

5.5 Five-Bay Five-Story RC Frame

The geometry and property group numbers of the five-bay five-story RC frame with 55 members is shown in Figure 5.24 and the loading details is given Figure 5.25. A frame has 25 beams and 30 columns, which are collected in five design groups: two groups for beams and three groups for columns. The compressive strength of concrete and yield strength of steel are 30 MPa and 400 MPa, respectively. The frame is loaded symmetrically and the loading details is given in Figure 5.26. The load combinations that are given in App. B (ACI 318-05) are considered in the optimum design. The seismic lateral forces are calculated according to ASCE 7-05 for given conditions. The unit weight of concrete and steel is taken as 25kN/m^3 and 78.5kN/m^3 , respectively. The input data of the frame considered are given in Table 5.12. For this example, the frame is considered as symmetrical and there are 18 design variables, 15 of which are beams, the remaining 3 is for columns, in the optimum design problem. In this example, the example frame is analyzed for two cases of the unit costs of materials. The different unit costs are used to investigate the effect of unit material costs on the optimum designs. The unit costs of concrete, steel and formwork are taken $100\text{ \$/m}^3$, $1\text{ \$/kg}$ and $25\text{ \$/m}^2$, respectively in the first case, and $110\text{ \$/m}^3$, $2.1\text{ \$/kg}$ and $27\text{ \$/m}^2$, respectively in the second case. The Harmony search method parameters HMS, HMCR, PAR are selected as 40, 0.80, and 0.20, respectively for the first case, and 50, 0.85, and 0.15 for the second case. The minimum costs of optimum designs obtained for these two different cases are found to be \$ 17838.99, and \$ 27159.44. The quantities of concrete, formwork and steel obtained in these two cases are given in Table 5.15. The optimum values of the design variables in these two cases are listed in Tables 5.13 and 5.14. The design histories of the harmony search algorithm in these two cases are shown in Figures 5.27 and 5.28.

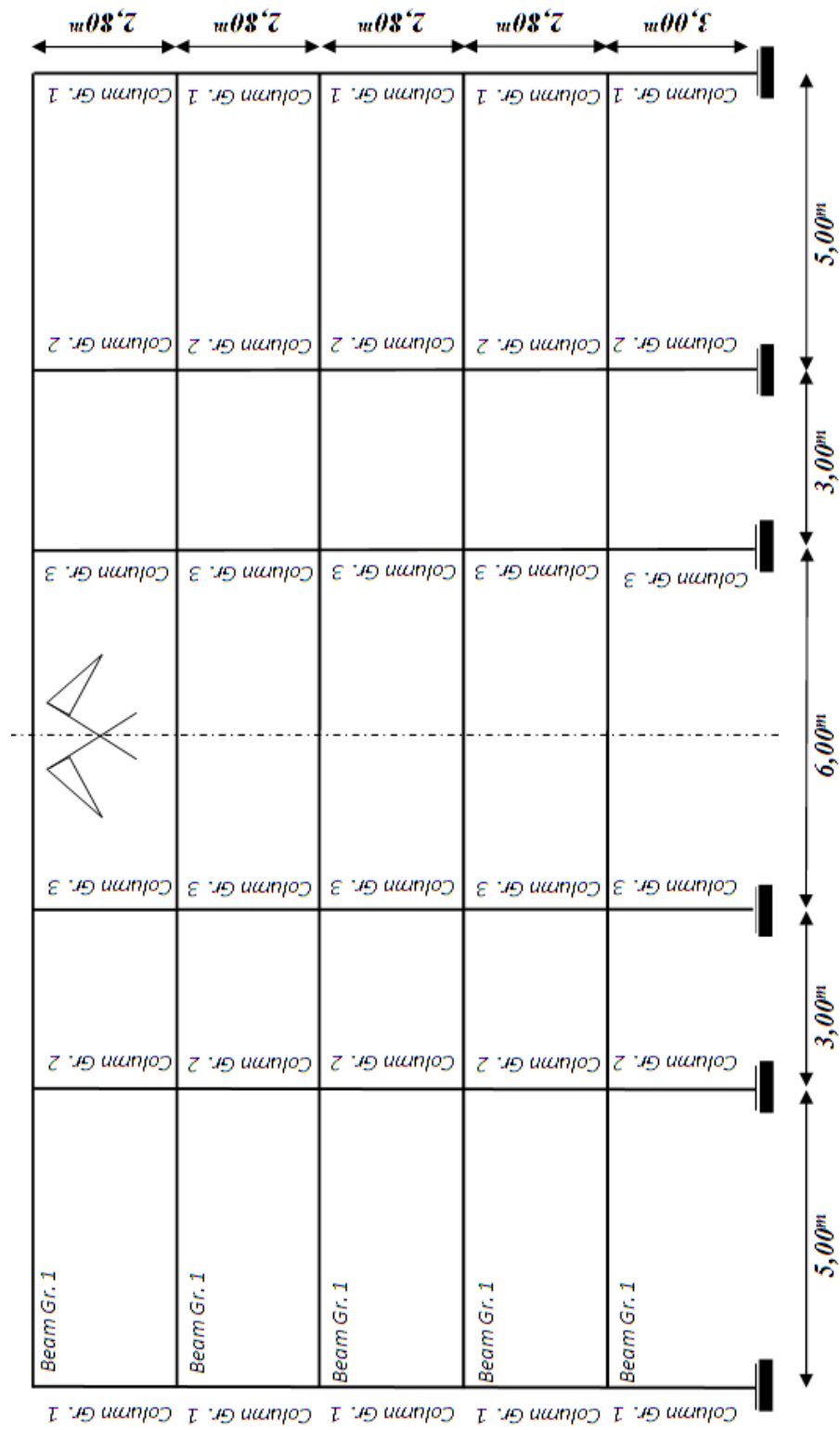
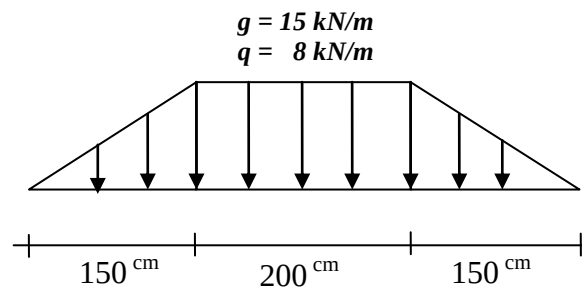
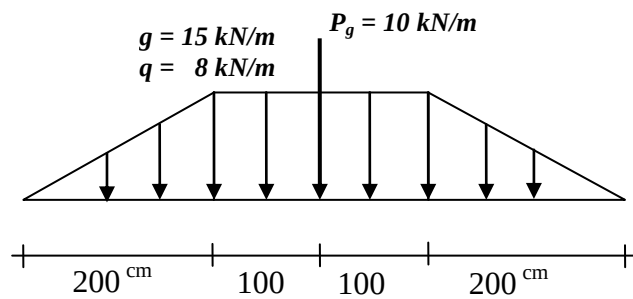


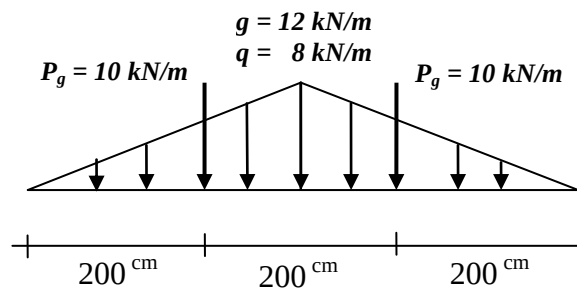
Figure 5.24 Five-bay five-storey RC plane frame.



(a) loading on the first and last span beams



(b) loading on the second and the fourth span beams



(c) loading on the third span beams

Figure 5.25 Gravity loading for five-bay five-storey frame (a) for beams at first and last spans, (b) for beams at second and fourth spans, (c) for beams at third span.

Table 5.12 Input data for five-bay five-story RC frame.

<i>Input datas for five-bay five-storey frame</i>	
Compressive strength of concrete, f_c (N/mm ²)	30
Yielding stress of steel, f_y (N/mm ²)	400
Cover for beams, mm	40
Cover for columns, mm	50
Total number of beam groups	1
Total number of beams	25
Total number of column groups	3
Total number of columns	30
<i>Seismic Parameters</i>	
Site Class	C
S_s	0.50
S_1	0.20
F_a	1.20
F_v	1.60
R	8
I	1.0
T_L	6.0
I	1
S_{DS}	0.533
S_{D1}	0.2133
<i>Harmony Search Algorithm Parameters</i>	
Case 1 – HMS =40, HMCR=0.80, PAR =0.20	
Case 2 – HMS =50, HMCR=0.85, PAR =0.15	
Max. Iteration for each case	100,000

Table 5.13 The optimum values of design variables of five-bay five-story RC frame example (for Case 1).

<i>Optimum design results for five-bay five-storey frame</i>			
<i>Optimum column design results</i>			
<i>Design Variables</i>	<i>Column Gr. 1</i>	<i>Column Gr. 2</i>	<i>Column Gr. 3</i>
Section number in design pool	(29)	(51)	(117)
The height of section, h (mm)	300	350	300
The height of section, b (mm)	550	600	750
The diameter of reinforcement bars, ϕ (mm)	18	16	20
The number of bars in x dir., n_1	3	6	5
The number of bars in y dir., n_2	1	3	1
Total reinforcement	12 ϕ 18	22 ϕ 16	16 ϕ 20
<i>Optimum beam design results</i>			
<i>Design Variables</i>	<i>Beam Gr. 1</i>		
x_1	25/60 (10)		
x_2	3 ϕ 12 (3)		
x_3	2 ϕ 16 (4)		
x_4	1 ϕ 14 (3)		
x_5	4 ϕ 12 (12)		
x_6	1 ϕ 14 (3)		
x_7	1 ϕ 12 (2)		
x_8	1 ϕ 14 (3)		
x_9	6 ϕ 14 (29)		
x_{10}	1 ϕ 14 (3)		
x_{11}	1 ϕ 12 (2)		
x_{12}	1 ϕ 14 (3)		
x_{13}	4 ϕ 12 (12)		
x_{14}	NNR* (1)		
x_{15}	1 ϕ 20 (8)		
x_{16}	1 ϕ 16 (4)		
x_{17}	NNR* (1)		
x_{18}	1 ϕ 16 (4)		
x_{19}	1 ϕ 22 (10)		
x_{20}	1 ϕ 16 (4)		
x_{21}	1 ϕ 22 (10)		
x_{22}	1 ϕ 16 (4)		
x_{23}	NNR* (1)		
x_{24}	NNR* (1)		
x_{25}	1 ϕ 20 (8)		

NNR*, no need reinforcement, (n_i), the sequence number of i^{th} design variable in the design variable pool.

