HUMAN ERRORS IN STRUCTURAL DESIGN AND CONSTRUTION

IN THE UINTED ARAB EMIRATES

by

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Abstract

The construction sector in the United Arab Emirates has witnessed an unparalleled growth in the past decade. In spite of recent challenges due to global recession, this sector remains strong in the Gulf region. Experience has shown that higher than-expected human errors are committed during the structural design and construction phases in the country. To address this problem, engineers have been surveyed on common errors, deterministic and reliability-based sensitivity analyses are performed, and design and construction checklists have been developed. The survey has shown that the engineers perceived different frequency of errors, depending on the years of experience and nature of work, and contractors are more reluctant than designers to report errors. Results of the sensitivity analysis showed that variations in concrete strength have minor effect on the reliability of beams in flexure, moderate effect on the reliability of beams in shear and severe effect on the reliability of columns in axial compression. Changes in the steel reinforcement yield strength, on the other hand, have great effect on the reliability of beams in flexure, moderate effect on the reliability of beams in shear and mild effect on the reliability of column in axial compression. The effective steel reinforcement depth, on the other hand, is critical to both flexural strength and shear strength, and cross-sectional dimensions of a column are very important factors to the axial compression capacity. The effect of variations in design variables on the capacity of reinforced concrete portal frame systems was investigated using static pushover analysis with the aid of Zeus-NL software. Results of the nonlinear analysis showed that the sensitivity of structural systems is highly dependent on the number of bays and stories the frame structure consists of. Moreover, reinforced concrete frames are most sensitive to reductions in the yield strength of the longitudinal steel reinforcement of the beams and columns. Finally, design and construction checklists were developed in order to have another layer of quality control and help reduce the frequency of human errors in structural design and construction. The checklists cover a wide range of design and construction activities related to both substructure and superstructure work.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

The United Arab Emirates, UAE, is located in the Middle East between Oman and Saudi Arabia, bordering the Gulf of Oman and the Persian Gulf. It includes 7 states, termed emirates, which are Abu Dhabi, Ajman, Dubai, Fujairah, Ras al-Khaimah, Sharjah and Umm al-Quwain. The country's population has grown rapidly in recent years, reaching 8.26 million in mid-2010, which represents a growth of 64.5% in 4 years. The UAE comprises an area equal to 83,600 km^2 and a 1318 km long coastline. The geography is mainly flat, barren coastal plain integrating into vast desert wasteland, with mountainous terrain in the east. It has a hot and dry climate, with minimal rainfalls. The important natural resources of the country are oil and natural gas [1].

The UAE economy has been among the most dynamic and fast-growing economies in the region for the past few years. The UAE government strategy to diversify its economy from an oil-and-gas to other industries has resulted in an unprecedented development in construction across all emirates [2]. The UAE's economic report for 2009 indicated that the construction sector was the second largest in the country, contributing about 10% to the UAE's gross domestic product [3].

Despite the sluggish growth in 2009 amidst the global financial crises, the UAE is still the biggest construction market in the Gulf Cooperation Council, GCC, with $715 billion worth of construction projects currently either in the planning stage or under way [3]. The UAE seems to be committed to further develop its projects in housing, tourism, industrial facilities, education, healthcare sectors, transportation, utilities, communications, ports and airports. Such activities are expected to change the face of urban landscape in the coming years.

Recent drop in property sale prices has lifted long-term demand, while new regulations governing property ownership have enhanced the image of the construction and real estate sectors. Additionally, some companies have switched their focus from private to public sector schemes and formed joint ventures to secure contracts in an increasingly competitive environment, resulting in a workface dominated by expatriates who have different qualifications, knowledge and experience.

1.2 Problem Statement

The fast growth in the construction sector had not come without a price. Rarely does a month pass without reading in the local newspapers or regional construction magazines about building failures and collapses in the UAE that led to fatalities and injuries due to human errors. In fact, this has been occurring very frequently to the extent that the public has accepted such tragic incidences as an “act of God” or “act of nature.” A quick web search on structural failures in the UAE in the last decade is documented in Table 1.

One of the reasons behind this fact is because the workface in the UAE is currently dominated by foreign consultants and contractors who lack the in-depth knowledge of the local design practices and construction environment. As a result, the quality of design and construction of some projects has been compromised, especially during construction.

Although local municipalities and departments of public work have in place quality control schemes, not to mention that they review the consultant’s and contractor’s work, they often have little time to go thoroughly over design calculations, reports and structural/shop drawings. Consequently, higher than-expected human errors committed during the design and construction phases are inevitable in this fast-growing environment. While experience has shown that most committed errors are on the conservative side, there are some errors that reduce the structural safety and reliability of the structure.

Based on the above, it becomes necessary to find an approach that efficiently measures the effects of human errors on structural reliability in the UAE, and assist in providing guidelines to control and improve the safety of constructed facilities during and after construction.

Table 1: Some failures from the past decade in the UAE reported by the media

Date	Place	Event / Article Title	Reference
Jan 22, 2012	Dubai	Four workers dead in UAE scaffolding collapses	http://www.constructionweekonline.com/article-15366-four-workers-dead-in-uae-scaffolding-collapses/
July 7, 2011	Ras Al Khaimah	3 workers hurt in roof collapse in Ras Al Khaimah	http://gulfnews.com/news/gulf/uae/emergencies/3-workers-hurt-in-roof-collapse-in-ras-al-khaimah-1.834519
August, 1, 2010	Abu Dhabi	Two killed by falling concrete slab in Abu Dhabi	http://www.constructionweekonline.com/article-8852-two-killed-by-falling-concrete-slab-in-abu-dhabi/
October 12, 2009	Sharjah	6 workers injured in Sharjah building collapse	http://www.arabianbusiness.com/570135-workers-injured-in-sharjah-building-collapse
October 3, 2009	Ajman	Five Injured in Ajman Building Collapse	http://www.khaleejtimes.com/DisplayArticle08.asp?xfile=data/theuae/2009/October/theuae_October56.xml&section=theuae
September 24, 2009	Ras Al-Khaimah	Worker dies in RAK building collapse	http://www.thenational.ae/apps/pbcs.dll/article?AID=/20090924/NATIONAL/909249980/1010/news
August 16, 2009	Dubai	Workers flee Dubai building collapse	http://www.arabianbusiness.com/565001-six-story-building-collapses-in-dubai
March 26, 2009	Dubai	Building collapses at Dubai Industrial City	http://www.constructionweekonline.com/article-4736-building_collapses_at_dubai_industrial_city/
November 11, 2008	Dubai	Crane collapse probe ongoing - RTA	http://www.arabianbusiness.com/537822-crane-collapse-probe-ongoing---rta
September 4, 2008	Abu Dhabi	Two labourers killed in well collapse	http://www.arabianbusiness.com/530009-two-labourers-killed-in-well-collapse
August 16, 2008	Dubai	Ibn Battuta ceiling fall still being investigated	http://www.arabianbusiness.com/527640-ibn-battuta-ceiling-fall-still-being-investigated
June 18, 2008	Dubai	Dubai bridge collapses	http://www.arabianbusiness.com/522347-dubai-bridge-collapses
June 2, 2008	Ajman	Six killed in Ajman hotel collapse	http://www.arabianbusiness.com/520946-six-killed-in-ajman-hotel-collapse
December 22, 2007	Dubai	Quay wall collapse hits D1 Tower site	http://www.arabianbusiness.com/506644-quay-wall-collapse-hits-d1-tower-site
December 13, 2007	Dubai	Collapse of sea wall causes Dubai site to flood	http://www.building.co.uk/story.asp?storycode=3102195
December 8, 2007	Dubai	Casualties in Marina scaffolding collapse	http://www.constructionweekonline.com/article-2069-casualties_in_marina_scaffolding_collapse/
November 8, 2007	Dubai	RTA says human error caused bridge collapse	http://www.arabianbusiness.com/503756-rta-says-human-error-caused-bridge-collapse
April 21, 2007	Ajman	Worker dies in wall collapse	http://www.arabianbusiness.com/11218-worker-dies-in-wall-collapse
February 8, 2007	Dubai	Lucky escape for 100 workers	http://gulfnews.com/news/gulf/uae/general/lucky-escape-for-100-workers-1.160417
January 27, 2007	Sharjah	Drilling reason for scaffolding collapse	http://www.arabianbusiness.com/6722-drilling-reason-for-scaffolding-collapse
January 20, 2007	Dubai	Site accident kills worker	http://www.arabianbusiness.com/6454-site-accident-kills-worker
October 20, 2006	Dubai	Fichtner employee goes to prison after Dewa collapse	http://www.constructionweekonline.com/article-79-fichtner_employee_goes_to_prison_after_dewa_collapse/1/print/
May 20, 2006	Dubai	Another worker dies on JBR site	http://www.arabianbusiness.com/486426
September 28, 2004	Dubai	5 Workers Die in Dubai Airport Collapse	http://www.arabnews.com/?page=4&section=0&article=52108&d=28&m=9&y=2004
February 13, 2004	Sharjah	Six workers killed in UAE building collapse	http://www.abc.net.au/news/stories/2004/02/13/1043993.htm
August 27, 2002	Dubai	Persons killed in building collapse in Jabal Ali, Dubai	http://www.arabicnews.com/ansub/Daily/Day/020827/2002082715.html
August 26, 2002	Dubai	Dubai Building Collapse Kills Seven	http://www.highbeam.com/doc/1P1-55659044.html

1.3 Objectives of the Study

This research addresses an important topic related to structural failures in the UAE. It aims to reduce cases of collapse of structures due to human errors in structural design and construction in the country. The results of the study will serve as a basis for an error control strategy and decision making in design and construction.

The main objectives of this study are to:

1. Survey local practicing construction companies, local structural design companies, and UAE municipalities on the common human errors committed during design and constructions stages.
2. Determine the most critical design parameters affecting the nominal capacity of structural elements using a deterministic approach.
3. Determine the most critical design parameters affecting the reliability index of structural elements using a nondeterministic approach.
4. Investigate the most critical design parameters affecting the lateral drift of reinforced concrete portal frame systems under the effect of gravity and lateral loading using static pushover analysis.
5. Develop checklists for different design and construction activities to help reduce human errors committed during design and construction stages.

1.4 Organization of the Thesis

This Chapter serves as an introduction to the research outlined in this thesis. It includes the problem statement, objectives of the study, and a brief summary of the content of each chapter.

Chapter 2 addresses the definition of human errors, as well as some previous studies made on human errors. Chapter 3 talks about structural reliability methods and on how to calculate the reliability index.

Chapter 4 presents the results that were obtained after surveying practicing construction and structural design companies working in the UAE, as well as different UAE municipalities. These results are classified with respect to the number of years of experience, average frequency, percentage frequency, and whether the engineer is a consultant or a contractor.

Chapter 5 covers deterministically-based sensitivity analysis for structural members (Beams under flexure, beams under shear, and axially loaded columns).

Chapter 6, on the other hand, addresses the topic of reliability-based sensitivity analysis. The same procedures followed in Chapter 5 are repeated in Chapter 6, but considering the effect of changes in design variables on the reliability index.

After that, a comparison is made in Chapter 7 between the results of deterministic and reliability approaches.

In Chapter 8, the sensitivity analysis is performed for the structural system as a whole. Static pushover analysis is used to investigate the reduction in the load factor due to changes in design variables.

Chapter 9 presents the checklists developed to reduce human errors committed during different design and construction stages.

The thesis closes with Chapter 10, which summarizes the research work and provides final conclusions derived from the various activities that were conducted in the previous chapters.

Three appendices complete the thesis. Appendix A includes the survey forms in English and Arabic, Appendix B comprises the structural design checklist sheets, and Appendix C contains the construction checklist sheets.

CHAPTER 2

REVIEW OF HUMAN ERRORS IN DESIGN AND CONSTRUCTION

2.1 Human Errors Definition

One of the earliest works on structural failures is Hammond's book "Engineering Structural Failures" published in 1956 in England. The book details the causes and results of modern failures of different structural forms and materials, particularly in the substructure [4]. While most of the chapters were dedicated to civil engineering, Hammond covered other topics related to naval architecture, aircraft engineering, and welding.

In the United States, the American Concrete Institute published in 1964 a monograph on structural failures with special emphasis on design deficiencies and inadequate construction practices [5]. The report showed that the most common design shortcomings leading to failures were attributed to errors in assumptions in loading and flexural conditions, poor detailing and drafting, lack of attention to connections between members, improper location and spacing of reinforcing bars and splices, and ignorance of treatment of thermal and shrinkage effects. Among the construction conditions that cause failures were insufficient supervision and inspection, poor concrete mixing and placing practices, and inexperience with concreting in extreme weather conditions.

However, the earliest research on the effect of human errors on structural safety dates back to the 1970s. However, recognition of human errors as a major contributor to structural failures was first acknowledged in 1979 when the calculated probability of failure of structures was found to be much lower than the actual rate of structural collapses. Since then, major studies on this subject were conducted around the world, particularly in the US, Europe and Australia [6]. Due to sensitivity of this issue, the Middle East lacks behind other countries with regard to reveals on human errors in structural design and construction.

Few definitions of human errors in structural engineering have been suggested in the past years. Nowak and Carr classified the structural uncertainty causes into two major categories [7]. The first category is the variation within acceptable technical practice, defined as acceptable if it was found acceptable by significant number of the most knowledgeable engineers. This variation includes:

- Natural hazards such as failures due to earthquakes or wind,
- Manmade hazards, including failure due to fire or vehicle collision,
- Variation within common practice, such as using end hooks on rebars placed around an opening in a floor, and
- Departure from common practice, such as the use of FRP to reinforce an opening in a floor.

The second category is the departure from the acceptable technical practice, which can be considered as Human Errors. Hence, human errors are defined as the departure from acceptable technical practice in design and construction that causes variation in strength, loading parameters or both beyond the acceptable limits [8].

Modeling human errors using structural reliability theory has many advantages. Firstly, the structural reliability theory provides a better understanding of the errors characteristics and their mechanism of occurrence and detection. Secondly, it provides a better estimation of the structural probability of failure. Thirdly, modeling human errors rationalizes the decision making related to quality control during design, construction and service lifetime of structures. Finally, the existing methods of control can be improved significantly and thus, new control strategies can be developed [9].

One approach to address the effect of human error on structural reliability was suggested by Nowak and Tabsh [10] in 1989, in which the authors used reliability-based sensitivity analysis. In that approach, the authors determined the effect of under-strength on the reliability index. They demonstrated their method on reinforced concrete, structural steel and timber beams.

2.2 Previous Studies on Human Errors

There has been a strong interest in the last 30 years to integrate human errors in the reliability methods. Frequencies, consequences and circumstances surrounding human error occurrence should be available before analyzing human errors [7]. Several models were suggested to account for human errors in structural reliability theory.

In a 1977 landmark study, Matousek [11] published the results of a comprehensive survey on 800 European structural failures. The analyzed failures totaled 40 million US Dollars in direct damage, 592 injured people, and 504 killed. The survey indicated that the cause of failures can always be related to human errors attributed to the misjudgment of unfavorable influences such as natural environment, construction procedures, and material properties. A similar survey of errors committed in the design and construction of concrete structure in North America was conducted by Fraczek [12]. In that study, 277 cases of human errors in concrete structures were reported. The survey showed notable agreement with the findings of the European Survey, as reported by Hauser [13] and Melchers et al. [14]. Hauser also found out that only very few errors are unavoidable, and in a majority of cases a small number of additional checks can help considerably in reducing mistakes by engineers and contractors.

Later in 1981, the Construction Industry Research and Information Association published the results of an investigation of 120 building failures in the United Kingdom [15]. The study indicated that failures caused by the serviceability limit state are primarily a result of extensive cracking and large deformations due to excessive settlement, shrinkage, creep, or thermal movement. A large number of failures appeared to be due to errors in the development of the structural system and resulting in inadequate load paths. Deficient erection procedures during construction were also responsible for many of the reported collapses.

An analysis of 604 structural failures in the United States during the period 1975-1986 was carried out by Eldukair and Ayyub in 1991 [16]. The study showed that the important causes of errors in the building process were technical procedures, management practices, and environmental effects. The common errors that are committed during construction were mainly attributed to inadequate coordination and communication procedures between the consultants and contractors. The authors concluded that the problem of failures is mainly related to deficiencies in checking and inspection procedures and has little to do with the lack of refinement of codes of practice or quality control of materials and construction work procedures.

Nowak and Carr classified errors according to their causes [17]. They used an error survey that was sent to design offices and construction companies. They also interviewed persons about the errors detected by themselves, their colleagues or by the checkers and supervisors. Three types of errors were recognized in the study: conceptual errors, execution errors, and intention errors.

Two-hundred and twenty-five building failures in the United States from 1989 to 2000 were documented by Wardhana and Hadipriono [18]. The result showed that the majority of the failures occurred in low-rise buildings and apartments. The major causes of failure were grouped into 3 categories, i.e. external events, construction deficiencies and maintenance deficiencies. External events include rain, wind, snow, vehicular impact, and collision. Construction deficiencies cover improper renovation, unplanned demolition, poor workmanship, and unsafe excavation operations. Maintenance deficiencies are associated with building deterioration that was either ignored or improperly repaired.

Atkinson addressed the human error causes of defects by considering the human factors related to failures [19]. He surveyed construction industry practitioners, investigated many house building sites and conducted unstructured interviews. The study showed that managerial influences underlie many errors leading to defects. The subject of variability in structural engineering in common practice was discussed by Saffarini [20]. Human error in structural design was reviewed, and correlation was observed between the level of knowledge of designers and reported structural failures. An attempt to model mathematically human errors in structural design and their effect on structural safety was carried out by Melchers and Stewart [21]. The model allows for better prediction of the actual reliability of a given structure. For serviceability limit states, the approach is used to improve the efficiency of inspection/maintenance scheduling.

The impact of human error on the safety of nuclear power plants and petro-chemical industries was studied by Shibata [22]. He addressed the operation stage under normal and strong earthquake conditions and the transient stage after the earthquake, with emphasis on human operability during earthquake conditions. Frangopol utilized probabilistic models for structural safety evaluation with the presence of human errors affecting both the random structural resistance and the random load effects [23]. He proposed a methodology for integrating errors, both non-conservative and conservative, in structural safety evaluation.

A probabilistic procedure is presented by Haldar [24] to address fabrication and construction deficiencies in civil engineering projects. He demonstrated that a project with higher quality workmanship may require a smaller sample size to extract the desired probabilistic characteristics. Thus, creative repair work could be used to satisfy all concerned parties and save millions of dollars.

An attempt to model mathematically human errors in structural design and their effect on structural safety was carried out by Melchers and Stewart [25]. The model allows for better prediction of the actual reliability of a given structure. Frangopol [26] used probabilistic models to estimate the structural risk resulting from the presence of human errors affecting the random structural resistance and load effects. Both fundamental studies and frameworks for applications are presented. He used a simple discrete model associated with a reliability index format to account for the effect of human errors. The framework for application was provided by studying the impact of human errors on the reliability of reinforced concrete beams in bending. Nowak and Tabsh [27] used reliability-based sensitivity functions to quantify the effect of human error in structural design and construction on the structural safety and also to develop an error-control strategy.

Lopez et al. stated that design errors are the main reason behind accidents that result in death and injuries. They addressed three types of errors, including performance based errors, knowledge based errors and intentional errors. They attributed these errors to the inadequate training of designers, ineffective utilization of design software, inadequate quality assurance and ineffective coordination between the design team [28].

Among the regional publications in the Middle East is a survey conducted by Bayazeed [29] on 30 existing buildings and 40 under construction in the city of Makkah, Saudi Arabia. He found that 22 of the existing buildings and almost all those under construction had structural defects. The common failures were serviceability related problems such as abnormal cracking and excessive deflection. Furthermore, wrong placement of reinforcing steel and improper concreting practices were the most dominant types of errors in the field.

Arafah et al. [30] studied building failures from 125 case histories in Saudi Arabia and identified their major causes and the resulting consequences. They pointed to the inadequacy of the local construction practice in fulfilling the minimum safety and serviceability requirements in the country. The study showed that the shortcomings are due to substandard professional practice, acts of omission and commission of negligence, and development of deficient specifications. The results indicated that there is an urgent need for formulation of an efficient code of practice to provide effective guidance and exercise control over professional conduct.

Al-Kaabi and Hadipriono [31] surveyed and interviewed 120 contractors in the UAE to determine how safe construction companies operate their site activities. The common types of accidents included fall from high elevation, striking by equipments/objects, as well as natural causes such as illness, electrocution, mishandling by equipment, drowning, and burning from fire. The findings suggest that several companies are deficient in providing worker's benefits, site orientation, personal protective equipment, accident prevention schemes, health and hygiene. Insufficient training and communication problems of foreign construction workers also contributed to the occurrence of these accidents. Lack of regulations and deficient codes required for safe construction operations aggravate the problems.

Love and Josephson [32] examined the task of error recovery in detecting human errors with reference to building construction projects. Over 2,500 human errors were identified and examined. It was shown that error reduction lies in improving communication between participants, introducing incentives to the involved parties, improving resourcing levels in projects during design, and the encouragement of individual and organizational learning.

Tabsh investigated errors in testing of concrete cylinders and cubes [33] and found out that nonuniform strain gradients imposed by uncalibrated test machines may show lower-than-actual concrete strength by up to 20%. Tabsh et al. [34-35] also examined common anomalies in drilled shaft foundation and showed that defects smaller in size than 15% of the cross-sectional area cannot be reliably detected by NDT methods.

Recently, Tabsh and Al Rahmani studied the structural failures that had occurred in the UAE in the past decade [36]. The study showed that 41 serious structural and construction failures took place in the UAE between 2001 and 2010. Most of the construction failures were due to improper shoring and weak scaffolding due to negligence, which means that some structural failures could have been avoided. In addition, it was shown in the study that the frequency of failures was higher during the construction boom (2005-2007) than any other period due to pressure imposed on consultants and contractors to deliver their projects in a short period of time. Also, 60% of the considered structural failures occurred in Dubai, which until recently has witnessed most of the construction activities. Furthermore, the study indicated that 90% of the failures were related to construction, while the remaining 10% of the failures were related to design.

CHAPTER 3

BACKGROUND ON STRUCTURAL RELIABILITY METHODS

3.1 Introduction

Reliability methods were first introduced in the 1940's during the World War II for military applications. These methods were developed further due to the increasing needs in electronics, mechanical and aerospace engineering during the 1950's. Still, the first paper on the application of probability theory of the civil engineering structures was written by A.M. Freudenthal in 1956 and titled "Safety and Probability of structures Failures" [37], whereas the first mathematical safety measure of uncertainties was introduced by Cornell in 1969 [38].

Structural reliability is defined as the probability that a structure will not attain the ultimate or serviceability limit states during a specified reference period. Structural reliability, in other words, is the ability of the structure to fulfill its design purpose for some specified time. Structural reliability theory is mainly concerned with the treatment of uncertainty related to structural safety and serviceability, not to mention that it provides rational approach to modeling uncertainties due to natural variation of loads and resistance parameters within acceptable limits [38]. Structural reliability theory is also essential in the development of LRFD-based design codes, since it provides a convenient tool to optimize the design load and resistance factors. Furthermore, structural reliability theory provides basic principles for the structural quality control strategies, including checking, inspection, loading control and inspection, so as to control human errors during design and construction processes and hence, bring the variations in structural strength and loading parameters within the allowable limits.

Modeling human errors using structural reliability theory has many advantages. Firstly, the structural reliability theory provides a better understanding of the errors characteristics and their mechanism of occurrence and detection. Secondly, it provides a better estimation of the structural probability of failure. Thirdly, modeling human errors rationalizes the decision making related to quality control during design, construction and service lifetime of structures. Finally, the existing methods of control can be improved significantly and thus, new control strategies can be developed [38].

The evaluation of structural reliability starts with identifying the limit state functions. Limit state is defined as the boundary between safety and failure beyond which the structure can no longer function. Limit state functions are divided into ultimate and serviceability limit states. The ultimate limit state is the boundary beyond which the structural members might collapse. The failure in the ultimate limit state usually occurs at extreme loading such as flexure, buckling and torsion. Serviceability limit state, on the other hand, is the limit that can result in damage accumulation or discomfort to users once it is attained. For the serviceability limit state, failure often occurs at service loading. Examples of the serviceability limit state are the excessive cracks, excessive vibrations and fatigue. It is worth mentioning here that analysis of different limit states should identify all failure modes, and determine all acceptable levels of safety against attaining any limit state [39].

The second step of the evaluation of structural reliability is the formulation of the limit state functions. In fact, the formulation of limit state function is not an easy task. The difficulty comes mainly from the definition of structural failure. Probabilistic studies have provided several useful principles for modeling theses parameter. For example, normal distribution is suggested for variation of structural dimensions, whereas material properties are modeled by lognormal distribution. After that, related and random design parameters should be identified and modeled. Finally, a reliability levels that suits the required accuracy must be selected [39].

Methods of structural reliability analysis can be classified on the basis of the type of approximations that are made. Current methods for checking the safety of structures tend to fall within Level III, Level II or Level I. These levels are explained below [7]:

- Level III: Methods that determine the exact probability of failure. They include safety checking methods based on probabilistic analysis for the whole structural system. Simulation methods may be applied, such as Monte-Carlo techniques, which allow to evaluate the failure probability for various distributions.
- Level II: Methods that involve approximate iterative procedures to find the probability of failure. They are set of methods that incorporate safety checks only

at selected points on the failure boundary. Level II models are used by building code committees to develop rational sets of partial safety factors for use in Level I codes.

- Level I: Methods derived from level II. They involve the use of partial safety factors to provide the necessary level of structural reliability.

Level II methods are commonly used to find the probability of failure. They are set of methods that incorporate safety checks only at selected points on the failure boundary. This level, which will be considered in the study, starts with the development of a limit state function, defined as the boundary beyond which a structural member can no longer function. The margin of safety, G, is the difference between the resistance of the structural member and the imposed load effects. For a member subjected to gravity loads only, it is represented by the function [39]

$$G = R - Q \tag{3.1}$$

where R is the load carrying capacity, whereas Q is the load effect.

Safety can be measured in terms of a reliability index, β, defined as:

$$\beta = \frac{\mu_G}{\sigma_G} \tag{3.2}$$

in which μ_G and σ_G are the mean and standard deviation of G, respectively.

Figure 1 shows a typical probability density function for a safety margin, including the definition of probability of failure and graphical representation of the reliability index.

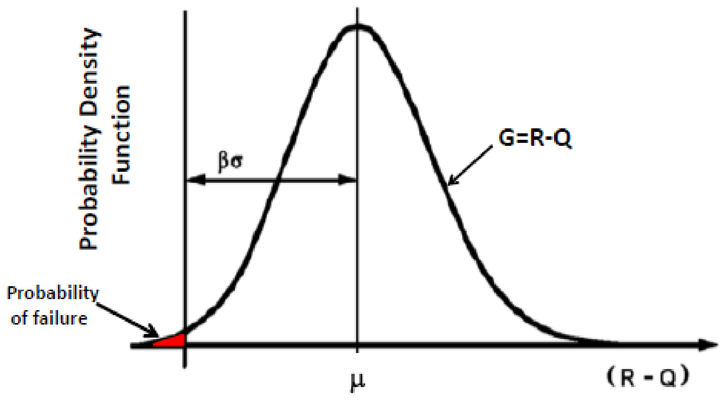

Figure 1: Definition of probability of failure and reliability index [38]

For the case of normally distributed load and resistance variables, β is computed from:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \tag{3.3}$$

where μ_R and $\mu_Q$are respectively the means of the resistance and load effect, and σ_R and σ_Q are respectively the standard deviations of the resistance and load effect.

For the case of log-normally distributed load and resistance variables, β is computed from:

$$\beta = \frac{ln(\mu_R/\mu_Q)}{\sqrt{V_R^2 + V_Q^2}} \tag{3.4}$$

where V_R and V_Q are respectively the coefficients of variation of the resistance and load effect. Table 2 shows the relationship between reliability index b and probability of failure for the cases of all variables normally distributed or longnormally distributed.

Table 2: Relationships between β and P_f for different probability distributions

Normal Distribution		Lognormal Distribution	
β	P_f	β	P_f
2.5	$0.62x10^{-2}$	2.5	$0.99x10^{-2}$
3.0	$1.35x10^{-3}$	3.0	$1.15x10^{-3}$
3.5	$2.33x10^{-4}$	3.5	$1.34x10^{-4}$
4.0	$3.17x10^{-5}$	4.0	$1.56x10^{-5}$
4.5	$3.40x10^{-6}$	4.5	$1.82x10^{-6}$
5.0	$2.90x10^{-7}$	5.0	$2.12x10^{-7}$
5.5	$1.90x10^{-8}$	5.5	$2.46x10^{-8}$

Statistics for the load and resistance variables for reinforced concrete building components are often taken from the available literature [40], and shown in Table 2. The bias factors, or mean-to-nominal ratios, for the resistance variables in the table are based on the nominal values in the ACI 318 code [41]. In this study, the variables are considered to uncorrelated.

Table 3: Statistics of building load and resistance variables [40]

Load or Resistance Variable	Bias	Coefficient of Variation	Distribution
Dead Load	1.05	0.1	Normal
Arbitrary point-in-time live load	0.24	0.65	Gamma
Maximum 50-year live load	1.00	0.18	Extreme Type-I
Flexural capacity of RC beam	1.19	0.089	Lognormal
Shear capacity of RC beam	1.23	0.109	Lognormal
Axial Compression capacity of tied column	1.26	0.107	Lognormal

In order to understand the procedure used to quantify structural safety, the flexural limit state of a simply supported reinforced concrete beam, shown in Figure 2, is considered. The beam will be used in a residential building to support a specified live load, plus the self-weight.

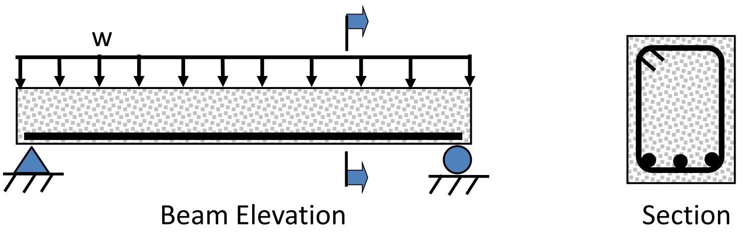

Figure 2: Beam considered in illustrating the reliability concept

The beam can be designed, constructed and then tested in the lab, as shown in Figure 3. In general, the actual capacity will be close to, but not the same as, the nominal (calculated) value specified in the structural design code (e.g. ACI 318-08).

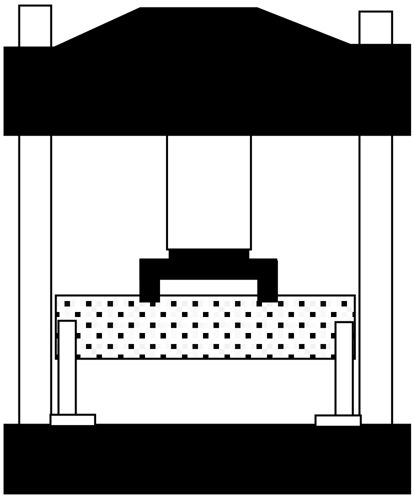

Figure 3: Testing of beam to determine its capacity

If 100 similar beams with the same nominal dimensions and material properties are constructed and tested, 100 different answers will most probably be obtained, although the answers will be close to each other. The answers can be plotted on a histogram after grouping them within ranges, as shown in Figure 4.

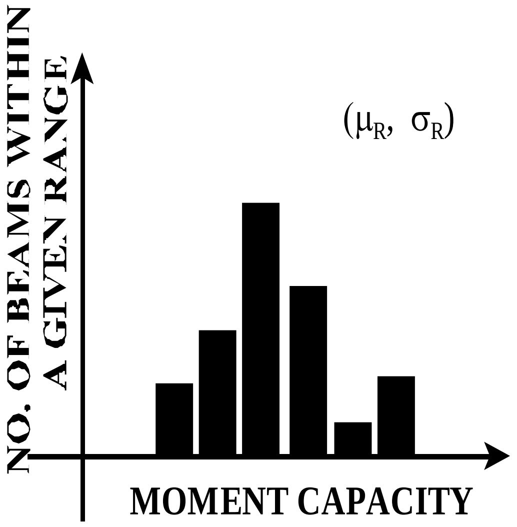

Figure 4: Histogram of capacity

Dead load on the beam can be obtained based on measurements of actual dimensions and material density of similar beams. The maximum bending moment due to dead load can be plotted on a histogram, as shown in Figure 5.

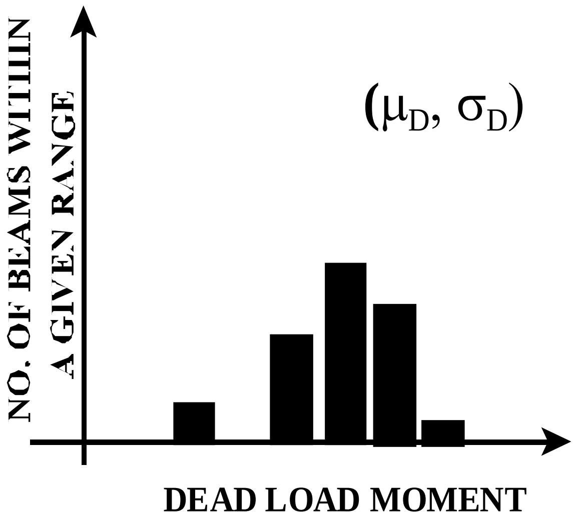

Figure 5: Histogram of moment due to dead load

The statistics of maximum live load on the beam can be obtained by field monitoring beams with similar tributary area and occupancy over an extended period of time, equivalent to the useful life of the structure. The maximum bending moment due to live load can be plotted on a histogram, as shown in Figure 6.

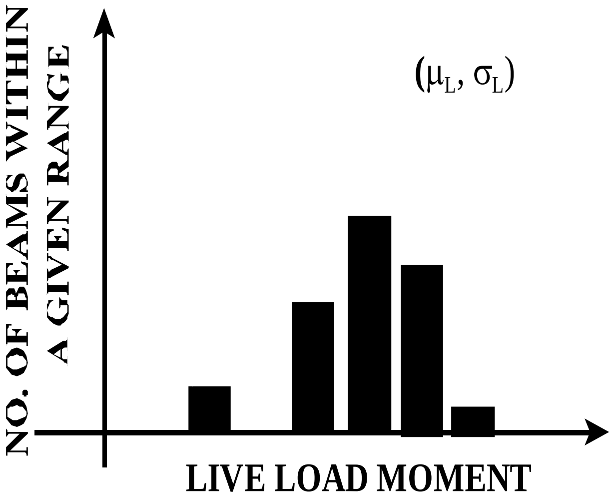

Figure 6: Histogram of moment due to live load

If all variables are normally distributed, then the reliability of the beam can be measured by a reliability index, β, computed from Equation 3.3, and if they are all lognormally distributed, then Equation 3.4 can be utilized. This concept will now be illustrated with the aid of two examples.

Example 1

Using the above equations, the reliability index of a beam under flexure, of which the resistance (R) parameters are μ_R=7500 kN-m and σ_R=550 kN-m and the load effect (Q) parameters are μ_Q=5000 kN-m and σ_Q=400 kN-m, for the cases of normally and lognormally distributed variables, can be computed as:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} = \frac{7500 - 5000}{\sqrt{550^2 \pm 400^2}} = 3.68$$

$$\beta = \frac{ln(\mu_R/\mu_Q)}{\sqrt{V_R^2 + V_Q^2}} = \frac{ln(7500/5000)}{\sqrt{\left(\frac{550}{7500}\right)^2 + \left(\frac{400}{5000}\right)^2}} = 3.74$$

Example 2

If a steel hanger with yield strength F_y (μ_{Fy}=250 MPa and V_{Fy}=8%) and deterministic cross sectional area A (μ_A =300 mm^2 and V_A=0) is subjected to a tensile dead load D (μ_D=30 MPa and V_D=10%) and tensile live load L (μ_L=20 MPa and V_L=20%), the reliability index assuming that all variables are normally distributed will be:

$$R = AF_y = 300F_y$$

$$\mu_R = 300 \times 250 = 75\ kN \qquad \sigma_R = V_{F_y} \times \mu_R = 6\ kN$$

$$\mu_Q = \mu_D + \mu_L = 30 + 20 = 50\ kN \qquad \sigma_Q = \sqrt{\sigma_D^2 + \sigma_L^2} = 5\ kN$$

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} = \frac{75 - 50}{\sqrt{6^2 + 5^2}} = 3.20$$

3.2 The Rackwitz-Fiessler Method

If all variables R and Q are neither normal nor lognormal, the Rackwitz-Fiessler method [40] can be used to compute the reliability index. The basic idea of this method is to transform the non-normal variables into equivalent normal variables. This transformation is accomplished by approximating the true distribution of the variables by normal distributions at the so-called design point. The design point is the point of maximum probability on the failure surface.

Let F_R and F_Q be the cumulative distribution functions (CDF) for the variables R and Q, respectively. Similarly, let f_R and f_Q be the density functions (PDF) for the variables R and Q respectively. The method starts by guessing an initial value for the design point, denoted by (R^*, Q^*). Since the design point is on the failure surface, R^* is equal to Q^*. Next, F_R and F_Q are approximated at the design point by normal distributions F_R' andF_Q', such that [12]:

$$F_R' = F_R(R^*) \tag{3.5}$$

$$F'_Q = F_Q(Q^*) \tag{3.6}$$

$$f'_R = f_R(R^*) \tag{3.7}$$

$$f'_Q = f_Q(Q^*) \tag{3.8}$$

The standard deviations of R' and Q' are computed from:

$$\sigma'_R = \frac{\emptyset\{\Phi^{-1}[F_R\,(R^*)]\}}{f_R(R^*)} \tag{3.9}$$

$$\sigma'_Q = \frac{\emptyset\{\Phi^{-1}[F_Q\,(Q^*)]\}}{f_Q(Q^*)} \tag{3.10}$$

where $\emptyset$ is the PDF of the standard normal random variable, and Φ is the CDF of the standard normal random variable.

This means than R' and Q' can be evaluated now using the following expressions:

$$\mu'_R = R^* - \sigma'_R\Phi^{-1}[F_R(R^*)] \tag{3.11}$$

$$\mu'_Q = Q^* - \sigma'_Q\Phi^{-1}[F_Q(Q^*)] \tag{3.12}$$

The reliability index is then computed using the formula:

$$\beta = \frac{\mu'_R - \mu'_Q}{\sqrt{{\sigma'_R}^2 + {\sigma'_Q}^2}} \tag{3.13}$$

After that, a new design point is calculated from the following:

$$R^* = Q^* = \mu'_R - \beta\frac{{\sigma'_R}^2}{\sqrt{{\sigma'_R}^2 + {\sigma'_Q}^2}} \tag{3.14}$$

Finally, the iteration is continued until the required accuracy is achieved.

In this study, a computer program was developed by Tabsh (unpublished) to calculate the reliability index using Rackwitz-Fiessler, without using the procedure explained above.

CHAPTER 4

SURVEY OF HUMAN ERRORS IN DESIGN AND CONSTRUCTION IN THE UAE

4.1 Introduction

In this chapter, the frequencies of common human errors committed in design and construction stages in the UAE will be investigated. In order to achieve this objective, a survey was developed and distributed to local practicing construction and consultant companies to get feedback on the perceived frequency of human errors committed during construction and design stages.

The survey, enclosed in Appendix A, was developed in English and in Arabic so that it could be filled by a large number of engineers. It consists of questions related to errors encountered during design stage, as well errors encountered during construction stage. The survey was made brief in order to encourage engineers to complete; it takes about 10-15 minutes for completion.

The survey was distributed to practicing construction and structural design companies in the UAE. Additionally, engineers working in Abu Dhabi municipality, Dubai municipality, and Sharjah municipality were surveyed. The Society of Engineers in Dubai was also approached in order to help distribute the survey to the largest number of engineers. Finally, a workshop sponsored by Emirates Foundation and organized by Drs. Sami Tabsh and Sherif Yehia was held at the American University of Sharjah campus in May 2011 to discuss the common human errors committed during design and construction stages in the country. At the end of that workshop, the survey was filled by the attendees of the workshop.

It should be noted that the questions on the survey were not derived from an extensive base of literature. Rather, it was largely developed from past experience, suggestions and recommendations of experienced people working in the field. Thus the various questions included in the survey are not exhaustive of the universe of possible errors in design and construction, but are expected to be commnonly encounted in practice.

The number of filled surveys was 107 surveys. The surveys were completed by engineers working in different private, semi-private and public design and construction companies in the UAE, as well engineers working in different municipalities within the country. Both junior and senior engineers completed the survey. Background details of the surveyed engineers on frequency of human errors in this study are summarized in Table 5.

Table 4: Number and characteristics of the surveyed engineers in this study

Nature of work	Number of Years of Experience		
	≤ 6 years	6 - 15 years	≥ 15 years
Design/consulting	17	31	23
Construction	11	10	15
Total Number	28	41	38

When analyzing the results obtained from the filled surveys, human errors encountered were classified to ranges of 5 cases per 100 cases, and then the percentage frequency of each error range was calculated. In addition, the number of years of experience of the engineers who completed the surveys was considered. Three "number of years of experience" groups were considered, i.e. "6 years of experience or less" (slightly experienced), "between 6 to 15 years of experience" (moderately experienced), and "15 years of experience or more" (highly experienced). Furthermore, results were categorized based on whether the surveys were filled by contractors or by designers. The average frequency for each error case was also calculated. The survey developed in English and Arabic is shown below in Figures 7, 8, 9 and 10, as well as in the appendix A.

Survey of Errors in Structural Design and Construction in the UAE

A. Background Information:

Title (e.g. senior structural engineer, project manager, project engineer, etc.): ……………………

Nature of work: □ Structural Design □ Construction □ Other (please Specify) …………....

Field of application: □ Buildings □ Infrastructure □ Other (please Specify) ……………

Number of years of experience: In the UAE: ……..… Outside of the UAE: …………

B. Questions related to construction:

How many times did you encounter unexpected problems in construction due to:

Q1: Improper soil investigation:

Frequency: ………………… per 100 cases

Q2: Improper sub-grade work, such as dewatering or water proofing:

Frequency: ………………… per 100 cases

Q3: Improper foundation work (e.g. wrong pile location, errors in drilled shaft construction, improper backfilling & soil compaction without following the specifications):

Frequency: ………………… per 100 cases

Q4: Using poor quality construction materials not complying with the specification:

Frequency: ………………… per 100 cases

Q5: Modifying details shown on drawings without referring to ………consultant/designer:

Frequency: ………………… per 100 cases

Q6: Improper formwork (e.g. shuttering and scaffolding):

Frequency: ………………… per 100 cases

Q7: Errors in designing/constructing temporary shoring and bracing during ………excavation:

Frequency: ………………… per 100 cases

Q8: Errors in details such as expansion/cold/construction joints:

Frequency: ………………… per 100 cases

Q9: Errors in reinforcement details (e.g. inadequate lap slices and end hooks):

Frequency: ………………… per 100 cases

Q10: Inadequate concrete cover and/or member sizes:

Frequency: ………………… per 100 cases

Q11: Improper concreting (e.g. inadequate concrete compaction or concrete placement):

Frequency: ………………… per 100 cases

Q12: Overloading the structure during construction or premature removal of formwork or scaffolding:

Frequency: ………………… per 100 cases

Q10: Inadequate concrete cover and/or member sizes:

Frequency: ………………… per 100 cases

Q11: Improper concreting (e.g. inadequate concrete compaction or concrete placement):

Frequency: ………………… per 100 cases

Q12: Overloading the structure during construction or premature removal of formwork:

Frequency: ………………… per 100 cases

Figure 7: Survey in English

C. Questions related to structural design:

How many times did you encounter unexpected errors in structural design due to:

Q1: Conceptual mistakes (e.g. load transfer, support boundary conditions, ………connection, etc.)

Frequency: …………………. per 100 cases of design

Q2: Unit related errors (e.g. using cm for m, using inches instead of cm):

Frequency: …………………. per 100 pages of calculations

Q3: Calculation mistakes:

Frequency: …………………. per 100 pages of calculations

Q4: Wrong extraction of information from tables, architectural drawings or ……....charts:

Frequency: …………………. per 100 tables/drawings/charts

Q5: Neglecting water table, buoyancy, soil weight or live load surcharge in ………calculations:

Frequency: …………………. per 100 cases of design

Q6: Mixing equations from different codes inappropriately (e.g. ACI code with BS standard):

Frequency: …………………. per 100 calculation steps

Q7: Wrong selection of factors of safety, load combinations, or load factors:

Frequency: …………………. per 100 cases of design

Q8: Wrong assumptions of wind load or seismic load (Not checking the governing case, wrong wind speed and wind factor, or wrong selection of seismic factors and accelerations):

Frequency: …………………. per 100 cases of design

Q9: Neglecting load eccentricity on a column, torsion, punching shear or uplift ………force:

Frequency: …………………. per 100 cases of design

Q10: Lack of knowledge with regard to use of software (e.g. wrong input or wrong interpretation of the output from the software):

Frequency: …………………. per 100 cases of design

Q11: Not checking the reinforcement limits according to the code (e.g. minimum & maximum reinforcement, rebar/stirrup spacing, etc.):

Frequency: …………………. per 100 cases of design

Q12: Not checking the required serviceability limits (e.g. minimum member thickness, crack width, temperature and shrinkage reinforcement, etc.):

Frequency: …………………. per 100 pages of calculations

Q13: Wrong transferring of results from design calculations to drawings:

Frequency: …………………. per 100 cases of transferring results

Q14: Wrong reinforcement around typical details (e.g. around openings, ………connections, etc.):

Frequency: …………………. per 100 cases of design

Please provide additional information, if you wish, on errors committed during the design and/or construction stages that were not addressed in Parts B and C. Indicate their frequencies as well.

………………………………………………………………………………………………..

Figure 8: Survey in English (Continued)

استبيان عن تأثير الأخطاء البشرية على سلامة المنشآت والمباني في دولة الإمارات العربية المتحدة

أولاً: المعلومات الشخصية:

المسمى الوظيفي (مثال: مهندس تصميم أول، مدير مشروع، مهندس مشروع، الخ): ....

طبيعة العمل: ☐ التصميم الهندسي ☐ تنفيذ الإنشاءات ☐ مجال آخر(يرجى ذكره): ....

مجال العمل: ☐ المنشآت ☐ البنية التحتية ☐ مجال آخر(يرجى ذكره): ....

عدد سنوات الخبرة: ☐ داخل دولة الإمارات: .... ☐خارج دولة الإمارات: ....

ثانياً: الأسئلة المتعلقة بأخطاء الإنشاءات

كم عدد المرات التي واجهت فيها مشاكل غير متوقعة أثناء تنفيذ الإنشاءات نتيجة:

Q1: أخطاء في فحص التربة

التكرار .... في 100 حالة

Q2: أخطاء في أعمال التربة كأعمال سحب الماء الجوفي أو العزل المائي

التكرار .... في 100 حالة

Q3: أخطاء في أعمال الأساسات (أخطاء في تحديد مكان الركائز "الخوازيق الأوتاد"، أخطاء في حفر الركائز "الخوازيق" أو في ردم ودك التربة دون مراعاة المواصفات)

التكرار .... في 100 حالة

Q4: استخدام مواد بناء غير متوافقة مع المواصفات

التكرار .... في 100 حالة

Q5: تعديل تفاصيل على المخططات دون الرجوع إلى الاستشاري/المهندس

التكرار .... في 100 حالة

Q6: أخطاء في شدات الطوبار (مثال القوالب والسقائل)

التكرار .... في 100 حالة

Q7: أخطاء في التصميم/التنفيذ للإسناد الجانبي المؤقت للتربة والتدعيم خلال الحفريات

التكرار .... في 100 حالة

Q8: أخطاء في بعض التفاصيل مثل فواصل التمدد/على البارد/ فواصل التنفيذ

التكرار .... في 100 حالة

Q9: أخطاء في تفاصيل حديد التسليح (عدم كفاية طول التثبيت "التشريك" أو طول الخطاف)

التكرار .... في 100 حالة

Q10: عدم كفاية الغطاء الخرساني حول حديد التسليح و/أو قياسات العنصر الإنشائي

التكرار .... في 100 حالة

Q11: أخطاء في صب الخرسانة (عدم دك الخرسانة بشكل جيد أو التوزيع السيء)

التكرار .... في 100 حالة

Q12: زيادة تحميل المبنى أثناء عمليات الإنشاء أو إزالة القوالب / السقائل قبل وقتها اللازم

التكرار .... في 100 حالة

Figure 9: Survey in Arabic

ثالثاً: أسئلة متعلقة بأخطاء التصميم الهندسي

كم عدد المرات التي واجهت فيها مشاكل غير متوقعة أثناء مراحل التصميم الهندسي نتيجة:

Q1: أخطاء في فكرة التصميم (طريقة انتقال الأحمال، سند الموقع المحيط، وصلات الربط)

التكرار .... في 100 حالة تصميم

Q2: أخطاء في وحدات القياس (استخدام السنتمتر بدل المتر، أو الإنش بدل السنتمتر)

التكرار .... في 100 صفحة حسابات

Q3: أخطاء في الحسابات (جمع، طرح، جذر تربيعي، الخ)

التكرار .... في 100 صفحة حسابات

Q4: أخطاء في استخراج المعلومات من الجداول، المخططات المعمارية أو المخططات البيانية

التكرار .... في 100 جدول/مخطط معماري/مخطط بياني

Q5: إهمال منسوب المياه الجوفية، وزن التربة، أو وزن المركبات المتحركة والحمولة الحية في الحسابات

التكرار .... في 100 حالة تصميم

Q6: استخدام معادلات من أكواد مختلفة بصورة غير ملائمة (استخدام الكود الأمريكي والبريطاني في الوقت نفسه)

التكرار .... في 100 خطوة حسابية

Q7: اختيار خاطئ لعوامل الأمان، حالات التحميل أو عوامل الحمولات

التكرار .... في 100 حالة تصميم

Q8: أخطاء في حساب حمولات الرياح والزلازل (عدم التأكد من الحالة المسيطرة، افتراض خاطئ لسرعة الرياح ومعامل الرياح، اختيار خاطئ لمعامل الزلازل والتسارع)

التكرار .... في 100 حالة تصميم

Q9: إهمال القوى اللامركزية على العمود، الفتل، الثقب، أو القوى الرافعة

التكرار .... في 100 حالة تصميم

Q10: قلة خبرة في استخدام برامج التصميم الهندسية (إدخالات خاطئة أو تفسيرات خاطئة لنتائج البرامج الهندسية)

التكرار .... في 100 حالة تصميم

Q11: عدم التأكد من موافقة نسب التسليح للكود (الحد الأدنى والأقصى من حديد التسليح، المسافات بين الوصلات)

التكرار .... في 100 حالة تصميم

Q12: عدم التأكد من حدود التشغيل (سماكة العنصر الإنشائي، سُمك الشق، درجات الحرارة وحديد التقلص)

التكرار .... في 100 صفحة حسابات

Q13: أخطاء في نقل النتائج من حسابات التصميم إلى المخططات

التكرار .... في 100 حالة نقل النتائج

Q14: أخطاء في حديد التسليح حول التفاصيل الخاصة (حول الفتحات والوصلات، الخ)

التكرار .... في 100 حالة تصميم

في حال كان لديكم معلومات إضافية عن الأخطاء الشائعة أثناء عمليات التصميم والإنشاءات والتي لم يُتطرق إليها في الاستبيان، فيرجى ذكرها إضافة إلى عدد مرات تكرارها

....

Figure 10: Survey in Arabic (Continued)

4.2 Human Errors Committed during Construction Stage

The possibility of human errors committed during construction is high, especially when considering the squeezed duration of the project, which often happened during the construction boom (2005-2007) in this country. Human errors in construction are not limited to a single activity, but rather to all activities stages of construction from mobilization to delivery of the finished product. To begin with, human errors could be committed during the soil investigation work. For example, water table level might not be measured properly because of poor monitoring of fluctuations in water table level, or because of errors in installing the piezometer. Poor monitoring of cavities by not paying attention to the loss of water, drop of tools, and fast drilling are also common sources of human errors during this stage of work.

Human errors could be also committed during foundation work. For instance, a pile might be drilled to a depth less than the approved one, the location of a pile might not be as per the approved drawing, or reinforcement provided in the pile or pile cap may not be as per the approved structural drawings.

Error may even happen when concrete is delivered to site, such as the site engineer neglect to check the delivery note of concrete to verify that the mix design is consistent with the approved one. The site engineer may also commit errors when checking that concrete strength, slump, and temperature are matching with specifications. The same thing could happen when steel is delivered to site if the Mill Certificate is not verified with the approved specifications, in terms of the yield strength, diameter of bars, chemical composition, and physical properties.

Even during concreting, human errors can happen if the engineer of record is not alert. If concrete is poured without maintaining an adequate vertical to horizontal concreting ratio, the concrete will apply an excessive lateral load on the formwork, which may reduce the factor of safety against stability and collapse. Also, if concreting in long span does not stop at the determined construction joints, cold joint will form at the location of large moment or large shear values.

There are other construction activities where human errors might be committed, and they are listed in the construction survey discussed earlier and included in Appendix A. In this section, the frequencies of committing errors in these activities as encountered or perceived by engineers with different years of experience were analyzed and the results are presented in Figures 11-22. In the following discussion and for the sake of brevity, engineers with less than or equal to 6 years of experience are denoted by "slightly experienced," those with more than 6 years but less than 15 years of experience are labeled "moderately experienced," and those with 15 or more years of experience are referred to "highly experienced."

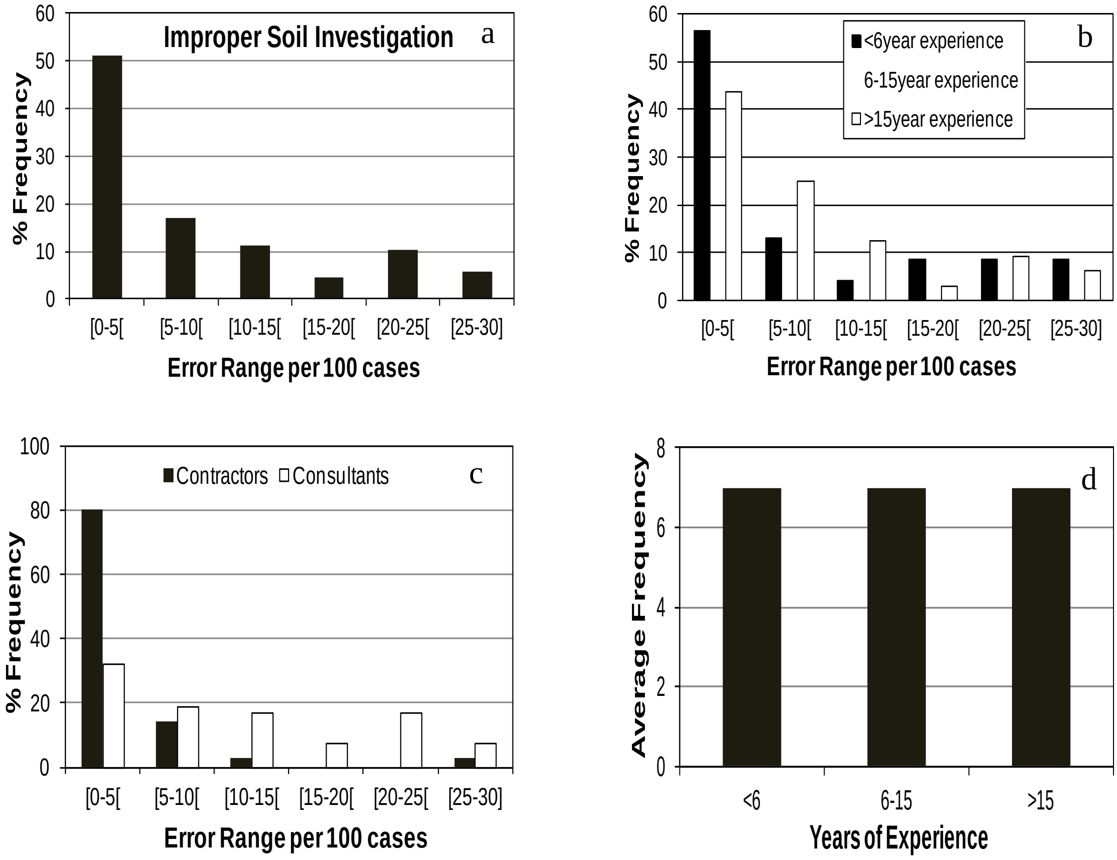

Figure 11: Survey results on improper soil investigation

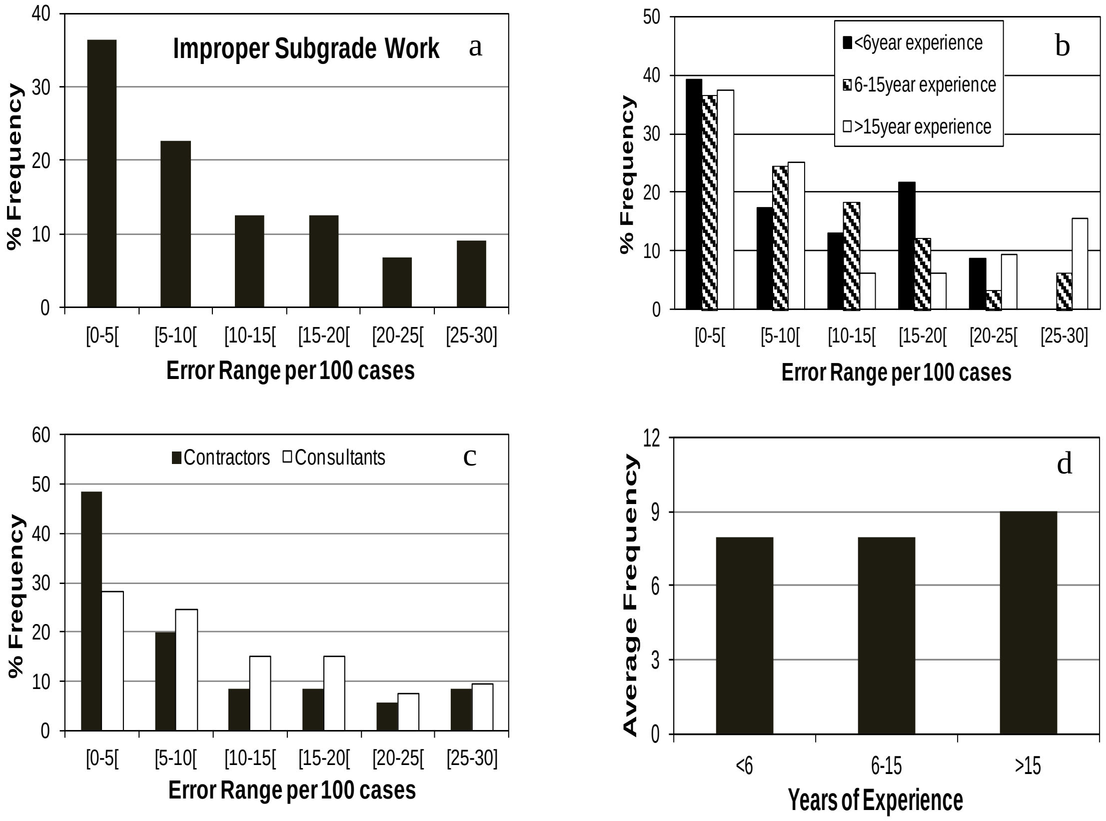

Figure 12: Survey results on improper subgrade work

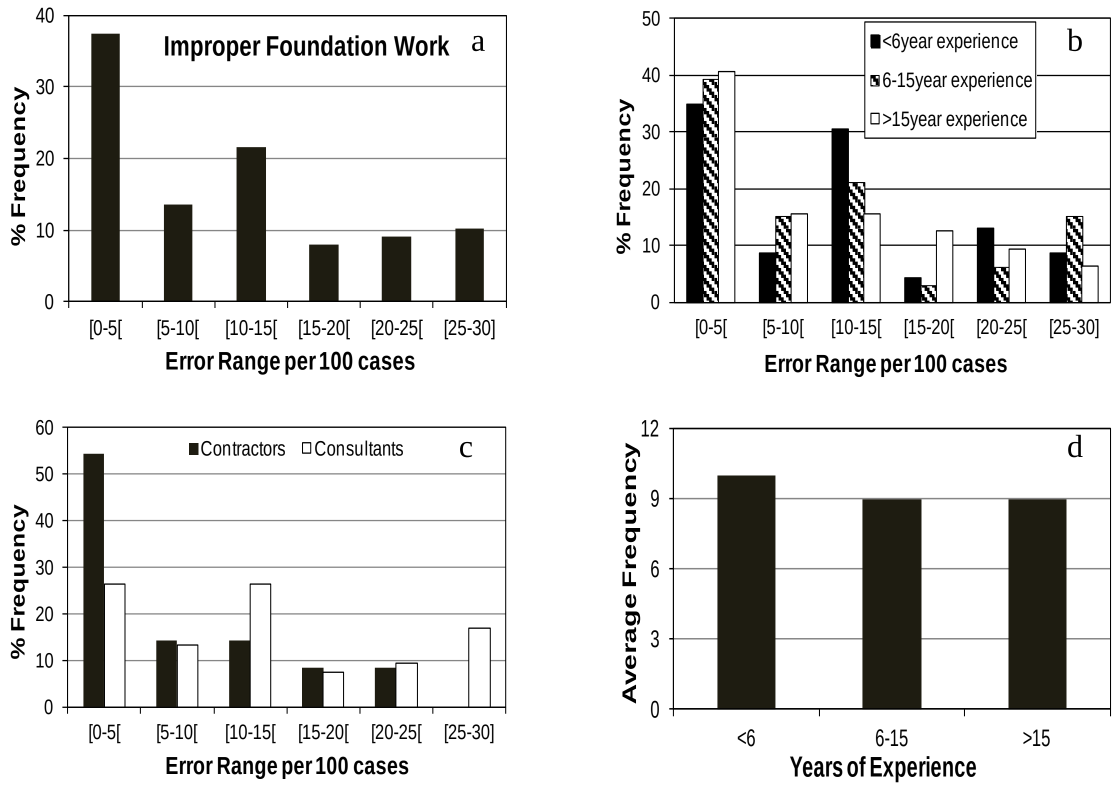

Figure 13: Survey results on improper foundation work

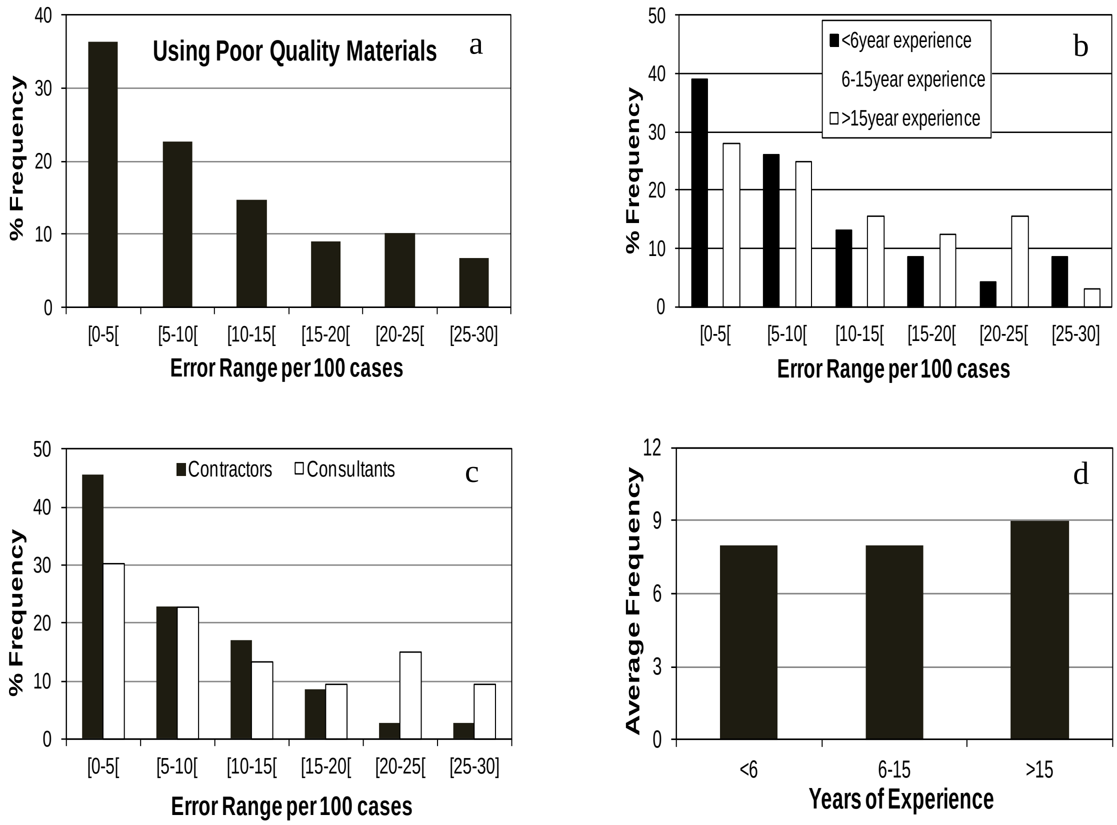

Figure 14: Survey results on use of poor quality materials

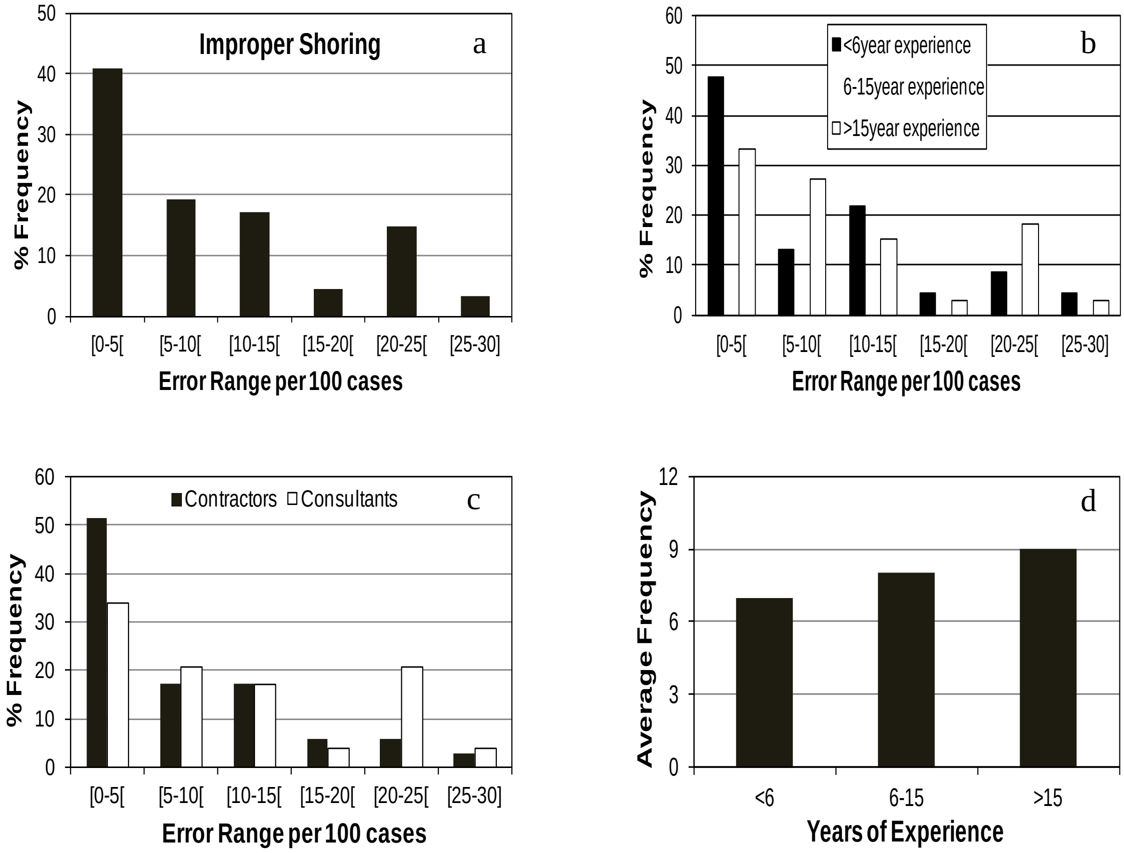

Figure 15: Survey results on improper shoring

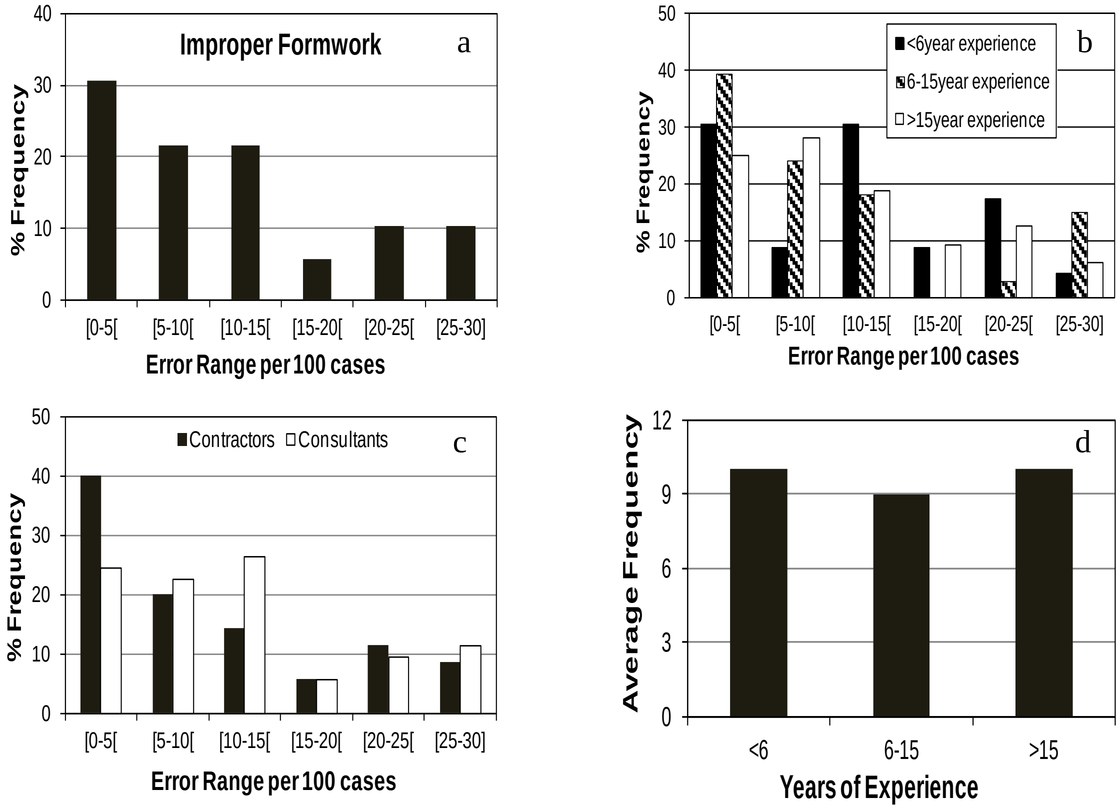

Figure 16: Survey results on improper formwork

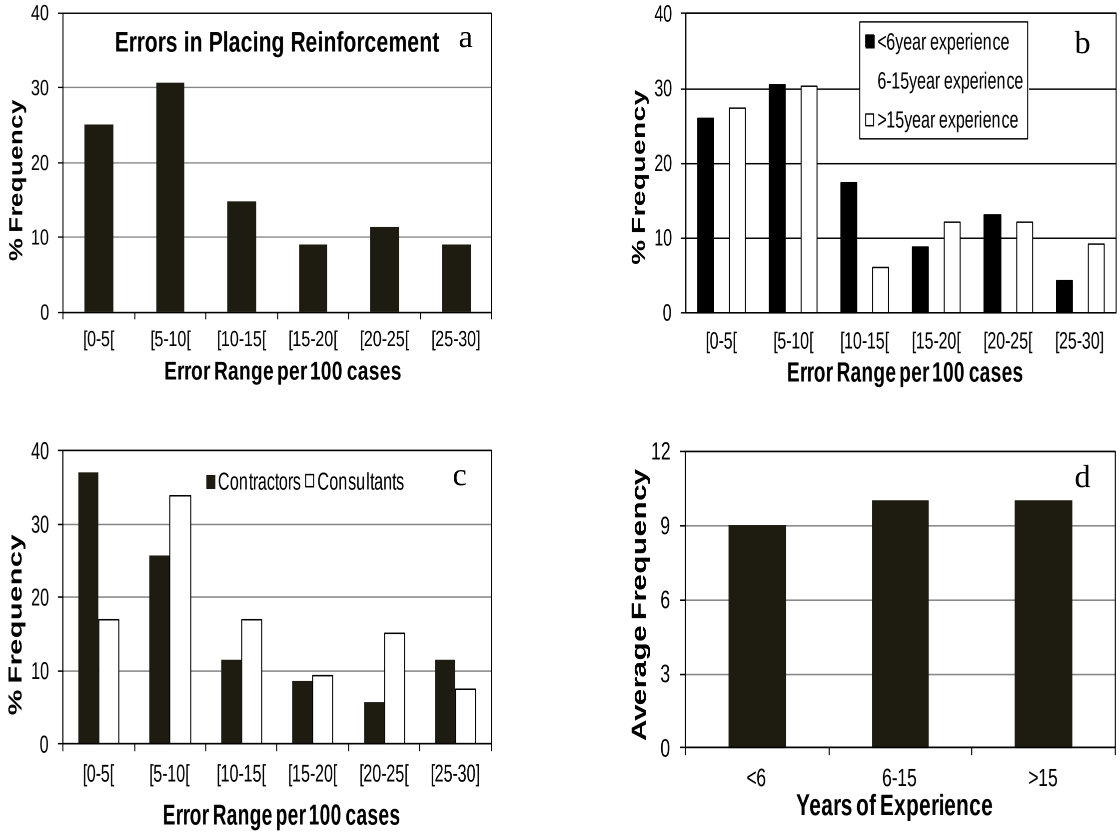

Figure 17: Survey results on errors in placing reinforcement

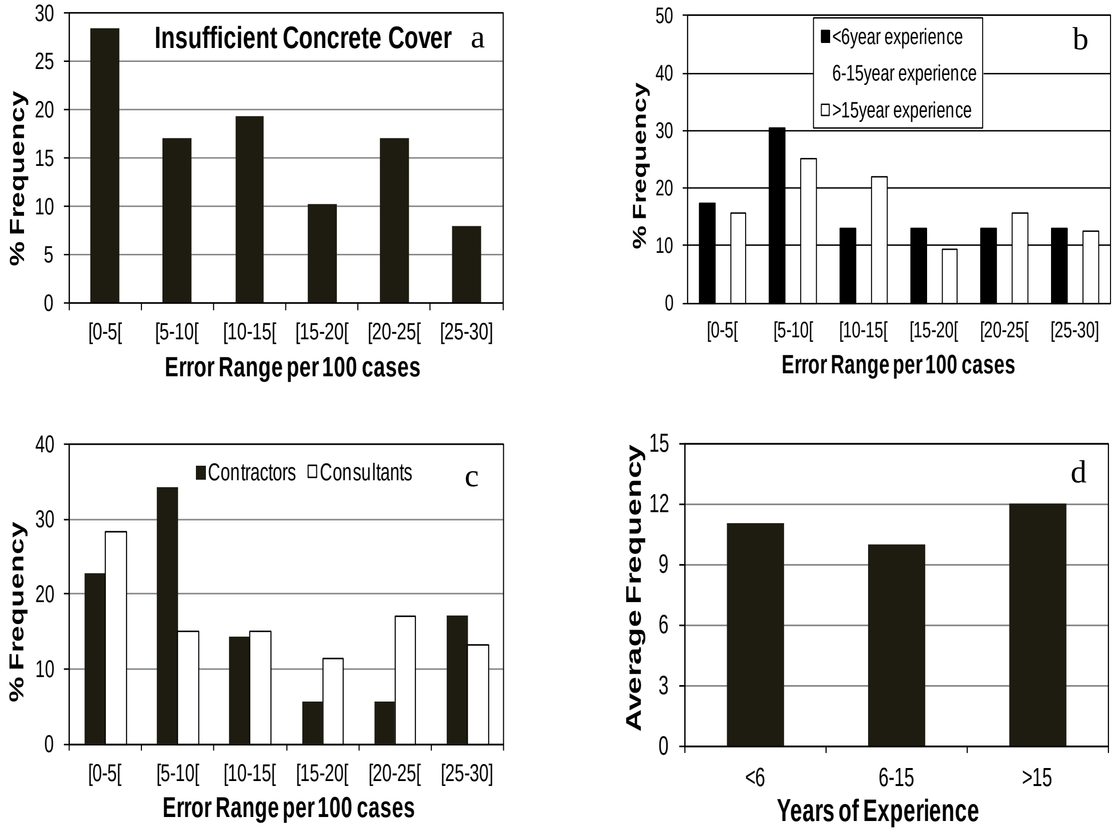

Figure 18: Survey results on insufficient concrete cover

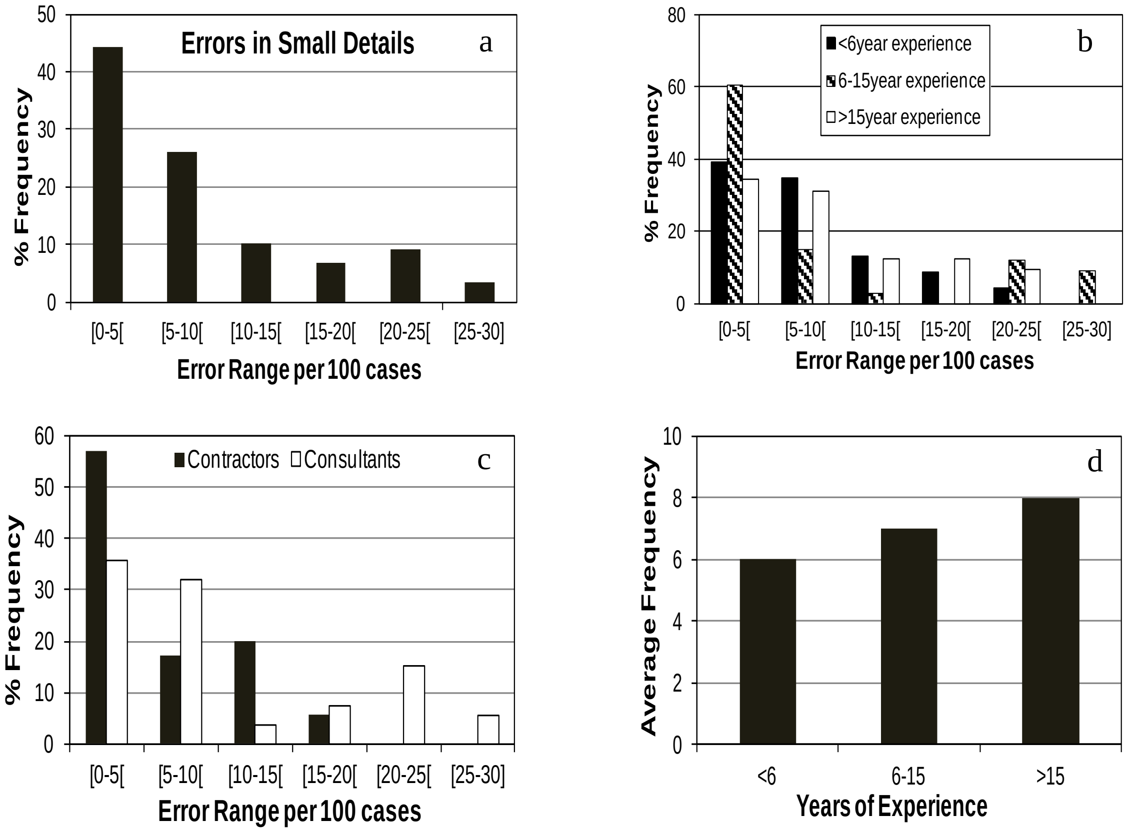

Figure 19: Survey results on errors in small details

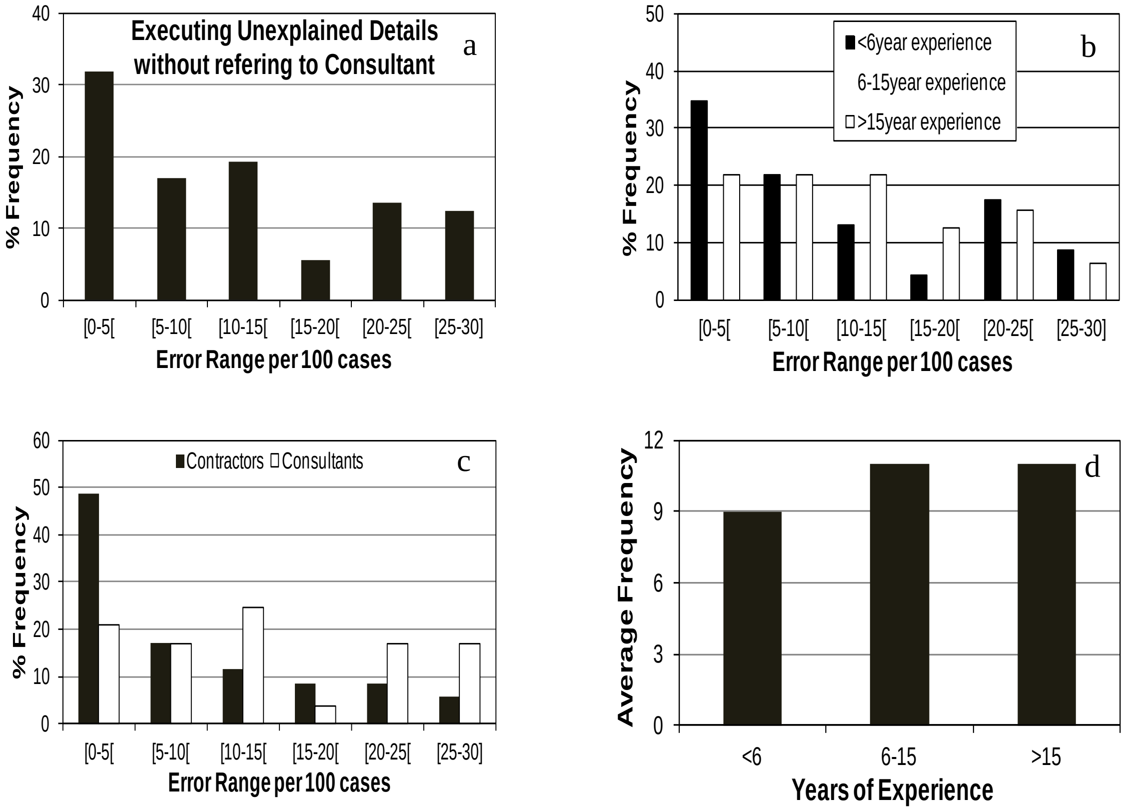

Figure 20: Survey results on executing details without referring to consultant

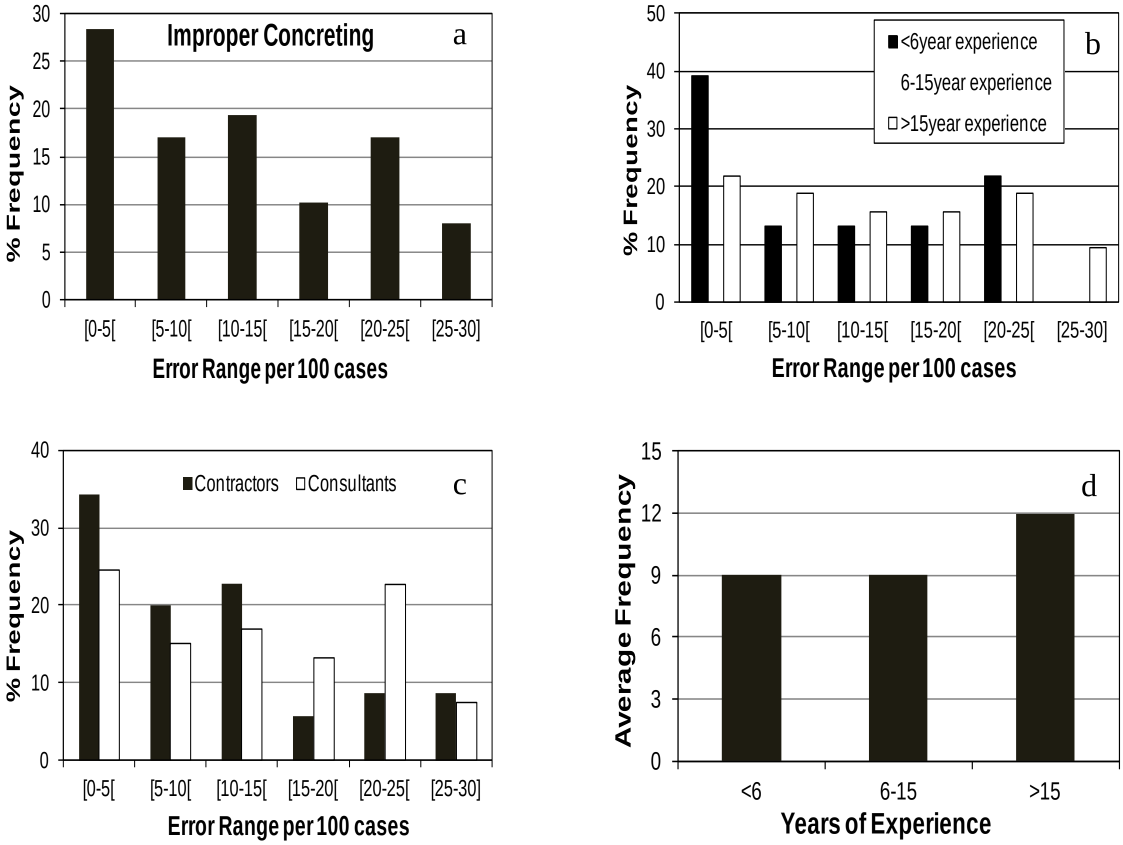

Figure 21: Survey results on improper concreting

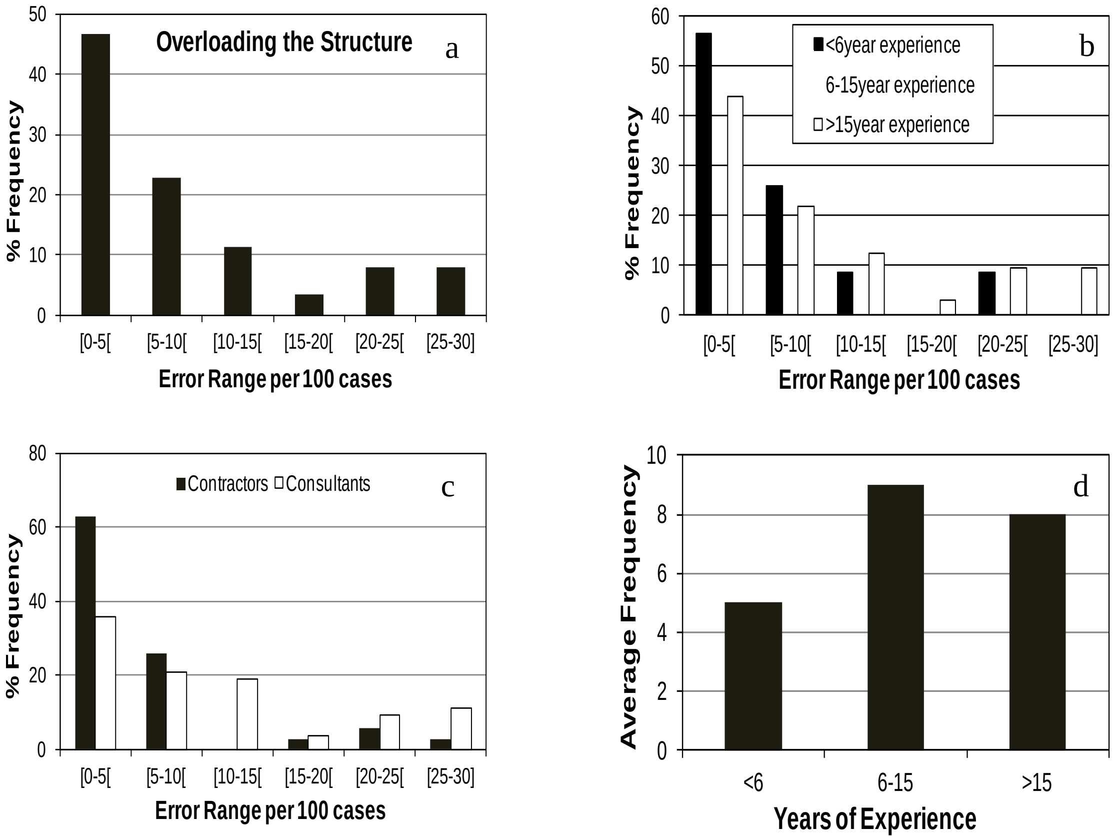

Figure 22: Survey results on overloading the structure during construction

The results obtained from the completed surveys have shown that different types of human errors are committed during construction stages. In all graphs (a), shown on the top left part of the figures, which presents the percentage frequency at which the error per 100 cases is occurring, without considering the number of years of experience for the engineers who filled the survey, the results have shown that the most frequent range of committed human errors was the "Less than 5 cases per 100 cases" range, except for human errors committed during placing the steel reinforcement, providing concrete cover, and concreting. Surveys have shown that the most frequent range of human errors committed during placing reinforcement was "5-10 cases per 100 cases" range. For improper concreting and insufficient concrete cover, analyzing the survey results has pointed out that error ranges occurred almost at the same frequency, between 25% and 30%.