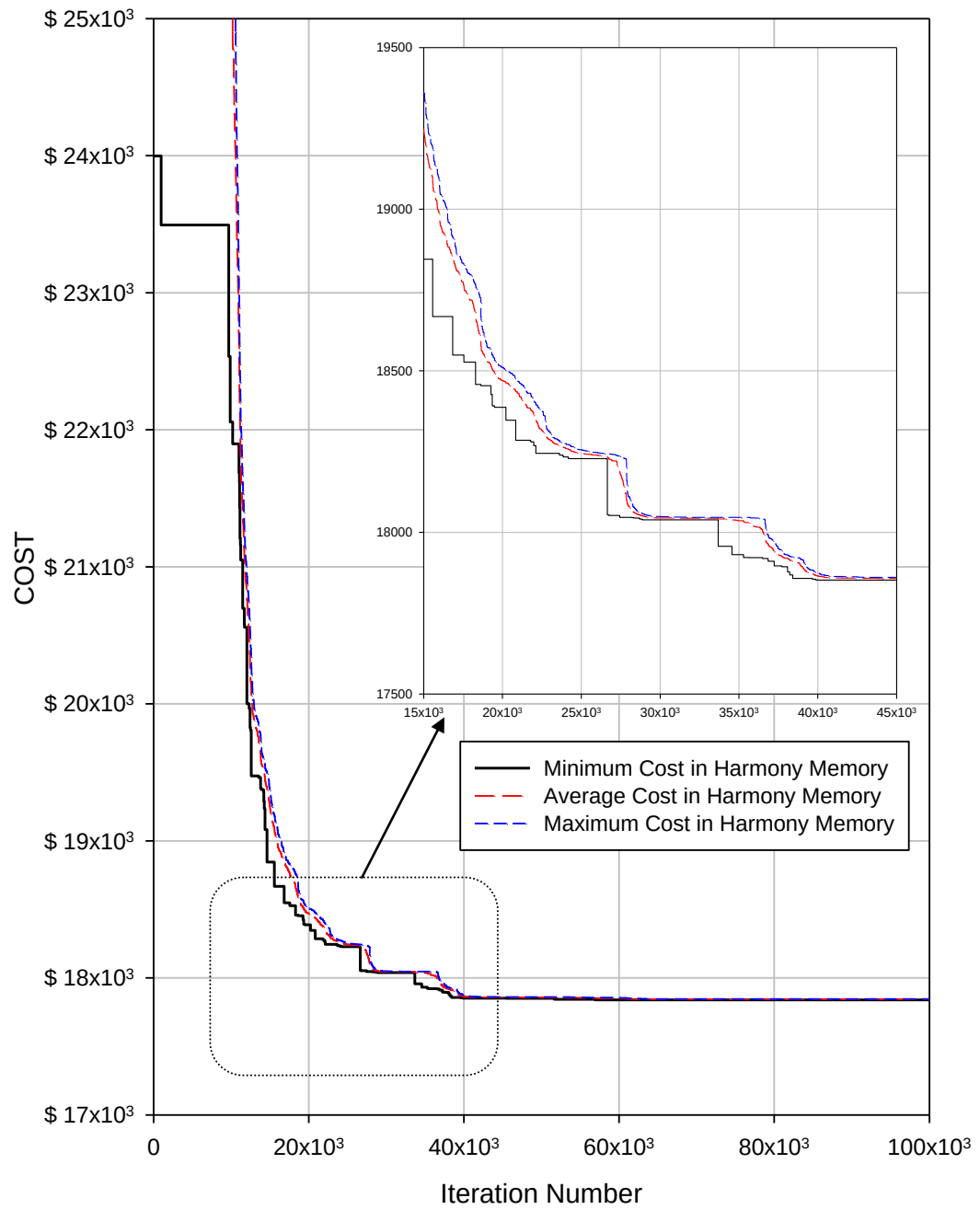


Figure 5.26 The design memory history for the design of five-bay five-storey RC plane frame (for Case 1).

Table 5.14 The optimum values of design variables for five-bay five-story RC frame (Case 2).

Optimum design results for five-bay five-storey frame			
<i>Optimum column design results</i>			
<i>Design Variables</i>	<i>Column Gr. 1</i>	<i>Column Gr. 2</i>	<i>Column Gr. 3</i>
Section number in design pool	(29)	(121)	(117)
The height of section, h (mm)	300	300	350
The height of section, b (mm)	550	750	800
The diameter of reinforcement bars, ϕ (mm)	18	14	20
The number of bars in x dir., n_1	3	6	4
The number of bars in y dir., n_2	1	3	2
Total reinforcement	12 ϕ 18	22 ϕ 14	16 ϕ 20
<i>Optimum beam design results</i>			
<i>Design Variables</i>	<i>Beam Gr. 1</i>		
X ₁	25/60 (10)		
X ₂	2 ϕ 12 (1)		
X ₃	3 ϕ 12 (3)		
X ₄	1 ϕ 18 (6)		
X ₅	1 ϕ 26 (16)		
X ₆	1 ϕ 18 (6)		
X ₇	1 ϕ 16 (4)		
X ₈	1 ϕ 18 (6)		
X ₉	5 ϕ 16 (31)		
X ₁₀	1 ϕ 18 (6)		
X ₁₁	1 ϕ 16 (4)		
X ₁₂	1 ϕ 18 (6)		
X ₁₃	1 ϕ 26 (16)		
X ₁₄	NNR* (1)		
X ₁₅	2 ϕ 14 (7)		
X ₁₆	1 ϕ 12 (2)		
X ₁₇	NNR* (1)		
X ₁₈	1 ϕ 12 (2)		
X ₁₉	3 ϕ 12 (9)		
X ₂₀	1 ϕ 12 (2)		
X ₂₁	3 ϕ 12 (9)		
X ₂₂	1 ϕ 12 (2)		
X ₂₃	NNR* (1)		
X ₂₄	NNR* (1)		
X ₂₅	2 ϕ 14 (7)		

NNR*, no need reinforcement, (n_i), the sequence number of i^{th} design variable in the design variable pool.

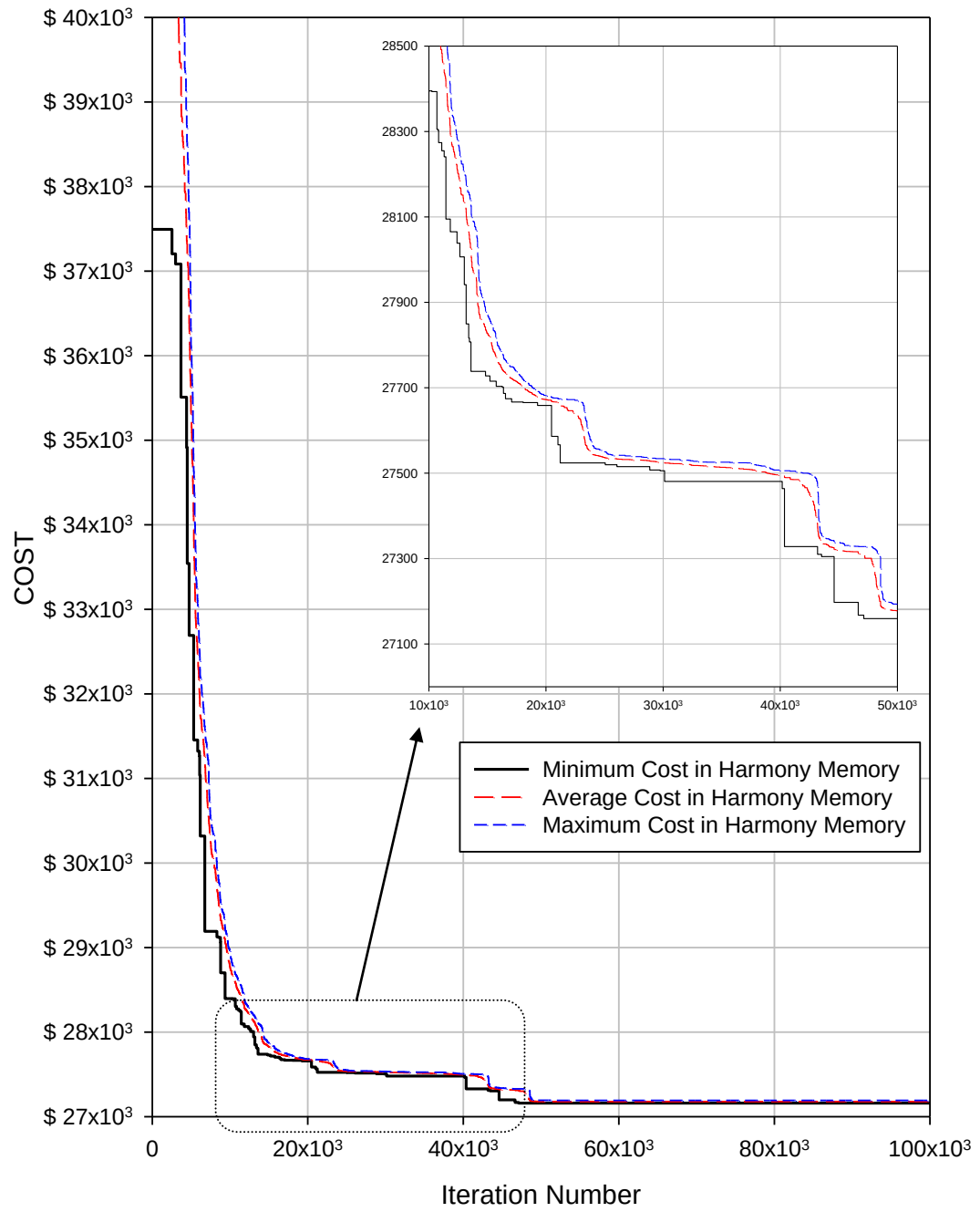


Figure 5.27 The design history for five-bay five-storey RC plane frame (for Case 2).

The optimum unit quantities of concrete, formwork and steel obtained for two cases are given in Table 5.15. Optimum sections for the given unit cost of concrete, steel and concrete are obtained for both cases. In case two, due to the increased unit cost of steel, the optimum sections are selected with less steel usage. Even though there is a small increase in the amount of steel and a small decrease in the amount of concrete for the beams, the amount of steel is decreased and the amount of concrete is increased for the columns. Since more steel is used in the columns in comparison to the beams, the column and beam cross-section dimensions that provide the optimum quantity of concrete, steel, and formwork are impacted. Using more steel in the columns in comparison to the beams, affects the optimum column and beam cross-section dimensions that provide the optimum quantity of concrete, steel, and formwork.

Table 5.15 The details of material quantities and costs of the optimum designs for five-bay five-story RC frame.

Components	Case 1	Case 2
Concrete (m³)		
Beam		
Column		
Steel (kg)		
Beam		
Column		
Formwork (m²)		
Beam		
Column		168.580
Total Cost (\$)	17838.99	27159.44

(1\$ =1.51 TL , \$; American Dollar, TL; Turkish Liras)

CHAPTER 6

DISCUSSION

6.1 Discussion

The major difference between this study and the ones available in the literature is the consideration of the reinforcement detailing in the optimum design of reinforced concrete structures. The articles published so far on the topic do not pay attention to the constructability of the reinforced concrete member. They generally obtain the optimum dimensions of the reinforced concrete members and the optimum areas of reinforcements required for the member. The selection of bar diameters and total number of bars at the cross sections are left to designer. In some cases, it becomes difficult to place required number of bars into the cross section which satisfies the optimum reinforcement area. Furthermore, in the studies available in the literature, the shear design calculations of concrete members are not considered and as result the cost of shear reinforcements is not considered in the objective function. In these studies, the design constraints included in the design formulation are not comprehensive but they are simple capacity and detailing requirements for flexural reinforcements. In this study, all the constraints described in ACI 318-05 [30] for Special Seismic Moment reinforced concrete frames are included in the optimum design problem. In previous studies, the arrangement of detailing of reinforcement bars is oversimplified and the development length of reinforcement bars are not considered in cost calculations. In this study, these considerations are taken into consideration. In some of the previous studies, the lateral loading on the frame for earthquakes is considered; however, the values of the lateral loads are taken

as a constant even though the value of lateral loads change with the weight. In this study, the seismic loads on the frame are considered and these lateral loads are updated for each frame with selected member sections according to the weight of frame and regulations in the seismic specification ASCE 7-05 [31]. In addition to the aforementioned design discrepancies, there is also a lack of application of modern meta-heuristic optimization methods in the optimum design of reinforced concrete structures. There are few publications which make use of meta-heuristic techniques in literature. This study is an addition to these few application.

The optimum frame designs obtained by the algorithm developed are checked by SAP2000 package program in order to verify its optimality. For this purpose two-bay five-storey RC frame is designed by SAP2000 using the optimum beam and column dimensions under the same loadings. SAP2000 has the facility of computing reinforcement areas for the longitudinal bars at the ends and middle section of beams as well as columns. These reinforcement areas are given in Figure 6.1. The optimum design algorithm proposed determines the total number of bars and bar diameters at the ends and middle sections of beams. These are converted to reinforcement areas and given in Figure 6.2 for the same frame. It is apparent from this figure that the values of reinforcement areas are close to those shown in Figure 6.1. Remembering the fact that the optimum design algorithm developed determines the arrangement of reinforcement bars, but not reinforcement areas, it will not be realistic to expect that the values of reinforcement areas will be exactly the same. However, the comparison of both figures clearly indicates that they quite close to each other. This verifies the optimum results obtained by the proposed algorithm. More detailed arrangement of reinforcement steel bars may lead to find better results for the optimum design, but this situation leads to an increase in the number of design variables. The increase in the number of design variables adversely affects the performance of optimization methods. For concrete column sections, the section database is created in this study.