The concept of all graphs (b), shown on the top right part of the figures, is similar to the concept of all graphs (a) in the figures. The only difference is that the number of years of experience of the engineers who filled the survey was taken into consideration. Analyzing the survey results leads to the following conclusions:

- For improper soil investigation, improper subgrade work, improper shoring, and overloading the structure, the survey results have indicated that the most frequent range of human errors committed for all three considered groups of experience was the “Less than 5 cases per 100 cases”.
- For the case of placing reinforcement, the most frequent committed error range was the “5-10 cases per 100 cases” range.
- Regarding errors encountered due to improper foundation work, the most frequent range of committed errors was the “Less than 5 cases per 100 cases” range, except for the group of “slightly experienced” engineers, as the error range of “Less than 5 cases per 100 cases”, and “5 to 10 cases per 100 cases” occurred almost at the same frequency.
- As for errors committed because of using poor quality materials, the responses “Less than 5 cases per 100 cases”, and “5 to 10 cases per 100 cases” were selected at the same frequency for the surveys filled by the “highly experienced” engineers. For the other two groups of engineers, “Less than 5 cases per 100 cases” was the most frequent encountered range.
- In addition, the results of the survey have indicated that “moderately experienced” engineers were on the conservative side for errors encountered because of providing insufficient concrete cover, or when executing small details. For the “slightly experienced” and “highly experienced” engineers groups, “5-10 cases per 100 cases” error range was the most frequent range for the insufficient concrete case, whereas “less than 5 cases per 100 cases”, and “5-10 cases per 100 cases” error ranges were selected almost at the same frequency for the small details cases.
- Furthermore, “slightly experienced” and “moderately experienced” engineers groups were very conservative in estimating the errors encountered because of executing unexplained details without referring to the consultant, or because of improper concreting. On the other hand, the surveys filled in by “highly experienced” engineers have shown that the frequency of different error ranges of error was almost the same.

- Finally, the frequency of errors encountered due to improper formwork was different among the three experience groups of engineers. For “slightly experienced” engineers, the most frequent range was the “10-15 cases per 100 cases” range, while the “less than 5 cases range per 100 cases” was the most frequent range for “moderately experienced” engineers. For “highly experienced” engineers, the “5-10 cases per 100 cases” range was the most frequent one.

Moving to graphs (c), shown at the bottom left, where the results shown in graphs (a) have now been classified according to whether surveys were filled by consultants or contractors. Except for the case of providing insufficient concrete cover shown in Figure 18, contractors were very conservative than consultants in estimating the frequency of encountered errors, since the frequency of “Less than 5 cases per 100 cases” range is much larger than other ranges. This is a predictable and logical result since contractors believe that most of the errors are not due to construction. On the other hand, the frequency of errors encountered by consultant was distributed somehow uniformly over the different ranges of errors. For the case of providing insufficient concrete cover, as mentioned above, consultants were more conservative in estimating the errors encountered due to human errors.

Graphs (d), shown at the bottom right, present the comparisons between engineers with different experience with respect to the average frequency of error occurrence. The comparison have shown that the average frequency between the three groups of experience for the engineers is almost the same for all cases, except for the case of improper concreting, where “highly experienced” engineers perceived more errors, and the case of overloading the structure, in which “slightly experienced” engineers were very conservative in estimating the number of error cases.

4.3 Human Errors Committed during Design Stage

Experience has shown that human errors are frequently committed during design stage. For instance, the limited time the designer has to complete the design might result in design mistakes. Also, the psychological state of the designer might influence his/her performance positively or negatively. As an illustration, if the designer is feeling tired, there is a higher possibility that he/she will commit more mistakes in design. Furthermore, the fewer the design review stages, the higher the possibility of committing human errors. However, the reasons behind committing human errors in design stage are beyond the scope of this study. This study focuses on the frequency of common human errors encountered in different design activities, as listed in the design survey shown Appendix A.

As stated in the survey related to design errors, human errors can be committed because of conceptual mistakes such as wrong assumption of boundary conditions, and wrong load path or structural system within the structure. Design errors are often committed by engineers who lack knowledge of the code of practice. For example, a European Engineer may be more familiar with the Eurocode, but less knowledgeable in the ACI318 code or AISC LRFD Steel Specification. Design errors are sometimes encountered due to lack of knowledge in using design software. For example, the designer might build the model, and design structural elements on ETABS without knowing the assumptions considered by the program; for example the difference between using membrane and shell elements in modeling the floor. Most importantly, the designer could be incapable of reading and interpreting the software output.

Using the wrong equation can lead to huge reduction in the nominal capacity of the structural element. As an illustration, the equation provided in the ACI code to calculate the nominal capacity of a beam under shear using the US unit system is:

$$\emptyset V_n = \emptyset(2\sqrt{f'_c} b_w d + A_v f_y d/s)$$

whereas the equation provided by the same code but for the metric system is:

$$\emptyset V_n = \emptyset(0.17\sqrt{f'_c} b_w d + A_v f_y d/s)$$

A large reduction in the nominal shear capacity would happen if the designer mistakenly uses the second equation to design a beam against shear using the US unit system.

Besides, human error can be committed because of neglecting the effect of water table (i.e. buoyancy) or because of neglecting the seismic or wind effect on a moderately to high-rise structures.

A very common source for human errors in design is the transferring of the design results from software or hand calculations into drawings. It might happen that the RC schedule and structural drawings are inconsistent with the design output. For instance, the beam width is recorded as 300mm in the RC schedule, while the design is showing that the width of the beam is 450mm. Another example is that the bar might be recorded as T25mm diameter although the diameter considered in the design is T32mm diameter.

The results of the survey are presented in Figures 23-36. They are also analyzed and discussed thereafter.

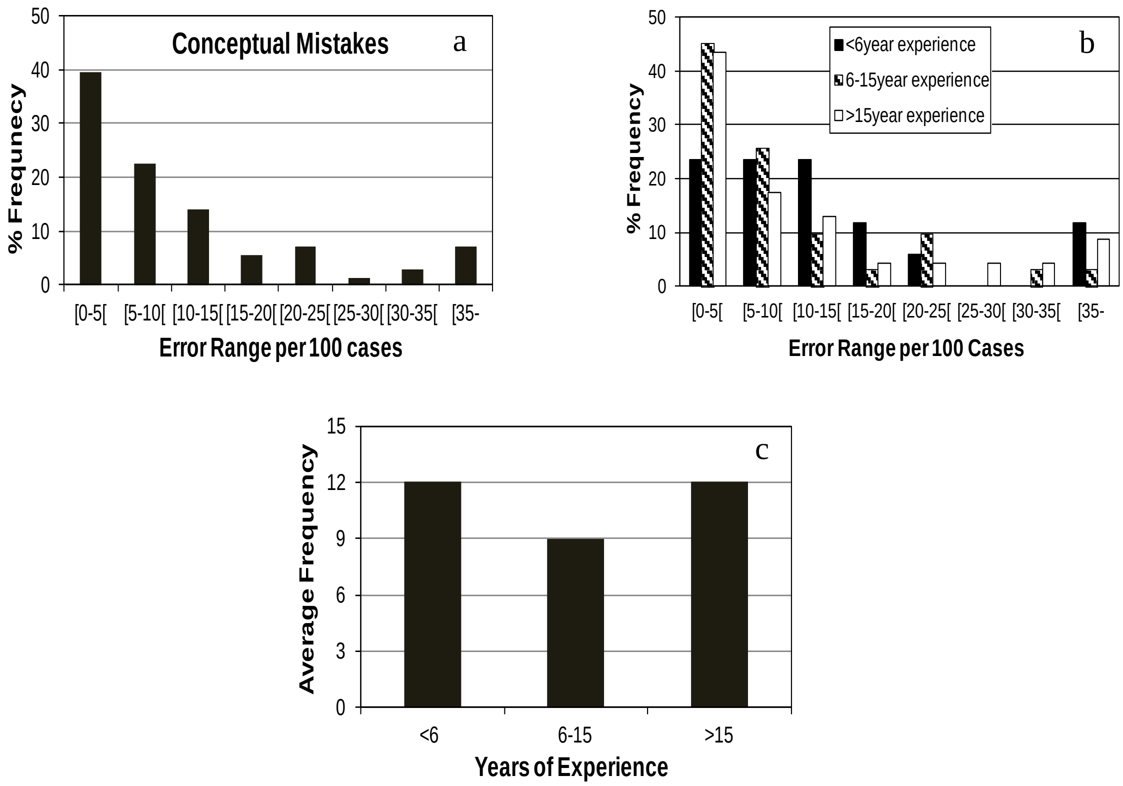

Figure 23: Survey results on conceptual mistakes in design

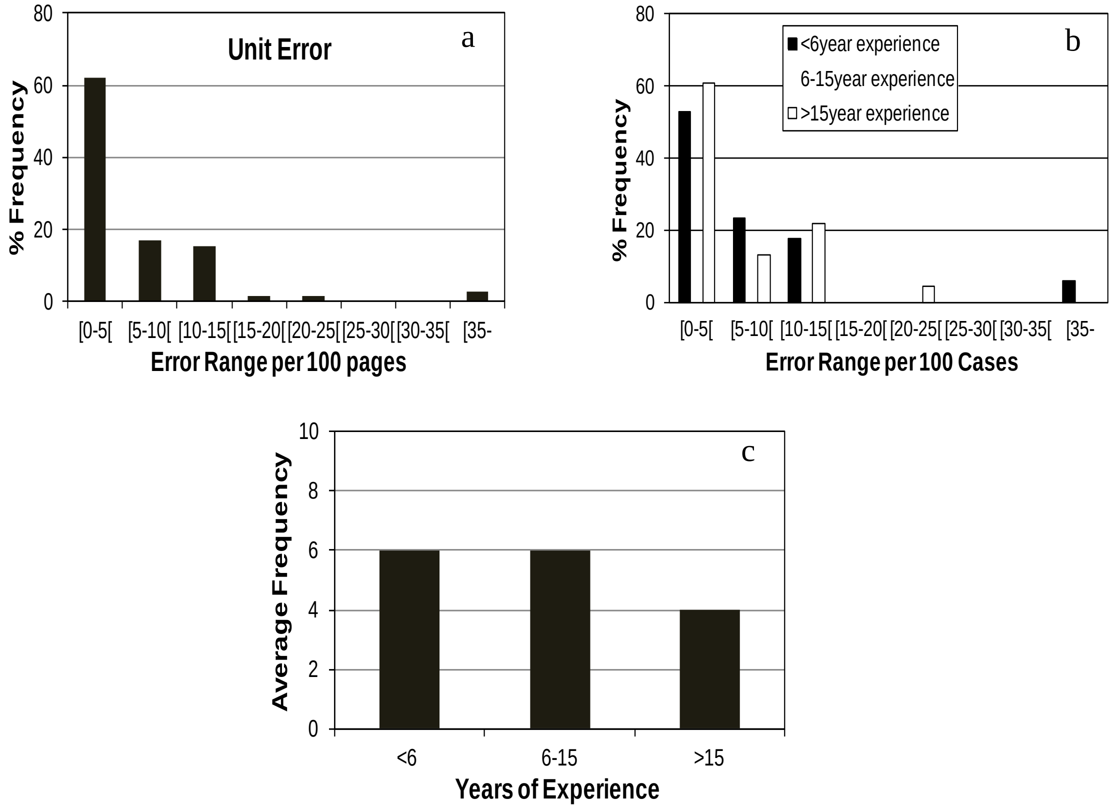

Figure 24: Survey results on units error

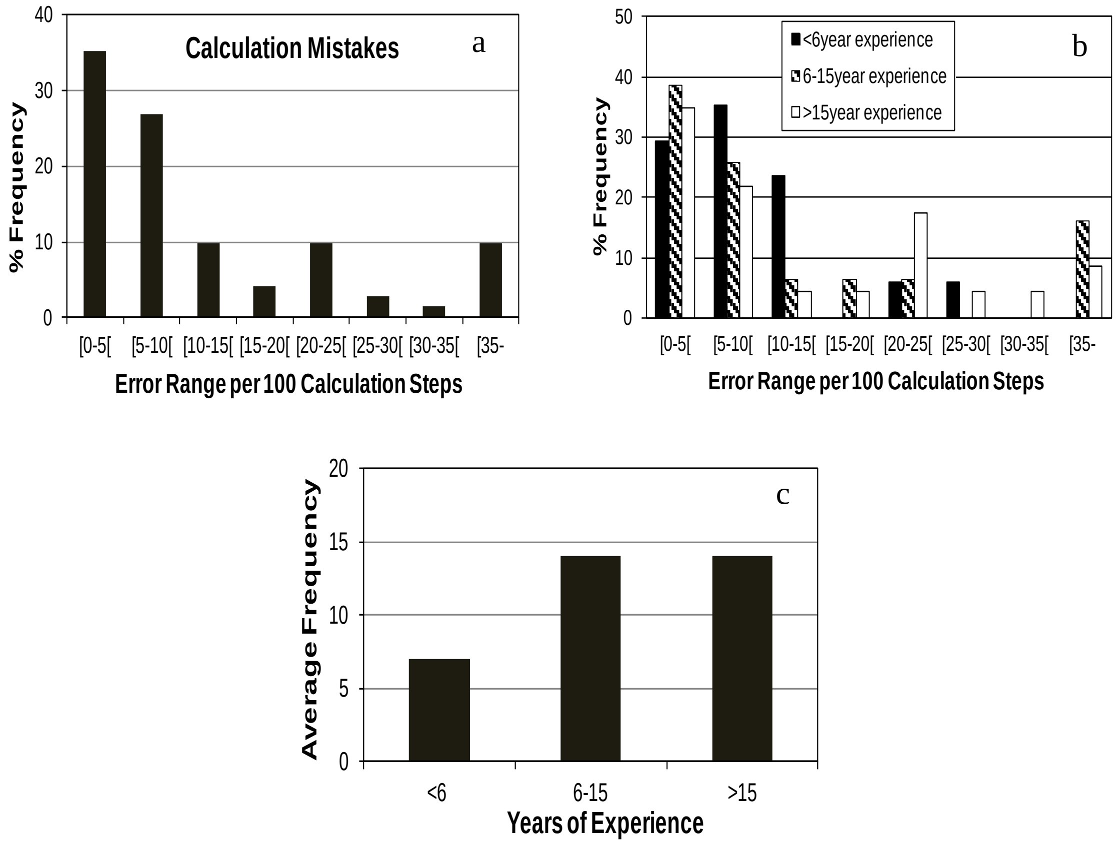

Figure 25: Survey results on calculation mistakes

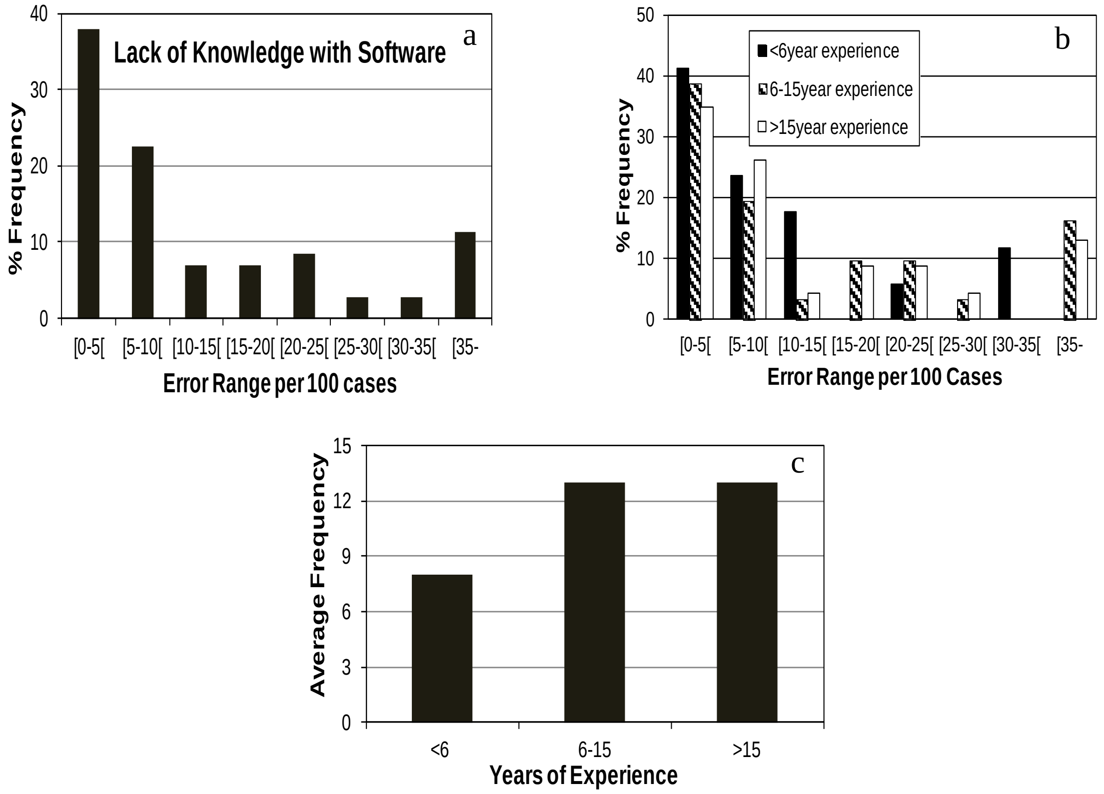

Figure 26: Survey results on lack of knowledge about software

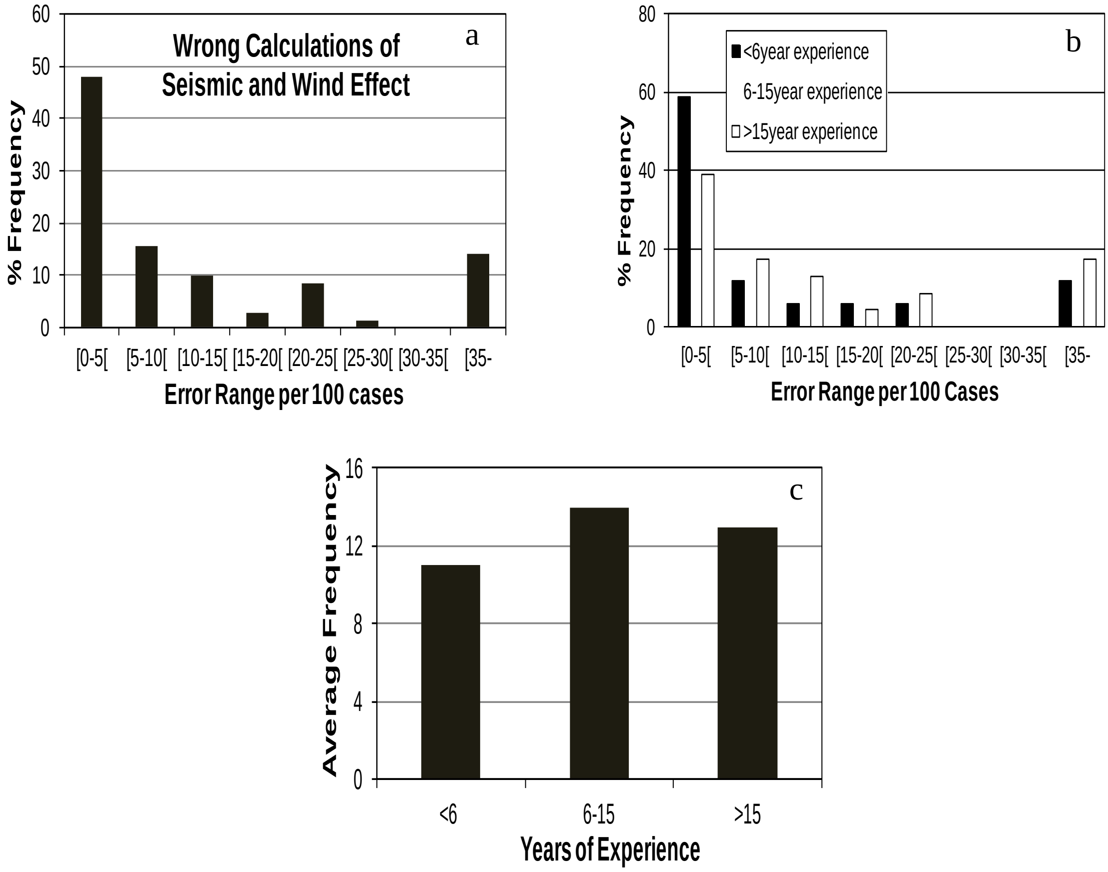

Figure 27: Survey results on wrong calculations of seismic and wind load effect

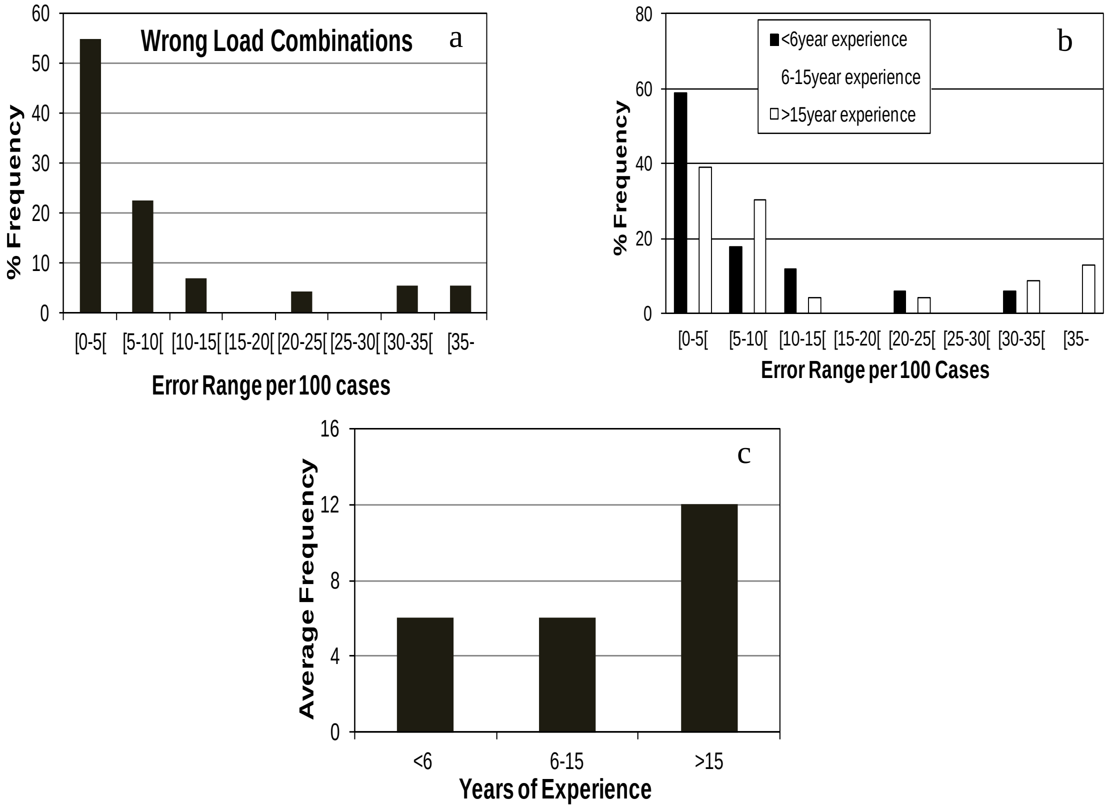

Figure 28: Survey results on wrong load combinations

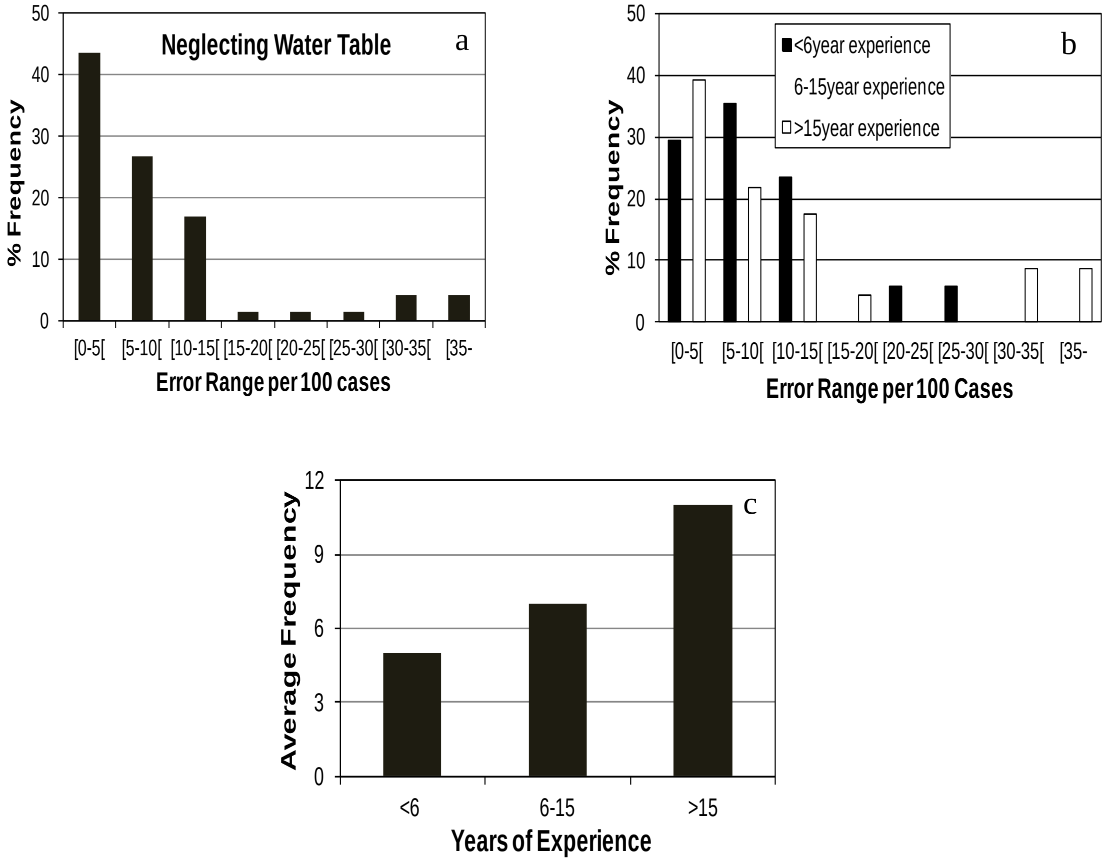

Figure 29: Survey results on neglecting water table in foundation design

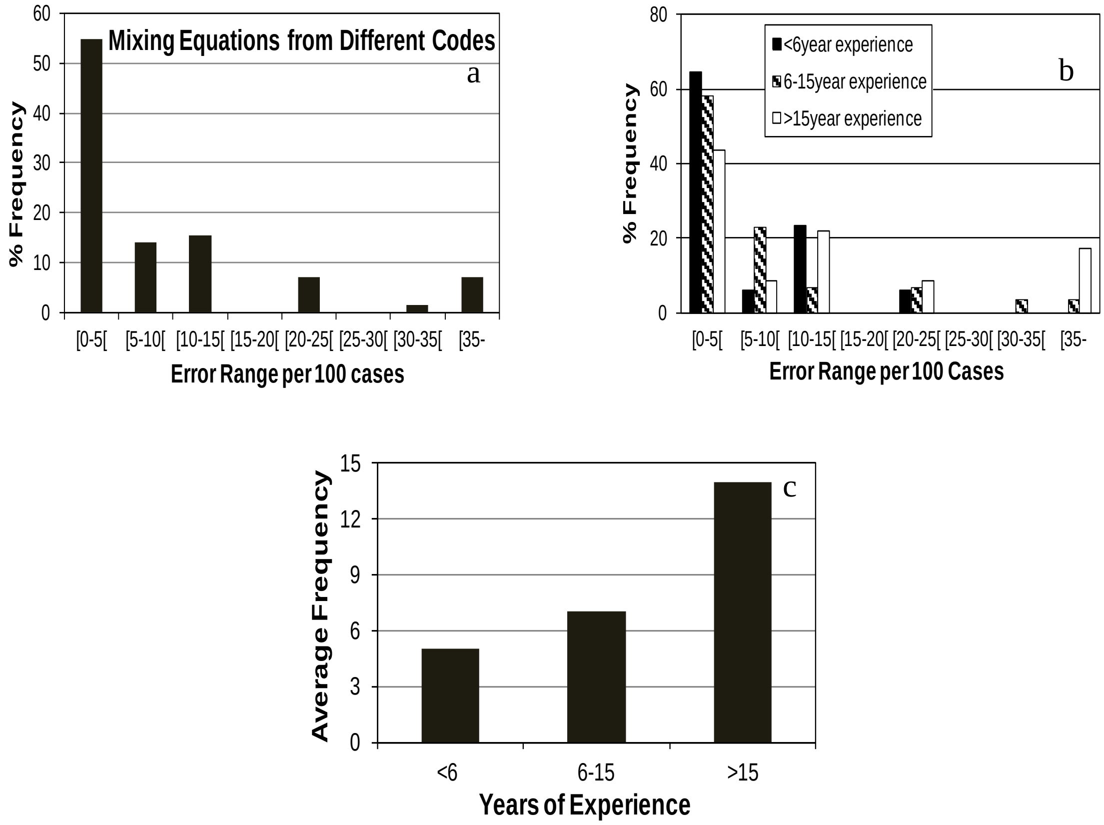

Figure 30: Survey results on mixing equations from different codes

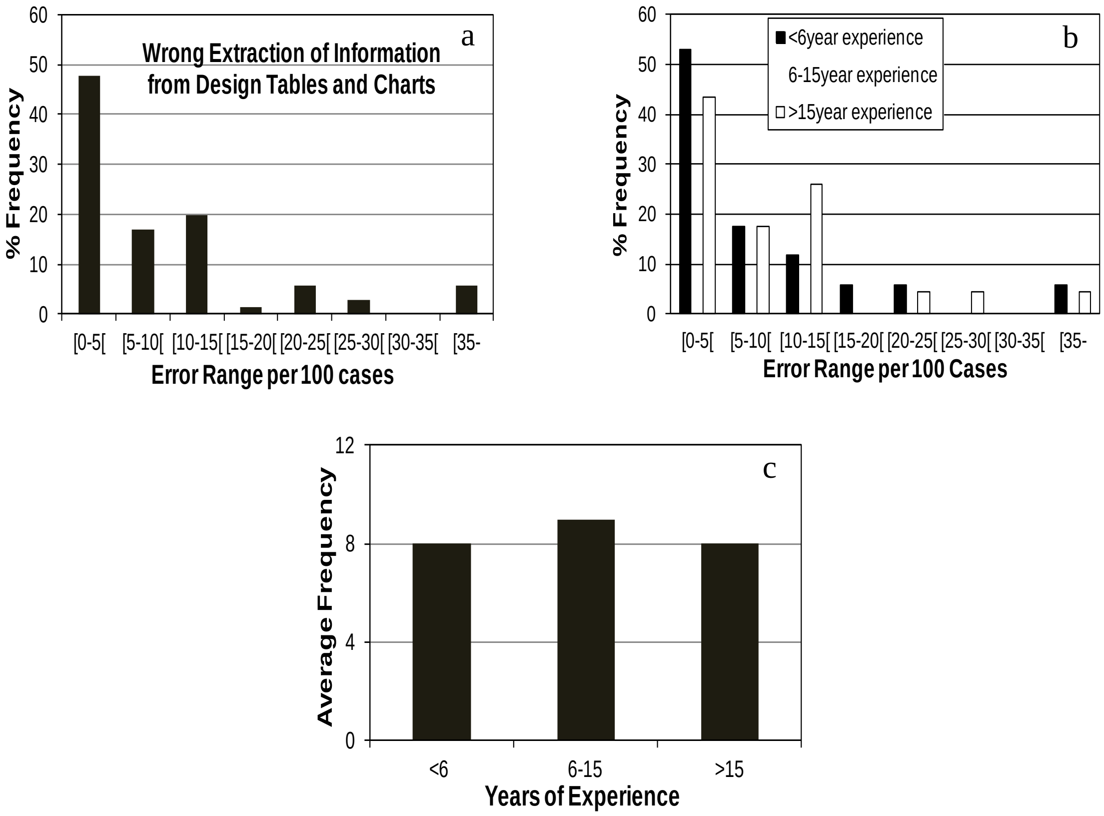

Figure 31: Survey results on wrong extractions of information from tables and charts

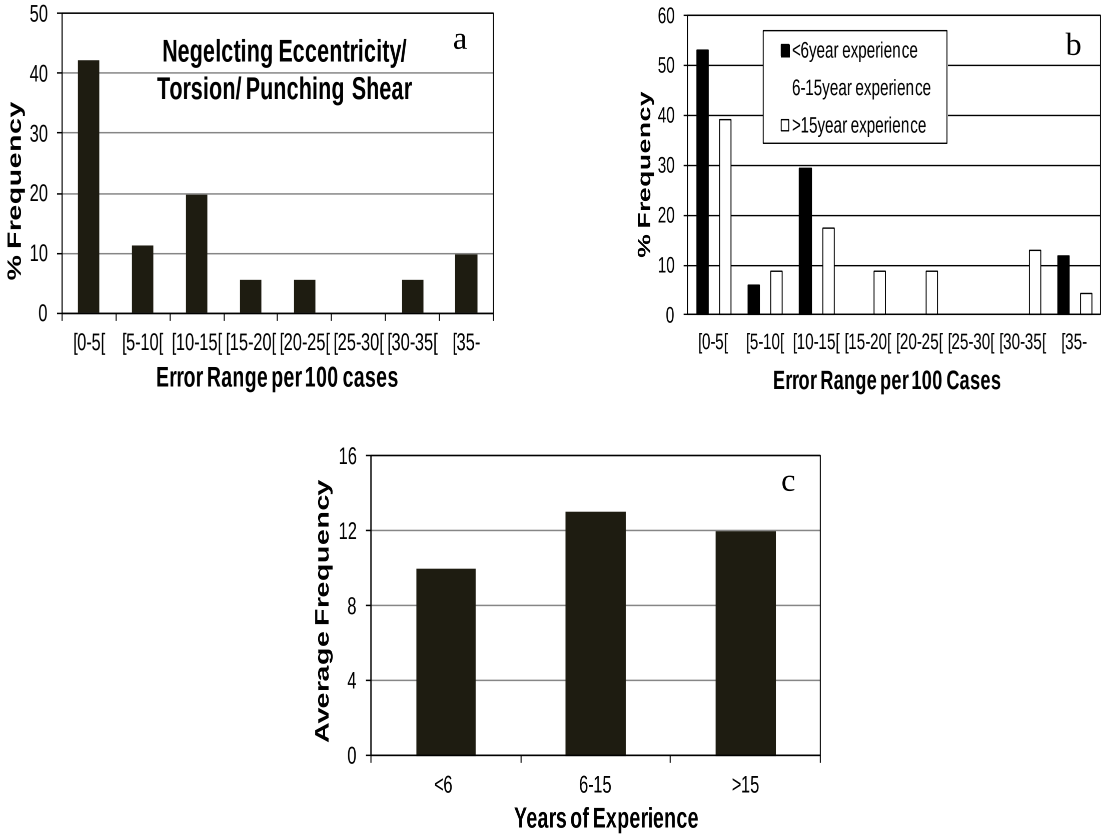

Figure 32: Survey results on neglecting load eccentricity, torsion or punching shear

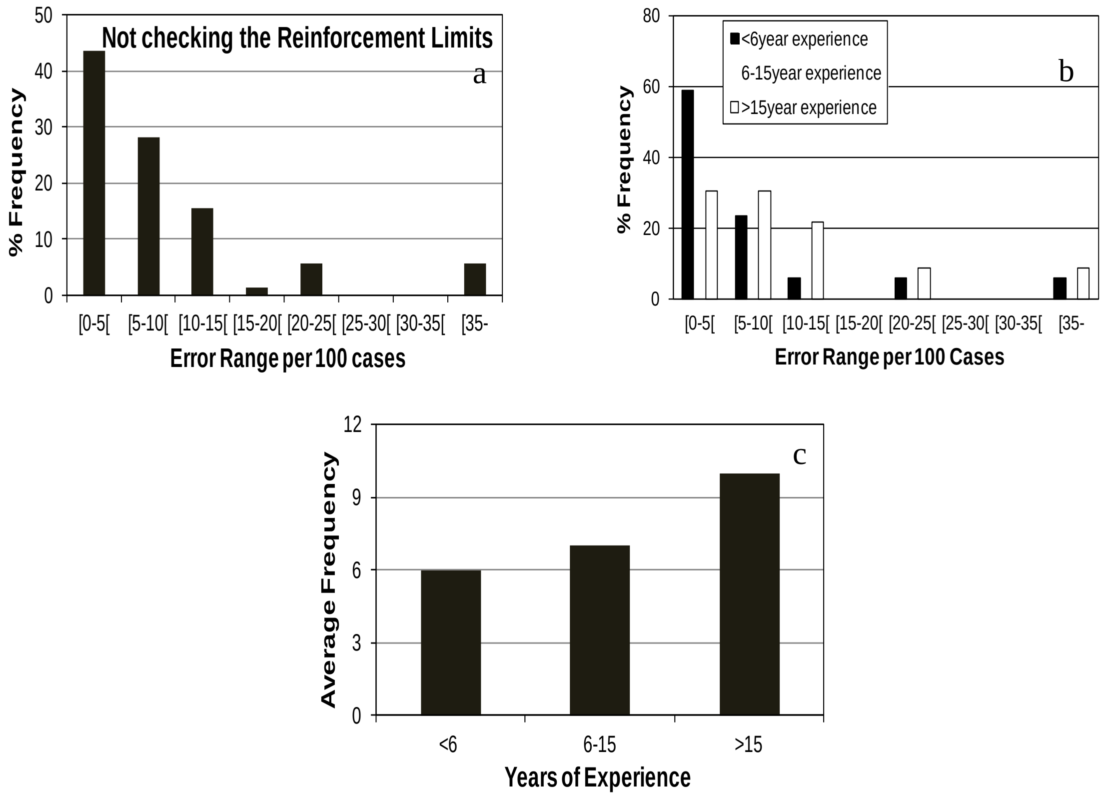

Figure 33: Survey results on lack of checking the reinforcement limits

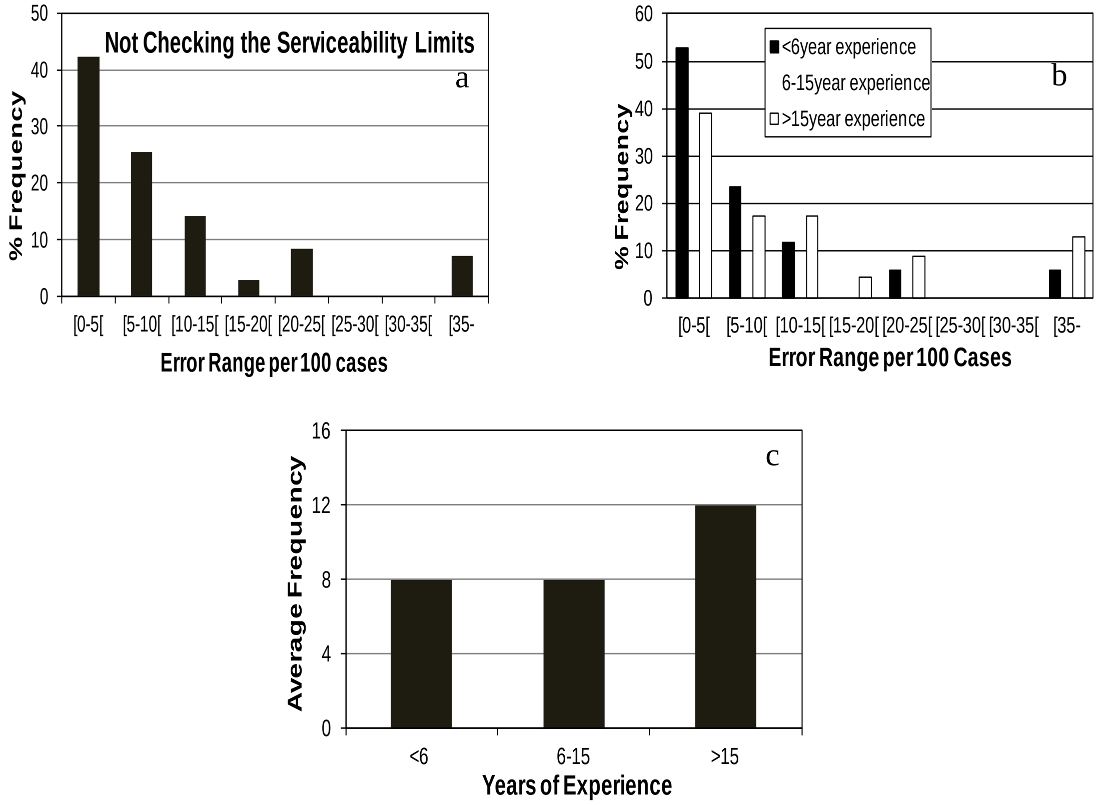

Figure 34: Survey results on lack of checking the serviceability limits

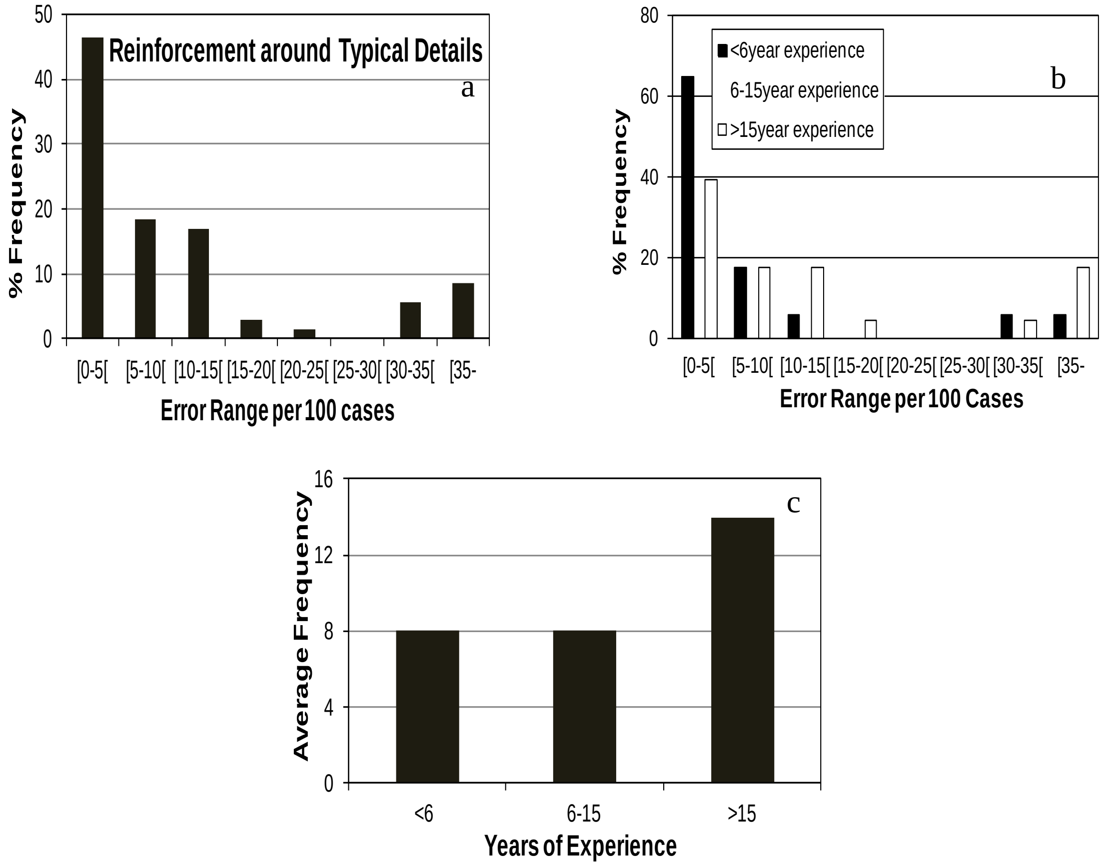

Figure 35: Survey results on reinforcement around typical details

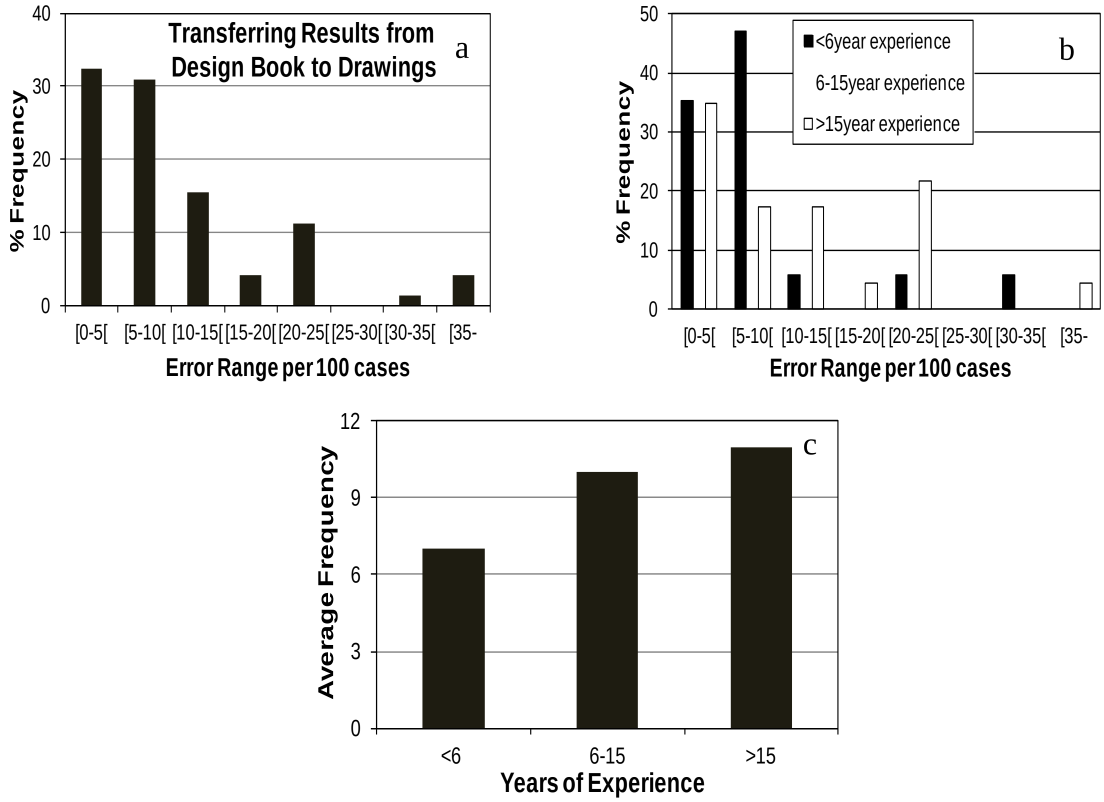

Figure 36: Survey results on misrepresenting results from design book into drawings

The surveys distributed to consultants (designers) have also shown that different types of human errors at different percentages are perceived to be committed during design stages. When comparing the ranges of error with respect to their frequency, without taking into account the number of years of experience, as highlighted in all graphs (a), shown in top left figure, it turned out that for all the investigated cases, the most frequent range of error was the “less than 5 cases per 100 cases” range. However, errors were also reported within other ranges, but at lower frequency. It shall be noted here that for the case of transferring results from design calculation book to drawings, shown in Figure 36.a, the error range of “5-10 cases per 100 cases” happened almost at the same frequency of the “less than 5 cases” range, which indicates that the possibility to commit errors at this stage is very high, compared to other cases. Note also that for the case of unit error, illustrated in Figure 24.a, committing more than “15 cases per 100 cases” error was very rare.