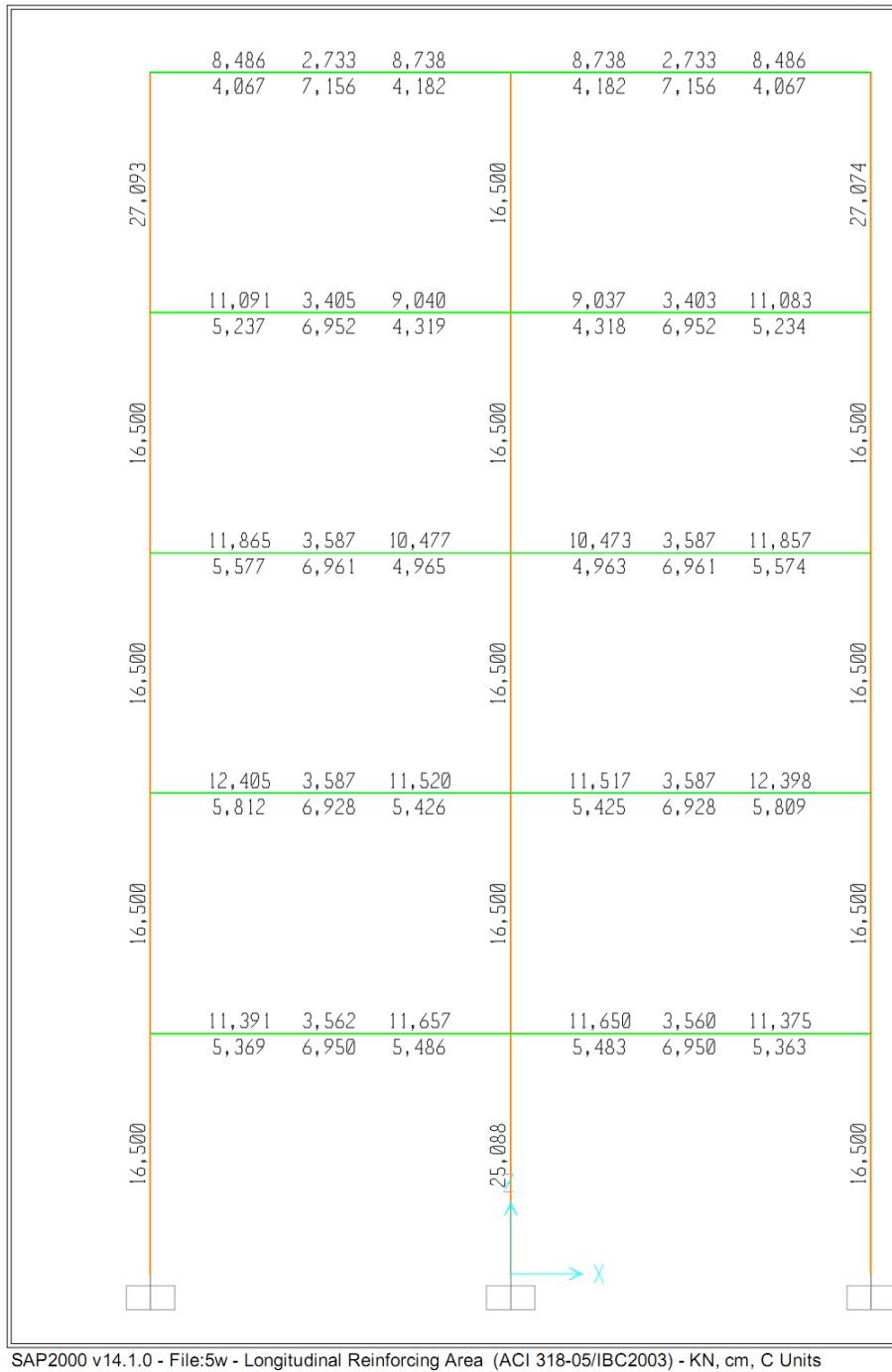


Figure 6.1 The requirement reinforcing steel areas (cm^2) for optimum sections of two-bay seven-storey RC plane frame example ($I=1.00$) (SAP2000 results).

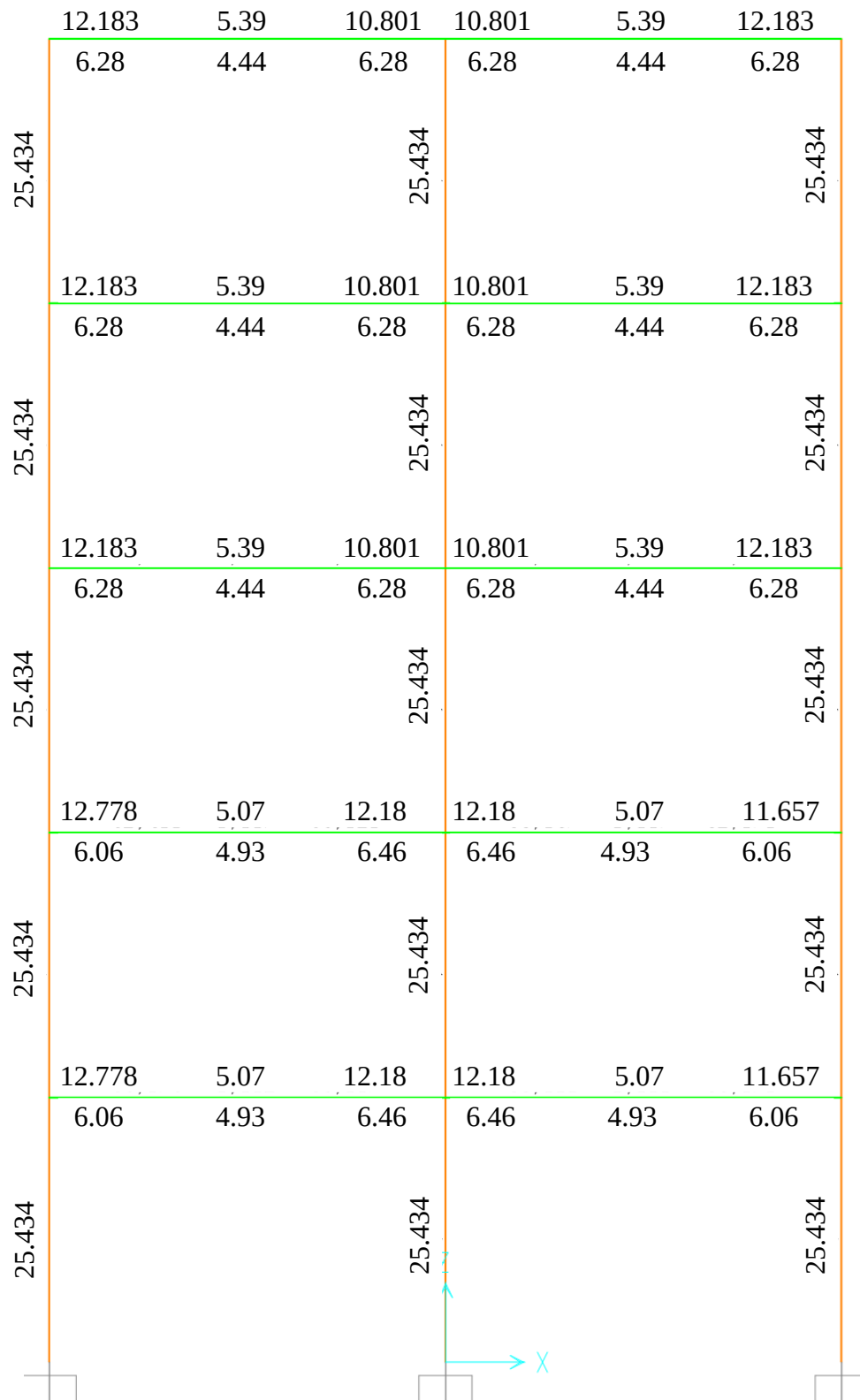


Figure 6.2 The placed reinforcement steel areas (cm^2) for the optimum design of two-bay seven-storey RC plane frame example ($I=1.00$).

In the conventional design, the arrangements of reinforcement bars in the column sections are determined using some assumptions and charts, such as the areas of reinforcement steel at four sides of column are accepted as equal to each other. Because of the axial load and flexural moment strength of column sections are depended to each other, finding of the optimum arrangement of the longitudinal reinforcement steel in columns is very hard and complex.

In conventional reinforced concrete frame design, designer tries to determine the width and depth of beams and columns, and the detailing of reinforcement by trial and error. The frame is analyzed several times by considering different dimensions for beam and column sections. After the each analysis the response of the frame is checked to find out whether the strength and serviceability requirements specified by the design code. If they are not satisfied, the design is rejected. If they are satisfied, the cost of RC frame is calculated. This process continued until the frame which satisfies the code requirements and its cost is the minimum is reached. However, it is apparent that without any guideline to determine such a frame is computationally very expensive and in some case it is even impossible. On the other hand, formulating the design problem as an optimum design problem and finding out its solution using one of met-heuristic optimization techniques overcomes this discrepancy. Such procedure reaches the optimum dimensions of beams and columns of the frame as well as reinforcement bars required for the selected dimensions where the design limitations described in the design code is satisfied and the cost of the frame is minimum. It is apparent that such result cannot be achieved by trial and error type of procedure. The optimum design algorithm presented considers the restrictions of the design code for special seismic moment frames which necessitates the strong column – weak beam designs. According to this design rule, columns should not only carry the design forces acting on the column but they should be stronger than beams framed into this joint. This criteria is satisfied in all the design examples considered in this study. Furthermore, when the cross sectional dimensions of a frame member changes, the seismic loads acting the frame laterally also changes. This is

automatically carried out by the program coded. Hence, this feature of the algorithm is also different from the studies available in the literature. In these algorithms, lateral loads are considered to be constants throughout the design process. Hence, it can be concluded that use of meta-heuristic optimization techniques in the design process of reinforced concrete frames automates the design and achieves the most economical frames which is not possible otherwise.

CHAPTER 7

SUMMARY AND CONCLUSIONS

7.1 General

The primary objective of structural design engineers is generally search for the optimal system under prescribed conditions. Because of this primary objective, the optimization researches become one of the major fields in structural engineering. The applicability of the optimization methods in engineering design problems attracted wide interest which yield large number of research works.

Structural design problems involve large number of design variables and necessitate finding the global optimum under the large number of design constraints. Accordingly, the computational time spent to find the optimum design increases and finding the optimum solution becomes quite hard. Thus, the computer programs based on optimization algorithms are used to determine the optimum solution of these design problems efficiently. The new optimization techniques dealing with discrete optimization are emerged in order to offer a solution to this problem. These new optimization techniques are based on the analogies of the natural events and they are the applications of artificial intelligence.

The harmony search algorithm as presented in detail in chapter 2 is one of the new meta-heuristic techniques applied to engineering optimization problems. The analogy of the harmony search method is based on principles of the musicians'

improvisation of the perfect state of harmony. The harmony search method is applied to a wide range of engineering optimization problem successfully and the attractive results are obtained by the researchers. In recent years, the number of publications in the application of harmony search algorithm to civil engineering design problems has increased exponentially.

7.2 This Study

In this thesis, the recently developed harmony search algorithm is applied to the optimum design problem of RC *Special Sesimic Moment Frames*. The optimum design problem of such frames has been formulated under the provision of ACI 318-05. The design variables in this optimization problem are selected as the section of column, the width and the depth of beams in each span, and the diameter and the number of longitudinal reinforcement bars along span and supports. The objective function to be minimized is the cost of RC frame. This function is subject to constraints implemented from ACI 318-05 for *Special Seismic Moment Frames*. These constraints covers the flexural and shear strength, serviceability, the minimum and maximum steel percentage for flexural and shear reinforcement, the spacing requirements for the stirrups and the upper and lower bound requirements for the width and the depth of the beam section. Because RC frames are statically indeterminate and generally have a large number of members and design parameters and members, the optimization problem involves a large number of variables and constraints.

In this study, the harmony search algorithm, which is one of the recent addition to stochastic search techniques of numerical optimization, is used to obtain the solution of the optimum design problem described. For this purpose, a computer program based on the harmony search method is developed using FORTRAN language and this computer program is designed to meet the requirements of the design engineers. The optimum design algorithm developed is tested on the

design of five different reinforced concrete frames. These frames are chosen to investigate the performance and efficiency of the harmony search algorithm. The design examples considered has shown that the harmony search method can be used in finding the solution of optimum design problem of RC frames where the reinforcement detailing and members cross sectional dimensions are taken as design variables. It is noticed that three main parameters of the harmony search algorithm namely, harmony memory size HMS, harmony memory considering rate HMCR, and the pitch adjustment rate PAR play an important role on the optimum designs obtained. These parameters are required to be selected before the harmony search method starts its search to find the optimum solution. One set of these parameters which gives good results in one optimum design problem may not lead to a good solution in another design problem. It is shown that the appropriate values of the parameters of harmony search algorithm are problem dependent. Therefore, it is necessary to run the design algorithm several times with different values of the harmony search algorithm parameters and determine the best solution. For this optimization problem, the harmony search algorithm with parameters, the high HMCR, especially from 0.70 to 0.95, the small PAR, generally from 0.15 to 0.20, and HMS, from 40 to 60, yields good performance in design optimization. The optimum design problem may be more complicated because the self weights of the column and beam sections affect the dead and earthquake loads acting on the RC frame. When the optimum designs are inspected, the optimum sections are obtained for the beams first and then optimum reinforcing detailing for the beams is obtained next. For the columns, the optimum sections are obtained directly. Moreover; the constraints that define the relationship between the column section dimensions and the beam section dimensions at the joints of the frame, and the unit costs of concrete, steel, and formwork play an important role in determining of the optimum column and beam section dimensions.