Classifying errors according to the designers’ experience, as highlighted in graphs (b), shown in top right figure, has given the following outcomes:

- For mistakes committed in units, calculation of seismic and wind forces, load combinations, mixing different design codes, extraction of information from design charts and tables, neglecting eccentricity, neglecting serviceability limit, lack of knowledge in using design software, and providing reinforcement around typical details, the surveys have shown that the most frequent range of error was the "less than 5 cases per 100 cases" range.
- Regarding conceptual mistakes, most frequent errors encountered by designers with more than 6 years of experience were the errors of "less than 5 cases per 100 cases". On the other hand, ranges "less than 5 cases per 100 cases", "5-10 cases per 100 cases", and "10-15 cases per 100 cases" were the most frequent committed ranges encountered by "slightly experienced" designers, and were almost committed at the same frequency.
- In addition, the filled surveys have pointed out that most frequent error range of calculation mistakes and neglecting water table in the calculation experienced by "moderately experienced" and "highly experienced" designers is "less than 5 cases per 100 calculation steps/cases". Yet, the most frequent error range encountered by "slightly experienced" designers was the "5-10 cases per 100 calculation steps/cases"
- Moreover, "slightly experienced" and "moderately experienced" designers were very conservative in estimating the errors encountered due to not checking the reinforcement limit. On the opposite, "highly experienced" designers encountered almost the same frequency for the "less than 5 cases per 100 cases" and "5-10 cases per 100 cases" error ranges.
- Furthermore, "highly experienced" designers were very conservative in estimating errors encountered when results are transferred from design calculation book to drawings, unlike designers with less experience, whose most frequent range of error in this regard was "5-10 cases per 100 cases".

The results obtained when comparing designers with different experiences in terms of the average frequency of errors they encountered, shown in graphs (c), at the bottom of the figure, can be grouped into the following categories:

- All are the same except for the "slightly experienced" designers group. This has been noticed in the errors encountered due to calculation mistakes (graph 25.c) and errors due to lack of knowledge in using design software (graph 26.c). In both cases, the “slightly experienced” designers claimed that they have encountered fewer errors.

- All are the same except for the “moderately experienced” designers group. This can be seen in errors encountered due to conceptual mistakes, graph 23.c, where the average frequency of errors encountered by “moderately experienced” designers were less than the other two surveyed groups. The opposite scenario was found in the reported errors due to wrong extraction of information from design tables and charts, graph 31.c, as the errors encountered by “moderately experienced” designers were more than the other two groups.
- All are the same except for the “highly experienced” designers group. This can be observed in the cases related to errors in units, errors in load combinations, errors due to not checking the serviceability limit, and errors in providing inappropriate reinforcement around typical details, shown in graphs 24.c, 28.c, 34.c, and 35.c. For the unit error case, the errors encountered by “highly experienced” designers were less than the other two groups. For the other three cases, “highly experienced” designers encountered more errors than the other two groups.
- No ordering. This is illustrated in graph 27.c, wrong calculation of seismic and wind loads effect, and graph 32.c, neglecting torsion, eccentricity, and punching shear. Note that the average frequency for the three groups is different, given that the largest average frequency was encountered by “moderately experienced” designers.
- Increase in the average frequency with the increase in number of years of experience. This was observed in the errors encountered because of neglecting water table, mixing equation from different codes, not checking the reinforcement limit, and wrong transferring of results from design book to drawings. Refer to graphs 29.c, 30.c, 33.c, and 36.c respectively.

4.4 Final Remarks

In conclusion, the surveys’ results from engineers and designers working in the UAE have shown that the range of errors differs from one engineer/designer to another, depending on which activity the error is committed, the number of years of experience the engineer/designer has, as well as whether the engineer is working as a contractor or as a consultant.

Although the most frequently committed range of error was the “less than 5 cases per 100 cases” range, the surveys have shown that some engineers/designers encounter errors at a higher range. The surveys have also pointed out that contractors are more conservative than designers in reporting the errors they have encountered.

CHAPTER 5

DETERMINISTICALLY-BASED SENSITIVITY ANALYSIS FOR STRUCTURAL MEMBERS

5.1 Introduction

Sensitivity analysis is essentially a parametric study that is used to identify the most sensitive parameters to the outcome of a given function or activity. In structural engineering and construction, sensitivity analysis helps determine the parameters that deserve more quality control during design or construction. In this study, this approach is used to identify the critical design and construction variables to the nominal capacity and structural safety [8]. Sensitivity analysis is also used to quantify the extent of loss of nominal capacity and reliability when a variable is compromised by a specified percentage.

There are two approaches to sensitivity analysis: (1) deterministically-based approach, and (2) reliability-based approach. In the deterministic approach, the effect of varying any of the design variables on the nominal capacity (as specified by the code) is measured, whereas the reliability based approach shows how sensitive the reliability index, and consequently the structural safety, is to the variations in the design variables. In this chapter, the deterministic approach will be used to investigate the sensitivity of nominal capacities of beams under flexure, beams under shear, and axially loaded columns. The nominal capacity is calculated using the ACI 318-08 structural concrete code [40], which is an approved code of design for reinforced concrete structures by all municipalities in the UAE.

5.1.1 Flexure in Beams

According to ACI 318-08, the design moment capacity of an under-reinforced beam with rectangular section, ϕM_n, subjected to gravity loads is calculated using the equation:

$$\emptyset M_n = \emptyset A_s f_y (d - 0.5a) \geq 1.2 M_D + 1.6 M_L \tag{5.1}$$

where the Whitney block depth from extreme compression fibers is given by:

$$a = \frac{A_s f_y}{0.85 f'_c b} \tag{5.2}$$

and ϕ = strength reduction factor that depends on the strain in the extreme layer of steel, M_n = nominal flexural capacity, A_s = area of tension steel reinforcement, f_y = steel yield strength, d = effective depth of tension steel reinforcement, M_D = applied nominal dead load moment, M_L = applied nominal live load moment, f'_c = specified compressive concrete strength, and b = width of beam.

In order to conduct the deterministic sensitivity analysis on a beam under flexure, a reference cross section was used in the study. Also, the reference beam was assumed to be subjected to dead moment equal to live moment, which is common in reinforcement concrete beams. From Equation 5.1, this assumption results in the following nominal flexural capacity:

$$M_n = A_s f_y (d - 0.5a) = 2.8 M_D / \emptyset \tag{5.3}$$

The reference cross section was assumed to have the following properties:

- 400 mm by 800 mm cross section.
- 50 mm clear concrete cover on stirrups.
- Compressive strength of concrete is 42 MPa.
- Yield strength of reinforcement is 420 MPa.
- Longitudinal tension steel is 4 No. 32 bars (i.e. A_s = 3217 mm^2).
- No. 12 closed stirrups at 200 mm spacing.

With these design parameters, the nominal moment capacity of this beam is 912 kN-m. Since dead moment and live moment were assumed to be equal, the nominal dead load moment and the live load moment are equal to 293 kN-m. The moment capacity that was just computed for this reference cross section represent the moment capacity assuming that the beam has been designed or constructed without any human error being committed.

Now, let us assume that when this beam was constructed at site, only 85% of the required area of steel was provided. This reduction in steel could be due to an error in design (such as mis-calculation) or construction mistake (providing fewer bars or smaller size bars than necessary). Using the bending moment capacity equation in the ACI 318-08 code, the nominal flexural capacity of this beam will be reduced to 783 kN-m, which is equivalent to a reduction of 14.1% in the design moment capacity of this beam.

Repeating the same calculations for 30% and 45% reductions in the area of tension steel reinforcement, the reductions in the nominal flexural capacity would be 28.1% and 43.9%, respectively. These results are summarized in Figure 37.

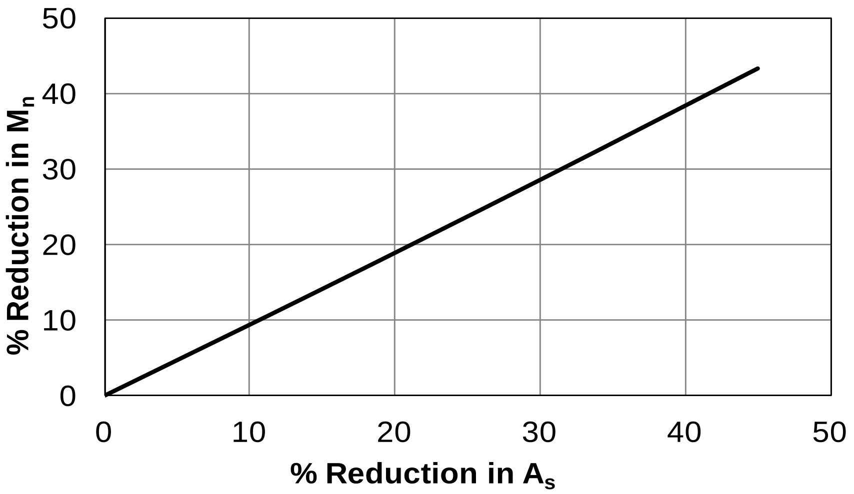

Figure 37: Effect of reduction in area of steel on the flexural capacity

When the same procedure is followed with the remaining design parameters, including the effective depth of tension steel (d), width of the beam (b), compressive strength of concrete (f'_c), yield strength of longitudinal reinforcement (f_y), dead load moment (M_D) and live load moment (M_L), the results obtained are as presented in Figure 38.

Note that reduction in depth of tension steel could be due to movement in rebar cage during concreting, reduction in beam width could be due to shifting of formwork during construction, and lower yield strength could be due to ordering wrong material on site. The reduction in dead load and live load moments could be due to miscalculation of load, moment diagram, misreading from software output, or misunderstanding of structural system, boundary condition or load path.

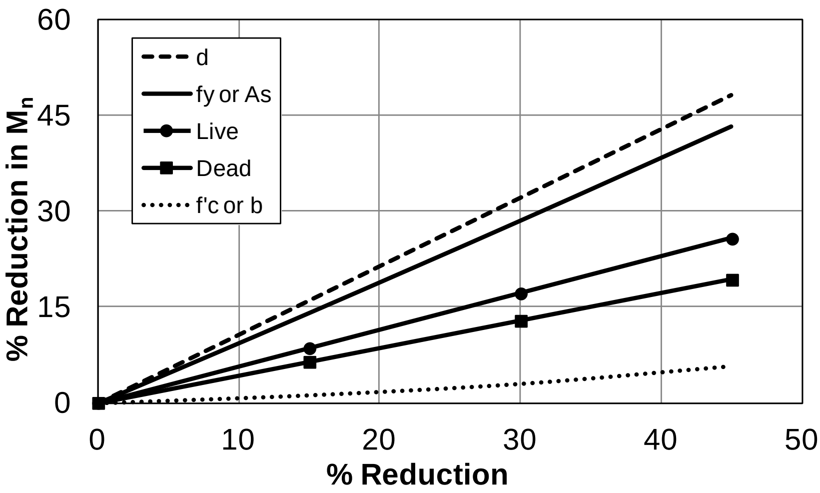

Figure 38: Effect of variations in design variables on nominal flexural capacity

Figures 38 indicates that beams under flexure are more sensitive to the reduction in the effective depth of tension steel, whereas they are less sensitive to the reduction in the compressive strength of concrete or the width of the beam. Also, beams under flexure have similar sensitivity to the reduction in the yield strength and the area of steel, and similar sensitivity to the reduction in the compressive strength of concrete and the width of the beam. This is because the two pair of variables (f_y and A_s, and f'_c and b) are always multiplied by each other in the equations of the nominal flexural capacity.

5.1.2 Shear in Beams

According to ACI 318-08, the design shear capacity of a beam with rectangular or flanged section and lateral reinforced with stirrups, ϕV_n, and subjected to gravity loads is calculated using the equation:

$$\emptyset V_n = \emptyset(0.17\sqrt{f'_c} b_w d + A_v f_y d/s) \geq 1.2V_D + 1.6V_L \quad (5.4)$$

where ϕ = strength reduction factor equal to 0.75, V_n = nominal shear capacity, A_v = total area of transverse steel reinforcement, f_y = transverse steel yield strength, d = effective depth of tension steel reinforcement, s = spacing of stirrups, V_D = applied nominal dead load shear, V_L = applied nominal live load shear, f'_c = specified compressive concrete strength, and b_w = narrowest with of the beam.

To investigate the effect of human errors committed during design and construction stages on the nominal shear capacity of beams, the following reference cross section was used, with the assumption that the beam is subjected to dead shear equivalent to live shear:

- 400 mm by 800 mm cross section.
- 50 mm clear concrete cover on stirrups.
- Compressive strength of concrete is 42 MPa.
- Yield strength of transverse reinforcement is 420 MPa.
- Longitudinal tension steel is 4No. 32 bars (i.e. $A_s = 3217\ mm^2$).
- No. 12 closed stirrups at 200 mm spacing.

The nominal shear capacity of this beam will be 661 kN, while the live load shear and the dead load shear are both equal to 177 kN. Again, the above calculated nominal shear capacity represents an "intact" condition, such that this beam has been designed and constructed without any human error being committed during either the design stage or construction stage.

When performing deterministically-based sensitivity analysis for all the design parameters contributing to the shear capacity of this cross section, the results are as shown in Figure 39.

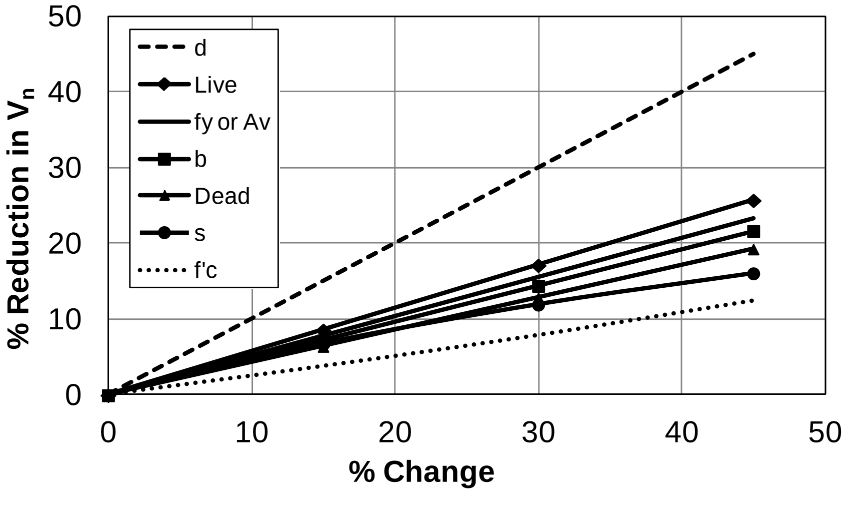

Figure 39: Effect of variations in design variables on nominal shear capacity

The above figure shows that beams under shear are most sensitive to the reduction in the depth of tension steel, whereas they are least sensitive to the reduction in the compressive strength of concrete. It can be observed also that the yield strength of stirrups and the area of stirrups have the same effect on the reliability index and the shear capacity of beams, as they are presented as a product within the capacity equation. The effective depth of longitudinal steel reinforcement is the most important parameter for shear strength due to the fact that it contributes to both the concrete shear strength and the stirrups shear strength. On the hand, the concrete strength's contribution is not that predominant because it affects only the shear strength of the concrete and it is under the square-root.

In summary, the deterministic analysis performed on beams under shear has indicated that the reduction in the depth of tension steel due to human errors has high impact on nominal shear capacity of beams. Changes in the live load shear, dead load shear, compressive strength of concrete, stirrup spacing, area of stirrups, and the yield strength of stirrups have moderate impact on the shear capacity of beams.

5.1.3 Axially Loaded Columns

The design axial capacity of a tied column is calculated using the equation:

$$\emptyset P_n = \emptyset 0.8\{0.85 f'_c (A_g - A_s) + A_s f_y\} \geq 1.2 P_D + 1.6 P_L \tag{5.5}$$

To examine the effect of human errors committed during design and construction stages on the nominal capacity of axially loaded columns, the following reference cross section was used:

- 500 mm by 500 mm square cross section
- Compressive strength of concrete is 42 MPa
- Yield strength of longitudinal reinforcement is 420 MPa
- Longitudinal reinforcement consists of 8 No. 32 bars (ρ = 2.57%)

The axial nominal capacity of this reference column is 9118 kN.

After carrying out the deterministic sensitivity analysis for other design parameters that may influence the axial load capacity of this column, the following results are obtained, as shown in Figure 40.

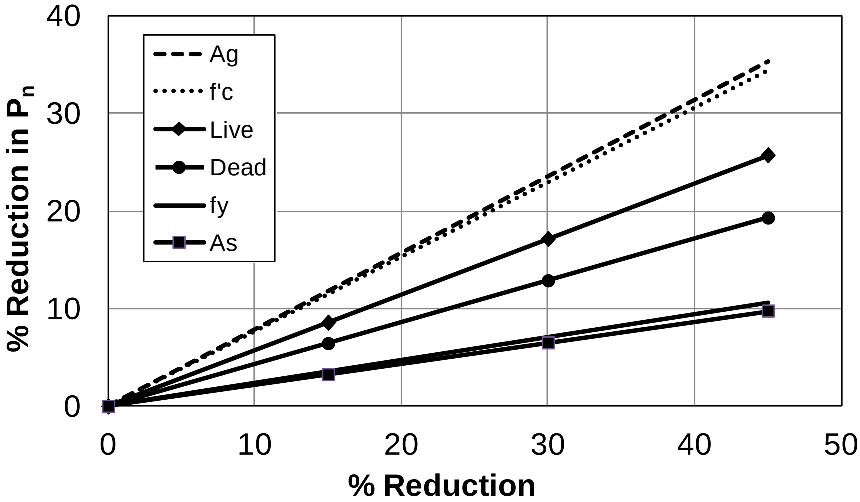

Figure 40: Effect of variations in design variables on compressive capacity

Sensitivity analysis results for the axially loaded columns, as summarized in the Figure 40, show that the axial capacity is more sensitive to the reduction in the column's gross cross-sectional area and the compressive strength of concrete. On the contrary, the capacity of axially loaded columns is less sensitive to the reduction in the area and yield strength of reinforcement. As for the effect of reducing the dead and live axial load on the nominal axial capacity of the column, the results show that it can be considered as moderate.

5.1.4 Comparisons between Members in Flexure, Shear, or Compression

In this section, comparisons are made amongst beams under flexure, beams under shear, and columns subjected to axial compression with respect to the reduction in the capacity due to human errors in the relevant design variables. The purpose of the analysis is to show how one variable, such as the steel reinforcement yield strength, can have more (or less) impact on one limit state, such as flexure, than another (such as shear or axial compression). The results of the comparisons are presented in Figures 41.

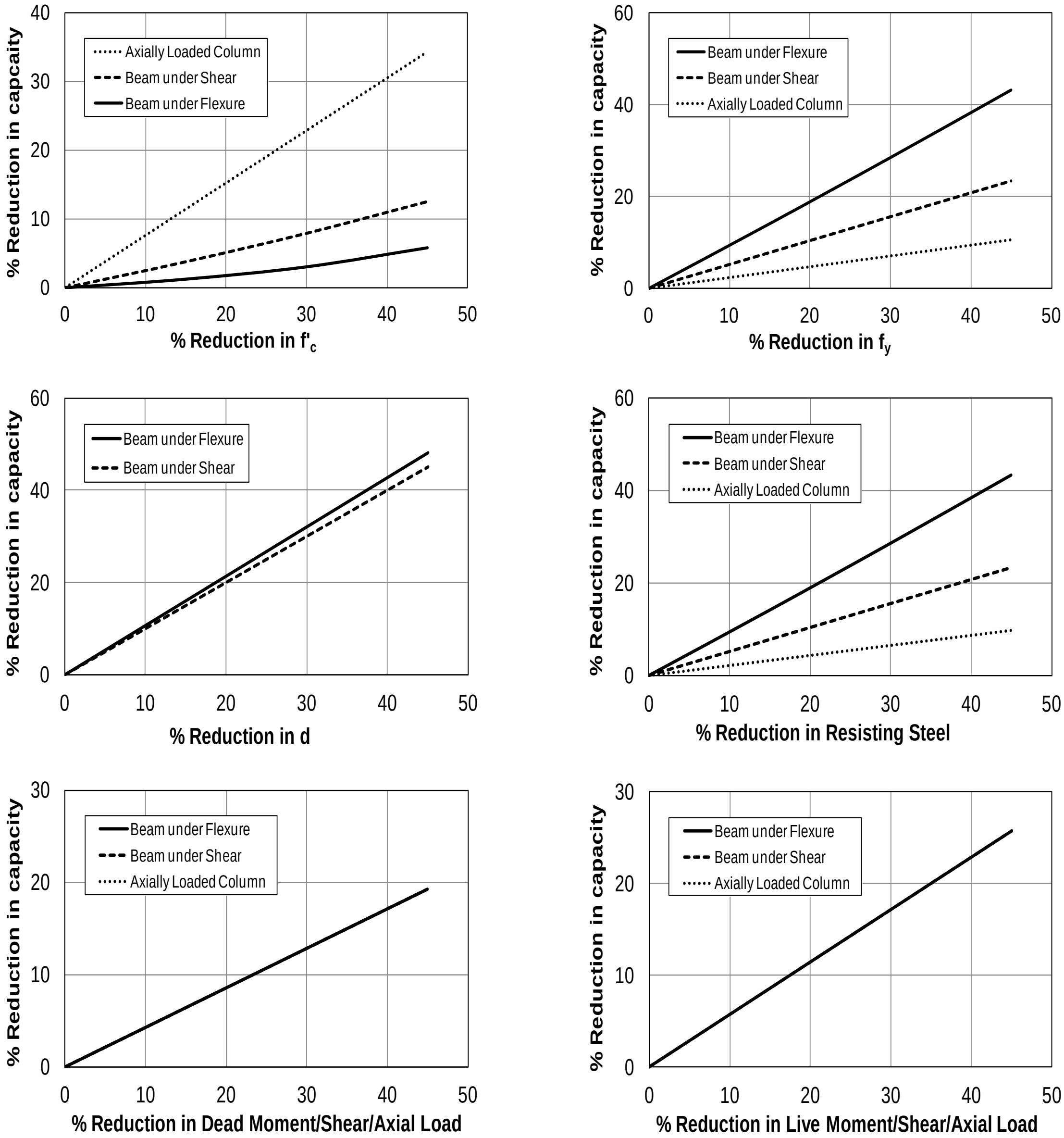

Figure 41: Comparison among flexure, shear, and axial compression results

Figure 41 shows that beams under flexure are most sensitive to reductions in reinforcement yield strength and area of longitudinal steel. Axially loaded columns are the most sensitive to changes in concrete compressive strength. For the remaining scenarios, the sensitivity is the same.

The previous analysis only considers one standard cross-section with given material properties, section dimensions and reinforcement. For reinforced concrete members having different characteristics than the considered (such as 21 MPa concrete compressive strength instead of 42 MPa, 250 MPa steel reinforcement yield strength instead of 420 MPa, 400 mm beam thickness instead of 800 mm, or live load being twice dead load instead of equal to it), one needs to determine whether the previous findings are applicable or not. To investigate this issue, sensitivity analysis was performed on different cross sections with various properties for beams under flexure, beams under shear, and columns under axial compression. This is discussed in details in the following sections.

5.2 Deterministically Based Sensitivity Analysis on Different Cross Sections

5.2.1 Beams under Flexure

In this section, deterministic approach is performed on different cross sections with different properties in order to examine the reduction in the nominal capacity from one cross section to another. The purpose of this analysis is to show how a given reduction in a design parameter affects the required nominal strength in the structural design code, as opposed to the reliability analysis which shows the reduction in structural safety due to changes in the design variables. Such an analysis is helpful for those who do not have background in reliability methods. The same cases that have been investigated earlier are considered in this section:

1. Three cross sections with the same design parameters, but with different area of steel (1608.5 mm^2, 3217 mm^2 and 6434 mm^2).
2. Three cross sections with the same design parameters, but with different compressive strength of concrete (21 MPa, 42 MPa and 84 MPa).
3. Three cross sections with the same design parameters, but with different yield strength of reinforcement (250 MPa, 420 MPa and 500 MPa).
4. Three cross sections with the same design parameters, but with different thickness (500 mm, 800 mm and 1100 mm).
5. Three cross sections with the same design parameters, but with different live-to-dead load fraction of the total load (M_D=2M_L, M_D=M_L, and M_L=2M_D).

The sensitivity analysis yields the results shown in Figures 42-46, which are discussed next.

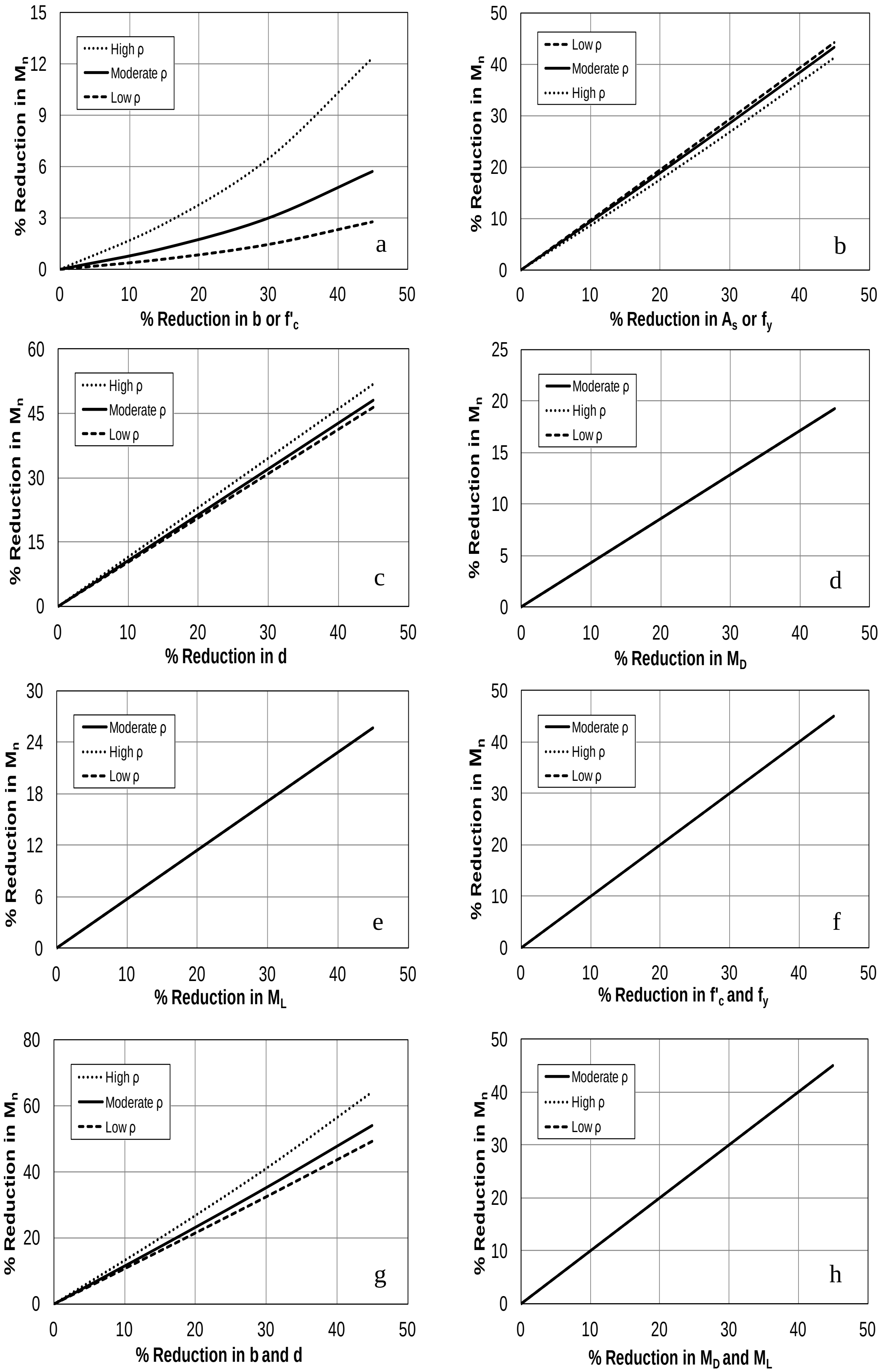

Figure 42: Effect of reinforcement ratio on the nominal flexural capacity of beams

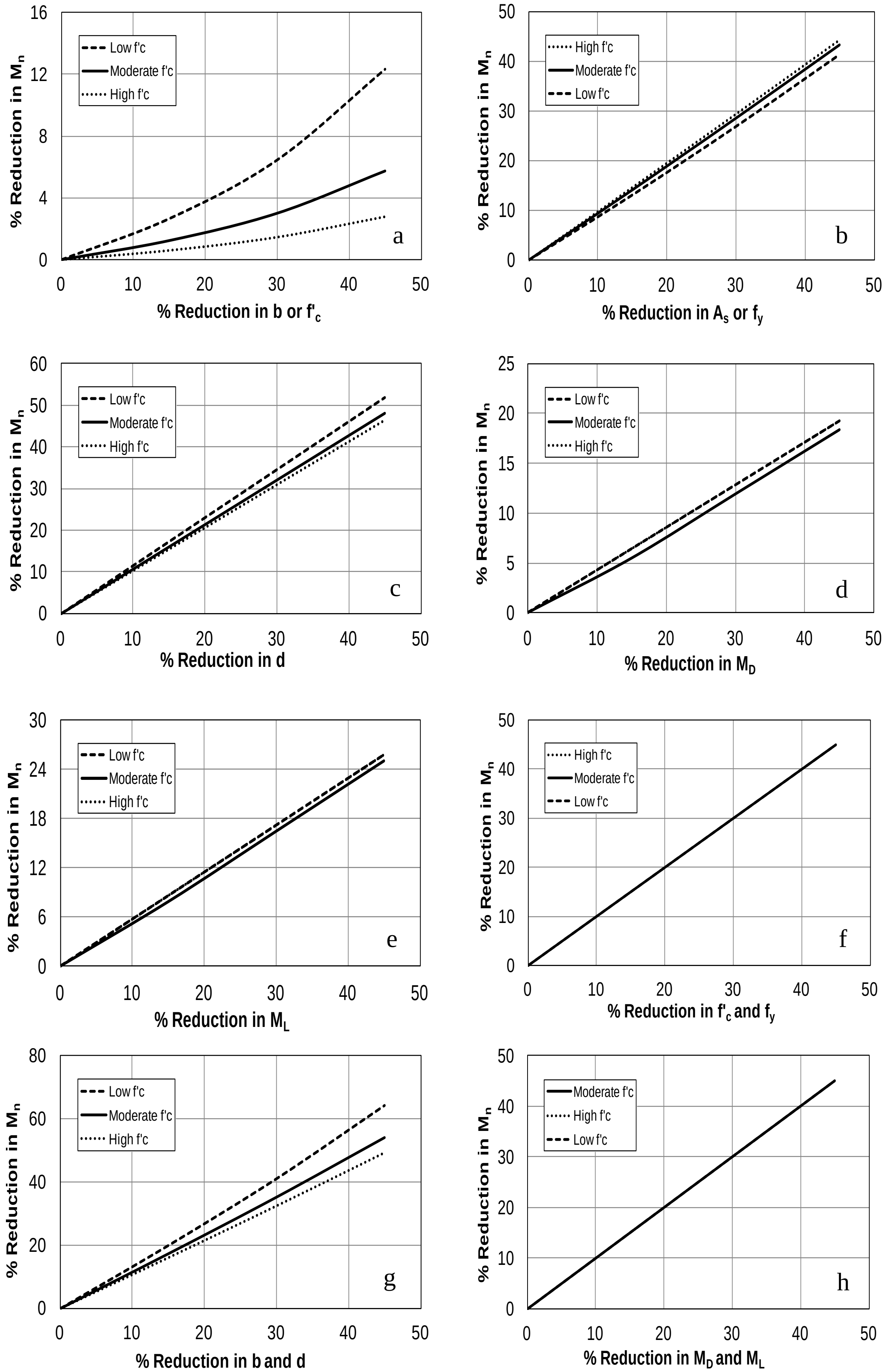

Figure 43: Effect of concrete strength on the nominal flexural capacity of beams

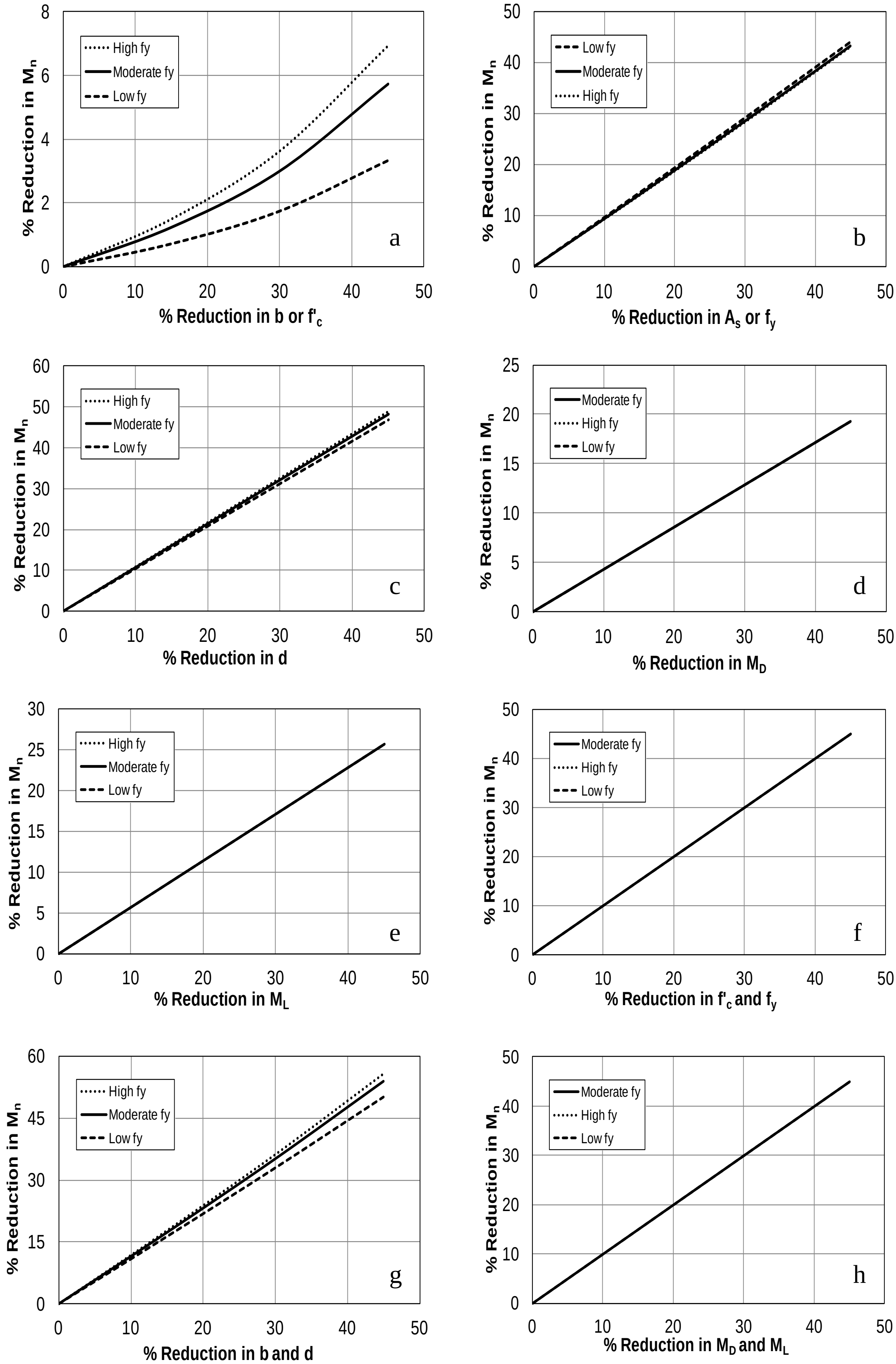

Figure 44: Effect of steel yield strength on the nominal flexural capacity of beams

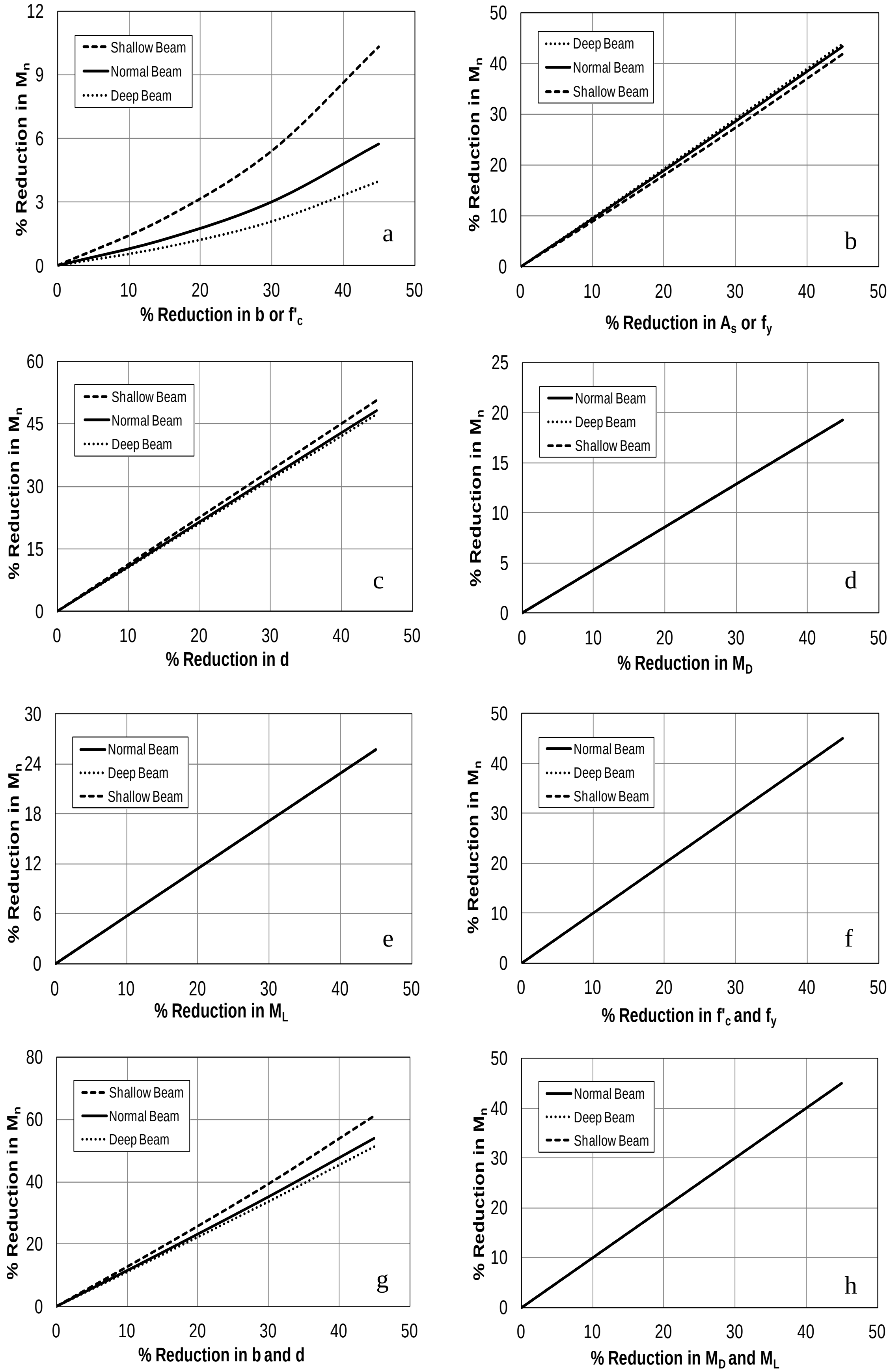

Figure 45: Effect of beam depth on the nominal flexural capacity of beams

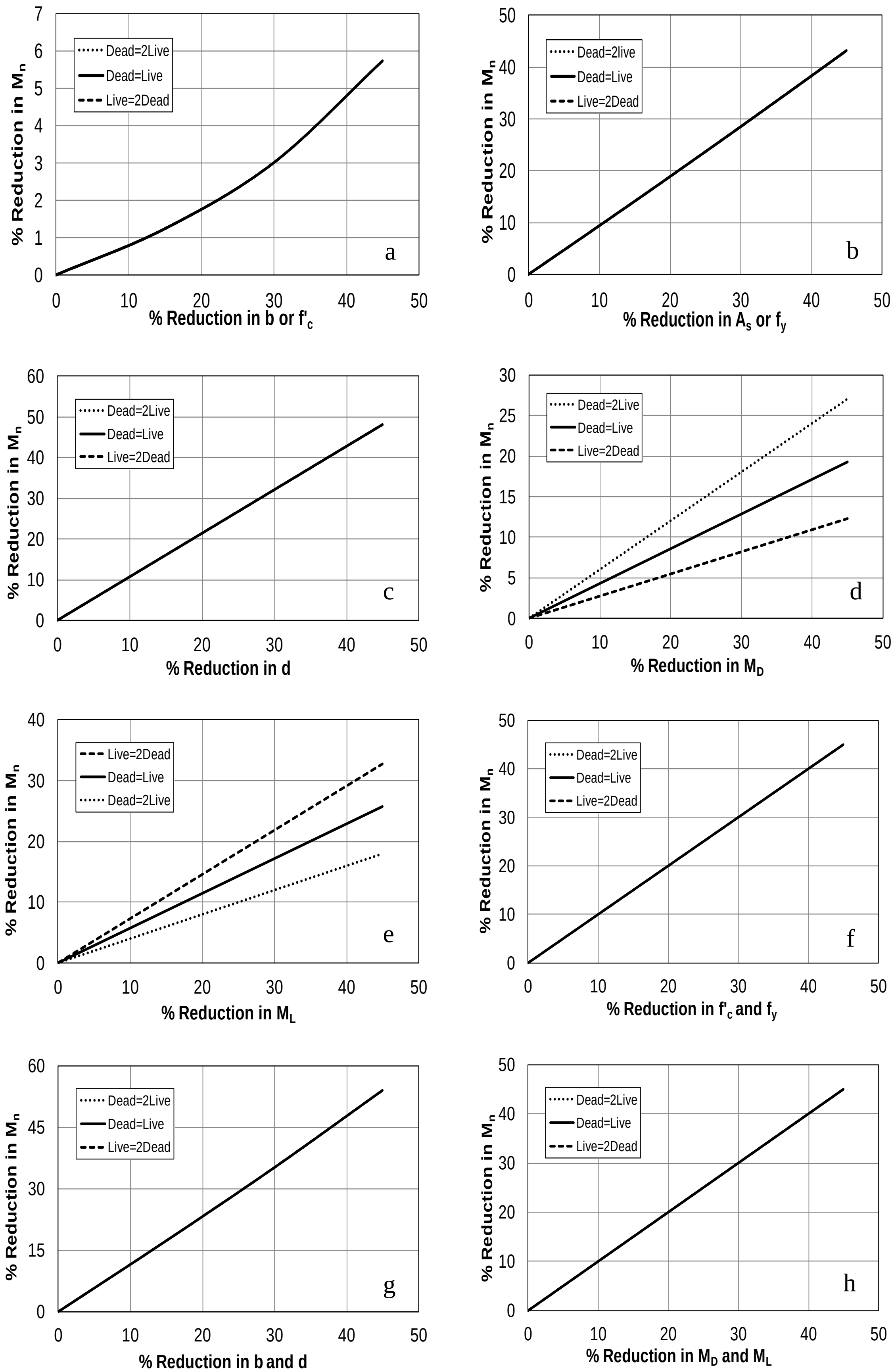

Figure 46: Effect of live-to-dead load fraction on the nominal flexural capacity of beams

The sensitivity analysis showed that the results varied for one cross section to another. To start with, when the area of longitudinal steel was the only difference among the considered cross sections, beams under flexure designed with higher reinforcement ratio were the most sensitive to the reduction in concrete compressive strength, beam width, and effective depth of tension steel.

The above conclusion is also valid for the cases in which concrete compressive strength, reinforcement yield strength, or beam depth is the controlling factor. In this extent, beams under flexure designed with low concrete compressive strength, beams under flexure designed with high reinforcement yield strength, and beams under flexure designed with shallow depth are the most sensitive to the reduction in the compressive strength of concrete, the width of the beam, or the effective depth of tension steel.

In the last case, in which the loading is governing the three considered cross sections, the sensitivity of the three cross sections was always the same, except when the error was committed in the dead moment or in the live moment. In the first scenario, the cross section designed with dead moment larger than live moment was the most sensitive to the errors committed in dead moment. On the contrary, the cross section designed with live moment larger than dead moment was the most sensitive to the reduction in the live moment.

5.2.2 Beams under Shear

Again, in order to check if the sensitivity analysis results obtained for the reference cross sections are also valid for beams under shear but with different cross sections and material properties, the deterministic sensitivity analysis approach was applied to the following cases:

1. Three cross sections with the same design parameters, but with different area of stirrups (113 mm^2, 226 mm^2 and 452 mm^2).
2. Three cross sections with the same design parameters, but with different compressive strength of concrete (21 MPa, 42 MPa and 84 MPa).

3. Three cross sections with the same design parameters, but with different spacing (100 mm, 200 mm and 350 mm).
4. Three cross sections with the same design parameters, but with different thickness (500 mm, 800 mm and 1100 mm).
5. Three cross sections with the same design parameters, but with different width of member (200 mm, 400 mm and 800 mm).
6. Three cross sections with the same design parameters, but with different live-to-dead load fraction of the total load (V_D=2V_L, V_D=V_L, and V_L=2V_D).

The results of the deterministically based sensitivity analysis are shown in Figures 47-52 and discussed next.

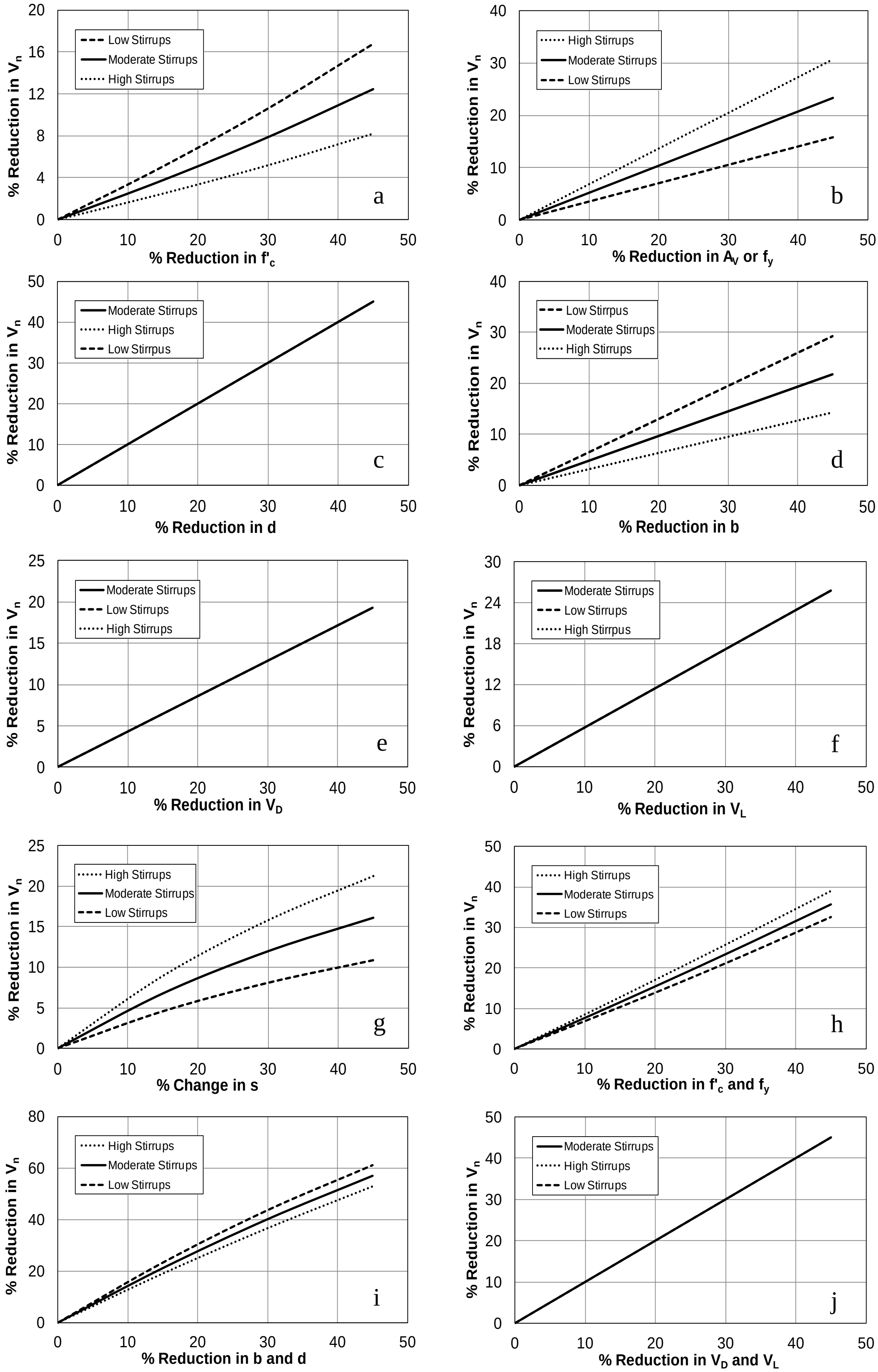

Figure 47: Effect of area of stirrups on the nominal shear capacity of beams

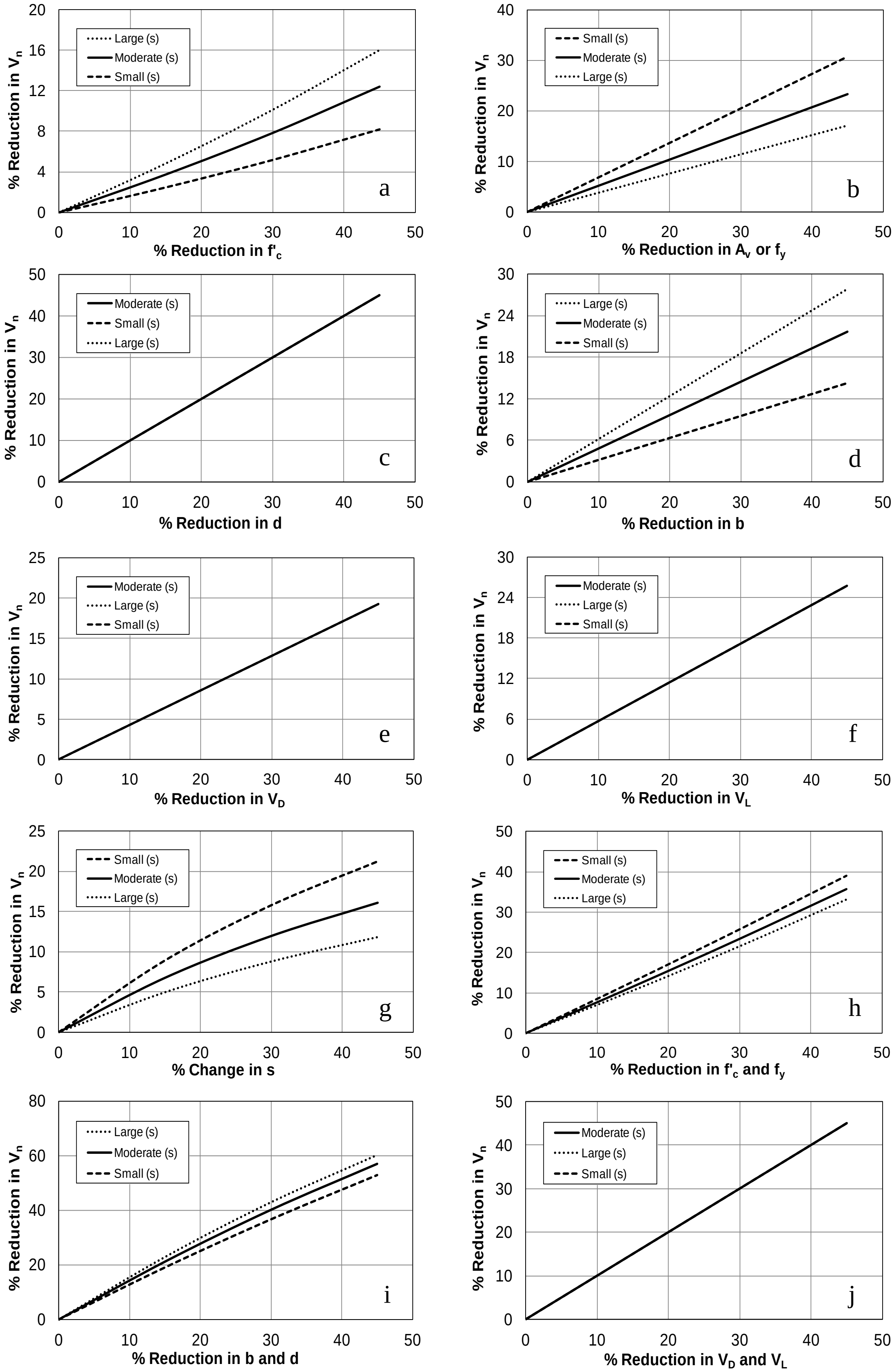

Figure 48: Effect of stirrup spacing on the nominal shear capacity of beams

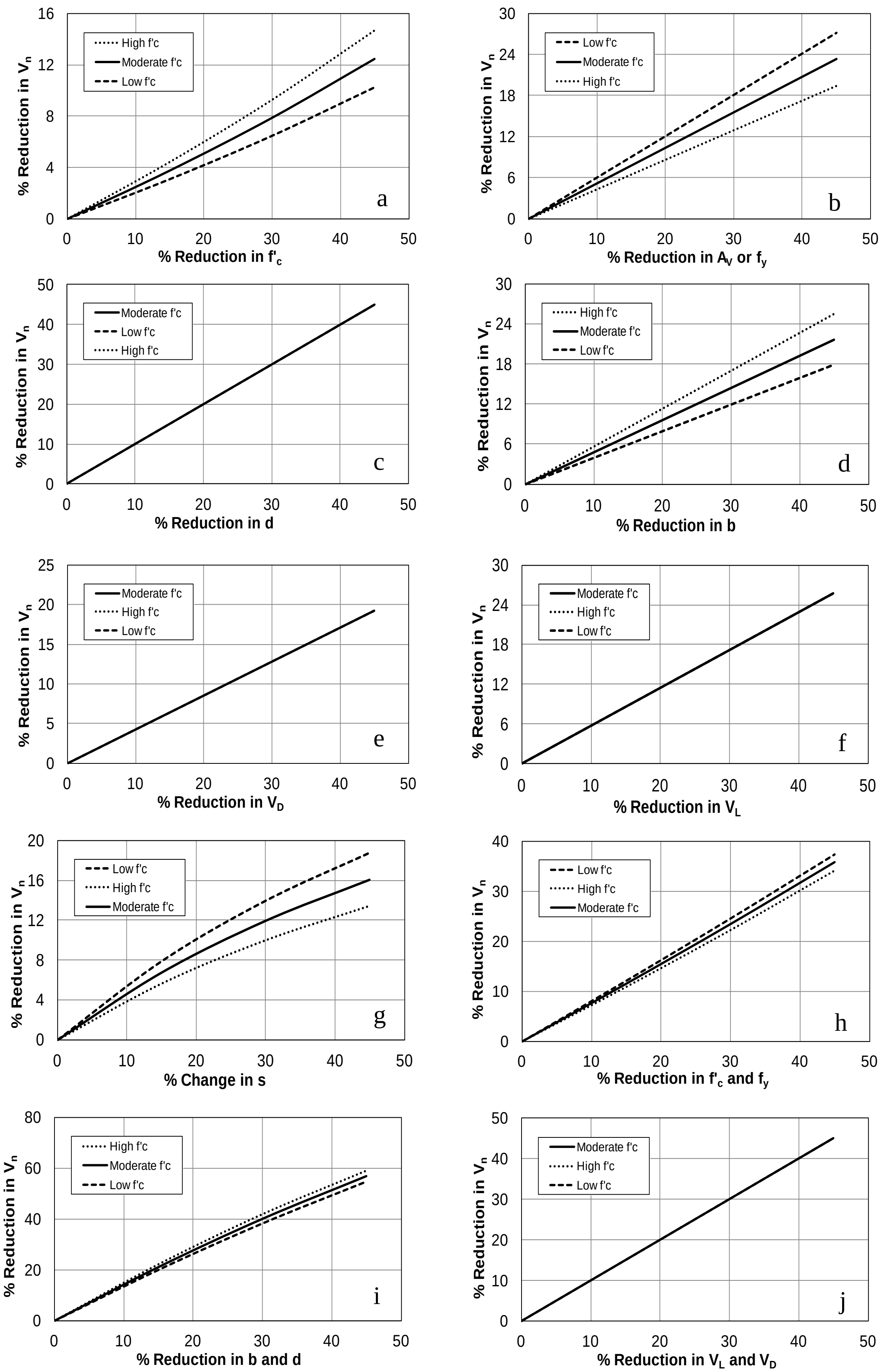

Figure 49: Effect of concrete strength on the nominal shear capacity of beams

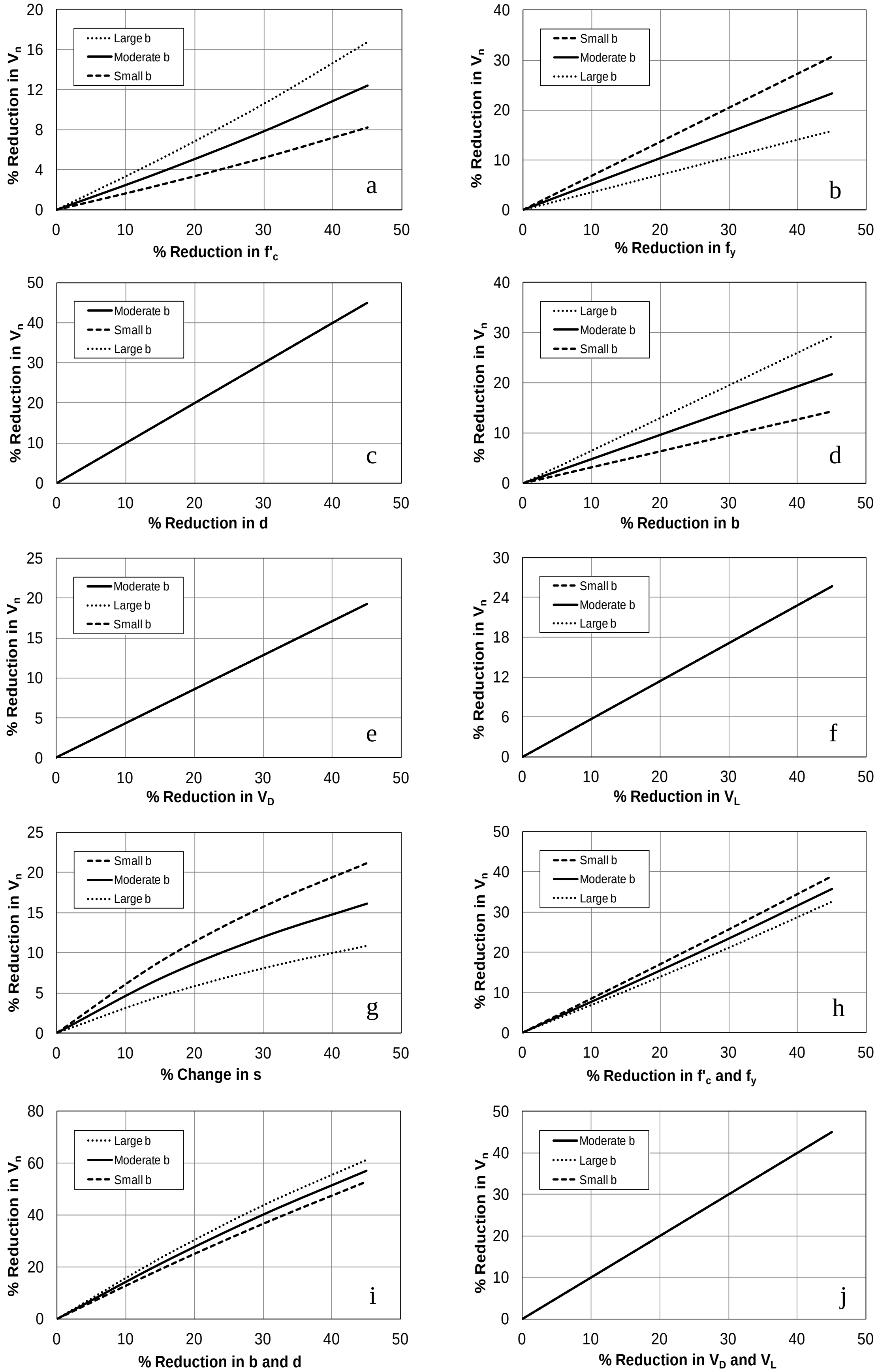

Figure 50: Effect of beam width on the nominal shear capacity of beams

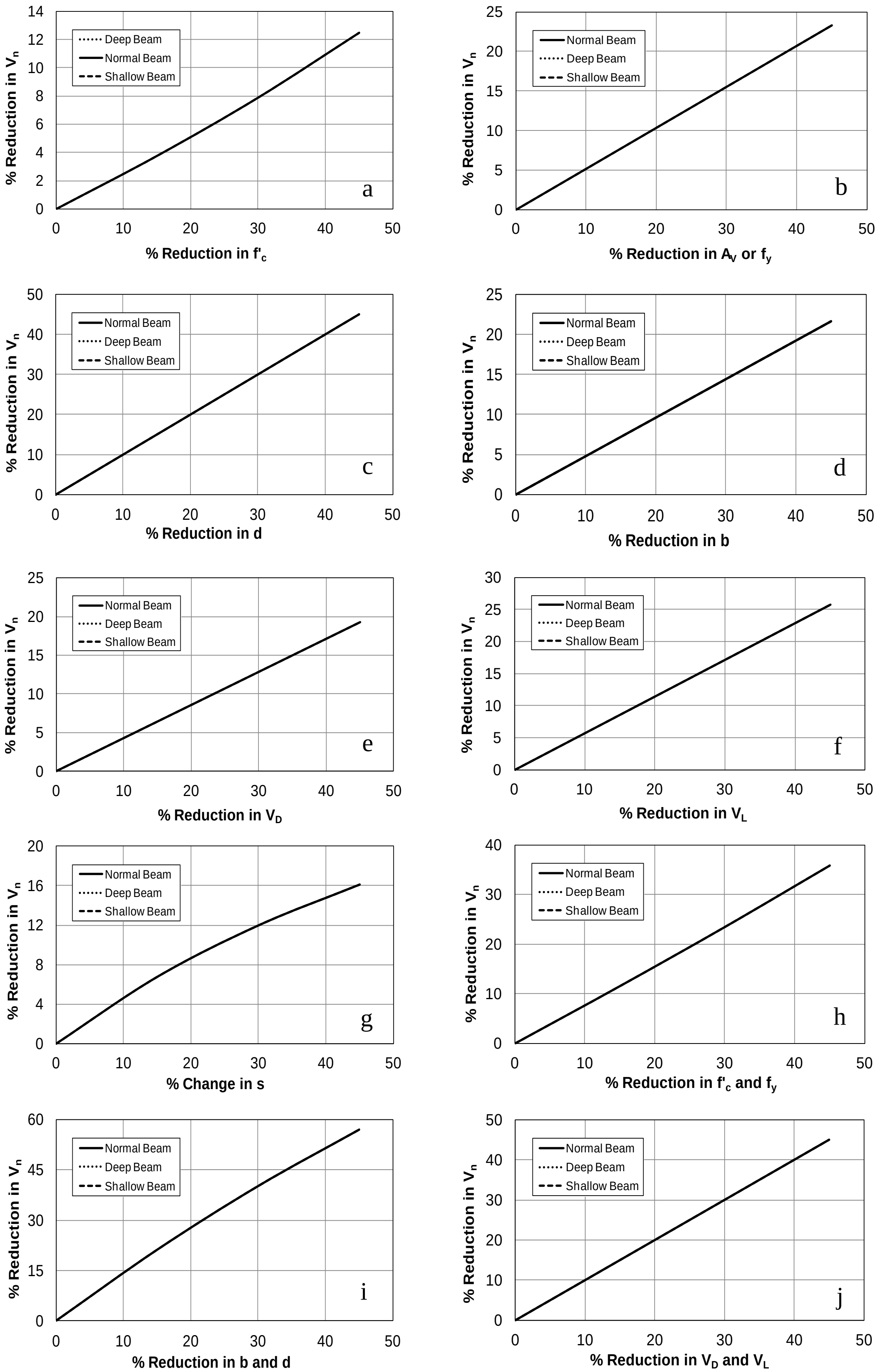

Figure 51: Effect of beam depth on the nominal shear capacity of beams

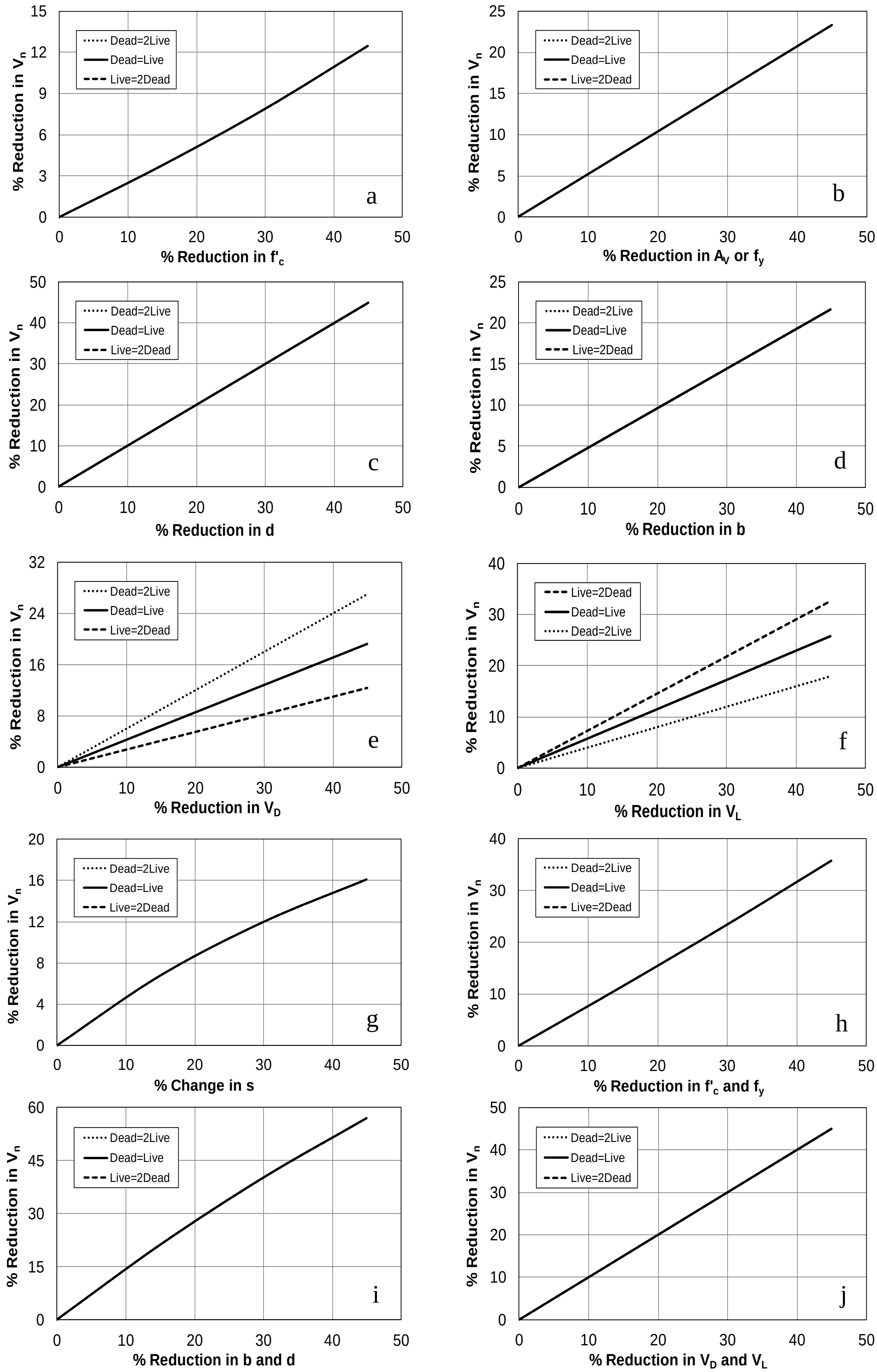

Figure 52: Effect of live-to-dead load fraction on the nominal shear capacity of beams

The sensitivity analysis has shown that the results for the reference cross section used in the previous section are not always valid for other cross sections with different geometry, materials and live-to-dead load ratio from the reference section.

Beams under shear designed with high concrete compressive strength are the most sensitive to changes in concrete compressive strength or beam width. Also, the sensitivity of beams under shear designed with low compressive strength is the most susceptible to the changes in the stirrups yield strength, area and spacing. Moreover, the considered cross sections have the same sensitivity to changes in effective depth of tension steel, dead shear and live shear.

Note that the above conclusion is also applicable for the beams that have similar properties, but with different stirrups spacing, low stirrups, or width. In this case, high concrete compressive strength (Figure 49) will correspond to large stirrups spacing (Figure 48), low stirrups (Figure 47), and large width (Figure 50), respectively.

When the depth of the beam was the controlling factor, as shown in Figure 51, the considered cross sections had similar sensitivity to changes in all design variables no matter what depth is considered during the design stage.

In Figure 52, the deterministically-based sensitivity analysis performed on cross sections with different loading scenarios has shown that sensitivity of nominal shear capacity was not affected by the reductions in design variables, except when changes happened in dead shear, live shear, or both.

5.2.3 Axially Loaded Columns

Similar to what has been carried out for beams under flexure and shear, many cross sections with different properties were evaluated in order to know if they are matching with the results found out for the reference cross section. These cross sections are as follows:

1. Three cross sections with the same design parameters, but with different reinforcement ratio (1%, 2.57% and 4%).

2. Three cross sections with the same design parameters, but with different compressive strength of concrete (21 MPa, 42 MPa and 84 MPa).
3. Three cross sections with the same design parameters, but with different yield strength of reinforcement (250 MPa, 420 MPa and 500 MPa).
4. Three cross sections with the same design parameters, but with different gross area (90000 mm^2, 250000 mm^2 and 360000 mm^2).
5. Three cross sections with the same design parameters but with different live-to-dead load fraction of the total load ($P_D=2P_L$, $P_D=P_L$, and $P_L=2P_D$).

The results of all sensitivity analyses are shown in Figures 53-57 and discussed next.

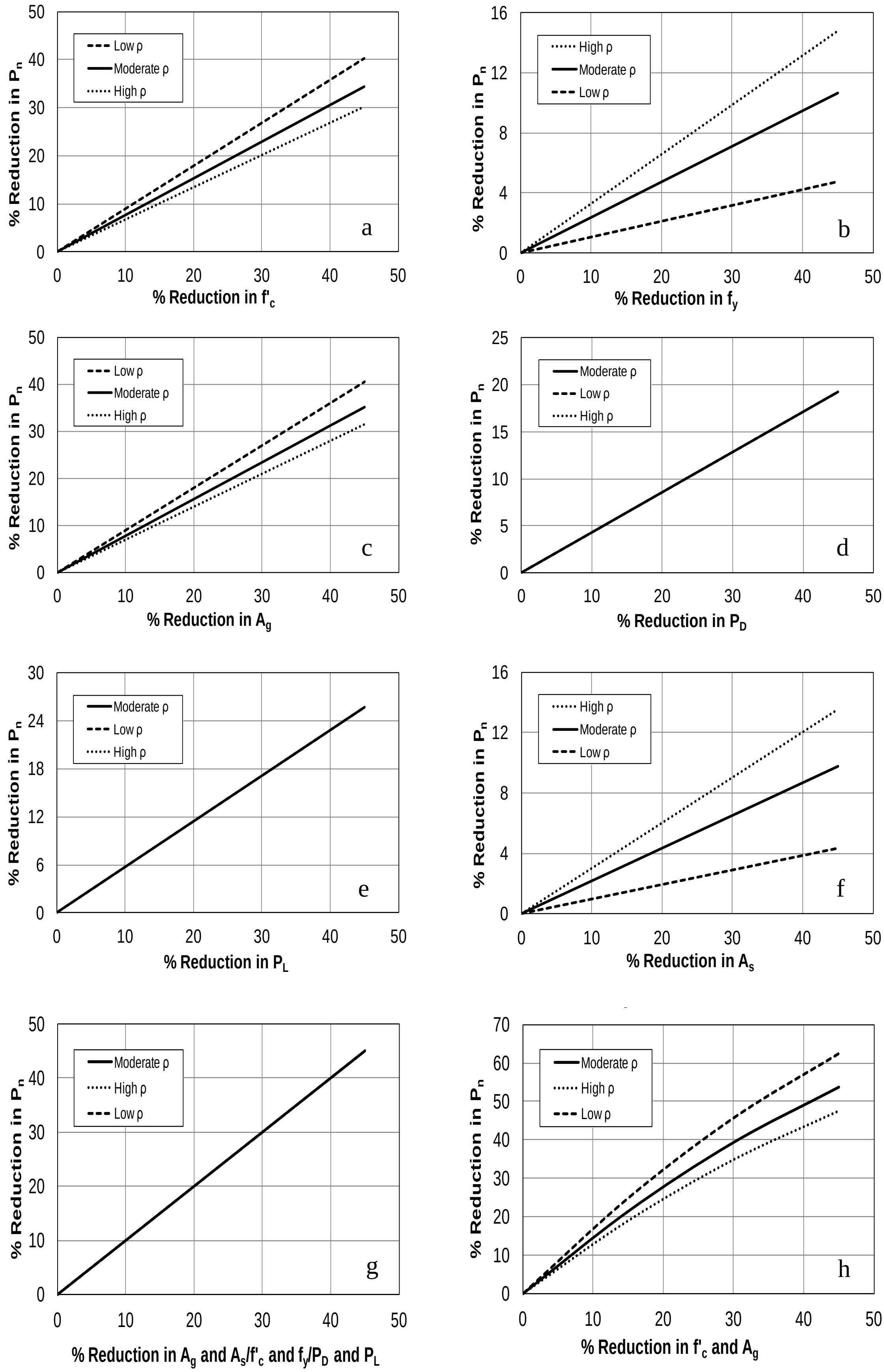

Figure 53: Effect of gross reinforcement ratio on the nominal axial capacity of columns

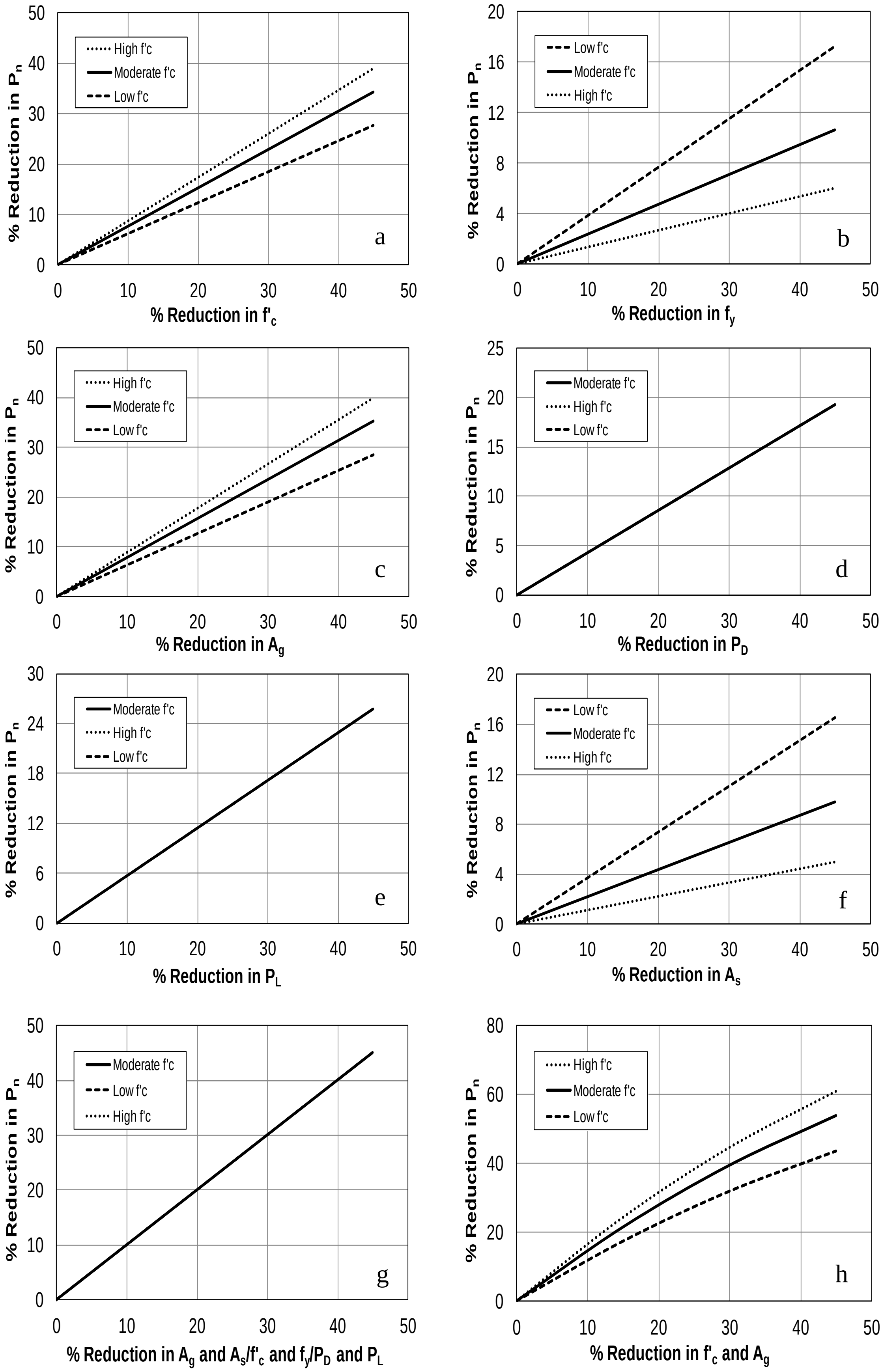

Figure 54: Effect of concrete compressive strength on the nominal axial capacity of columns

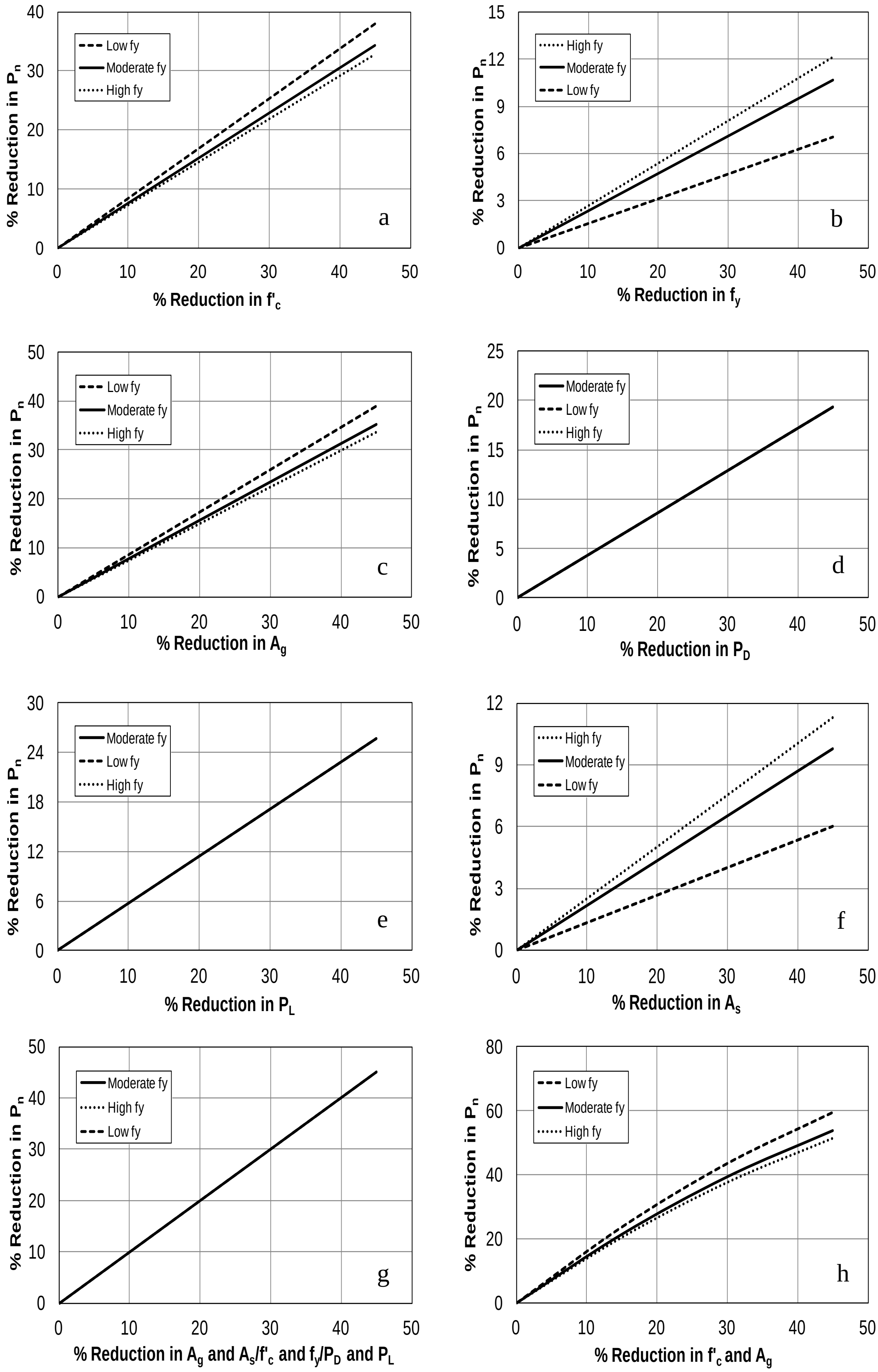

Figure 55: Effect of steel yield strength on the nominal axial capacity of columns

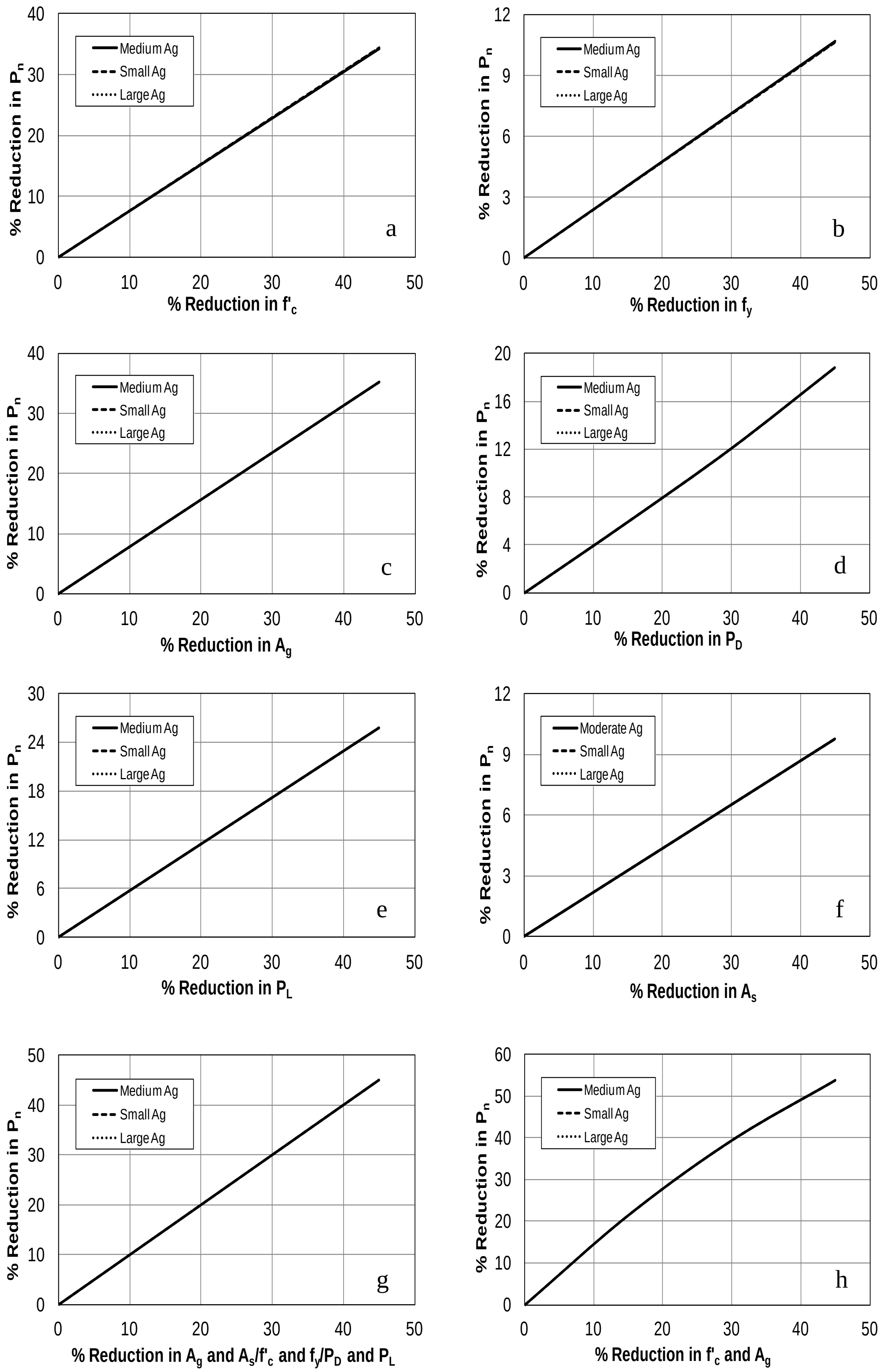

Figure 56: Effect of gross cross-sectional area on the nominal axial capacity of columns

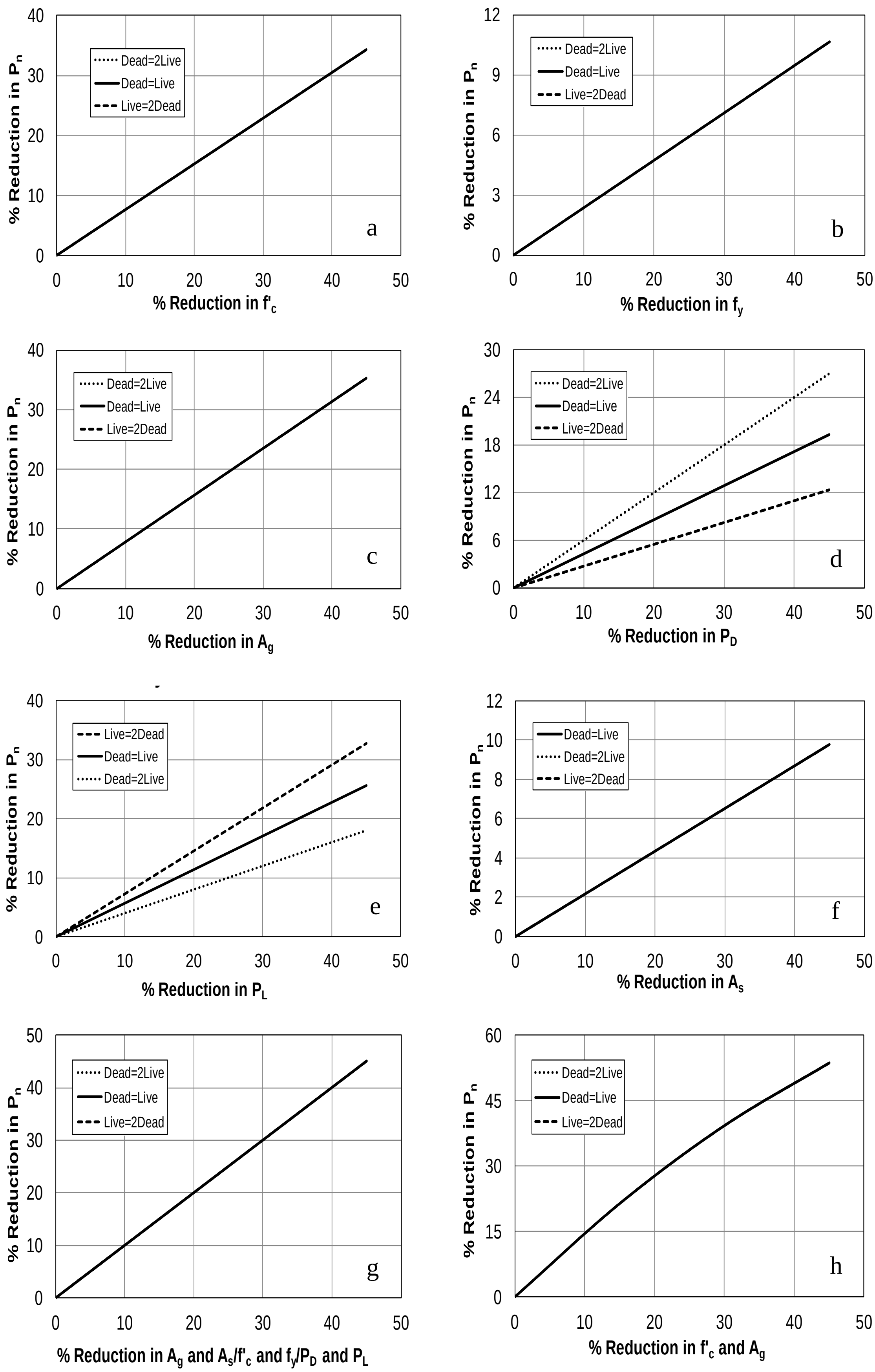

Figure 57: Effect of live-to-dead load fraction on the nominal axial capacity of columns

As shown in Figures 52-57, the deterministic sensitivity analysis for the considered cross sections indicates that the considered cross sections have different sensitivity behavior in relation to the design variables.