The results obtained from the design examples are shown that harmony search method is a reliable and robust technique that can be effectively used in finding

the optimum detailed design of reinforced concrete frames. The design algorithm performs effectively in finding the optimum values of the design variables. The optimization algorithm arrives at rational and realistic design solutions that are directly constructible. The proposed optimization algorithm is mathematically less complex and easier than those that employed traditional optimization techniques.

7.3 Recommendations for Future Studies

The optimum performance-based design of concrete structures may be studied with this new meta-heuristic algorithm. The harmony search algorithm may be used for optimum design of prestressed concrete beams and the optimization problem may be extended to box-girder bridges. Adaptive harmony search method which arranges the harmony search parameters automatically during the design process can be used in order to improve the performance of classical harmony search method.

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APPENDIX A

ANALYSIS OF PLANE FRAMES WITH MATRIX DISPLACEMENT METHOD

A.1 Degree of Freedoms

There are six degrees of freedoms at a member of plane frames. These are the usual the two translations at first end (U_1, U_2), two translations at far end (U_4, U_5) and two rotations (U_3, U_6) at both ends of the member shown in Figure A.1 in global axis. These are represented as vector $\vec{U} = U_1, U_2, U_3, U_4, U_5, U_6$. Load vector applied on member is described as $\vec{P} = P_1, P_2, P_3, P_4, P_5, P_6$. P_1, P_4 are the horizontal forces, P_2, P_5 are the vertical forces and P_3, P_6 are the moments acting on both ends of a member in global axis.

In matrix displacement method stiffness relations between these vectors are obtained. Joint displacements, member end forces and moments are calculated by using these relations. Displacements vector of the member in local coordinates can be represented as $\vec{u} = u_1, u_2, u_3, u_4, u_5, u_6$. Member end forces vector of the plane frame member in local coordinates can be represented as $\vec{F} = F_1, F_2, F_3, F_4, F_5, F_6$. These vectors are shown in Figure A.2. Calculation of these vectors will be described in the following.

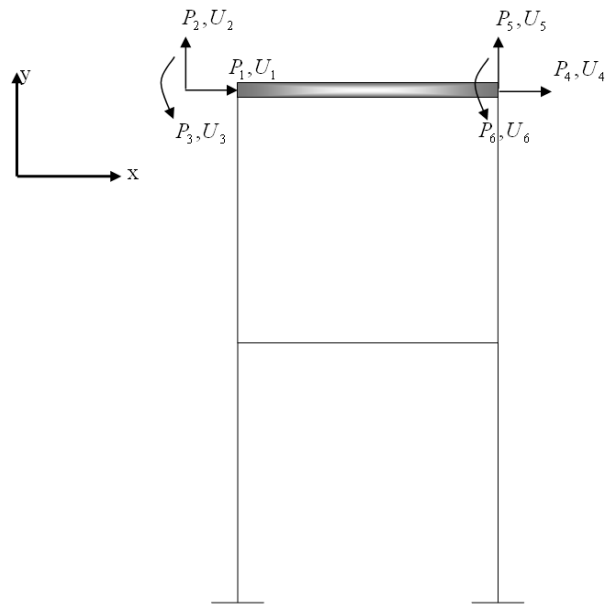


Figure A.1 Plane frame.

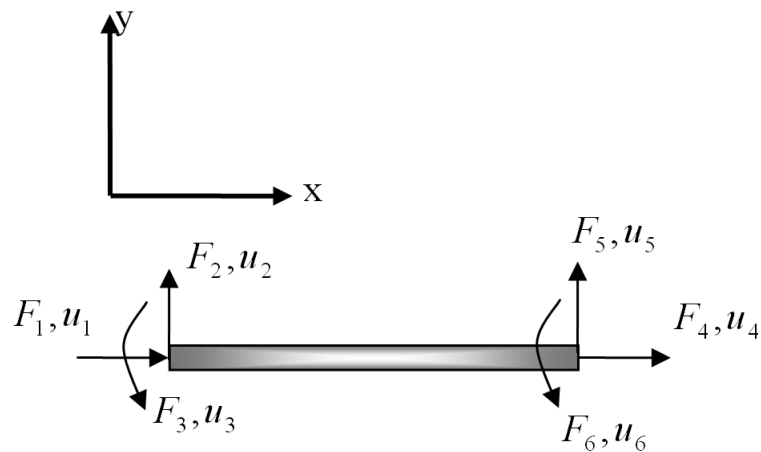
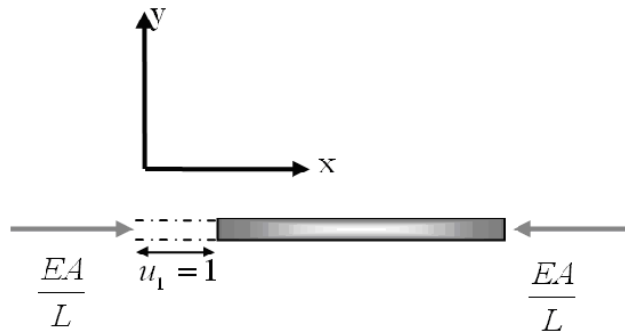


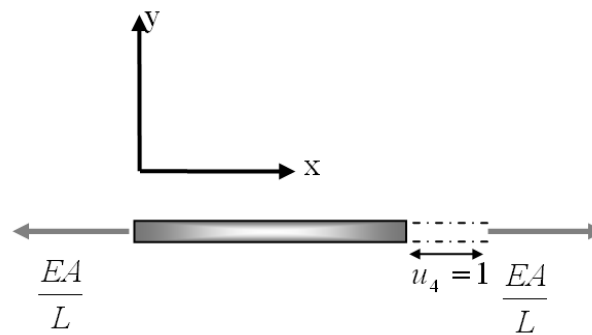
Figure A.2 Joint end displacements, end forces and moments in frame members.

A.2 Member Stiffness Matrix

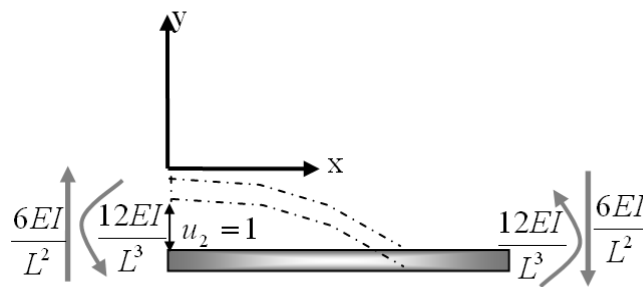
Member stiffness matrix describes the member's resistance against deformations. this matrix has six rows and six columns for plane frames. Each column of this matrix can be obtained by assigning unit value to each degree of freedom respectively shown in Figure A.3.



(a) Unit displacement for x axis at first end.

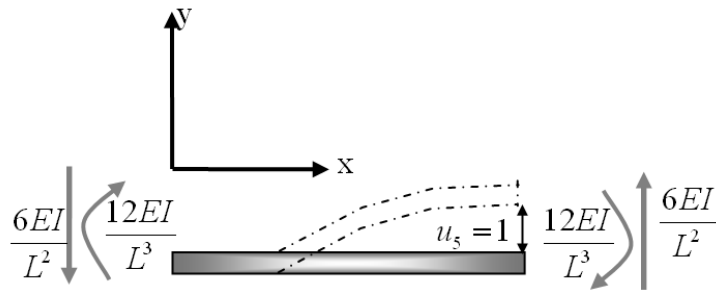


(b) Unit displacement for x axis at far end.

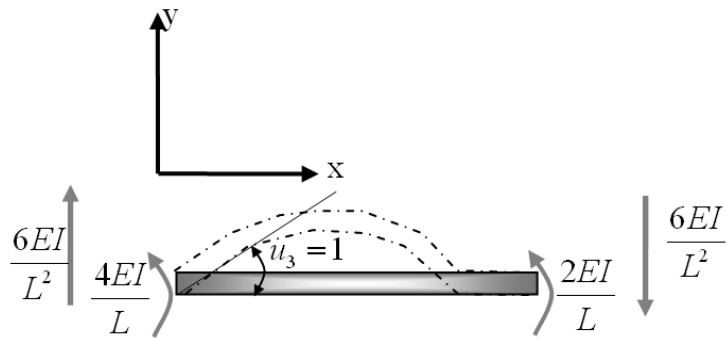


(c) Unit displacement for y axis at first end.

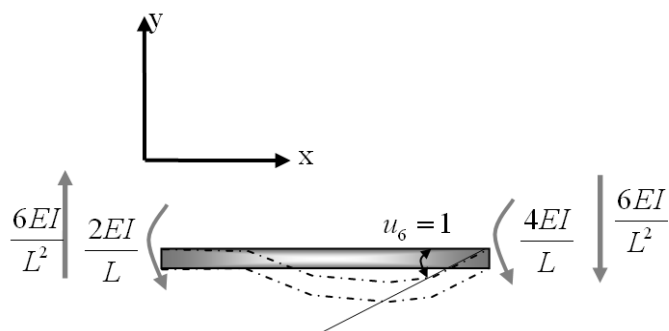
Figure A.3 Member forces and moments for degree of freedoms.



(d) Unit displacement for y axis at far end.



(e) Unit rotation at first end.



(f) Unit rotation at far end.

Figure A.3 Cont. Member forces and moments for degree of freedoms.

Stiffness terms obtained from Figure A.3 substituted to the corresponding rows for each column member stiffness matrix in local coordinate system is obtained.

$$\mathbf{k} = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & -\frac{12EI}{L^3} & \frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{2EI}{L} \\ -\frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & -\frac{12EI}{L^3} & -\frac{6EI}{L^2} & 0 & \frac{12EI}{L^3} & -\frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{4EI}{L} \end{bmatrix}$$

A.3 Displacement and Force Transformation Matrices

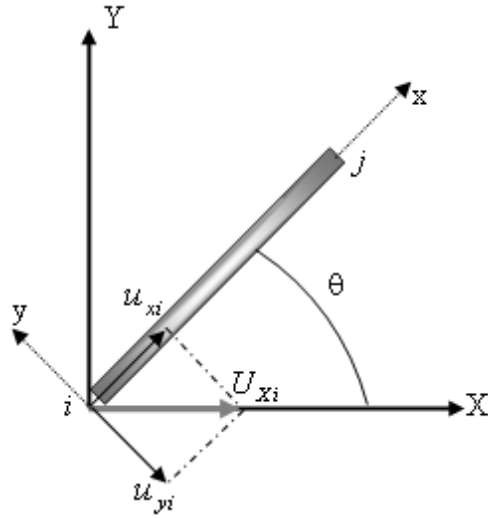
In matrix displacement the internal member loads (q) and deformations (u) from local x, y, z coordinates should be transformed to global X, Y, Z coordinates. Therefore, transformation matrices are needed.

A.3.1 Displacement transformation Matrix

Consider a frame member shown in Figure A.1. It is seen that from that figure a global coordinate displacement along X axis for joint i U_{xi} has two components in local coordinate systems. These are:

$$u_{xi} = U_{xi} \cdot \cos \theta \quad (A.1)$$

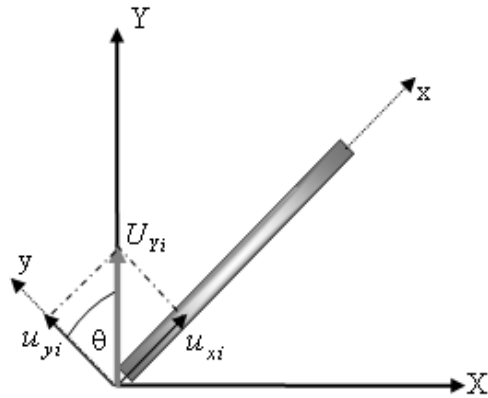
$$u_{yi} = -U_{xi} \cdot \sin \theta$$



Similarly, It is seen that from Figure A.2 a global coordinate displacement in Y axis for first end U_{yi} creates local coordinate displacement.

$$u_{xi} = U_{yi} \cdot \sin \theta \quad (A.2)$$

$$u_{yi} = U_{yi} \cdot \cos \theta$$



Finally, since z and Z axes are coincident, that is directed out of page, a rotation at first end U_{zi} about Z causes a corresponding rotation z about Z. Thus,

$$u_{zi} = U_{Zi} \quad (A.3)$$

Similarly, if global displacements U_X, U_Y and a rotation U_Z are applied on the far end of the member, the resulting transformation equations are obtained as follows:

$$\begin{aligned} u_{xj} &= U_{Xj} \cdot \cos \theta & u_{yj} &= -U_{Xj} \cdot \sin \theta \\ u_{xi} &= U_{Yi} \cdot \sin \theta & u_{yi} &= U_{Yj} \cdot \cos \theta \end{aligned} \quad (A.4)$$

$$u_{zj} = U_{Zj}$$

If transformation equations obtained above is written the superposition of displacements in matrix form, following equation system is obtained.

$$\begin{bmatrix} u_{xi} \\ u_{yi} \\ u_{zi} \\ u_{xj} \\ u_{yj} \\ u_{zj} \end{bmatrix} = \begin{bmatrix} \cos \theta & \sin \theta & 0 & 0 & 0 & 0 \\ -\sin \theta & \cos \theta & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & \cos \theta & \sin \theta & 0 \\ 0 & 0 & 0 & -\sin \theta & \cos \theta & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \cdot \begin{bmatrix} U_{Xi} \\ U_{Yi} \\ U_{Zi} \\ U_{Xj} \\ U_{Yj} \\ U_{Zj} \end{bmatrix}$$

Equation (2.4) can be shown shortly as $u = T \cdot U$. By inspection T transforms the six global displacements.