Axially loaded columns designed with low reinforcement ratio, high concrete compressive strength, or low reinforcement yield strength were the most sensitive to the errors committed in the compressive strength of concrete, or in the dimensions of the cross section. See Figures 53, 54, and 55.

On the other hand, axially loaded columns designed with high reinforcement ratio, low concrete compressive strength, or high reinforcement yield strength were the most sensitive to the errors committed in the reinforcement yield strength, or area of longitudinal steel. As for the remaining scenarios shown in Figures 53, 54, and 55, the reduction in these scenarios did not differ among the three cross sections.

Reduction in the nominal capacity of axially loaded columns with different gross cross sectional area was always the same, regardless of the design variable that is reduced, as shown in Figure 56.

Regarding the case where the loading is the only difference among the considered cross sections, presented in Figure 57, the sensitivity of the considered cross sections varied only in case the error was committed in dead axial load or in live axial load. In the first scenario, columns with dead load larger than live load were the most prone to changes in dead axial load, whereas the opposite was encountered in the second scenario.

CHAPTER 6

RELIABILITY-BASED SENSITIVITY ANALYSIS FOR STRUCTURAL MEMBERS

6.1 Introduction

As explained in the previous chapter, sensitivity analysis is used to identify the most sensitive parameters contributing to the strength or reliability of structural elements. Deterministic approach was investigated earlier in the Chapter 5. In this chapter, reliability-based approach will be examined in detail for the same cross-sections and limit states addressed in Chapter 5. Similar to the deterministically-based method, the reliability-based sensitive analysis methods is also used to investigate the most critical parameters affecting the structural safety for flexure, shear, and axial compression limit states.

The Rackwitz-Fiessler method [41] is used in the nondeterministic analysis to compute the change in reliability index due to a change in the design variable. As discussed in Chapter 2, this method is often used when the random variables in a given safety margin are neither normal nor lognormal. The method is based upon approximating the actual distribution by a normal function at the point of maximum probability on the failure surface (referred to the design point). In the reliability analysis, the statistics for the load and resistance variables for reinforced concrete building components are taken from the available literature [41] and are shown below in Table 4. These variables are assumed uncorrelated (or independent) in the analysis.

Table 5: Statistics of building load and resistance random variables [41]

Load or Resistance Variable	Bias	Coefficient of Variation	Distribution
Dead Load	1.05	0.1	Normal
Arbitrary point-in-time live load	0.24	0.65	Gamma
Maximum 50-year live load	1.00	0.18	Extreme Type-I
Flexural capacity of RC beam	1.19	0.089	Lognormal
Shear capacity of RC beam	1.23	0.109	Lognormal
Axial Compressive capacity of tied RC column	1.26	0.107	Lognormal

The magnitude of a target reliability index for a structural member in a given design code is affected by the philosophy of the code, considered limit state, relative costs of safety measures and consequences of failure. In order to judge whether the value of a reliability index of a deficient structure is satisfactory or not, we consider the Annex of the “ISO 2394:1998, General principles on reliability for structures” document [42], which includes guidance on the target reliability index for different groups of relative costs of safety measures and different categories of consequences of failure. For example, the target reliability index is 3.8 for a structure that requires moderate safety measures and has great consequences of failure.

Table 6: suggested target reliability index values [42]

Relative costs of safety measure	Consequences of failure			
	Small	Some	Moderate	Great
High	0	1.5	2.3	3.1
Moderate	1.3	2.3	3.1	3.8
Low	2.3	3.1	3.8	4.3

6.1.1 Flexure in Beams

The first step in calculating the reliability index for a member based on a given design code is to calculate the design moment capacity as specified by the code. Therefore, Equations 5.1 and 5.2 are used again:

$$\emptyset M_n = \emptyset A_s f_y (d - 0.5a) \geq 1.2M_D + 1.6M_L$$

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

In addition, the reference beam used in Chapter 5 was also used in the reliability approach. That reference beam had the following properties:

- 400 mm by 800 mm cross section.
- 50 mm clear concrete cover on stirrups.
- Compressive strength of concrete is 42 MPa.
- Yield strength of reinforcement is 420 MPa.
- Longitudinal tension steel is 4No. 32 bars (i.e. A_s = 3217 mm^2).
- No. 12 closed stirrups at 200 mm spacing.

Using a computer program that is based on Rackwitz-Fiessler method and developed by Tabsh (unpublished), along with the statistical data shown in Table 4, the reliability index of the beam for the flexural limit state is equal to 3.33. This reliability index represents the reliability index assuming that the beam has been designed or constructed without any human error being committed.

If the area of steel was reduced by 15% due to negligence, the reliability index of this beam will be reduced to 2.61, which is equivalent to a reduction of 21.6% in the reliability index of this beam.

Repeating the same calculations for a 30% and 45% reduction in the area of longitudinal steel reinforcement, the reductions in the reliability index would be 50.1% and 95.9%, respectively. These results are illustrated in Figure 58.

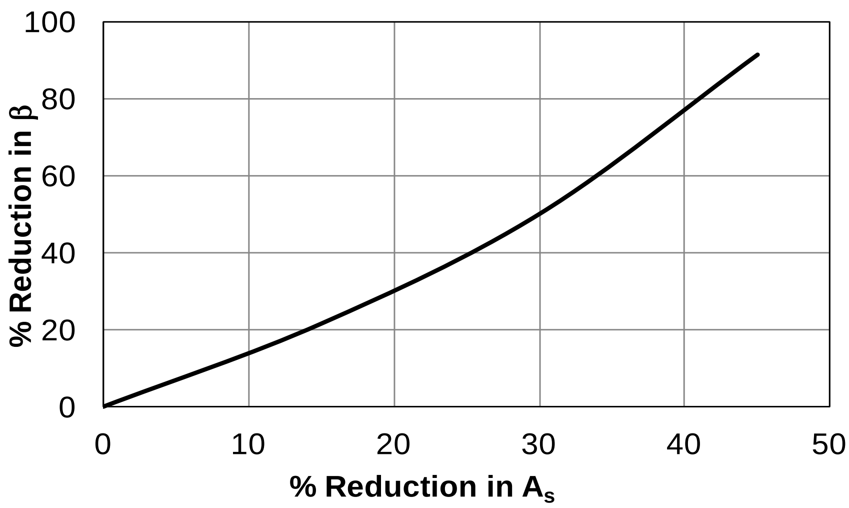

Figure 58: Effect of reduction in area of steel on reliability index in flexure

Figure 59 represents the reliability results when the same procedure above is followed with the remaining design parameters such as the effective depth of tension steel, width of the beam, compressive strength of concrete, yield strength of reinforcement, dead load moment and live load moment.

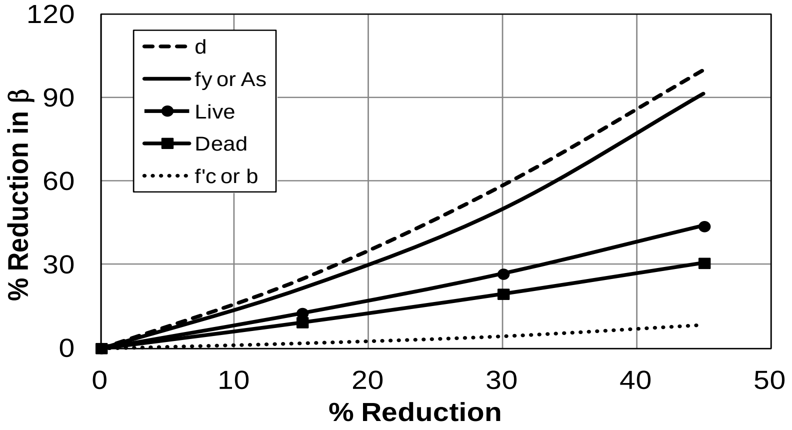

Figure 59: Effect of variations in design variables on reliability index in flexure

The trend of sensitivity shown in Figure 59 is similar to that in Figure 38 in Chapter 5, but with different reductions in the considered functions. The reliability analysis, however, provides more information in relation to structural safety and probability of failure. Beams under flexure are more sensitive to the reduction in the effective depth of tension steel reinforcement, while they are less sensitive to the reduction in concrete compressive strength or beam width.

In summary, Figure 59 shows that reductions in the depth of tension steel, yield strength of reinforcement, or the area of longitudinal steel due to human errors have high impact on the reliability index of beams under flexure. One the contrary, reductions in the compressive strength of concrete or the width of the beam have minor impact on the reliability of beams under flexure. Miscalculations that result in reductions in the dead load moment or the live load moment, during the design stage, have moderate impact on the reliability index.

6.1.2 Shear in Beams

In order to develop reliability-based sensitivity functions for shear, the same reference beam considered earlier for flexure was used again:

- 400 mm by 800 mm cross section.
- 50mm clear concrete cover on stirrups.
- Compressive strength of concrete is 42 MPa.
- Yield strength of transverse reinforcement is 420 MPa.
- Longitudinal tension steel is 4 No. 32 bars (i.e. A_s = 3217 mm^2).
- No. 12 closed stirrups at 200 mm spacing.

Then, nominal shear capacity is calculated from:

$$\emptyset V_n = \emptyset(0.17\sqrt{f'_c} b_w d + A_v f_y d/s) \geq 1.2V_D + 1.6V_L$$

The reliability index of this beam for the shear limit state is equal to 4.14. When conducting reliability-based sensitivity analysis approaches for all the design parameters contributing to the shear capacity of this cross section, the results are as shown in Figure 60.

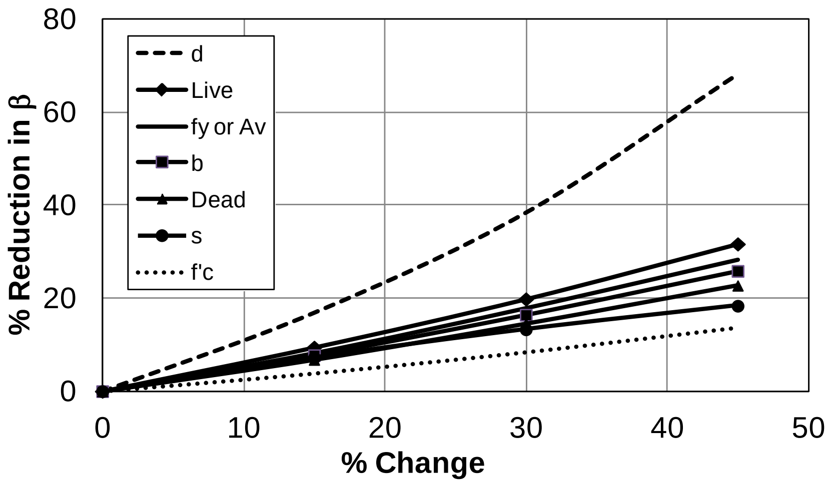

Figure 60: Effect of variations in design variables on reliability index in shear

The reliability-based sensitivity analysis performed on beams under shear has led to the same findings of the deterministically-based sensitivity analysis highlighted in Figure 39 but with different effect. More specifically, reduction in the depth of tension steel due to human errors has high impact on the reliability index and nominal shear capacity of beams. Changes in live load shear, dead load shear, concrete compressive strength, stirrup spacing, area of stirrups, and the yield strength of stirrups have moderate impact on the reliability index and the shear capacity of beams. The reliability analysis, however, quantifies the structural safety and gives more insight into the risk of collapse.

6.1.3 Axially Loaded Columns

The design axial capacity of a tied column is calculated using the equation:

$$\emptyset P_n = \emptyset 0.8\{0.85f'_c(A_g - A_s) + A_s f_y\} \geq 1.2P_D + 1.6P_L$$

To examine the effect of human errors committed during design and construction stages on the reliability index of axially loaded columns, the same reference cross section used in Chapter 5 was utilized again:

- 500 mm by 500 mm square cross section
- Compressive strength of concrete is 42 MPa
- Yield strength of longitudinal reinforcement is 420 MPa
- Longitudinal reinforcement consists of 8No. 32 bars ($\rho = 2.57\%$)

Conducting the reliability analysis on all design parameters contributing to the column strength gave the results shown in Figure 61.

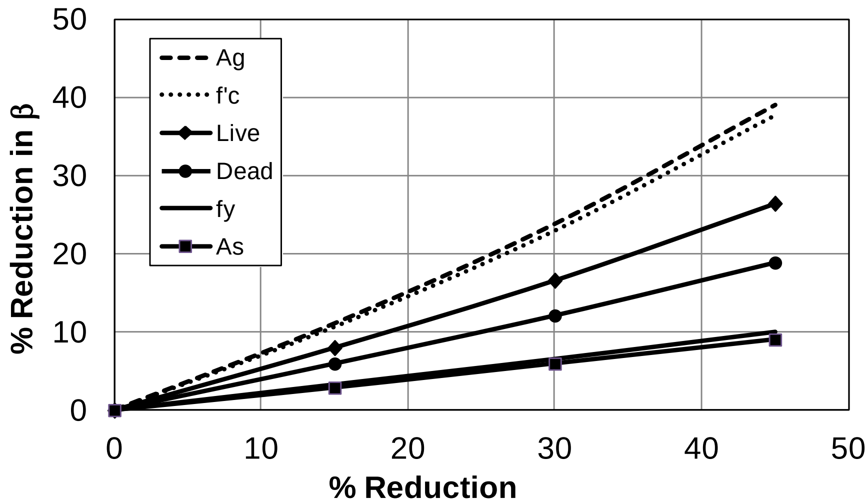

Figure 61: Effect of variations in design variables on reliability index in compression

Reliability-based sensitivity analysis results for the axially loaded columns, as summarized in the Figure 61, have pointed out that the axial capacity is more sensitive to the reduction in the column's gross cross-sectional area and the compressive strength of concrete. The capacity of axially loaded columns, on the other hand, is less sensitive to the reduction in the area and yield strength of the longitudinal steel reinforcement.

6.1.4 Comparisons between Members in Flexure, Shear, or Compression

Comparison among beams under flexure, beams under shear, and columns subjected to axial compression with respect to the reduction in the reliability index due to human errors in the relevant design variables is made in this section. The results are summarized in Figure 62. The objective of this comparison is to show how one design variable might impact various limit states by different degrees.

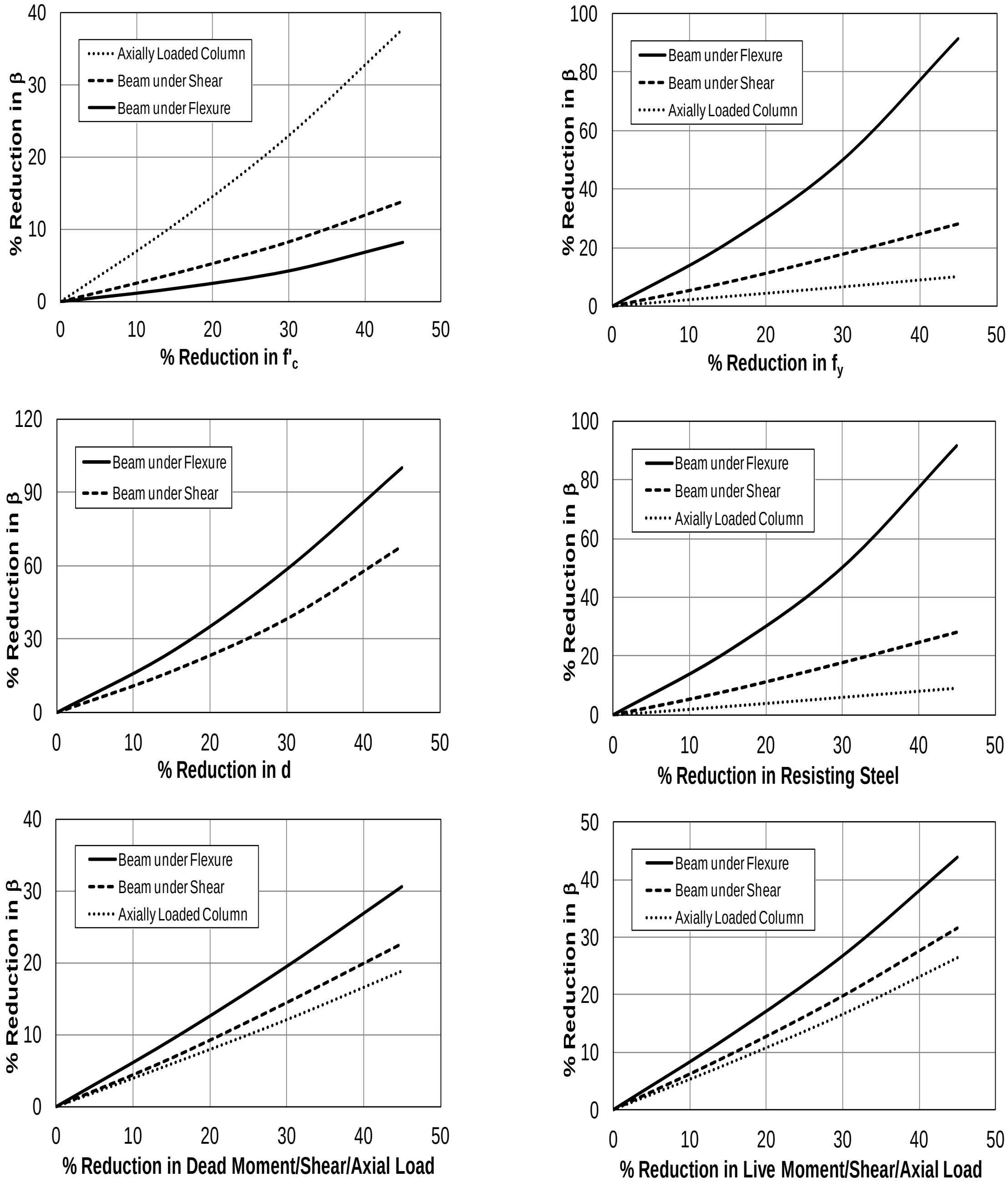

Figure 62: Comparison among reliability results for flexure, shear, and axial compression

As illustrated in Figure 62, beams under flexure are the most sensitive to reductions in the considered design variables, except for changes in concrete compressive strength, where axially loaded columns are the most sensitive to such variations. When compared with flexure and axial compression, the effect of change in design variables on shear strength is found to fall between the two other limit states.

The previous analysis only considers one standard cross-section with given material properties, section dimensions and reinforcement. Again, for reinforced concrete members having different characteristics than the considered (such as 21 MPa concrete compressive strength instead of 42 MPa, 250 MPa reinforcement yield strength instead of 420 MPa, 400 mm beam thickness instead of 800 mm, or live load being twice dead load instead of equal to it), one needs to determine whether the previous findings are applicable or not.

To investigate this issue, sensitivity analysis was performed on different cross sections with various properties for beams under flexure, beams under shear, and columns under axial compression. This issue is discussed in details in the following sections.

6.2 Reliability Based Sensitivity Analysis on Different Cross Sections

6.2.1 Beams under Flexure

In this section, reliability-based sensitivity analysis was carried out on different cross sections with different material properties and steel reinforcement in order to determine whether the reduction in the reliability index differ from one cross section to another. For beams under flexure, the sensitivity analysis has been investigated for the following cases, which cover a wide practical spectrum:

1. Three cross sections with the same design parameters, but with different area of steel (1608.5 mm^2, 3217 mm^2 and 6434 mm^2).
2. Three cross sections with the same design parameters, but with different compressive strength of concrete (21 MPa, 42 MPa and 84 MPa).
3. Three cross sections with the same design parameters, but with different yield strength of steel reinforcement (250 MPa, 420 MPa and 500 MPa).
4. Three cross sections with the same design parameters, but with different thickness (500 mm, 800 mm and 1100 mm).
5. Three cross sections with the same design parameters, but with different dead-to-live load moment ratio (M_D=2M_L, M_D=M_L, and M_L=2M_D).

The results of the reliability-based sensitivity analysis for the considered cross-sections under flexure are shown in Figures 63-67.

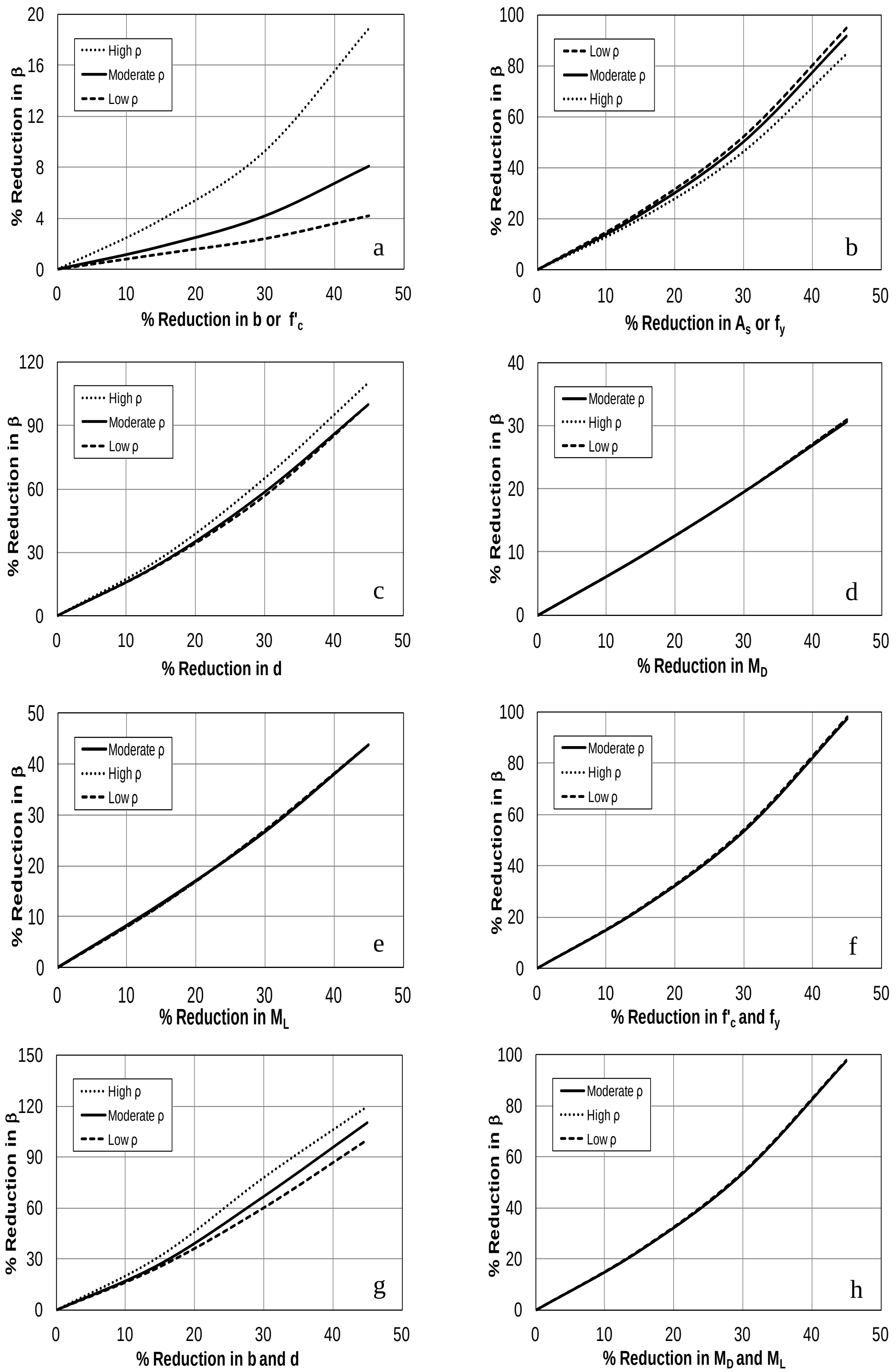

Figure 63: Effect of steel reinforcement ratio on the reliability of a beam under flexure

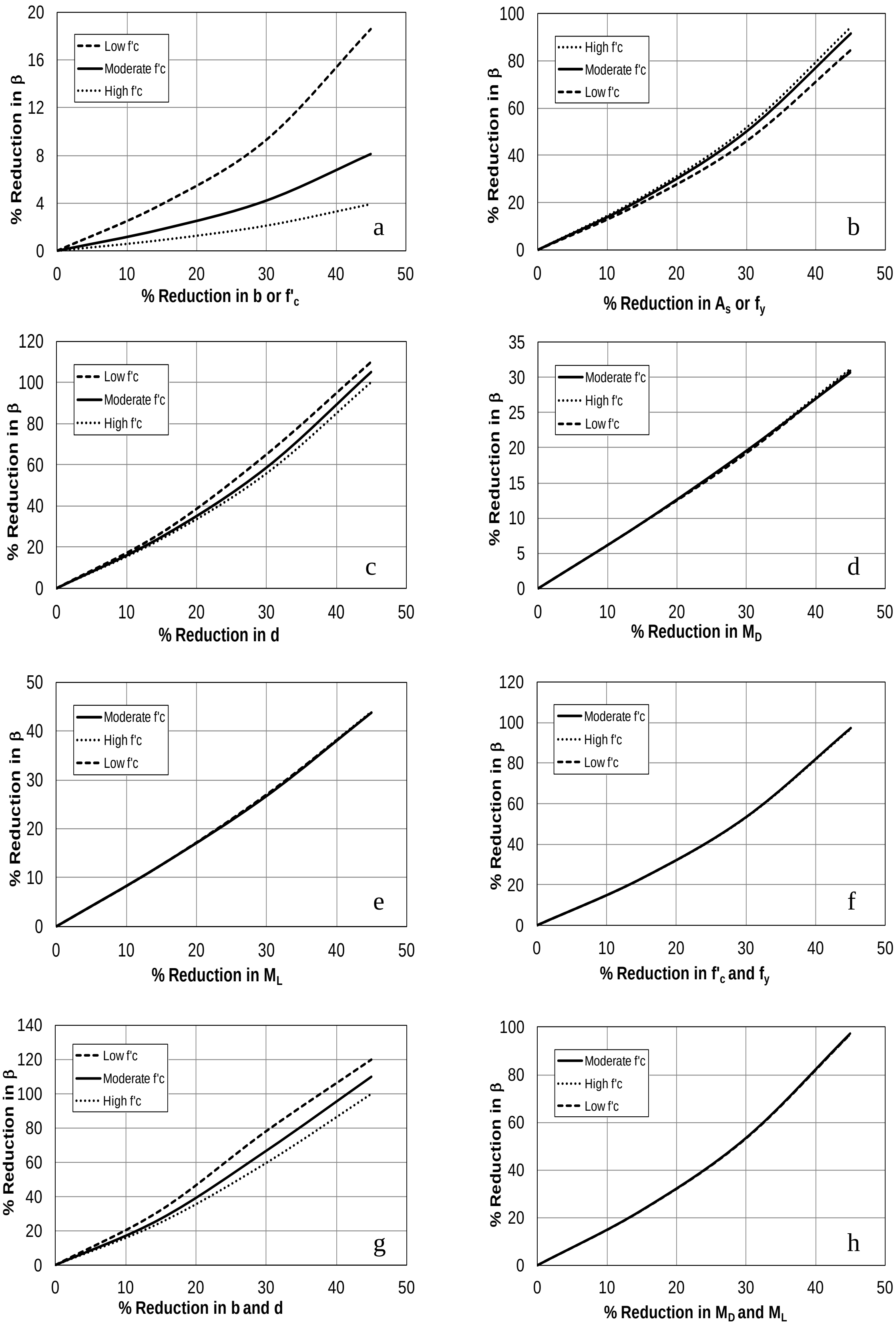

Figure 64: Effect of concrete compressive strength on the reliability of a beam under flexure

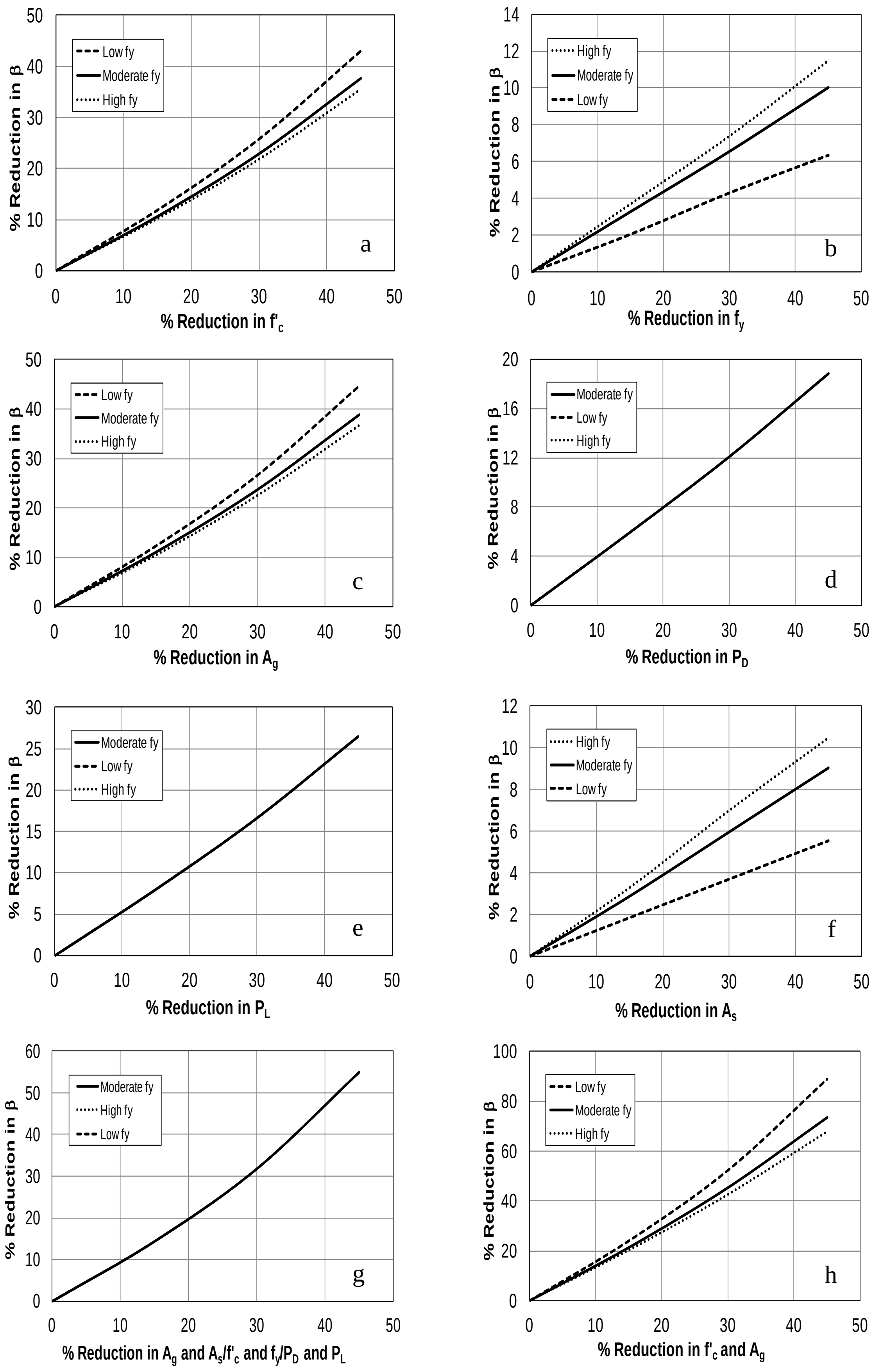

Figure 65: Effect of steel yield strength on the reliability of a beam under flexure

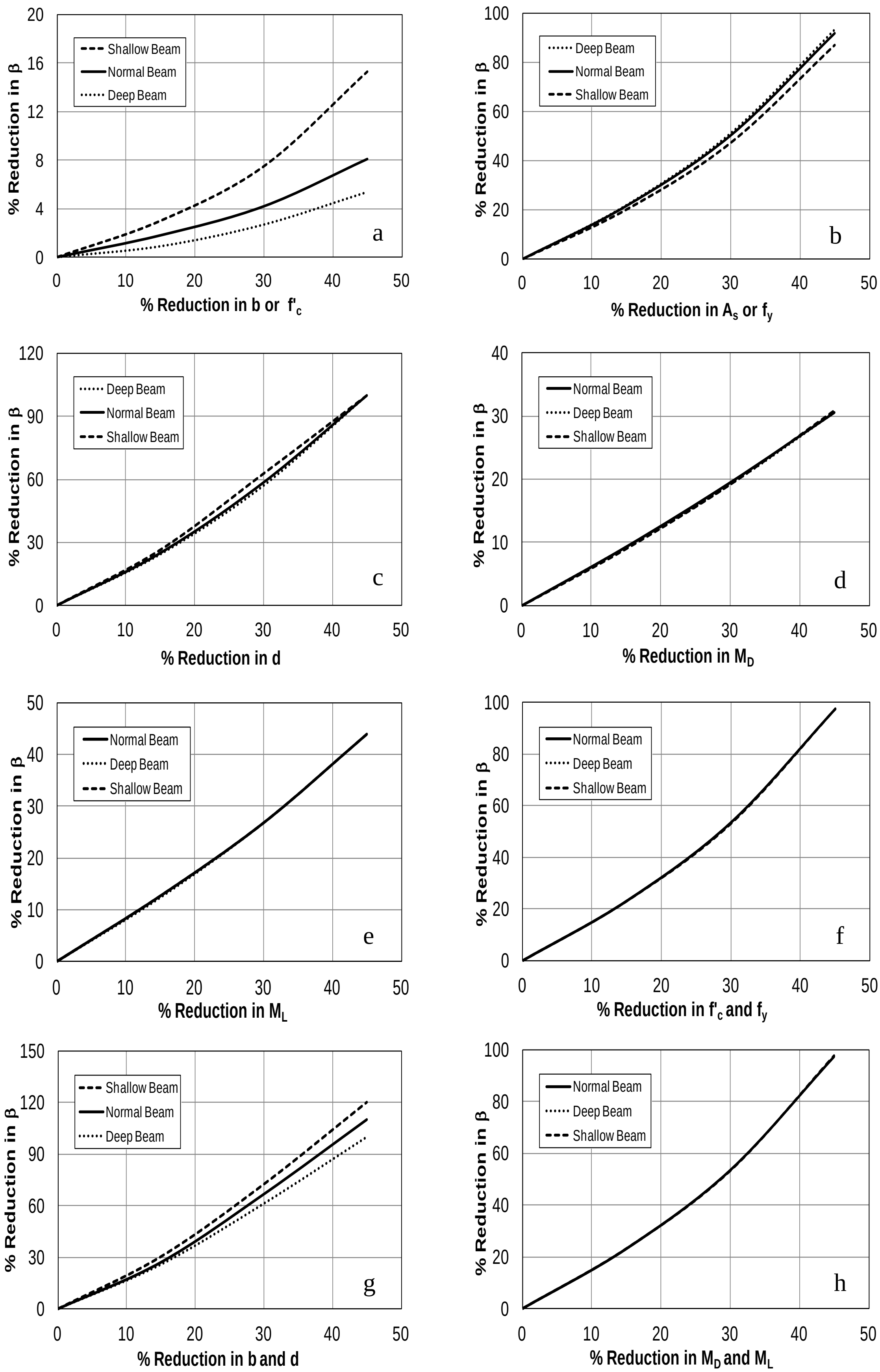

Figure 66: Effect of member thickness on the reliability of a beam under flexure

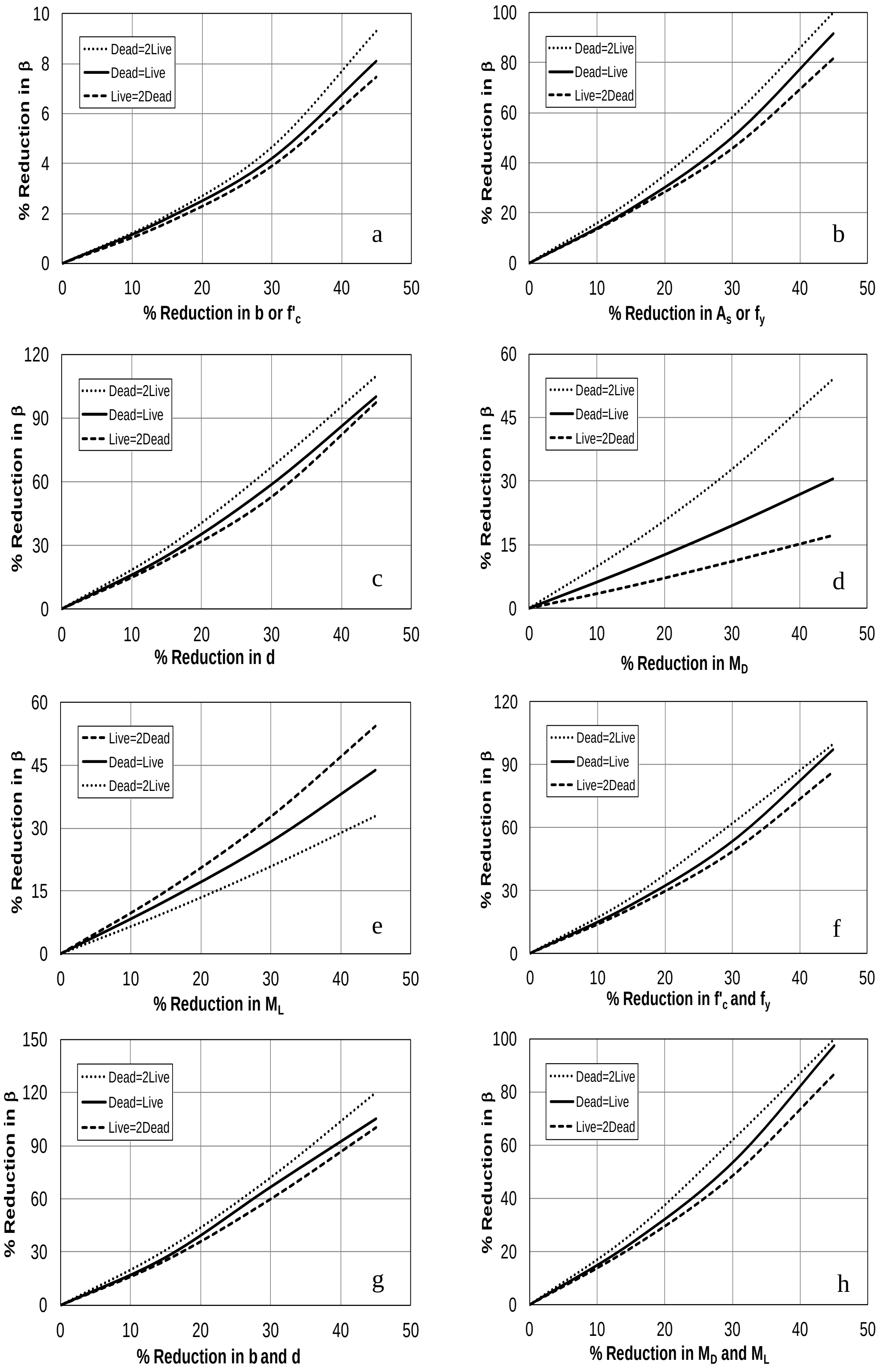

Figure 67: Effect of dead-to-live load ratio on the reliability of a beam under flexure

The reliability-based sensitivity analysis for beams under flexure, with consideration of the five different cases of parameters (ρ, f'_c, f_y, d, and M_D/M_L) leads to the following observations:

- The reliability index of beams with high area of longitudinal steel, low concrete compressive strength, high yield strength of reinforcement, or shallow depth was more sensitive to changes in concrete compressive strength or beam width.
- The effect of reduction in the area of longitudinal steel or yield strength of reinforcement on the reliability index of beams under flexure did not vary with changes in area of longitudinal steel, compressive strength of concrete, or yield strength of reinforcement.
- The reliability index of beams with high longitudinal steel area, lower concrete compressive strength, or higher yield strength was more sensitive to changes in effective depth of tension steel, especially when the change in the variable was 20% or more.
- The reduction in the reliability index was constant due to changes in dead load moment, live load moment, combination of live and dead load moments, and combination of concrete compressive strength and yield strength of reinforcement, regardless of the reinforcement ratio, concrete compressive strength, and reinforcement yield strength the beam is designed with.
- When changes happened simultaneously in the width and the effective depth of tension steel, beams designed with higher area of steel, lower concrete compressive strength, or higher reinforcement yield strength was ~~a~~ little more sensitive to such changes.
- Where the load components are the only difference among the three considered cross sections, it was observed that cross-sections subjected to larger dead load than live load witnessed the most reduction in reliability due to changes in all design variables, except when changes happened in live load, and live load was larger than dead load.

In Figure 68, the effect of reducing the area of longitudinal steel or the yield strength of reinforcement for the considered five cases is summarized. Note that the first 4 figures, denoted by a, b, c and d, have the same pattern, as opposed to graph e. Hence, it can be concluded that for beams under flexure, the reduction in the longitudinal area of steel or in the yield strength of reinforcement will have the same effect on the reliability index, regardless of the value of the steel reinforcement ratio, yield strength of steel, effective depth of reinforcement, and compressive strength of concrete. However, when it comes to the externally applied moment on the cross-section, beams subjected to larger dead load than live load are more sensitive to the reduction in the area of longitudinal reinforcement or the yield strength of steel, although the sensitivity is more predominant at large reductions in the two design variables.

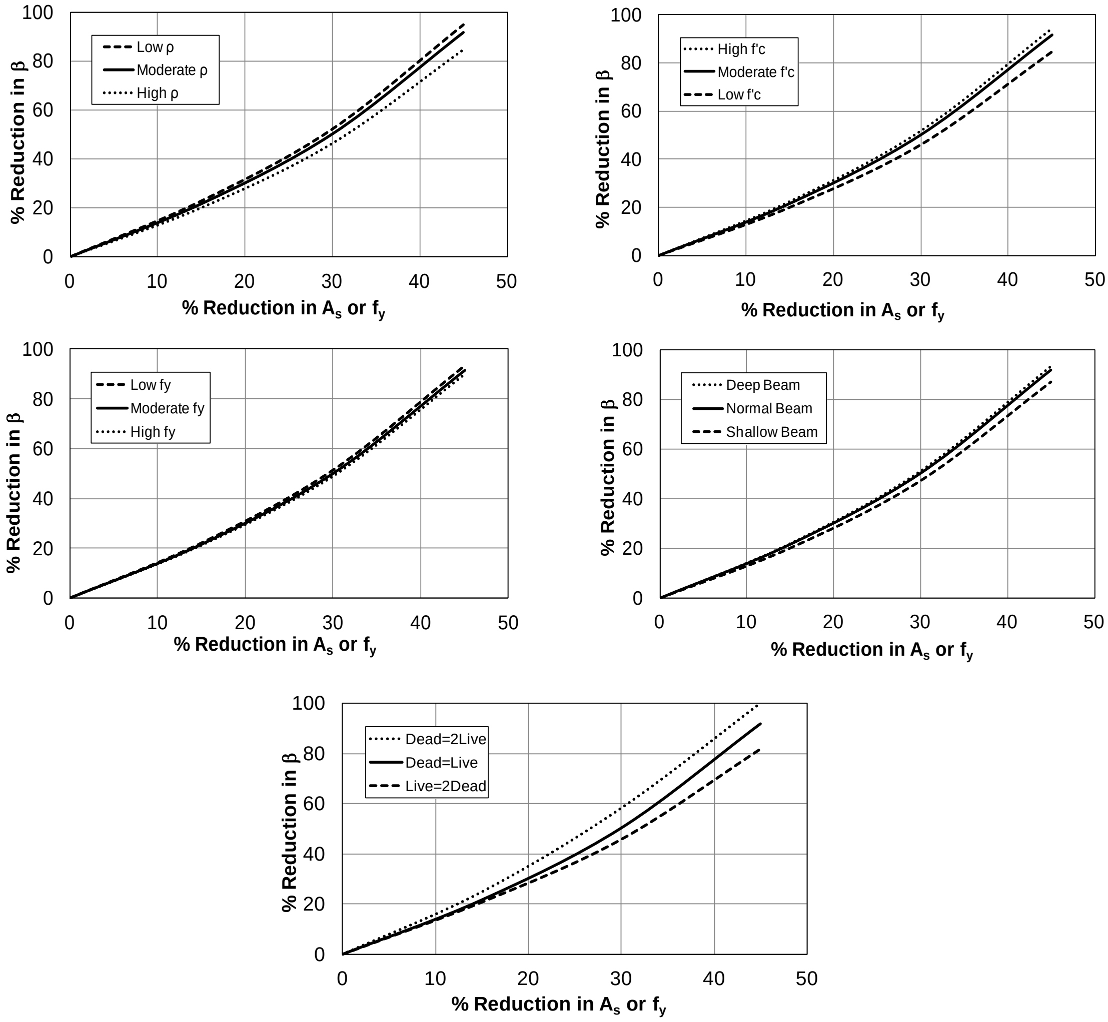

Figure 68: Effect of area of reinforcement on the reliability index of beams under flexure

The same trend is observed for the case of under-estimating the dead load effect on the structure, which is illustrated in Figure 69 below, redrafted from Figures 63.d, 64.d, 65.d, 66.d and 67.d. Figure 69 shows that the reduction in the reliability index due to changes in dead moment has the same effect on the cross-section for all cases, except when the live load fraction of the total load is the only difference among the three considered cross sections.

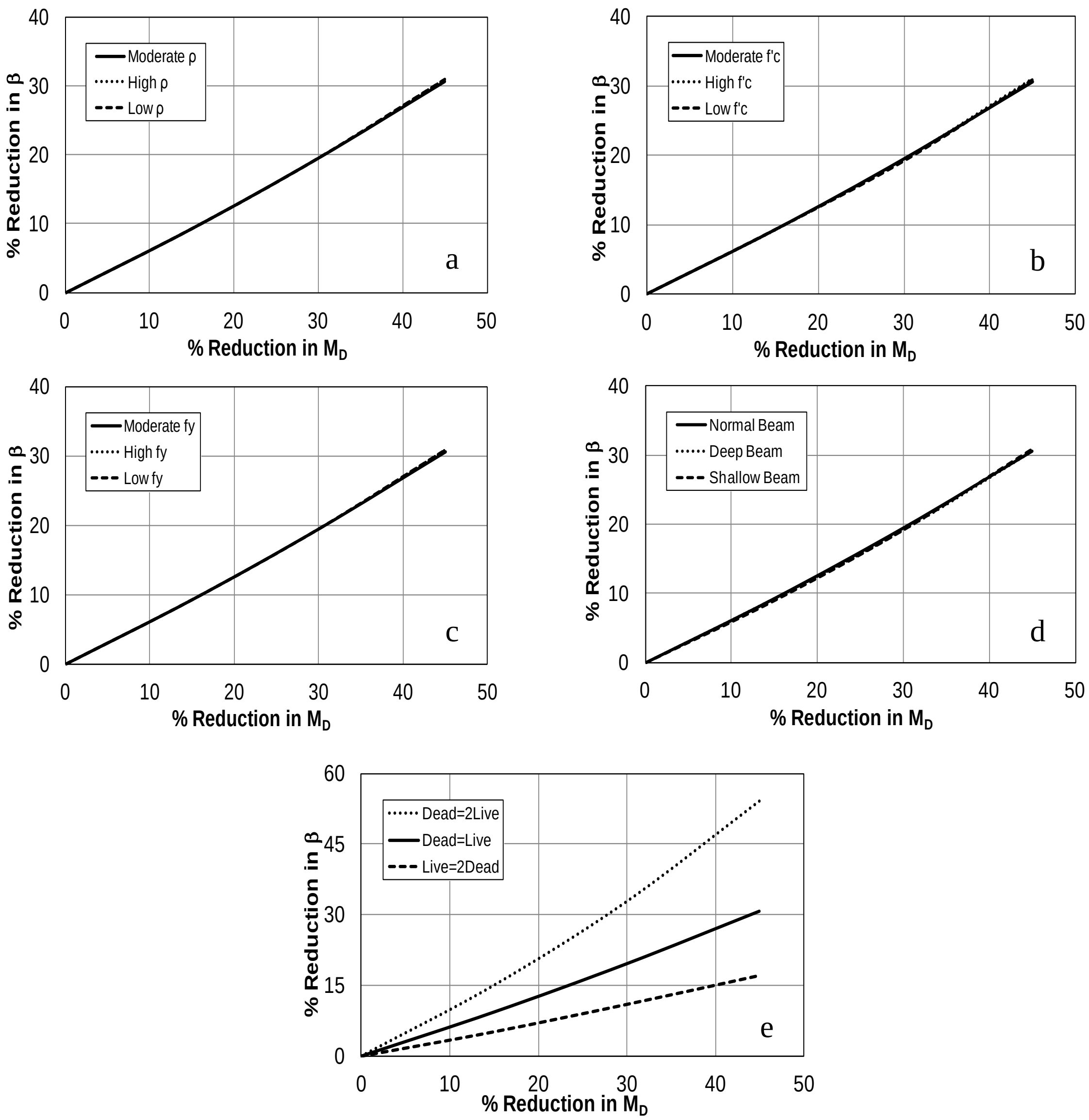

Figure 69: Effect of reduction in dead load on the reliability index of beams under flexure

If the same comparison is made for the remaining graphs with the same alphabetical letter (for example, 63.c, 64.c, 65.c, 66.c, and 67.c), the same outcome is obtained.

6.2.2 Beams under Shear

In order to check if the results obtained for the reference cross sections are valid for beams under shear with different cross sections dimensions, material properties, and steel reinforcement, reliability analysis approach was applied to cross-sections different from the considered reference:

1. Three cross sections with the same design parameters, but with different area of stirrups (113 mm^2, 226 mm^2 and 452 mm^2).
2. Three cross sections with the same design parameters, but with different compressive strength of concrete (21 MPa, 42 MPa and 84 MPa).
3. Three cross sections with the same design parameters, but with different stirrup spacing (100 mm, 200 mm and 350 mm).
4. Three cross sections with the same design parameters, but with different cross-section thickness (500 mm, 800 mm and 1100 mm).
5. Three cross sections with the same design parameters, but with different width of member (200 mm, 400 mm and 800 mm).
6. Three cross sections with the same design parameters, but with applied shear due to different dead-to-live load ratio (V_D=2V_L, V_D=V_L, and V_L=2V_D).

The results of the reliability-based sensitivity analysis are presented in Figures 70-75 and discussed next.

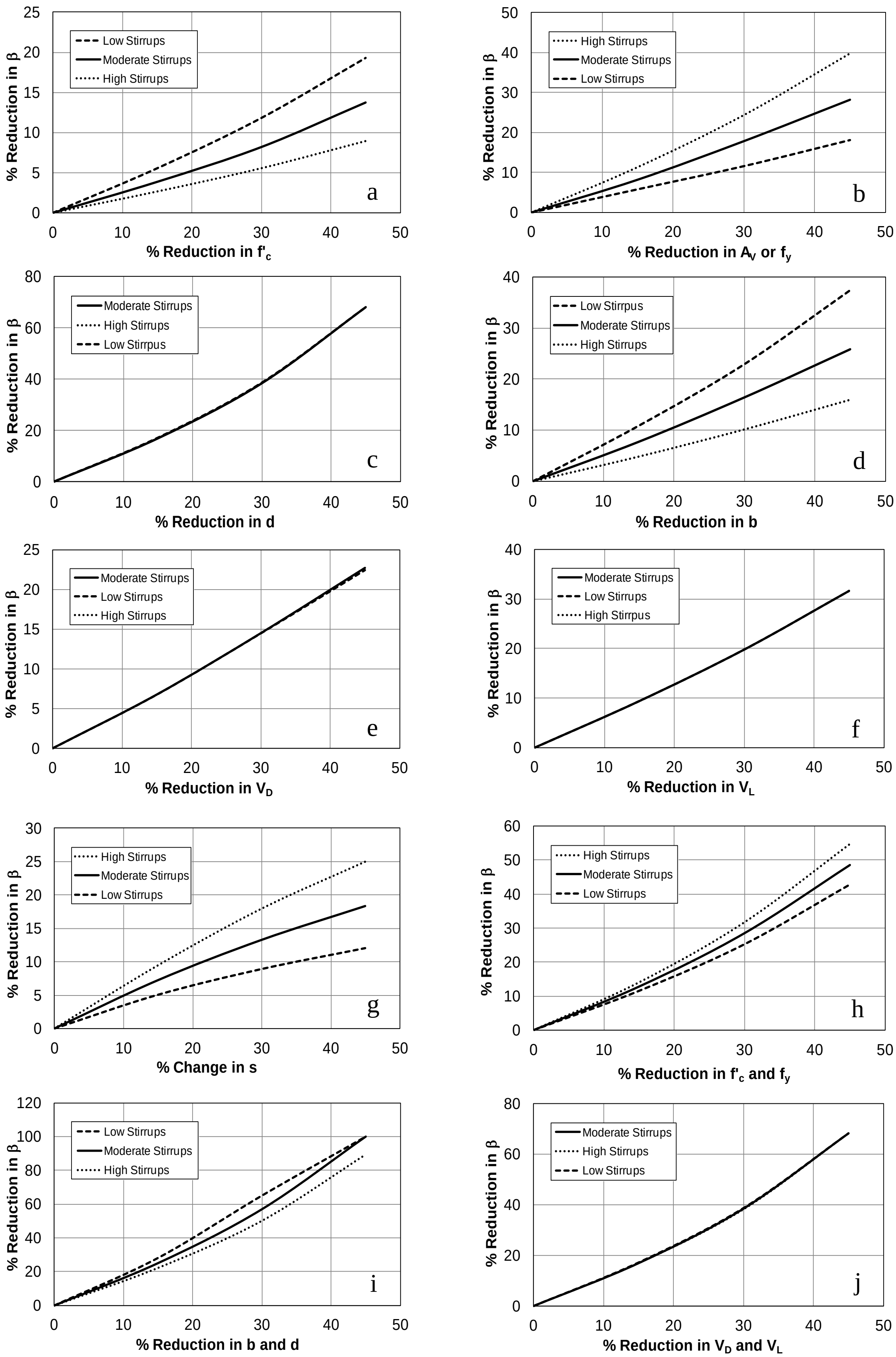

Figure 70: Effect of area of stirrups on the reliability of a beam under shear

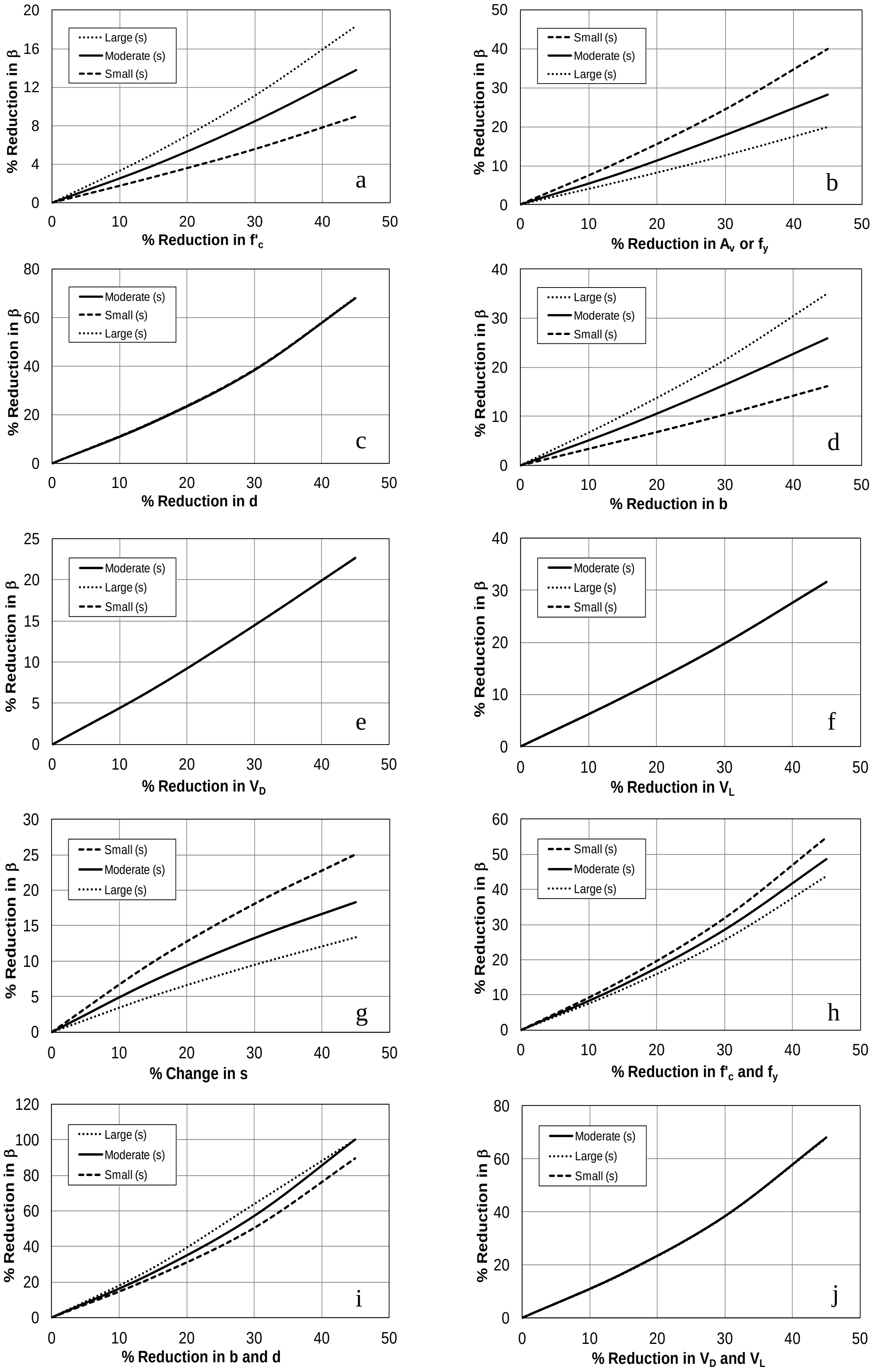

Figure 71: Effect of spacing of stirrups on the reliability of a beam under shear

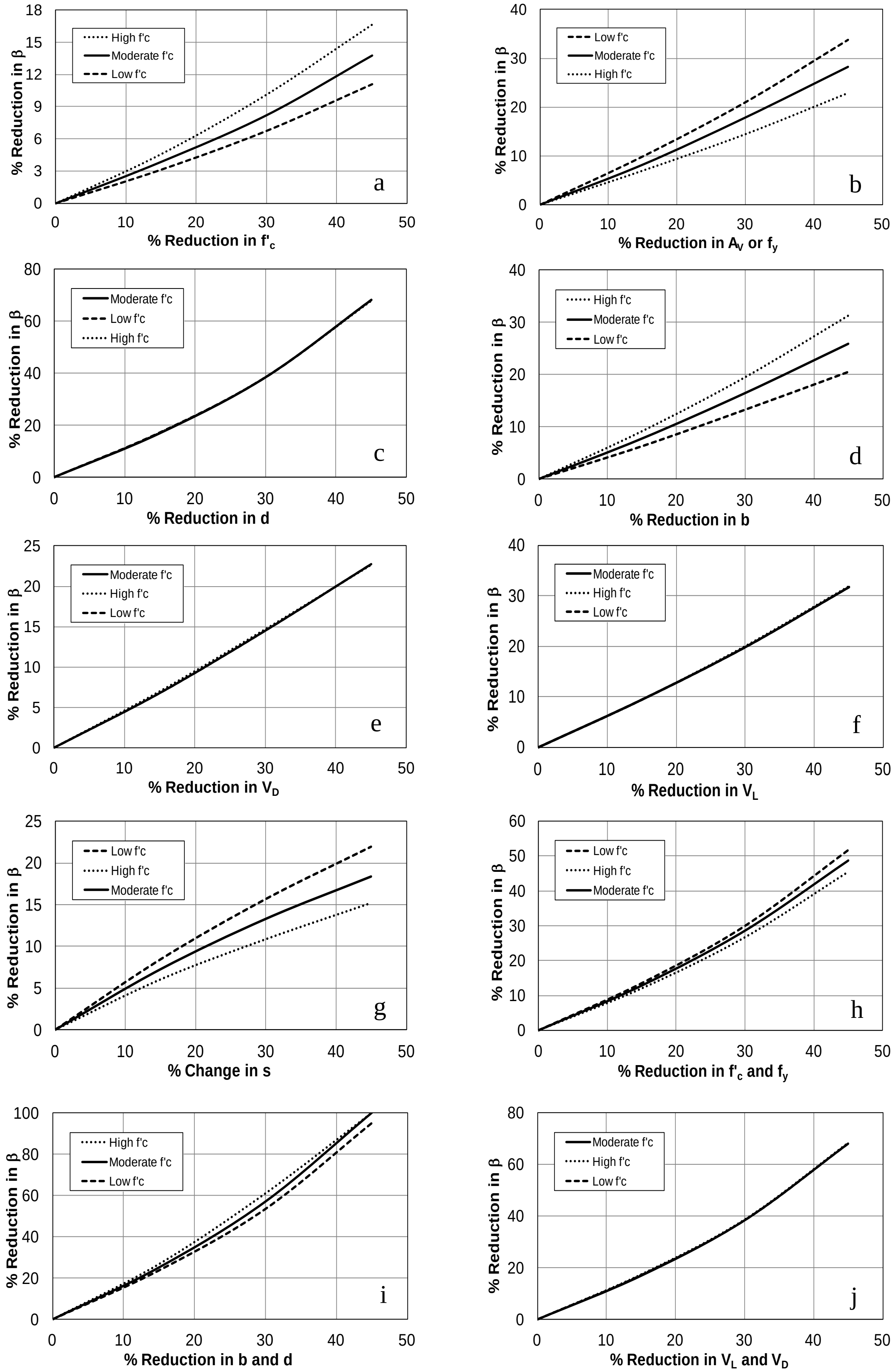

Figure 72: Effect of concrete strength on the reliability of a beam under shear

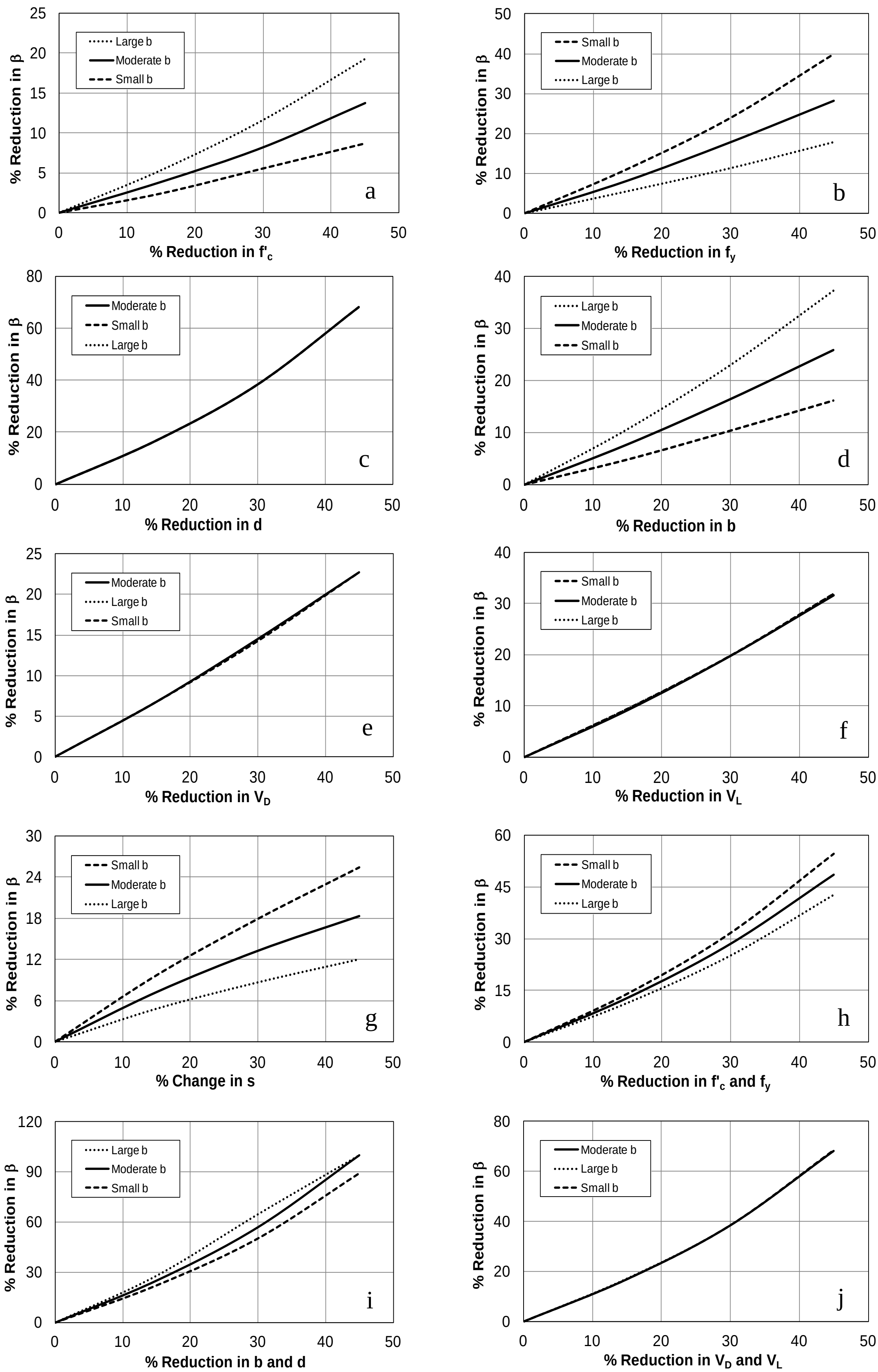

Figure 73: Effect of cross-section width on the reliability of a beam under shear

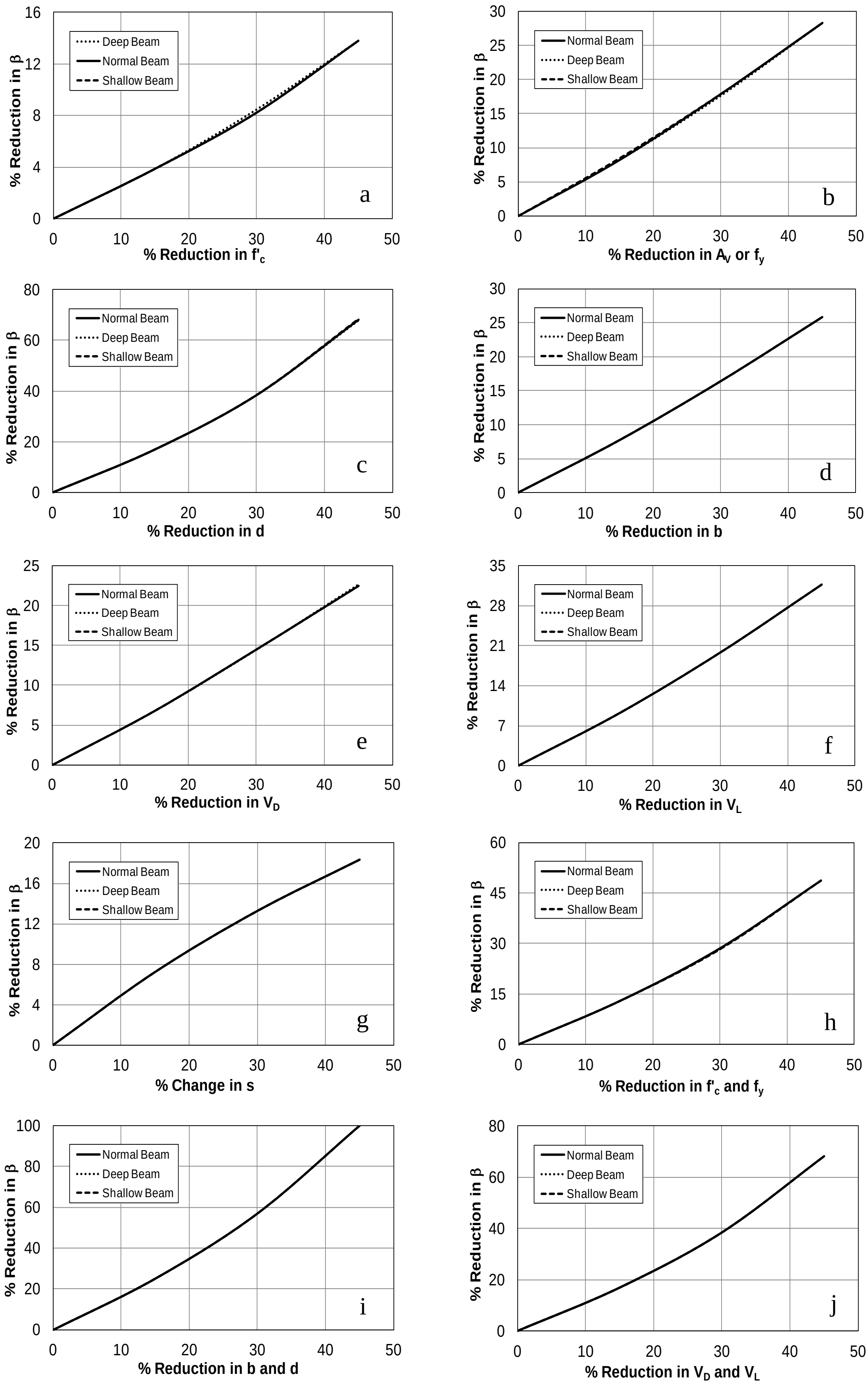

Figure 74: Effect of cross-section depth on the reliability of a beam under shear

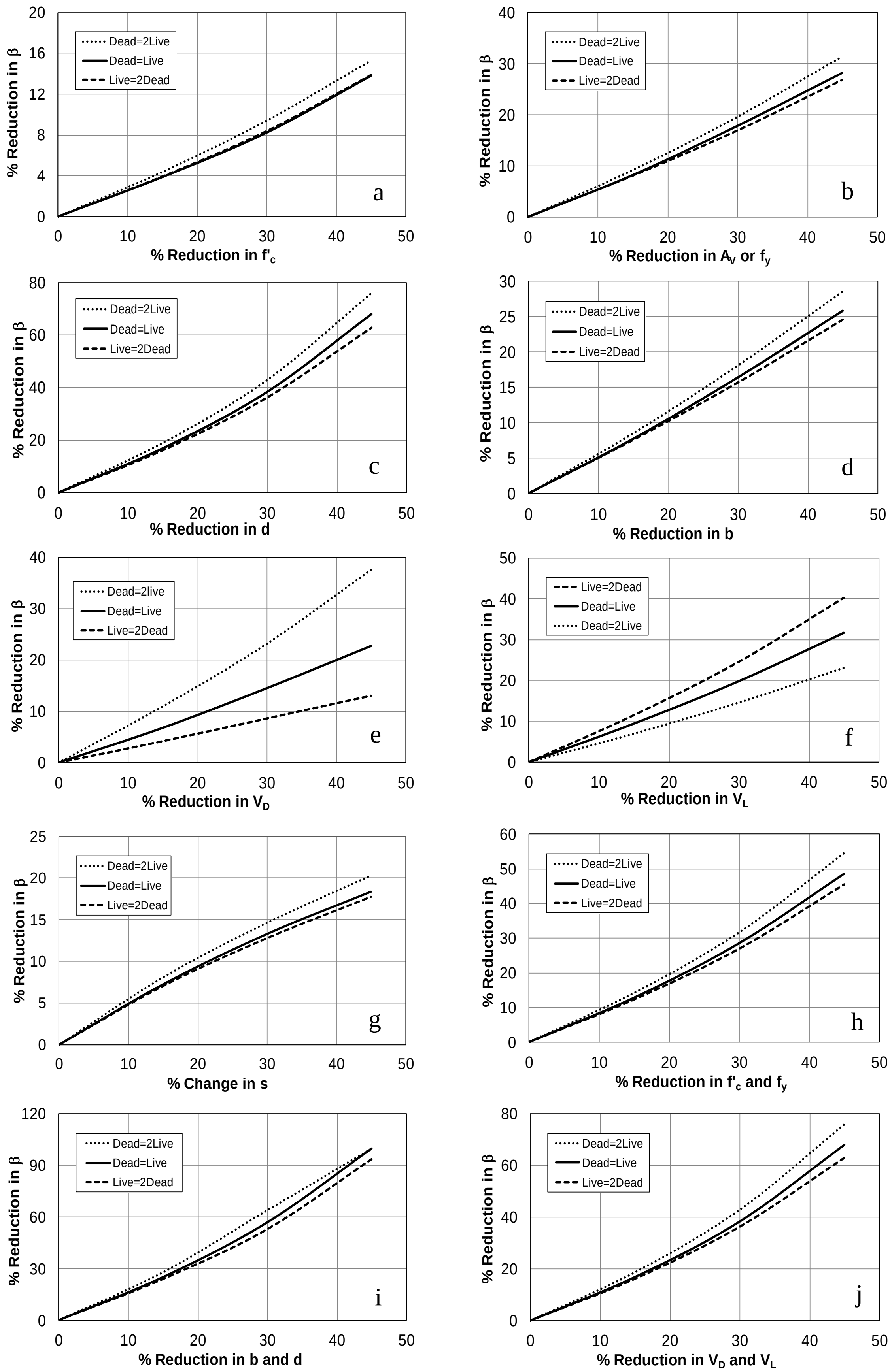

Figure 75: Effect of live-to-dead load ratio on the reliability of a beam under shear

Similar to what was noticed for beams under flexure, the reliability-based analyses carried out on the six different cases of beams under shear have shown that the sensitivity of beams under shear varies if the design parameters of the cross section change.

In Figure 70, where the area of vertical stirrups is the controlling factor, the cross section designed with the lowest area of stirrups was the most sensitive to the reductions in the compressive strength of concrete or the width of the beam. On the contrary, the cross section with the highest area of stirrups was the most sensitive to the errors committed in the yield strength of stirrups, area of stirrups, or the spacing between stirrups. As for variations in the effective depth of the tension steel reinforcement, dead load, live load and combined dead and live load, the sensitivity analysis has indicated that the area of stirrups did not affect the reduction in the reliability index in this regard. This means that beams transversely reinforced with a lot or few stirrups are equally likely to be affected by the same reduction in the reliability index due to reduced cross-section depth, or due to under-estimation of shear live or dead load.

When both the compressive strength of concrete and the yield strength of the steel stirrups are reduced by the same percentage, the three considered cross sections with small, moderate and large area of stirrups had similar sensitivity until the reduction reached 20%, after which beams with large area of stirrups became the most sensitive. This confirms that beams under shear are more sensitive to changes in the yield strength of stirrups than to changes in the compressive strength of concrete, especially when the area of stirrups is large.

Figures 71, 72, and 73 address the effect of spacing of stirrups, compressive strength of concrete, and the width of the beam, respectively. The conclusions that were drawn from Figure 70, as explained above, are valid for these three figures as well, given that the parameter “small area of stirrups” in Figure 70 corresponds to “large stirrup spacing” in Figure 71, and to “high compressive concrete strength” in Figure 72, and to “large beam width” in Figure 73.

In Figure 74, where the depth of the beam is the only difference among the three considered cross sections, the reliability-based sensitivity analysis conducted in this case has shown that the reduction in the reliability index of beams under shear was not affected at all by whether the beam was designed as shallow, normal or deep beam. This is due to fact that the beam depth contributes to both the shear strength provided by concrete, as well as the shear strength provided by the stirrups.

When the loading is the controlling factor among the three cross sections, as indicated in Figure 75, beams designed based on larger dead-to-live load shear ratio were the most sensitive to the reduction in all design variables, excluding the case of reduction in live load shear. Still, the difference in the reduction of reliability index was minimal, except when the reduction was in the dead load shear or the live load shear, as shown in Figure 75.

6.2.3 Axially Loaded Columns

Similar to what was performed on beams under flexure and shear, cross sections with different geometries, material properties and reinforcement were examined in order to determine if the results derived from the analysis of the standard case can be generalized. For columns under pure axial compression, the sensitivity analysis has been investigated for the following cases, which cover a wide profile of cases:

1. Three cross sections with the same design parameters, but with different gross longitudinal steel reinforcement ratio (1%, 2.57% and 4%).
2. Three cross sections with the same design parameters, but with different compressive strength of concrete (21 MPa, 42 MPa and 84 MPa).
3. Three cross sections with the same design parameters, but with different yield strength of steel reinforcement (250 MPa, 420 MPa and 500 MPa).
4. Three cross sections with the same design parameters, but with different gross cross-sectional area (90000 mm^2, 250000 mm^2 and 360000 mm^2).
5. Three cross sections with the same design parameters, but with applied axial compression due to different dead-to-live load ratio (P_D=2P_L, P_D=P_L, and P_L=2P_D).

The results of all reliability analyses are shown in Figures 76-80 and presented next.

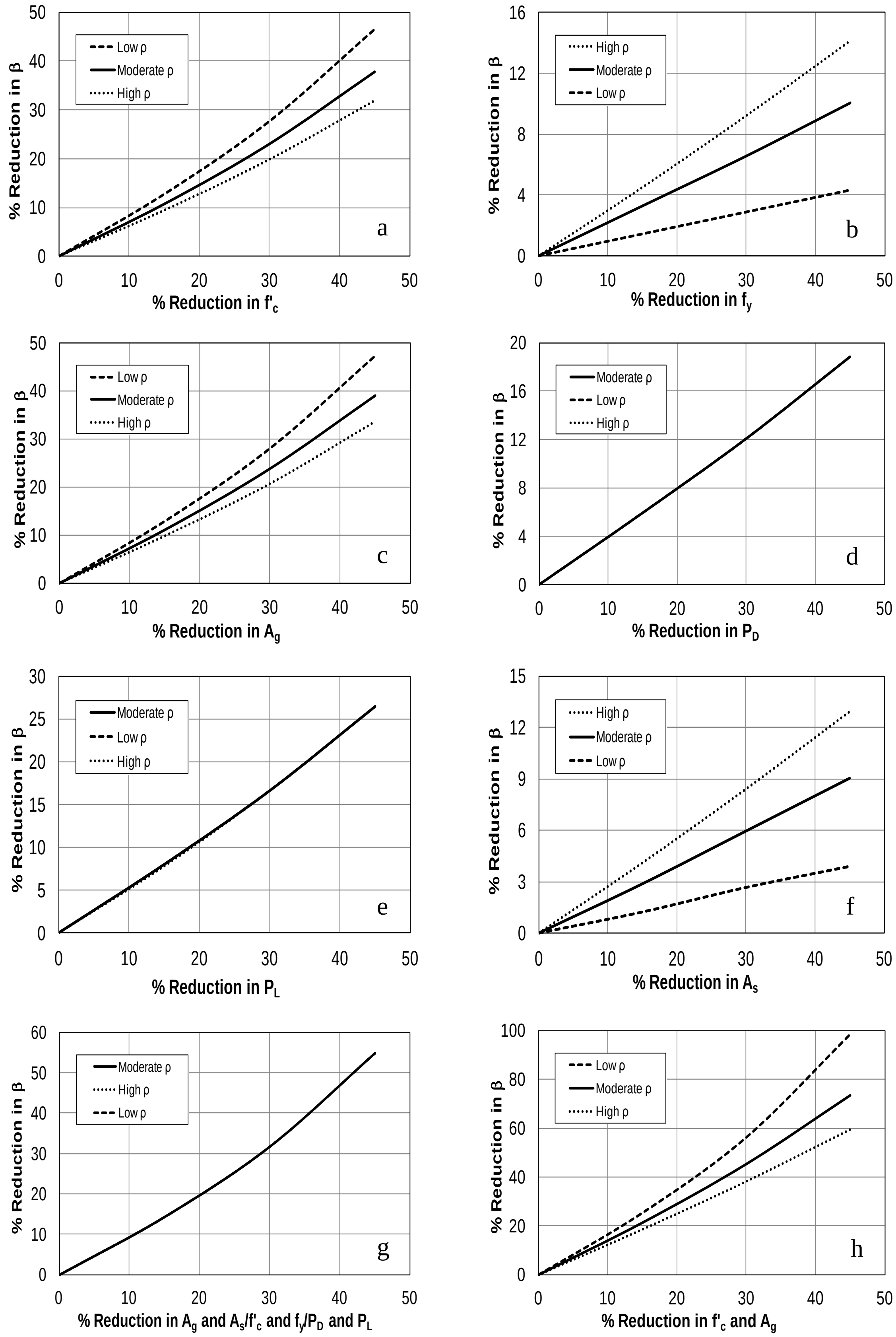

Figure 76: Effect of gross reinforcement ratio on the reliability of axially loaded columns

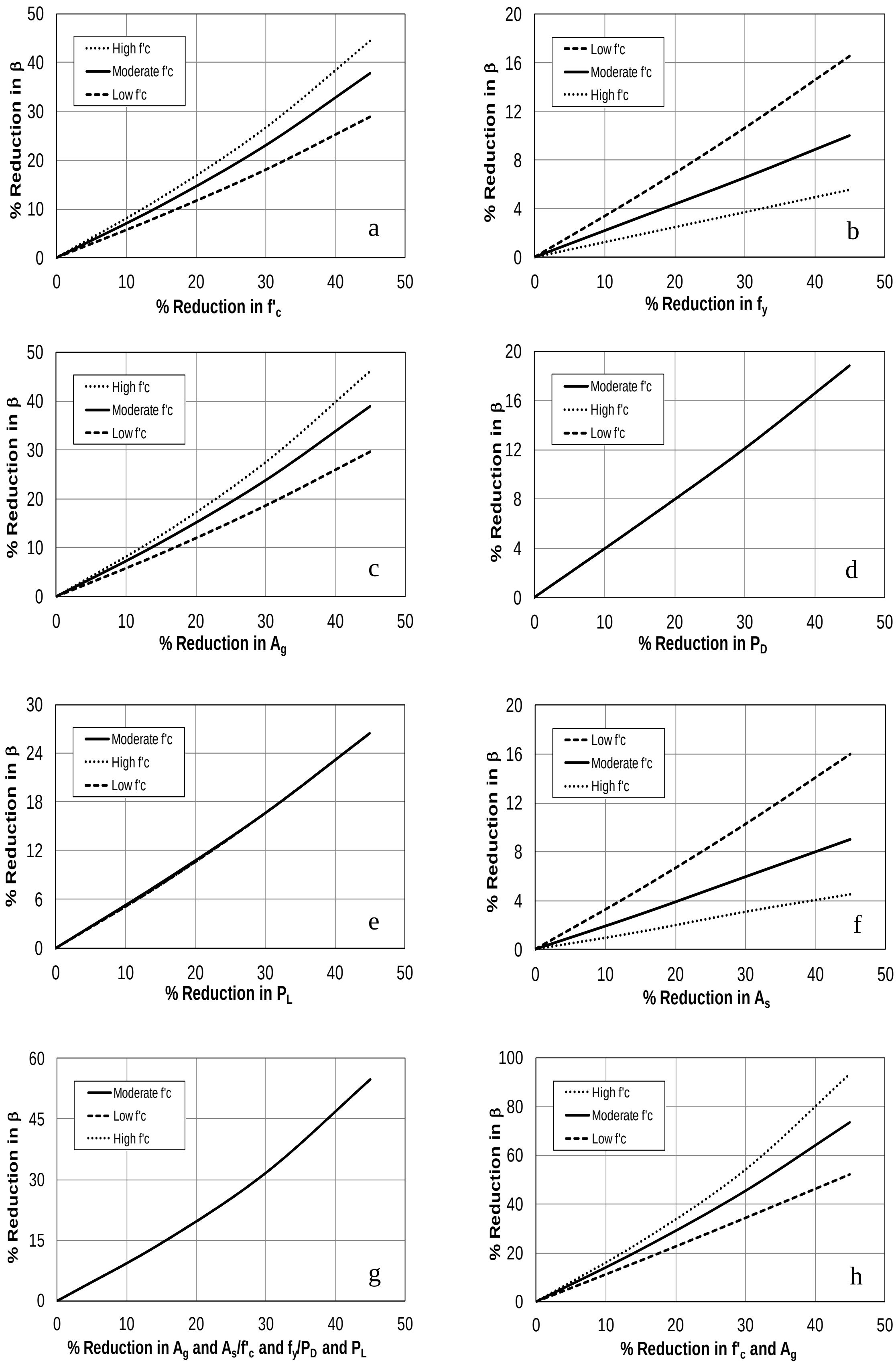

Figure 77: Effect of concrete strength on the reliability of axially loaded columns

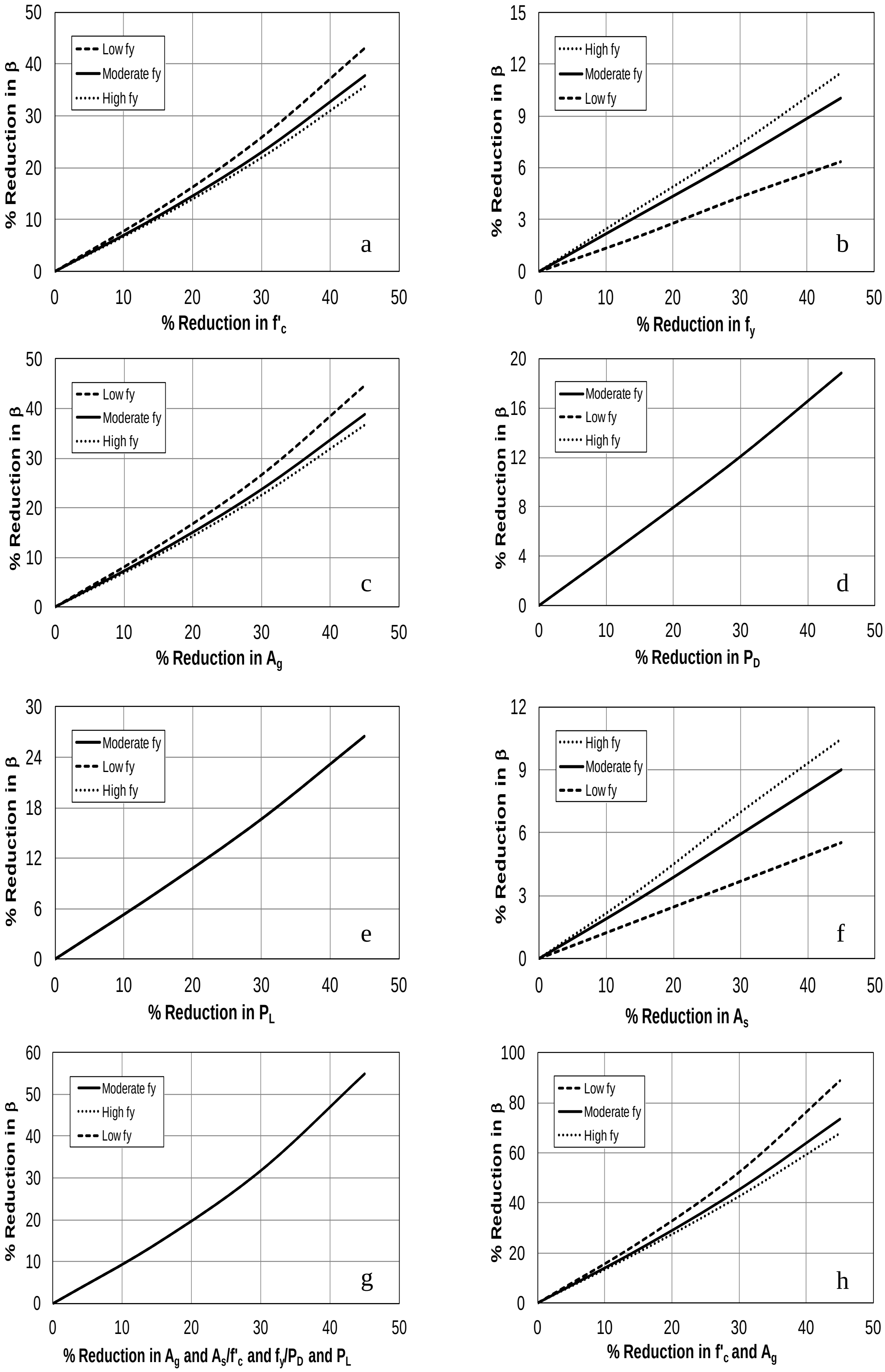

Figure 78: Effect of yield strength on the reliability of axially loaded columns

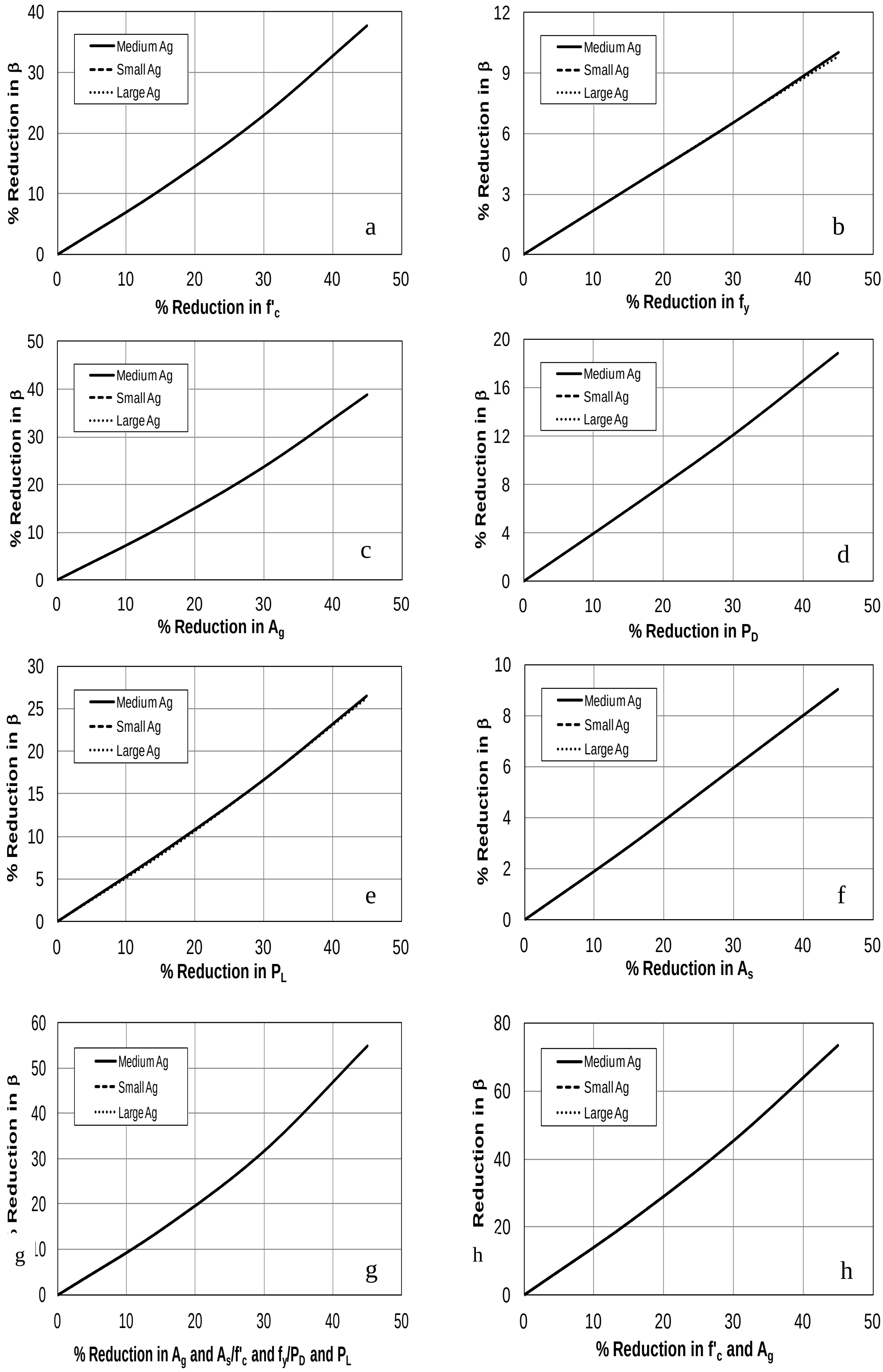

Figure 79: Effect of gross sectional-area on the reliability of axially loaded columns