A.3.2 Force Transformation Matrix

Relationship between external loads and member end forces is shown in following equation.

$$[R]^T [P] = [F] \quad (A.5)$$

where, P is the external load vector, F is the member end forces vector and T is transformation matrix described above.

A.3.3 Overall Stiffness Matrix

Relationships between the external load vector $[P]$ and joint displacement vector $[U]$ can be obtained by using Equations $[F] = [T]^T [P]$ and $[F] = [k][u]$. If these expressions are substituted in (2.6);

$$[P] = [T][k][u] \quad (A.6)$$

is obtained where, $[K]$ is overall stiffness matrix of the frame. Calculation of this matrix is shown in the following.

$$[K] = \sum_{i=1}^n [k_i] = \sum_{i=1}^n [T_i]^T [k_i] [T_i] \quad (A.7)$$

where, m is the number of members in space frame.

A.4 General steps for plane frame analysis

Step 1: The loading and geometric properties of the frame, material properties and member section dimensions are read from program. By using these values structure stiffness matrix $[K_s]$ and load vector $[P]$ are calculated in equation (A.6).

Step 2: By solving equation (A.6) $(\{R\} = [K]^{-1}\{F\})$ joint displacements in global coordinates $(\{U\} = [K]^{-1}\{F\})$ are calculated.

Step 3: After finding $\{U\}$ vector member end forces and moments are calculated by using $\{R\} = [K]^{-1}\{F\}$ equality. Consequently, by using frame analysis program in addition to the joint displacements and the rotations, axial forces shear forces and moments are computed.

APPENDIX B

CONCRETE DESIGN of SPECIAL SEISMIC MOMENT FRAMES (ACI 318-05 / IBC 2006)

B.1 Design Load Combinations

The design load combinations are various combinations of the load cases for which the structure is required to be designed. The load combination factors are applied to the forces and moments obtained from the analyses cases to form the factored design forces and moments for the load combination.

The design load combinations are the various combinations of the analysis cases for which the structure is required to be checked. In this study, it is considered that the structure is only subjected to dead load (D), live load (L), and earthquake load(E), and earthquake forces are reversible. Consequently, the following load combinations are needed to be considered for the design of reinforced concrete frame;

$$1.4 D \quad (ACI 9-1) \quad (B.1)$$

$$1.2 D + 1.6 L \quad (ACI 9-2) \quad (B.2)$$

$$0.9 D + 1.0 L \pm 1.0 E \quad (ACI 9-5) \quad (B.3)$$

$$0.9 D \pm 1.0 E \quad (ACI 9-7) \quad (B.4)$$

B.2 Column Design

For the design of columns, the geometry of the reinforcing bar configuration adopted for each concrete column section has been defined to determine the column capacity. The design procedure for the reinforced concrete columns of the structure involves in calculating the uniaxial axial force-moment capacity and shear capacity. To determine uniaxial moment capacity of a concrete section, the uniaxial interacting diagram is generated for all of the different concrete section types of the model. The capacity ratio is calculated using the obtained interaction diagrams for the factored axial force and uniaxial bending moments obtained from the design load combinations at each station of the column. The column axial force-bending moment interaction diagram of concrete sections is numerically defined by a series of discrete points that are generated on the uniaxial interaction failure surface. The interaction surfaces are generated for the range of allowable reinforcement; 1 to 8 percent for Ordinary and Intermediate Moment Resisting Frames (ACI 10.9.1) and 1 to 6 percent for Special Moment Resisting Frames (ACI 21.4.3.1). A typical uniaxial interaction surface is shown in Figure B.1.

The coordinates of the points on the uniaxial interacting diagram are determined by rotating a plane of linear strain on the concrete column section as shown in Figure B.2. The formulation of column capacity is based on the general principles of ultimate strength design (ACI 10.3). The maximum concrete strain (ϵ_c) at the extremity of the section is limited to 0.003 (ACI 10.2.3) on the linear strain diagram. And the stress in the steel (given by the product of the steel strain (ϵ_s) and the steel modulus of elasticity (E_s)) is limited to the yield stress of the steel (f_y) (ACI 10.2.4).

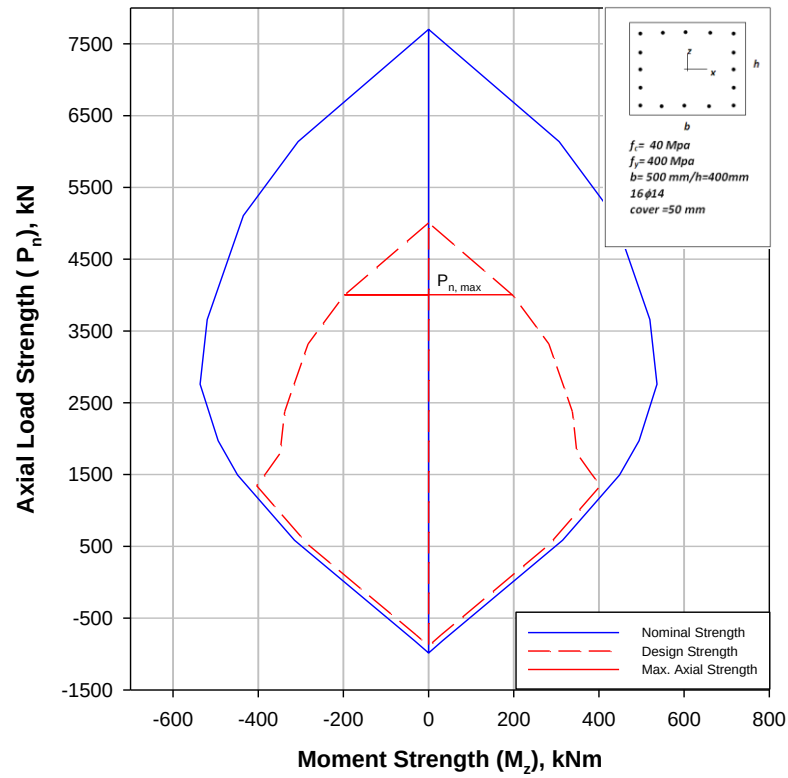


Figure B.1 Typical Axial Load-Moment interaction diagram.

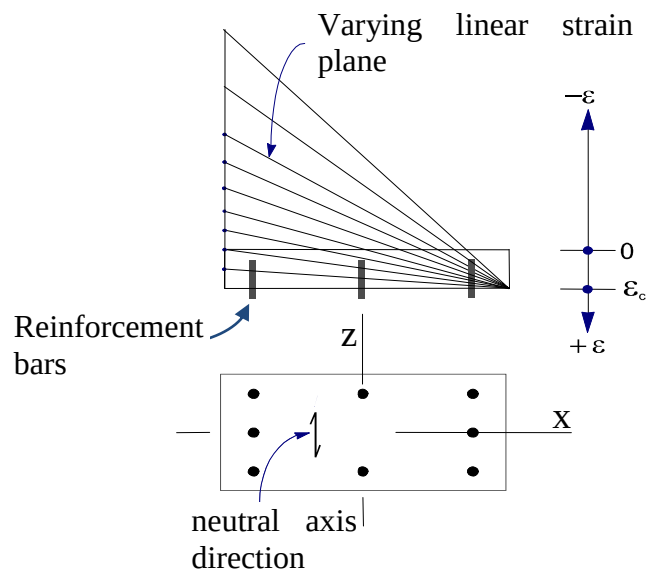


Figure B.2 Idealized strain distribution for generation of interaction surface.

The concrete compression stress block is assumed to be rectangular as shown in Figure B.3. The stress value of compression stress block is taken $0.85f_c$ (ACI 10.2.7.1). The depth of the equivalent rectangular stress block, a , is computed with:

$$a = \beta_1 c \quad (\text{ACI 10.2.7.3}) \quad (\text{B.5})$$

where, c is the depth of section in compression strain and,

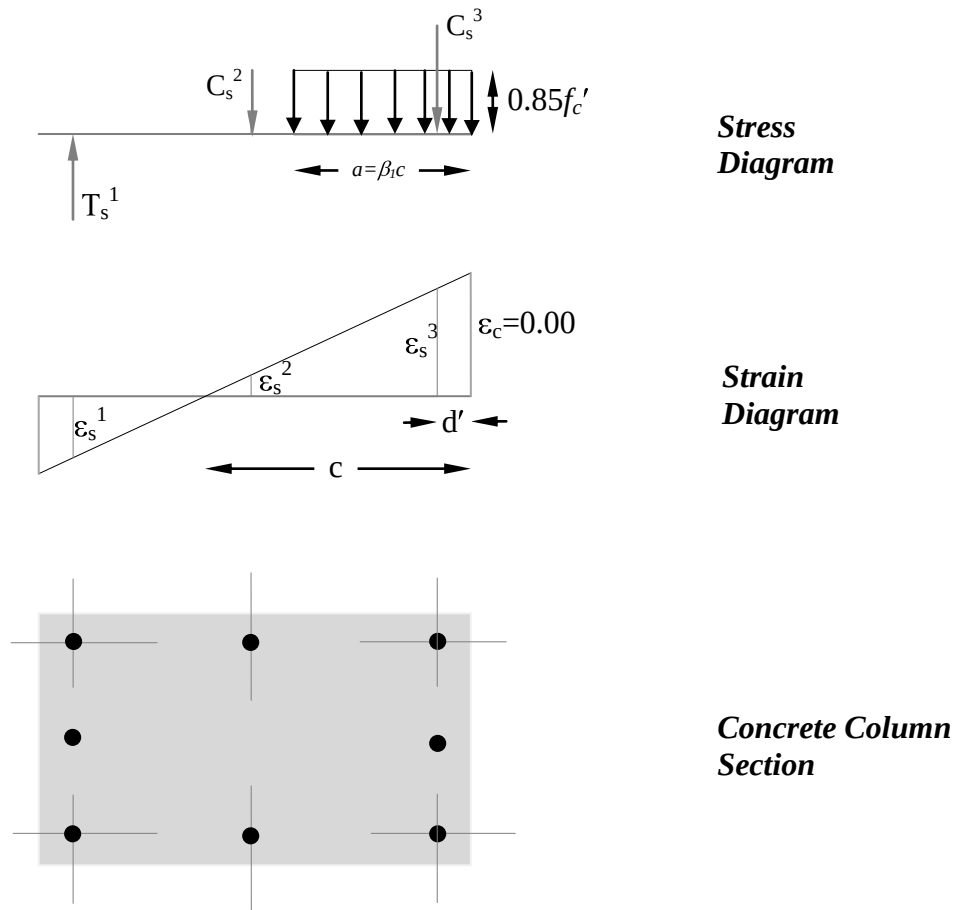


Figure B.3 Idealization of stress and strain distribution in a column section.

$$\beta_1 = 0.85 - 0.05 \left(\frac{f'_c - 4000}{1000} \right), \quad 0.65 \leq \beta_1 \leq 0.85 \quad (\text{ACI 10.2.7.3}) \quad (\text{B.6})$$

The strength reduction factor ϕ , is used for the generation of the interaction surface. The value of ϕ used in the interaction diagram varies from compression controlled section to tension controlled section according to the maximum tensile strain in the reinforcing bars at the extreme edge (ε_t) (ACI 9.3.2.2).

The definition of compression controlled section is used for the sections when the tensile strain in the extreme tension steel in the section is equal to or less than the compression controlled strain limit at the time the extreme concrete strain in compression reaches its assumed strain limit of ($\varepsilon_{c,max}$), which is 0.003. Sections are tension controlled when the extreme tensile strain in the steel bars is equal to or greater than 0.005, just as the extreme concrete strain in compression reaches its assumed strain limit of 0.003 (ACI 10.3.4). The sections are considered to be in a transition region between compression controlled and tension controlled sections according to the value of the maximum tensile strain in the reinforcing bars at the extreme edge (ε_t) (ACI 10.3.4). The strength reduction factor ϕ , is linearly interpolated between two values for the section within the transition region as shown in the following (ACI 9.3.2);

$$\phi_c = \begin{cases} \phi_c & \text{if } \varepsilon_t \leq \varepsilon_y \\ \phi_t - (\phi_t - \phi_c) \left(\frac{0.005 - \varepsilon_t}{0.005 - \varepsilon_y} \right) & \text{if } \varepsilon_y < \varepsilon_t \leq 0.005 \\ \phi_t & \text{if } \varepsilon_t \geq 0.005 \end{cases} \quad (\text{ACI 9.3.2}) \quad (\text{B.7})$$

where, $\phi_t = 0.90$ for tension controlled sections, $\phi_c = 0.65$ for compression controlled column sections with tied reinforcement.

The maximum compressive axial load for tied column sections, $\phi P_{n, max}$, is calculated as;

$$\phi P_{n(max)} = 0.80 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] \quad (\text{ACI 10.3.6.2}) \quad (\text{B.8})$$

The column bending moment capacity for each design load combination is found by using the obtained uniaxial interacting diagram at both end of the column member. By this way, the design point defined by the resulting axial load and bending moment set (obtained from load combinations) is determined whether it lies within the interaction surface or not. If the design point lies within the interaction surface, the column capacity is adequate, or else, the design point lies outside the interaction surface, the column is overstressed.

The shear design of columns involves the shear reinforcing for a particular column for a particular design load combination resulting from shear forces. Firstly, the factored forces, P_u and V_u , acting on the column section are determined to calculate the reinforcement steel required to carry the acting loads. In addition to the factored shear forces, the shear design of the columns is also based on the maximum probable moment strengths and the nominal moment strengths of the members or special and Intermediate moment resisting frames (Ductile frames) (IBC 2006, ACI 21.4.5.1, 21.12.3).

In the shear design of Special moment resisting frames (i.e., seismic design), in addition to the factored design loads, the shear capacity of the column is also checked for the capacity shear design force based on the maximum probable moment strengths of the column and beams framed to the column.

The shear capacity force in the column, V_u , is determined from consideration of the acting shear forces on the column section. Two different capacity shears are calculated for the column section. The first design force is based on the probable moment strength of the column, while the second is computed from the probable moment strengths of the beams framing into the column. The design strength is taken as the minimum of these two values, but never less than the factored shear forces obtained from the design load combinations.

$$V_u = \min V_e^c \quad V_e^c \geq V_{u, factored} \quad (\text{ACI 21.4.5.1, IBC 2006}) \quad (\text{B.9})$$

Where, V_e^c = the shear capacity force of the column based on the probable maximum flexural strengths of the two ends of the column, V_e^b = the shear capacity force of the column based on the probable moment strengths of the beams framing into the column. The shear capacity of the column, V_e^c , is calculated using the maximum probable flexural strength at the two ends of the column for existing factored axial loads. The shear capacity force is computed for clockwise rotation of the joint at one end of column and the associated counter-clockwise rotation of the other joint of column. These situations produce two shear capacity force, and the maximum of these two values is taken as the V_e^c . The maximum probable positive and negative moment capacities, M_{pr}^+ and M_{pr}^- , of the column under the influence of the axial force, P_u , are computed using the uniaxial interaction diagram in the corresponding direction. Therefore, V_e^c is the maximum of V_{e1}^c and V_{e2}^c .

$$V_e^c = \max V_{e1}^c \quad V_{e2}^c \quad (\text{ACI 21.4.5.1}) \quad (\text{B.10})$$

where,

$$V_{e1}^c = \frac{M_I^- + M_J^+}{L} \quad (\text{ACI 21.4.5.1}) \quad (\text{B.11})$$

$$V_{e2}^c = \frac{M_I^+ + M_J^-}{L} \quad (\text{ACI 21.4.5.1}) \quad (\text{B.12})$$

M_I^+ and M_I^- = positive and negative probable maximum moment capacities at end I of the column using a steel yield stress value of αf_y ($\alpha = 1.25$) and no reduction factor ($\phi = 1.0$), M_J^+ and M_J^- = positive and negative probable maximum moment capacities at end j of the column using a steel yield stress value of αf_y ($\alpha = 1.0$) and no reduction factor ($\phi = 1.0$), and L = the clear span of the column.

The capacity shear of the column based on the flexural strength of the beams framing into the column, V_e^b , is computed using the maximum probable positive and negative moment capacities of each beam framing into the top joint of the column. Both clockwise and counter-clockwise rotations are considered separately, then the sum of the beam moments is calculated as a resistance to joint rotation. The capacity shear of the column due to the flexural strength of the beams in the columns is determined assuming that the point of inflection occurs at mid-span of the columns above and below the joints. The effects of load reversals are investigated and the maximum of the joint shears obtained from the two cases is selected as the capacity shear.

$$V_e^b = \max V_{e1}^b \quad V_{e2}^b \quad (\text{ACI 21.4.5.1}) \quad (\text{B.13})$$

where, V_{e1}^b = the column capacity shear for clockwise joint rotation, V_{e2}^b = the column capacity shear for counter-clockwise joint rotation,

$$V_{e1}^b = \frac{M_{r1}}{H} \quad (\text{B.14})$$

$$V_{e2}^b = \frac{M_{r2}}{H} \quad (\text{B.15})$$

M_{r1} = sum of beam moment resistances with clockwise joint rotations, M_{r2} = sum of beam moment resistances with counter-clockwise joint rotations, and H = distance between the inflection points (this distance is equal to the mean height of the columns above and below the joint, the distance is taken as one-half of the height of the column at the bottom of the joint for the columns at top story).

In the shear design of columns, the required shear reinforcement in the form of ties is determined according to the principles of ultimate strength design. After the concrete shear capacity and required shear design capacity are determined, the area of shear reinforcement, A_v and the spacing between ties s , are calculated to carry the factored shear forces obtained from the design load combinations safely.

The concrete shear capacity is calculated separately, for axial compression and axial tension cases. For axial compression loadings, the nominal shear carried by the concrete is given by,

$$V_c = 0.17\sqrt{f'_c} \left(1 + \frac{P_u}{14A_g} \right) A_{cv} \quad (\text{ACI 11.3.1.2}) \quad (\text{B.16})$$

where,

$$V_c \leq 0.29\sqrt{f'_c} b_w d \sqrt{1 + \frac{0.29P_u}{A_g}} \quad (\text{ACI 11.3.2.2}) \quad (\text{B.17})$$

and the nominal concrete shear capacity for axial tension loadings is given by,

$$V_c = 0.17\sqrt{f'_c} \left(1 - \frac{0.29P_u}{A_g} \right) A_{cv} \quad (\text{ACI 11.3.2.3}) \quad (\text{B.18})$$

where, P_u = the factored axial load at a section, A_{cv} = the area of concrete used to determine shear stress (shown in Figure B.4), A_g = the gross area of concrete, b_w = the width of the column section, d = the distance from compression face to tension reinforcement.

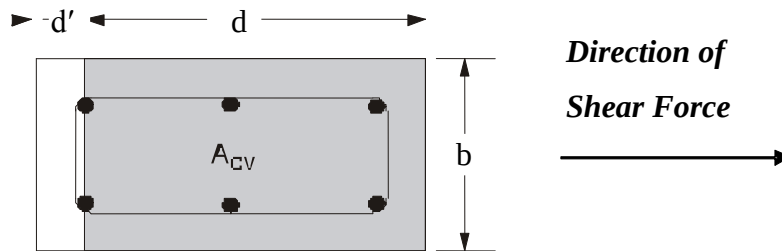


Figure B.4 Shear stress area A_{cv} for rectangular column section.

The required shear reinforcement in the form of ties within a spacing s obtained from factored loads and probable member moment capacities is calculated by the

following design procedure for rectangular columns.

$$\text{If } V_u \leq \phi(V_c / 2) \Rightarrow \frac{A_v}{s} = 0 \quad (\text{ACI 11.5.6.1}) \quad (\text{B.19})$$

$$\text{If } \phi(V_c / 2) \leq V_u \leq \phi(V_{\max}) \Rightarrow \frac{A_v}{s} = \frac{(V_u - \phi V_c)}{\phi f_y d} \quad (\text{ACI 11.5.7.1}) \quad (\text{B.20})$$

$$\frac{A_v}{s} \geq \max \left\{ \frac{0.062 \sqrt{f'_c} b_w}{f_y}, \frac{0.35 b_w}{f_y} \right\} \quad (\text{ACI 11.5.6.3}) \quad (\text{B.21})$$

$$\text{If } V_u > \phi(V_{\max}) \text{ a failure condition is declared.} \quad (\text{ACI 11.5.7.9}) \quad (\text{B.22})$$

$$V_{\max} = V_c + 0.66 \sqrt{f'_c} b_w d \quad (\text{ACI 11.5.7.9}) \quad (\text{B.23})$$

For special moment frames, column ends require adequate confinement and adequate shear reinforcement in order to ensure column ductility and to prevent shear failure prior to the development of flexural yielding at the column ends. The rectangular hoop reinforcement is used as the transverse reinforcement in this design study and regulations given in ACI design code for special seismic moment frames are shown in Figure B.5.

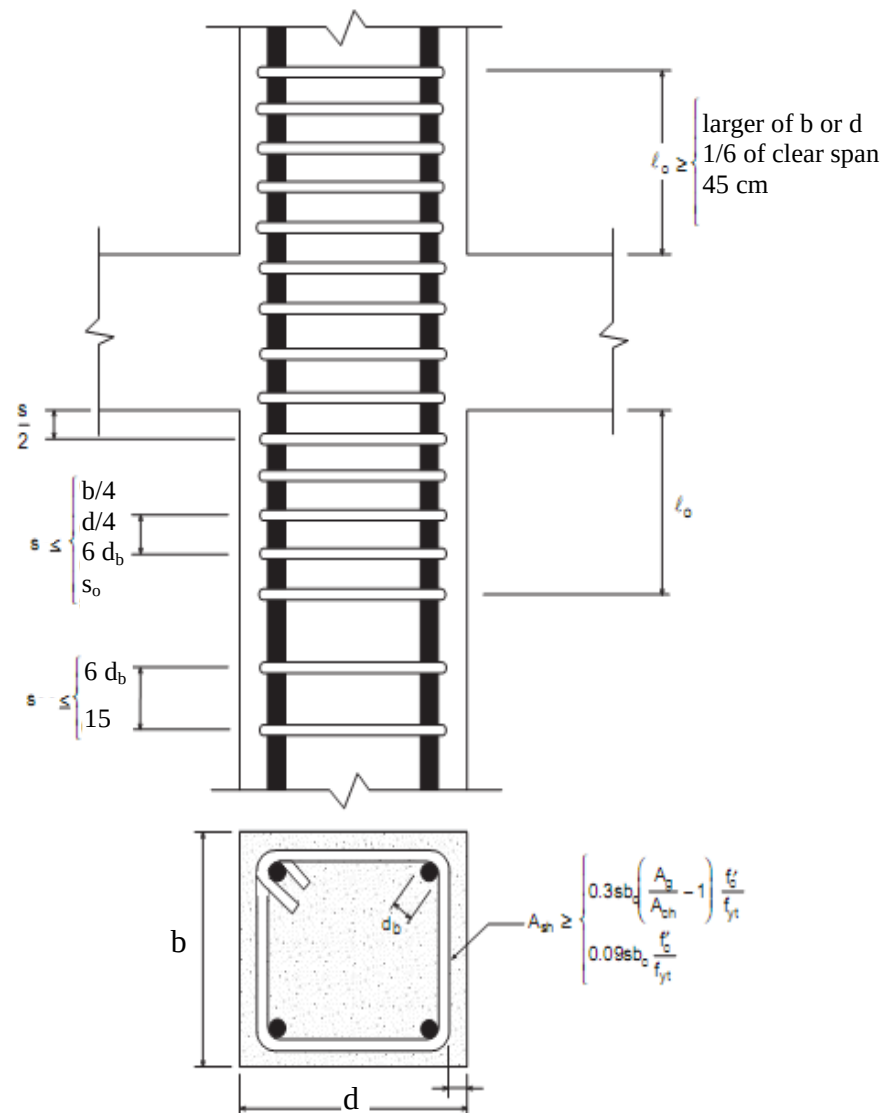


Figure B.5 Confinement requirements at column ends [30].

B.3 Beam Design

In the design of concrete beams, the beams are designed for flexure and shear to calculate the required areas of steel for flexure and shear based on the beam moments which are computed under the factored loads,. The beams are divided three equal parts along their span and the design flexural moments and the design shear forces are calculated for each part. The section of beams is considered as singly reinforced rectangular section.

The factored moments for each design load combination at a particular beam section (for three part of beam) are obtained by factoring the corresponding moments for different analysis cases. The moment capacities of beam sections (parts at both ends and the middle) are checked for the factored design moments obtained from all of the design load combinations. Positive moment capacity is produced by bottom steel and the negative moment capacity is produced by top steel. In such cases, the beam is always designed as a rectangular section.

The design procedure of beam sections is based on the simplified rectangular stress block (ACI 10.2) in Figure B.6. Furthermore, it is assumed that the net tensile strain of the reinforcing steel shall not be less than 0.005 (tension controlled) (ACI 10.3.4). All of the beams are designed ignoring axial force, because it is assumed that the design ultimate axial force does not exceed $\phi(0.1f_cA_g)$ (ACI 10.3.5).

The moment capacity of beams, M_u (for positive and negative moments), are calculated by following procedure.

$$M_u = A_s f_y \left(d - 0.59 \frac{A_s f_y}{f'_c b_w} \right) \quad (\text{ACI 10.2}) \quad (\text{B.24})$$

or

$$M_u = A_s f_y \quad d - 0.5a \quad (\text{ACI 10.2}) \quad (\text{B.25})$$

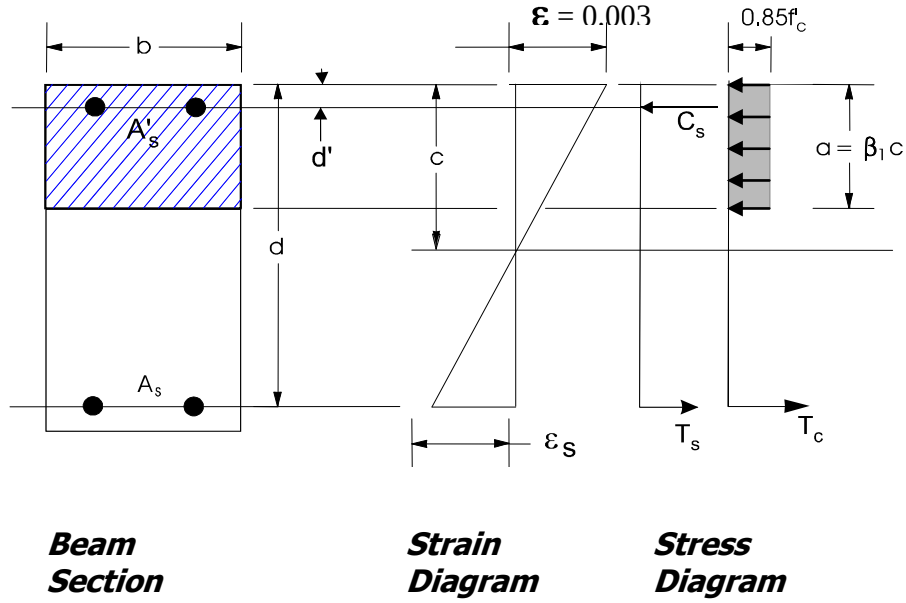


Figure B.6 Idealization of stress and strain distribution in a beam section.

where, the depth of the compression block, a ,

$$a = \frac{A_s f_y}{0.85 f'_c b_w} \quad (\text{ACI 10.2}) \quad (\text{B.26})$$

and the depth of the compression block, a , should satisfy the following conditions,

$$a_{\max} = \beta_1 c_{\max} \quad (\text{ACI 10.2.7.1}) \quad (\text{B.27})$$

$$\beta_1 = 0.85 - 0.05 \left(\frac{f'_c - 28}{7} \right), \quad 0.65 \leq \beta_1 \leq 0.85 \quad (\text{ACI 10.2.7.3}) \quad (\text{B.28})$$

$$c_{\max} = \frac{\varepsilon_{c,\max}}{\varepsilon_{c,\max} + \varepsilon_{s,\min}} d \quad \varepsilon_{c,\max} = 0.003, \varepsilon_{s,\min} = 0.005 \quad (\text{ACI 10.2.2}) \quad (\text{B.29})$$

where, A_s = area of tension reinforcement, a = the depth of compression block, $a_{\max}$ = maximum allowed depth of compression block, b_w = the width of the beam section, c = the depth to neutral axis, d = the distance from compression face to tension reinforcement, f_y = specified yield strength of flexural reinforcement, f'_c = specified compressive strength of concrete, β_1 = factor for obtaining depth of

compression block in concrete, $\epsilon_{c,max}$ = maximum usable compression strain allowed in extreme concrete fiber (0.003), $\epsilon_{s,min}$ = minimum tensile strain allowed in steel rebar at nominal strength for tension controlled behavior (0.005).

The minimum flexural tensile steel required in a beam section shall not be less than the following two limits;

$$A_{s(min)} \geq \max \left\{ \frac{\sqrt{f'_c}}{4 f_y} b_w d \quad \text{or} \quad \frac{1.4}{f_y} b_w d \right\} \quad (\text{ACI 21.3.2.1, ACI 10.5.1}) \quad (\text{B.30})$$

The flexural steel in a beam section is limited to the maximum given by

$$A_s \leq 0.025 b_w d \quad (\text{ACI 21.3.2.1}) \quad (\text{B.31})$$

For Special moment resisting concrete frames (seismic design), the beam design satisfies the following additional conditions. At any end (support) of the beam, the beam positive moment capacity (i.e., associated with the bottom steel) would not be less than 1/2 of the beam negative moment capacity (i.e., associated with the top steel) at that end (ACI 21.3.2.2). Neither the negative moment capacity nor the positive moment capacity at any of the sections within the beam would be less than 1/4 of the maximum positive or negative moment capacities of any of the beam end (support) stations (ACI 21.3.2.2).

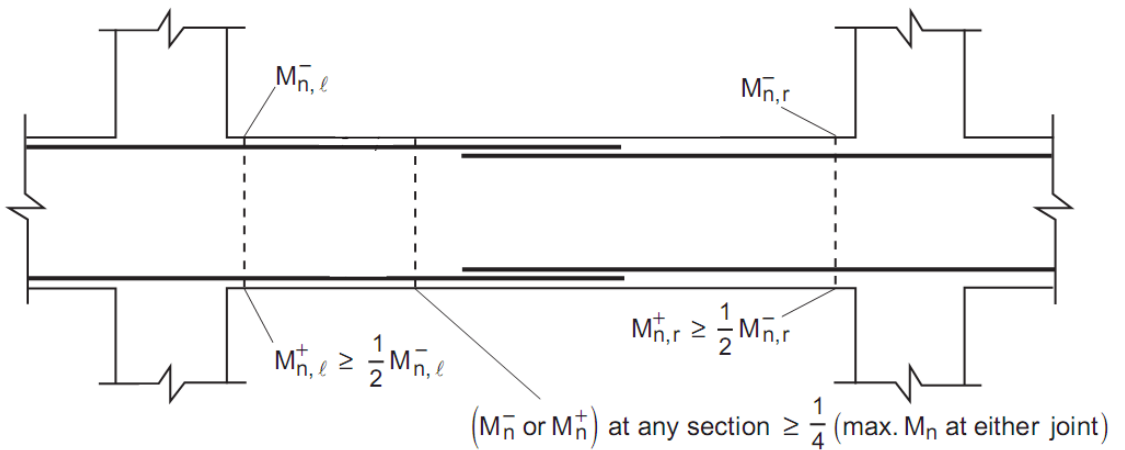


Figure B.7 Reinforcement requirements for flexural members of Special Seismic Moment Frames [30].

The shear reinforcement is designed for each design load combination at three part along the beam span. For shear design, the effects of axial forces on the beam section are neglected. For Special and Intermediate moment frames (ductile frames), the shear design of the beams is also based on the maximum probable moment strengths and the nominal moment strengths of the members, respectively, in addition to the factored design. In the design of Special moment resisting concrete frames (i.e., seismic design), the capacity shear force, V_p , is calculated from the maximum probable moment capacities of each end of the beam and the gravity shear forces. The procedure for calculating the design shear force in a beam from the maximum probable moment capacity is the same as that described for a column earlier in this section.

The design shear force is then given by (ACI 21.3.4.1, IBC 2006);

$$V_u = \max V_{e1} \quad V_{e2} \quad \text{(ACI 21.3.4.1)} \quad \text{(B.32)}$$

$$V_{e1} = V_{p1} + V_{D+L} \quad \text{(ACI 21.3.4.1)} \quad \text{(B.33)}$$

$$V_{e2} = V_{p2} + V_{D+L} \quad \text{(ACI 21.3.4.1)} \quad \text{(B.34)}$$

where, V_p , is the capacity shear force obtained by applying the calculated maximum probable ultimate moment capacities at the two ends of the beams acting in two opposite directions. V_{D+L} is the contribution of shear force from the in-span distribution of dead and live loads with the assumption that the ends are simply supported.

$$V_{p1} = (M_i^- + M_j^+) / L \quad \text{(B.35)}$$

$$V_{p2} = (M_i^+ + M_j^-) / L \quad \text{(B.36)}$$

where, M_i^- = the moment capacity at end I, with top steel in tension, using a steel yield stress value of αf_y and no reduction factors ($\phi = 1.0$), M_j^+ = the moment capacity at end J, with bottom steel in tension, using a steel yield stress value of αf_y and no reduction factors ($\phi = 1.0$), M_i^+ = the moment capacity at end I, with bottom steel in tension, using a steel yield stress value of αf_y and no

reduction factors ($\phi = 1.0$), M_J^- = the moment capacity at end J, with top steel in tension, using a steel yield stress value of αf_y and no reduction factors ($\phi = 1.0$), L = the clear span of beam.

The moment strengths are determined using a strength reduction factor of 1.0 and α is equal to 1.25 (ACI 2.1, R21.3.4.1).

After the design shear forces are calculated, the required shear reinforcement is calculated by the following design procedure for three parts of the beam.

The allowable concrete shear capacity is given by;

$$V_c = 0.17\sqrt{f'_c}b_wd \quad (\text{ACI 11.3.1.1}) \quad (\text{B.37})$$

$$\text{If } V_u \leq \phi(V_c / 2) \Rightarrow \frac{A_v}{s} = 0 \quad (\text{ACI 11.5.6.1}) \quad (\text{B.38})$$

$$\text{If } \phi(V_c / 2) \leq V_u \leq \phi(V_{\max}) \Rightarrow \frac{A_v}{s} = \frac{(V_u - \phi V_c)}{\phi f_y d} \quad (\text{ACI 11.5.7.1}) \quad (\text{B.39})$$

$$\frac{A_v}{s} \geq \max \left\{ \frac{0.062\sqrt{f'_c}b_w}{f_y}, \frac{0.35b_w}{f_y} \right\} \quad (\text{ACI 11.5.6.3}) \quad (\text{B.40})$$

$$\text{If } V_u > \phi(V_{\max}) \text{ a failure condition is declared.} \quad (\text{ACI 11.5.7.9}) \quad (\text{B.41})$$

$$V_{\max} = V_c + 0.66\sqrt{f'_c}b_wd \quad (\text{ACI 11.5.7.9}) \quad (\text{B.42})$$

At the ends of flexural members, where plastic hinges are likely to form, adequate confinement is required in order to ensure sufficient ductility of the members under reversible loads. Furthermore, the transverse reinforcement is required at the end of beam to assist the concrete in resisting shear and to maintain lateral support for the reinforcing bars. For flexural members of special moment frames,

the transverse reinforcement requirements are given in Figure B.8.

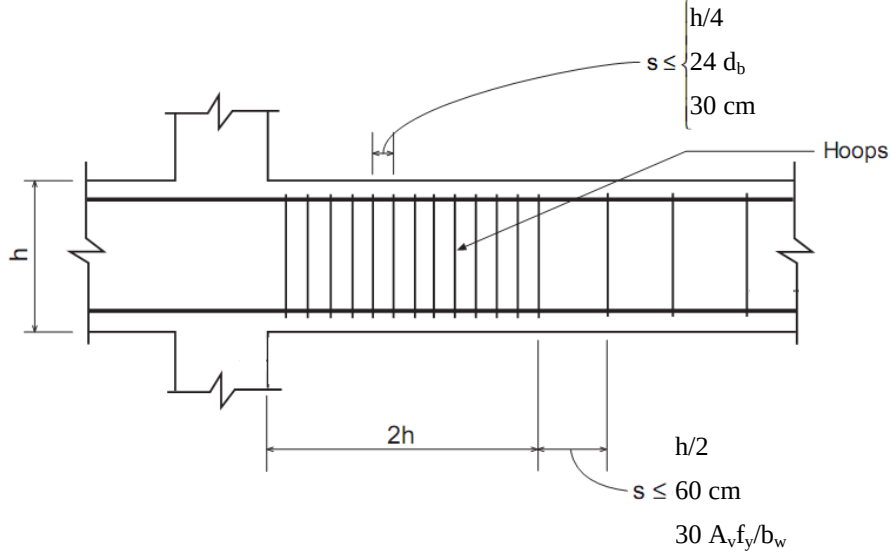


Figure B.8 Transverse reinforcement requirements for flexural members of Special Seismic Moment Frames [30].

In the shear design of the beam and column members, the diameter of shear reinforcements (ties) is taken as 8 mm, if the shear capacity of member or the shear constraints doesn't satisfy the requirements, the the diameter of shear reinforcements (ties) is taken as 10 mm.

Additionally, for Special moment resisting frames, at a particular joint for a particular column direction the ratio of the sum of the beam moment capacities to the sum of the column moment capacities should satisfy following condition (ACI 21.4.2.2);

$$\sum M_{nc} \geq \frac{6}{5} \sum M_{nb} \quad (\text{ACI 21.4.2.2}) \quad (\text{B.43})$$

$\sum M_{nc}$ = sum of the nominal flexural strengths of columns framing into the joint, evaluated at the faces of the joint; individual column flexural strength is calculated for the associated factored axial force, $\sum M_{nb}$ = sum of nominal flexural

strengths of the beams framing into the joint. The capacities are calculated with no reinforcing overstrength factor α , $\alpha = 1$, and with no ϕ factors ($\phi = 1.0$).

Spacing of lateral supports for a beam shall not exceed 50 times the width of beam, b_w . Minimum thickness of beam, h should satisfy following conditions (L_{beam} ; the length of beam).

$$b_{w,min} = \frac{L_{beam}}{50} \quad (B.44)$$

$$h_{min} = \frac{L_{beam}}{18.5} \quad \text{for one end continuous} \quad (B.45)$$

$$h_{min} = \frac{L_{beam}}{21} \quad \text{for both ends continuous} \quad (B.46)$$

Detailing of beams is arranged in respect of requirements on ACI 315-99. For non-perimeter beams, typical details in code, the development lengths of bars used at spans 0.3 times of adjacent spans. The length of reinforcement bars are calculated according to these specifications, and hook lengths at the end of some reinforcement bars is taken as $12d_b$ (d_b ; the diameter of bar).

The development lengths, L_d , for reinforcement bars at column sections are computed with following formula.

$$L_d = \frac{f_y \Psi_t \Psi_e \lambda}{2\sqrt{f_c}} d_b \quad \text{for } d_b \leq 20 \text{ mm} \quad (B.47)$$

$$L_d = \frac{5}{8} \frac{f_y \Psi_t \Psi_e \lambda}{\sqrt{f_c}} d_b \quad \text{for } d_b > 20 \text{ mm} \quad (B.48)$$

where, the material properties coefficients, Ψ_t, Ψ_e, λ are taken as 1.0.

APPENDIX C

SEISMIC LOADS (IBC 2006 / ASCE 7-05)

It is stated in section 1613 of the 2006 IBC that earthquake loads shall be determined in accordance with ASCE Standard 7-05. In ASCE 7-05, there are three options to calculate the building period. In this study, the calculation of the building period T_A is based on ASCE 7-05 Eqn. 12.8-7.

$$T_A = C_T (h_n)^x \quad (\text{Eqn. 12.8-7 in ASCE 7-05}) \quad (\text{C.1})$$

A typical range of values for C_T is 0.0466 to 0.0731, while x varies from 0.75 to 0.90. In case of moment resisting reinforced concrete frames the values of 0.0466 and 0.90 given as building period coefficient for C_T and x respectively. The height, h_n is measured from the base to the highest level of a structure.

The response modification factor R and the system over-strength factor Ω are specified in Table 12.2-1 of ASCE 7-05. A typical range of values for R is 2 to 8 and for Ω is 2 to 3. The response modification factor R is given as 5 for intermediate reinforced concrete moment frames and 8 for special reinforced concrete moment frames. The system over-strength factor Ω is 3 for both type RC moment frames. An importance factor I shall be assigned to each structure in accordance with Table 11.5-1 based on the Occupancy Category from Table 1-1. The importance factors are given in Table C.1.

Table C.1 Importance factors (Table 11.5-1 of ASCE 7-05) [31].

Occupancy Category	<i>I</i>
I or II	1.0
III	1.25
IV	1.5

The site classes are divided five parts from A to E, and the site class definitions are given Table 20.3-1 in ASCE 7-05. The seismic coefficients are specified with site class. The mapped maximum considered earthquake (MCE) spectral acceleration for short periods, S_s is determined in ASCE 7-05 Section 11.4.1. A typical range of values for S_s is 0 to 3. The seismic maps show S_s in % g with a typical range of 0 % to 300 %. Thus the map values should be divided by 100 when they are input. For example, if the map value is 125 % g it should be input as 1.25g. The mapped MCE spectral acceleration for a one second period, S_1 is determined in ASCE 7-05 Section 11.4.1. A typical range of values for S_1 is 0 to 1. The seismic maps show S_1 in % g with a typical range of 0 % to 100 %. Thus the map values should be divided by 100 when they are input. For example, if the map value is 100 % g it should be input as 1.0g. The parameters S_s , and S_1 shall be determined from the 0.2 and 1.0 s spectral response accelerations shown on Figs.22-1 through 22-14, respectively. Where S_1 , is less than or equal to 0.04 and S_s is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A and is only required to comply with Section 11.7.

The algorithm for determining ASCE 7-05 seismic loads is based on Section 12.8 (Equivalent Lateral Force Procedure) of ASCE 7-05. The algorithm begins by calculating the design spectral response acceleration at short period S_{DS} using (Eqs. 11.4-1 and 11.4-3 of ASCE 7-05). And the design spectral response acceleration is calculated at one second period, S_{D1} , using (Eqns. 11.4-2 and 11.4-3 of ASCE 7-05).

$$S_{DS} = \frac{2}{3} F_a S_s \quad (\text{ASCE 7-05 Eqn. 11.4-1 and 11.4-3}) \quad (\text{C.2})$$

$$S_{D1} = \frac{2}{3} F_v S_1 \quad (\text{ASCE 7-05 Eqn. 11.4-2 and 11.4-3}) \quad (\text{C.3})$$

The short-period site coefficient (at 0.2 s-period), F_a is determined in accordance with code (from the site class) and S_s based on Table 11.4-1 of ASCE 7-05. A typical range of values for F_a is 0.8 to 2.5. The long-period site coefficient (at 1.0 s-period), F_v is determined in accordance with code (from the site class) and S_1 based on ASCE 7-05 Table 11.4-2. If site coefficients are user defined, F_v is input directly by the user. A typical range of values for F_v is 0.8 to 3.5. The long-period transition period, T_L is determined in Section 11.4.5 of ASCE 7-05. The site coefficients F_a and F_v are defined in Table C.2 and Table C.3.

Table C.2 Site coefficient, F_a (Table 11.4-1 of ASCE 7-05) [31].

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_s \leq 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1.0$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

Table C.3 Site coefficient, F_v (Table 11.4-2 of ASCE 7-05) [31].

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

A seismic design category is determined based on S_{DS} using Table 11.6-1 of ASCE 7-05. A seismic design category also is determined based on S_{D1} using

Table 11.6-2 of ASCE 7-05. The more severe of these two seismic categories is chosen as the seismic design category for the building. Initially a seismic response coefficient C_s is calculated using (Eqn. 12.8-2 of ASCE 7-05). This base shear value is then checked against the limits specified in (Eqns. 12.8-3, 12.8-4, 12.8-5, and 12.8-6 of ASCE 7-05) and modified as necessary to obtain the final base shear.

$$C_s = \frac{S_{DS}}{R \frac{I}{I_0}} \quad (\text{ASCE 7-05 Eqn. 12.8-2}) \quad (\text{C.4})$$

where S_{DS} = the design spectral response acceleration at short periods, R = the response modification factor specified in Table 12.2-1 of ASCE 7-05, I = the occupancy importance factor determined in accordance with Table 11.5-1 of ASCE 7-05.

The seismic response coefficient, C_s , should not exceed the value specified in (Eqn. 12.8-3 of ASCE 7-05). Also, C_s shall not be less than 0.01 (Eqns. 12.8-5 of ASCE 7-05). In addition, for structures located in a region where S_1 is equal to or greater than 0.6g, C_s , shall not be less than value that specified in (Eqns. 12.8-6 of ASCE 7-05).

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I}\right)T} \quad \text{for } T \leq T_L \quad (\text{Eqn. 12.8-3 of ASCE 7-05}) \quad (\text{C.5})$$

$$C_s = \frac{S_{D1}T_L}{\left(\frac{R}{I}\right)T^2} \quad \text{for } T > T_L \quad (\text{Eqn. 12.8-4 of ASCE 7-05}) \quad (\text{C.6})$$

$$C_s \geq 0.01 \quad (\text{Eqn. 12.8-5 of ASCE 7-05}) \quad (\text{C.7})$$

$$C_s = \frac{0.5S_1}{\left(\frac{R}{I}\right)} \quad S_1 \geq 0.6g. \quad (\text{Eqn. 12.8-6 of ASCE 7-05}) \quad (\text{C.8})$$

where, S_{D1} = the design spectral response acceleration at a one second period, T =

the building period used for calculating the base shear, T_L = the long-period transition period, S_1 = the mapped MCE spectral acceleration for a one second period.

The seismic base shear, V , in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \quad (\text{Eqn. 12.8-1 of ASCE 7-05}) \quad (\text{C.9})$$

where, C_s = the seismic response coefficient determined in accordance with Section 12.8.1.1, W = the effective seismic weight as per Section 12.7.2.

The base shear, V , is distributed over the height of the building in accordance with (Eqns. 12.8-11 and 12.8-12 of ASCE 7-05). The lateral seismic force induced at any level (F_{story}) shall be determined from the following equations:

$$F_{story} = \frac{V w_{story} h_{story}^k}{\sum_{story=1}^n w_{story} h_{story}} \quad (\text{Eqn. 12.8-11 and 12.8-12 of ASCE 7-05}) \quad (\text{C.10})$$

where, F_{story} = the portion of base shear applied to a story level, V = Building base shear, w_{story} = the weight of story level (based on specified mass), h_{story} = the story height, distance from base of structure to story level, k = Exponent applied to building height. (The value of k depends on the value of the building period, T , used for determining the base shear. If $T \leq 0.5$ seconds, $k = 1$. If $T \geq 2.5$ seconds, $k = 2$. If 0.5 seconds $< T < 2.5$ seconds, k is linearly interpolated between 1 and 2.), n = the number of story levels in the structure.

The design story drift at level (story) x , (Δ_x) shall not exceed the allowable story drift (Δ_a) as obtained from Table C.4 for any story. The story drift is calculated with following equation:

$$\Delta_x = \frac{C_d \delta_{xe}}{I} \quad (\text{Eqn. 12.8-15 of ASCE 7-05}) \quad (\text{C.10})$$

where, δ_x = the relative deflections of level (story) x , C_d = the deflection amplification factor ($C_d = 5.50$ for special seismic RC moment frames.), I = the occupancy importance factor determined in accordance with Table C.1 (Table 11.5-1 of ASCE 7-05).

Table C.4 Allowable story drift (Table 12.12-1 ASCE 7-05) [31].

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}^c$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures ^d	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

where, h_{sx} = is the story height below level x .

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PUBLICATIONS

1. Akın A., Saka M.P. "Optimum Design of Short RC Columns Subjected to Axial Force and Uniaxial/Biaxial Bending Using Harmony Search Algorithm", 9th International Congress on Civil Engineering, September 2010, Trabzon, Turkey.
2. Akın A., Saka M.P. " Optimum Design of Concrete Cantilever Retaining Walls Using Harmony Search Algorithm", The Tenth International Conference on Computational Structures Technology, 14-17 September 2010, Valencia, Spain.

HOBBIES

Football, Swimming, Gourmet, Computer Technologies, Movies,