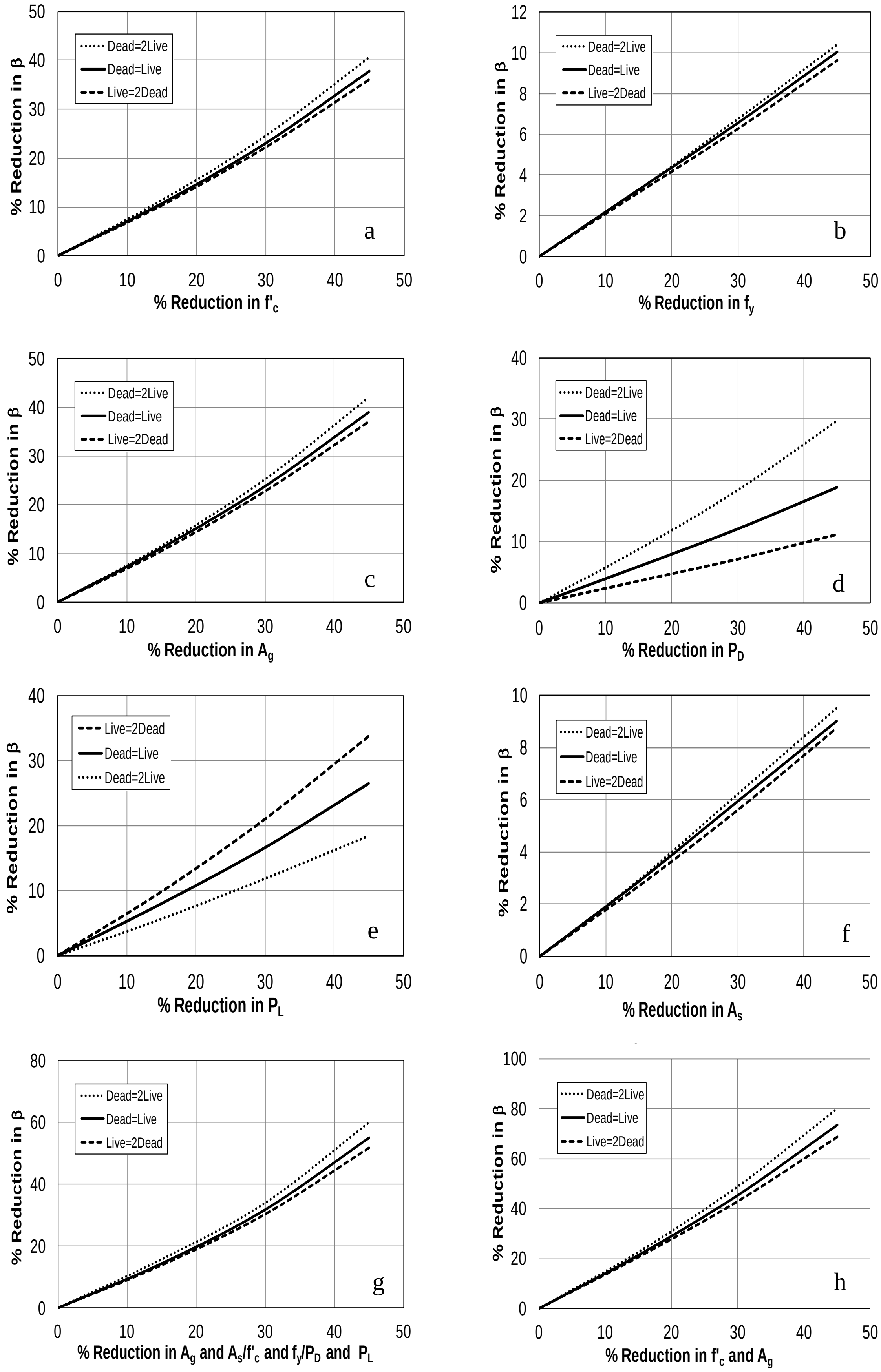

Figure 80: Effect of live-to-dead load fraction on the reliability of axially loaded columns

The sensitivity analysis performed on the different five cases has indicated that the results obtained for the reference cross section in the previous section are not always valid for other cross sections with different characteristics.

For the case where the three cross sections are similar in all properties, except for the reinforcement ratio, as shown in Figure 78, the reliability index of axially loaded columns with low reinforcement ratio was most sensitive to changes in the compressive strength of concrete, or in the dimensions of the column cross section. This is because the strength of lightly reinforced columns is dominated by the concrete capacity and gross sectional area, rather than the steel capacity.

Axially loaded columns with high reinforcement ratio, on the other hand, were the most susceptible to changes in reinforcement yield strength and area of longitudinal steel. Nevertheless, when changes were in dead load, live load, both the area of steel and the gross cross-sectional area, both the concrete compressive strength and steel yield strength of reinforcement, or a combination of dead and live loads, the reduction in the sensitivity of the three considered cross sections was constant.

The same finding applies to the cases in which concrete compressive strength or reinforcement yield strength is the controlling factor among the three cross sections, as represented in Figures 79 and 80, respectively. Axially loaded columns designed with low reinforcement ratio had the same sensitivity behavior to that of axially loaded columns designed with low reinforcement yield strength or high concrete compressive strength. Similarly, axially loaded columns designed with high reinforcement ratio, high reinforcement yield strength, or low concrete compressive strength had similar sensitivity behavior.

Except for the changes in the live load fraction of the total load, Figure 80 shows that columns subjected to large axial dead load, compared to live load, were the most sensitive to reductions in all other design variables, such as concrete strength, steel yield strength, gross sectional area. Also, unlike the previously discussed cases, the reduction in the reliability index for the considered cross sections with similar properties, but with different gross cross sectional area, was the same for all cross sections. In other words, when the cross sectional area is the controlling factor, axially loaded columns will have the same sensitivity to reductions in all design variables, as shown in Figure 79.

CHAPTER 7

COMPARSION BETWEEN DETERMINSTIC AND RELAIBILITY-BASED ANALYSES

7.1 Introduction

The analyses in Chapters 5 and 6 have led to a conclusion that the results of deterministic and the reliability based approaches are similar, in the sense that different cross sections characteristics have different sensitivity behaviors. Nonetheless, there are some cases in which the deterministic approach causes different reductions in load-carrying capacity, when compared with the reduction in the reliability index.

In this chapter, comparisons between the reduction in the reliability index and the reduction in the nominal capacity of beams under flexure, beams under shear, and axially loaded columns are made. The chapter makes use of the findings of all the cases discussed in the previous two chapters. The purpose of these comparisons is to show also the similarities and differences between the results of the two approaches.

7.2 Beams under Flexure

For beams under flexure, the main results are presented in Figure 81, which shows the percentage change in nominal capacity and reliability index from the intact member, when the member goes through reductions in some design variables. The figures are presented for the cases of changes in: (1) beam width b, (2) concrete compressive strength f'_c, (3) area of longitudinal steel reinforcement A_s, (4) reinforcement yield strength f_y, (5) effective depth of tension steel d, (6) dead load moment M_D, (7) live load moment M_L, (8) both concrete compressive strength f'_c and reinforcement yield strength f_y, (9) both beam width b and effective depth of tension steel d, and (10) both dead load moment M_D and load live moment.

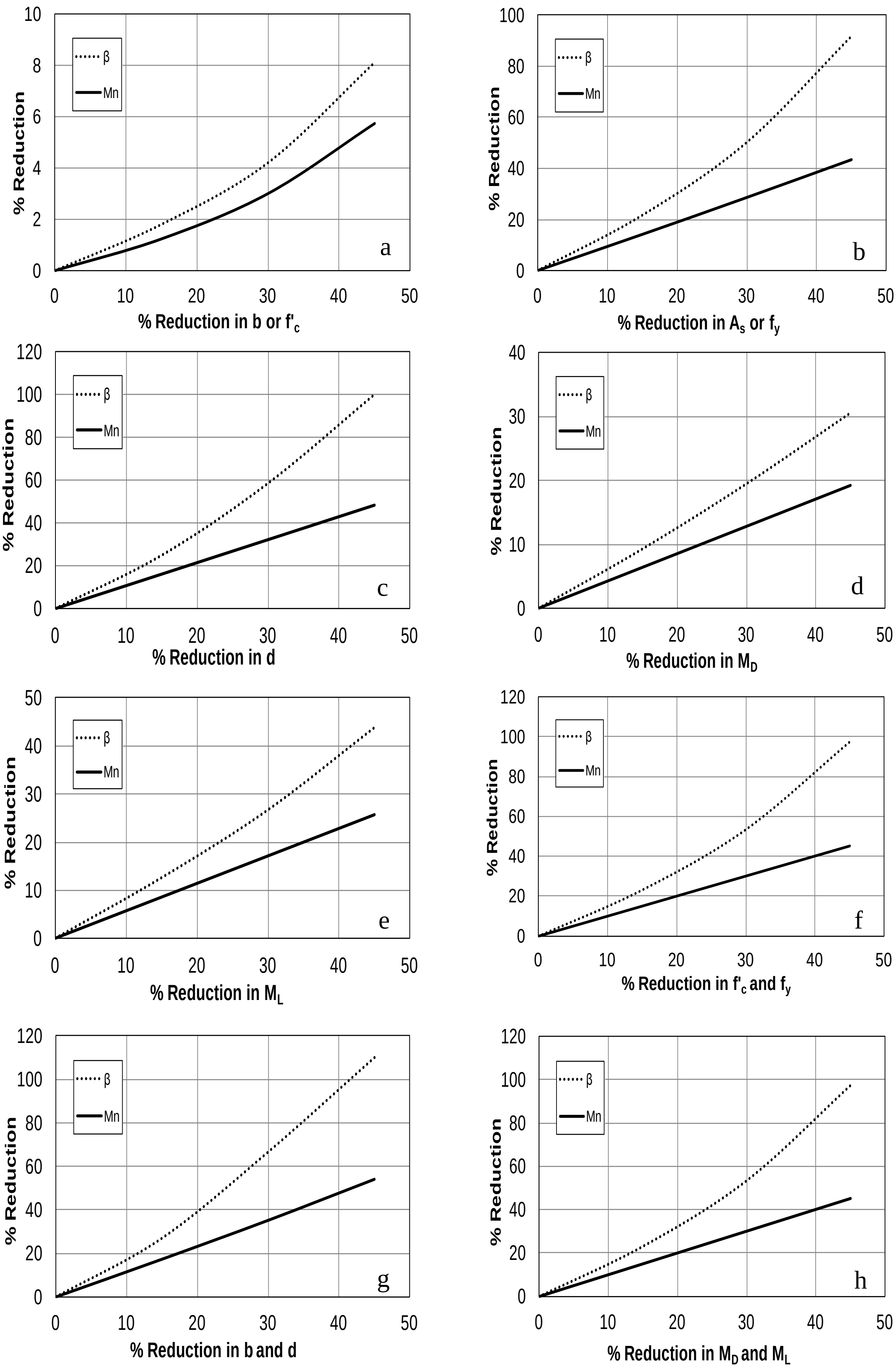

Figure 81: Comparison of deterministic and reliability analyses for beams under flexure

Figure 81 indicates that the reduction in the reliability index of beams under flexure is much larger than the reduction in the nominal capacity for all the considered scenarios. Therefore, use of the deterministic approach to examine the sensitivity behavior of beams under flexure does not help in assessing the loss of structural safety. Note also the slope of sensitivity functions is often nonlinear in the reliability analysis, as opposed to almost straight line in the deterministic analysis.

When comparing the results of the deterministic and reliability-based approaches in terms of the sensitivity of the three cross sections to the changes in design parameters, it was found that the three cross sections had the same sensitivity ordering, except for the case in which loading is the controlling factor (i.e. live-to-dead load ratio).

With regard to the case where the live load fraction of the total load being the controlling factor, the three cross sections had similar sensitivity ordering only when the change was either in the live load moment or dead load moment, as shown in Figure 82.

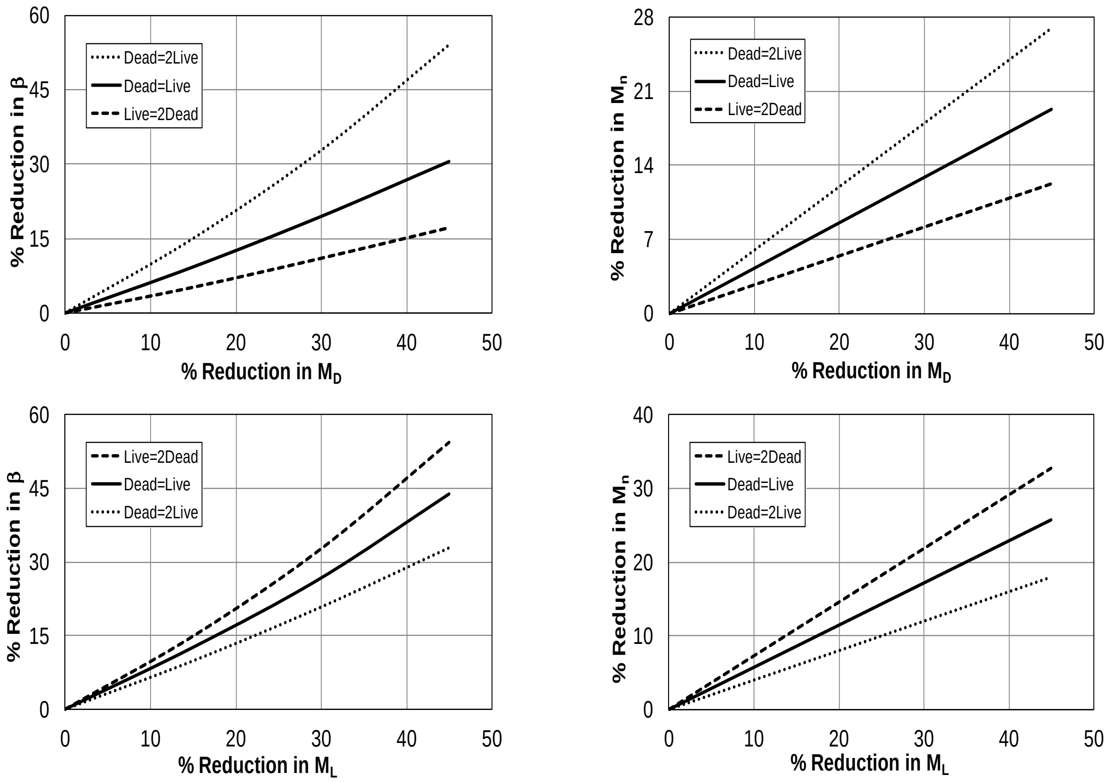

Figure 82: Effect of live-to-dead load ratio on reliability and nominal capacity in flexure

Although the sensitivity functions ordering is the same in Figure 82 for both deterministic and nondeterministic approaches, the reduction in the reliability index increased significantly with the change in dead load or live load moment when compared with the reduction in nominal moment capacity. For beams designed with different live-to-dead load moment, the percent reduction in the reliability index was just about the same as the percent reduction in the nominal flexural moment capacity.

When the change in live load and dead load moment occurred at the same time, both the deterministic and reliability-based approaches did not give the same pattern of the sensitivity functions, as illustrated in Figure 83. In the reliability analysis, there were some variations in the results for the different live-to-dead load ratios, which were not observed in the deterministic analysis.

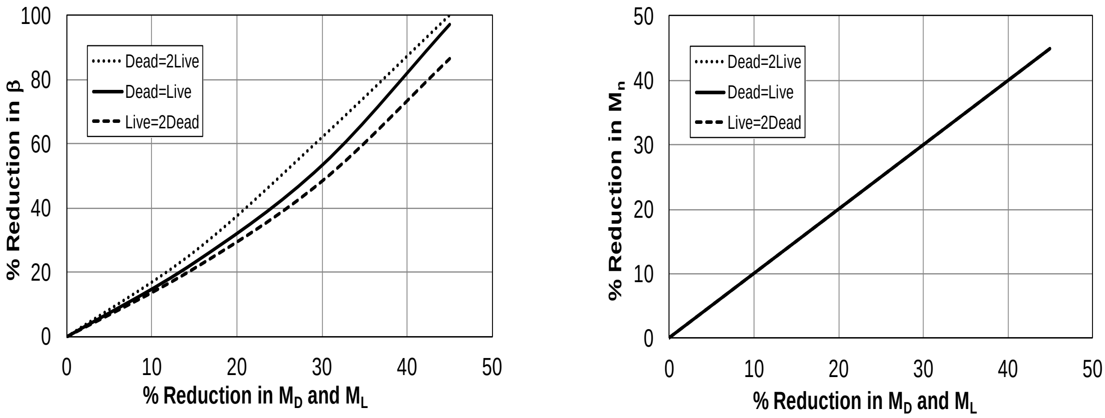

Figure 83: Effect of both live and dead loads on reliability and nominal capacity in flexure

For other cases related to live-to-dead load ratio, the two approaches did not give the same pattern, as highlighted in Figure 84.

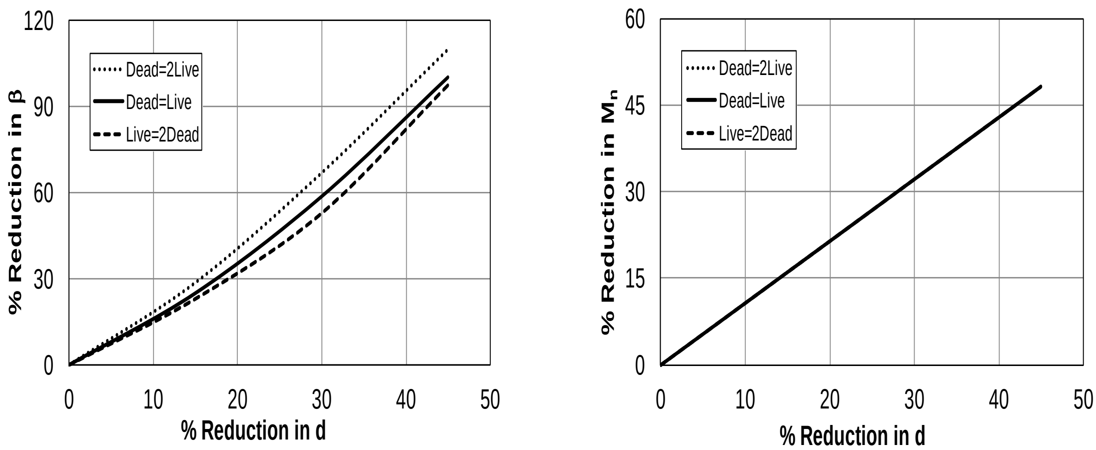

Figure 84: Effect of depth of tension steel on reliability and nominal capacity in flexure

7.3 Beams under Shear

For beams under shear, the major results are presented in Figure 85, which shows the percentage change in nominal capacity and reliability index from the intact member, when the member goes through reduction in some design variable. The figures are presented for the cases of changes in: (1) concrete compressive strength f'_c, (2) area of stirrups A_v, (3) reinforcement yield strength f_y, (4) effective depth of tension steel d, (5) beam width b, (6) dead load shear V_D, (7) live load shear V_L, (8) stirrups spacing s, (9) concrete compressive strength f'_c and reinforcement yield strength f_y, (10) both beam width b and effective depth of tension steel d, and (11) both dead load shear V_D and live load shear.

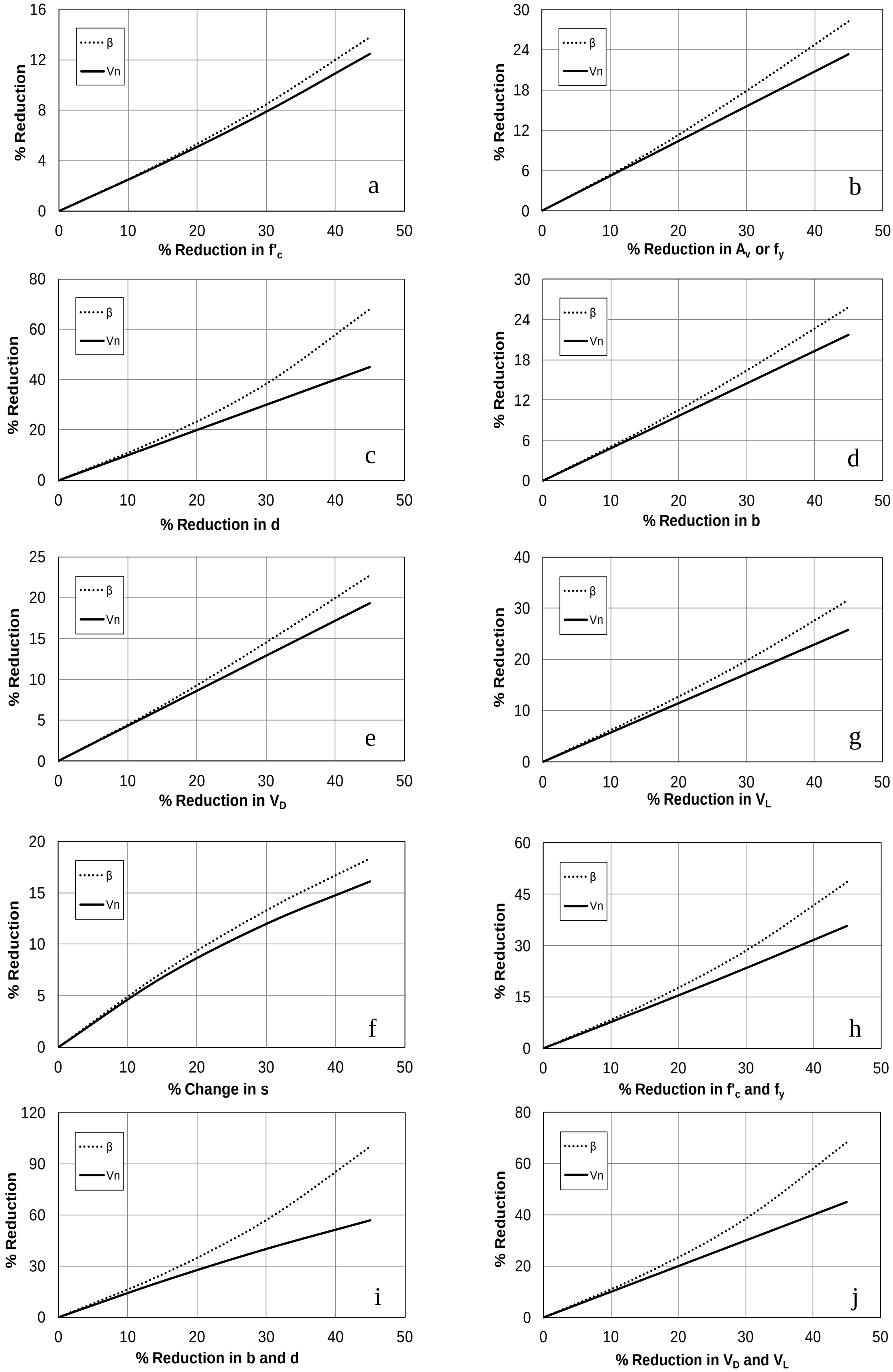

Figure 85: Comparison of deterministic and reliability analyses for beams under shear

The comparison between the reduction in the nominal capacity and the reduction in the reliability index for beams under shear due to changes in the design parameters, as summarized in Figure 85, has indicated that the reduction in both of them remained the almost the same until the reduction reached 20%, after which the reduction in the reliability index became larger. Still, this difference for the case of shear was very mild compared to the difference observed for beams under flexure. The only cases where the difference in the reduction between the two approaches can be considered large is when the reduction occurs in the effective depth of tension steel, and in the combination of the depth of tension steel and width of the beam, given that the change in them is 20% or more.

Regarding the slopes of the two sensitivity functions, the comparison has shown that slopes were matching in some scenarios and different in others. Hence, no definite conclusion can be reached in this regard.

The comparison between the deterministic and reliability-based approaches in terms of the sensitivity of the three considered cross sections to the changes in design parameters for beams under shear has indicated that the conclusions derived for beams under flexure are valid as well. The three cross sections had the same sensitivity ordering for all cases, with the exception of the case when load composition is controlling.

More specifically, the three cross sections had similar sensitivity ordering when the change is either in the dead load shear or in the live load shear, as shown in Figure 86.

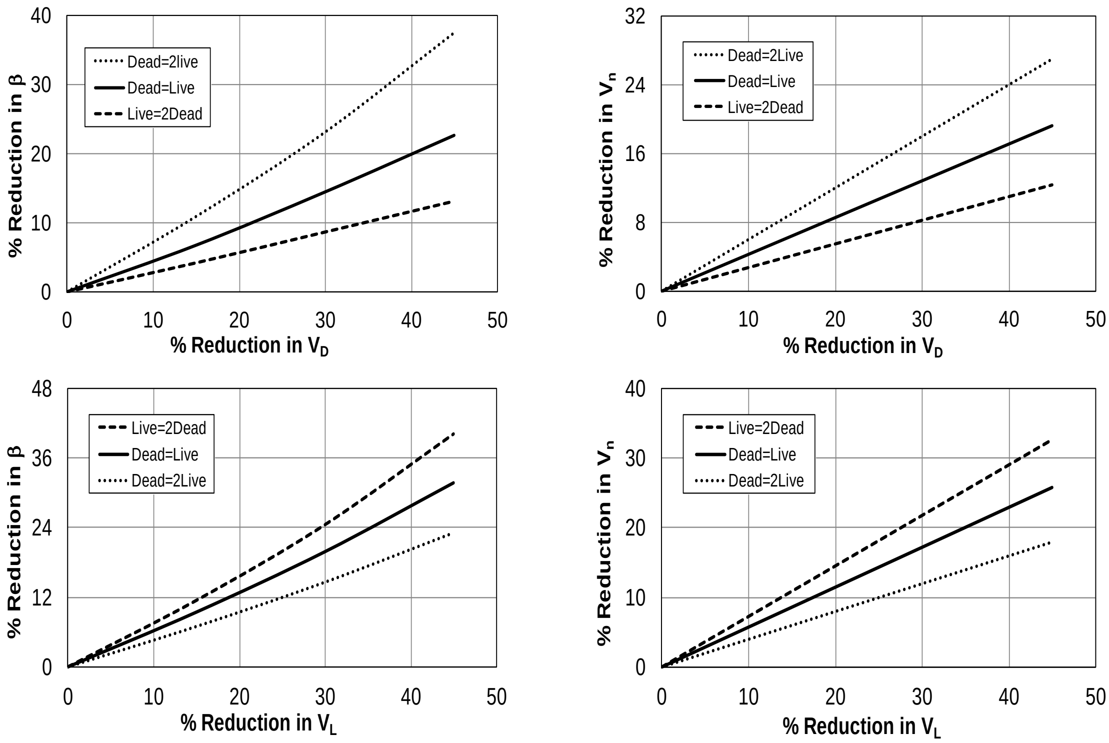

Figure 86: Effect of live-to-dead load ratio on reliability and nominal capacity in shear

Similar to beams under flexure, the reduction in the reliability index of beams under shear increased significantly when the dead load considered in the design increased. Likewise, the sensitivity of the three cross sections was not the same when the change occurred in both the dead load and live load at the same time, as shown in Figure 87.

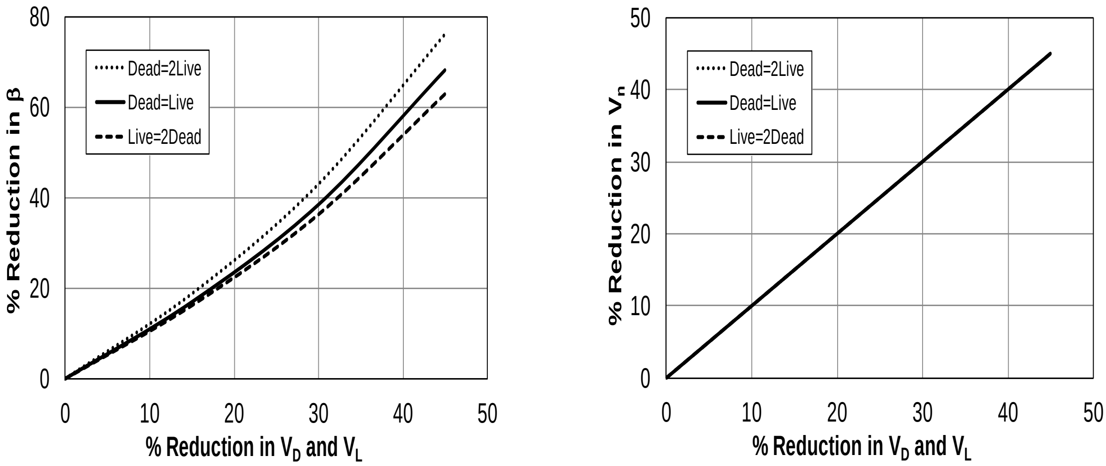

Figure 87: Effect of both live and dead loads on reliability and nominal capacity in shear

For other cases of the live-to-dead load ratio, the two approaches did not give the same sensitivity behavior, as presented in Figure 88.

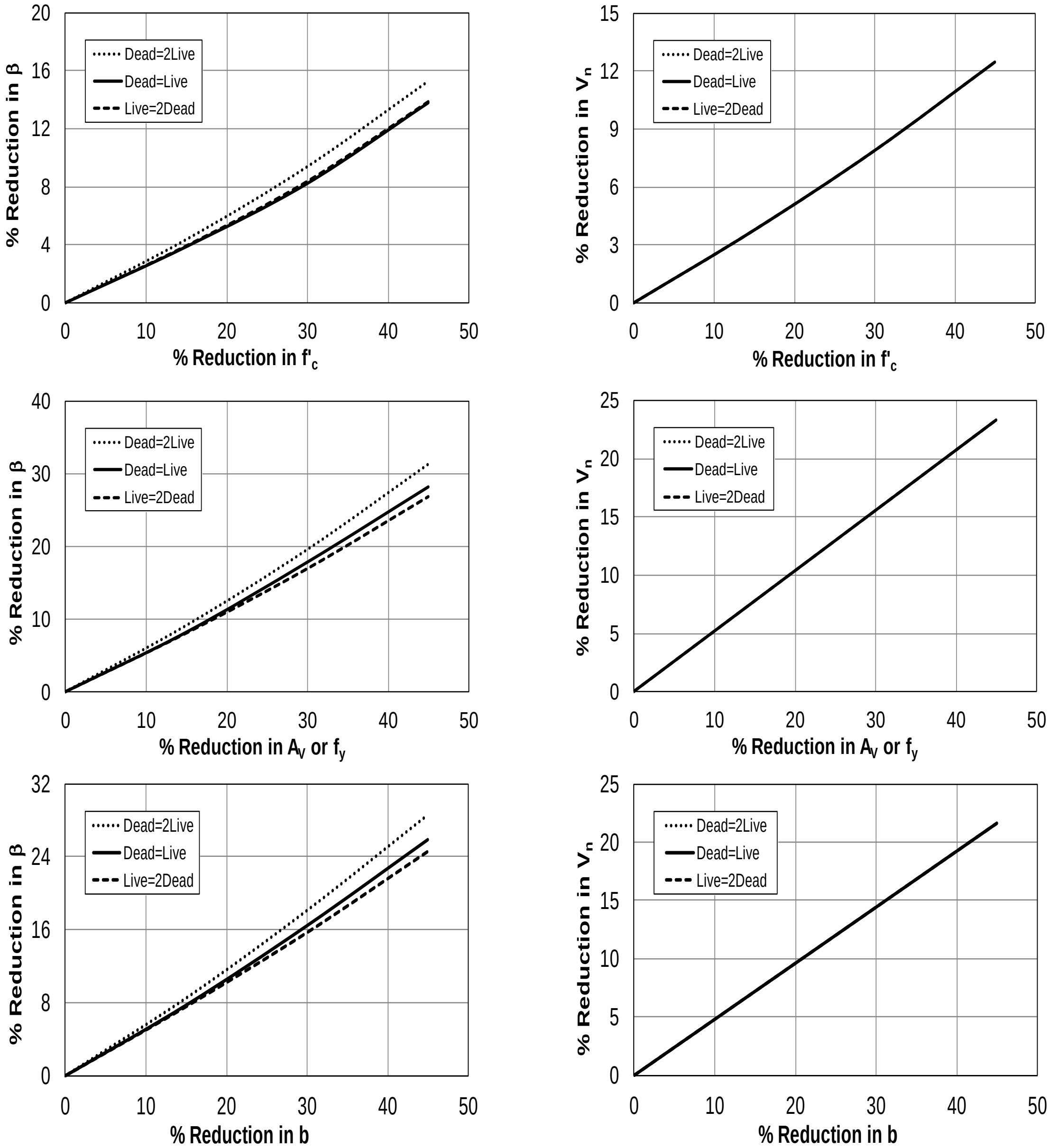

Figure 88: Reduction in reliability index and nominal capacity for beams under shear

As for the cases where there was significant reduction in the reliability index or in the nominal capacity among the three considered cross sections, this was only encountered in the reliability based approach and not in the deterministic analysis, as shown in Figure 89. In the deterministic analysis, the change in the nominal shear capacity was observed to be moderate.

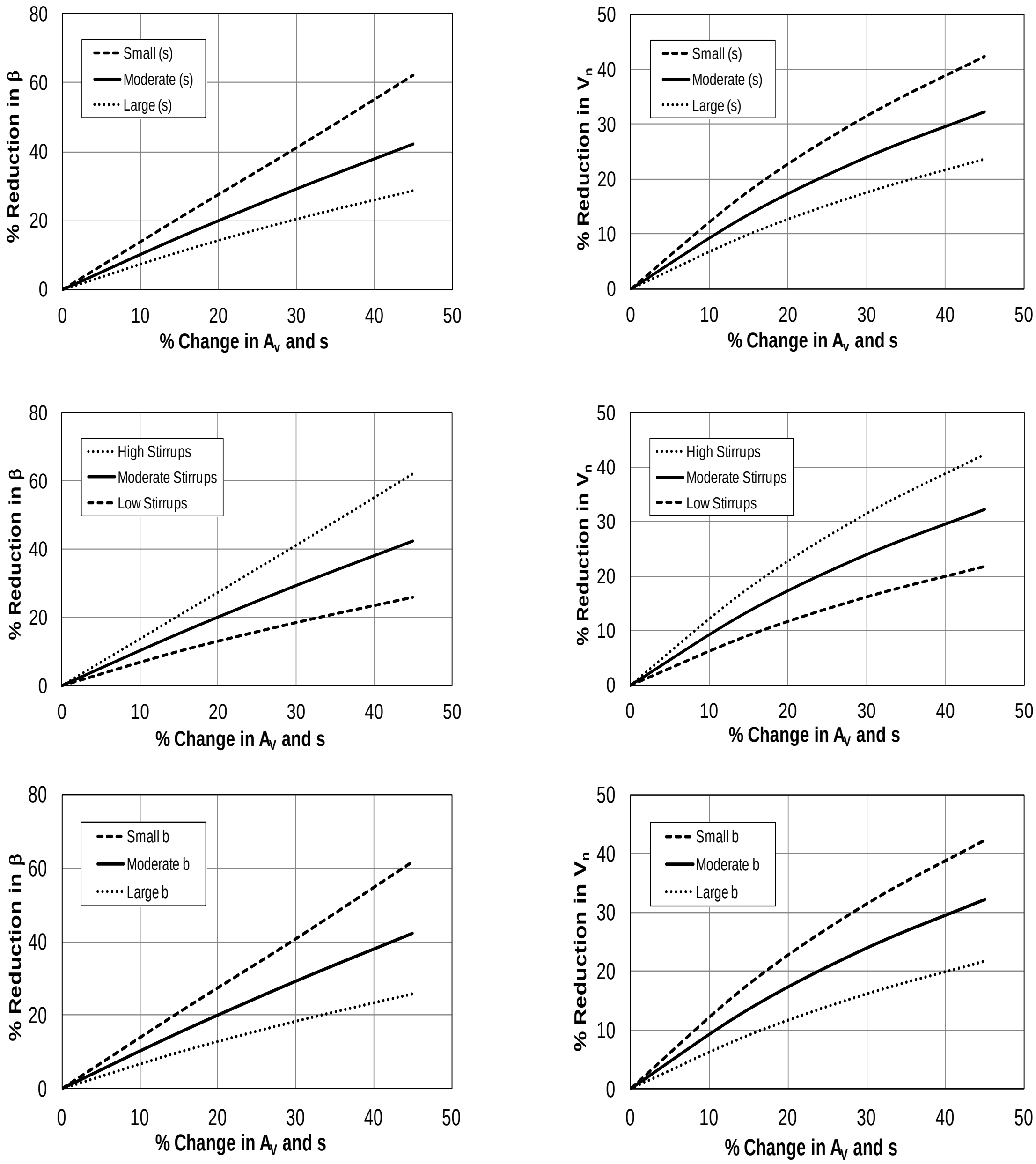

Figure 89: Significant reduction in reliability index and capacity for beams under flexure

7.4 Axially Loaded Columns

For columns under concentric axial compression, the major results are presented in Figure 90, which shows the percentage change in nominal capacity and reliability index from the ideal member, when the member goes through reduction in some design variable. The figures are presented for the cases of changes in: (1) concrete compressive strength f'_c, (2) yield strength of reinforcement f_y, (3) gross area of column A_g, (4) axial dead load P_D, (5) axial live load P_L, (6) area of longitudinal steel

A_s, (7) both gross area of column A_g and area of longitudinal steel A_s, (8) both concrete compressive strength f'_c and reinforcement yield strength f_y, (9) both axial dead load P_D and live load P_L, and (10) both concrete compressive strength f'_c and gross cross-sectional area A_g.

When the comparison was made between the reliability based and deterministic approaches for axially loaded columns, the reductions in the nominal capacity and reliability index were found to be almost the same. In most scenarios, the reduction in the nominal capacity was slightly larger than the reduction in the reliability index. When the change in concrete compressive strength or in the dimensions of the cross section were considered, their effect on the nominal capacity was more predominant until the change reached about 25%; thereafter, the reduction in the reliability index started to become slightly larger. Yet, the difference between the two approaches was always very small compared to the effects on beams under flexure and beams under shear, discussed earlier.

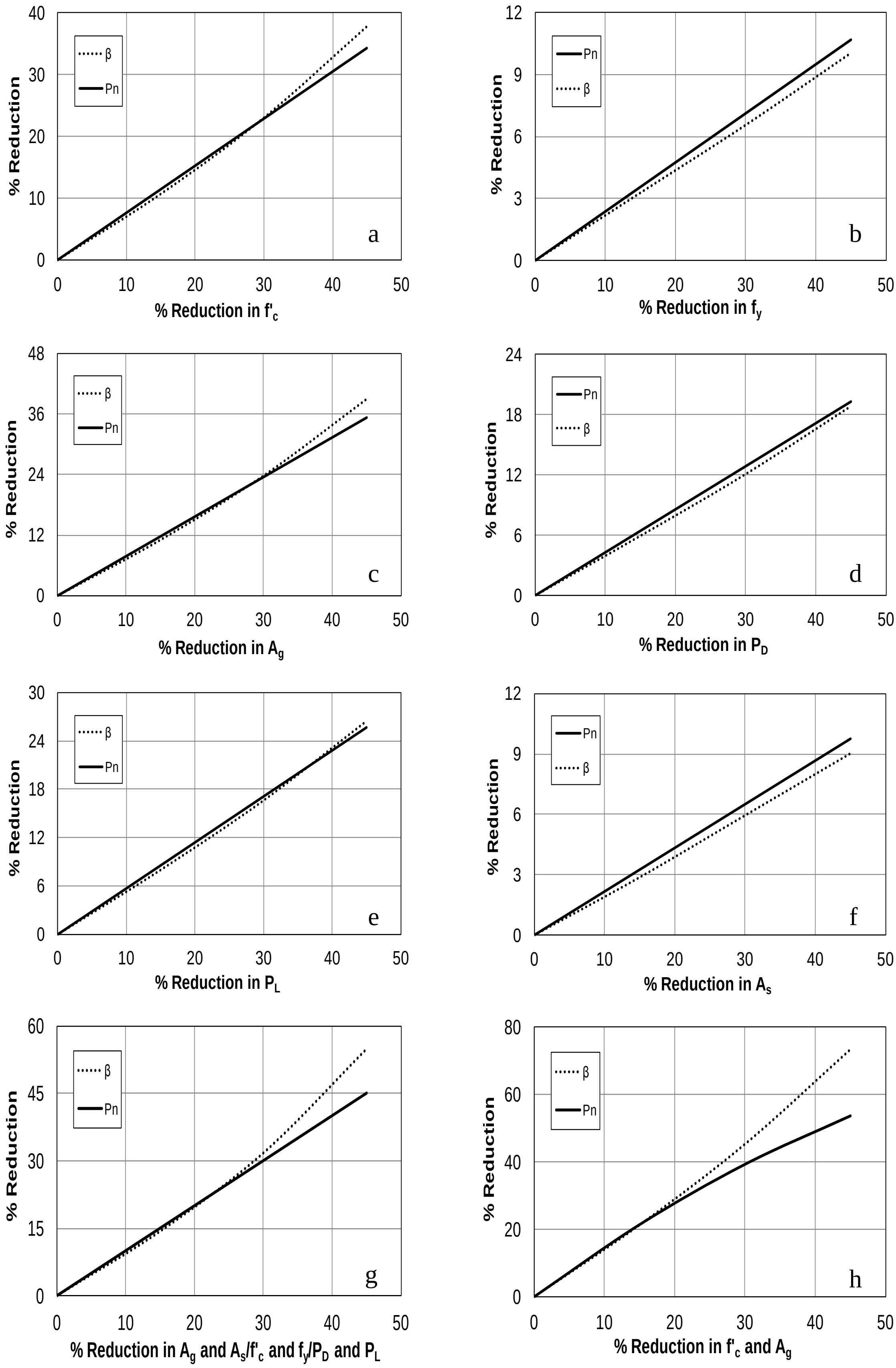

Figure 90: Comparison of deterministic and reliability analyses for axially loaded columns

As for the differences between the sensitivity of the three considered cross sections in both deterministic and reliability-based approaches with respect to the changes in the design variables, the comparison leads to the same conclusions previously made on beams under flexure and beams under shear (i.e. sensitivity ordering was only different for the case controlled by live-to-dead load ratio). In that case, the two approaches showed similarities in results when the reduction occurred in dead load or live load, as highlighted in Figure 91.

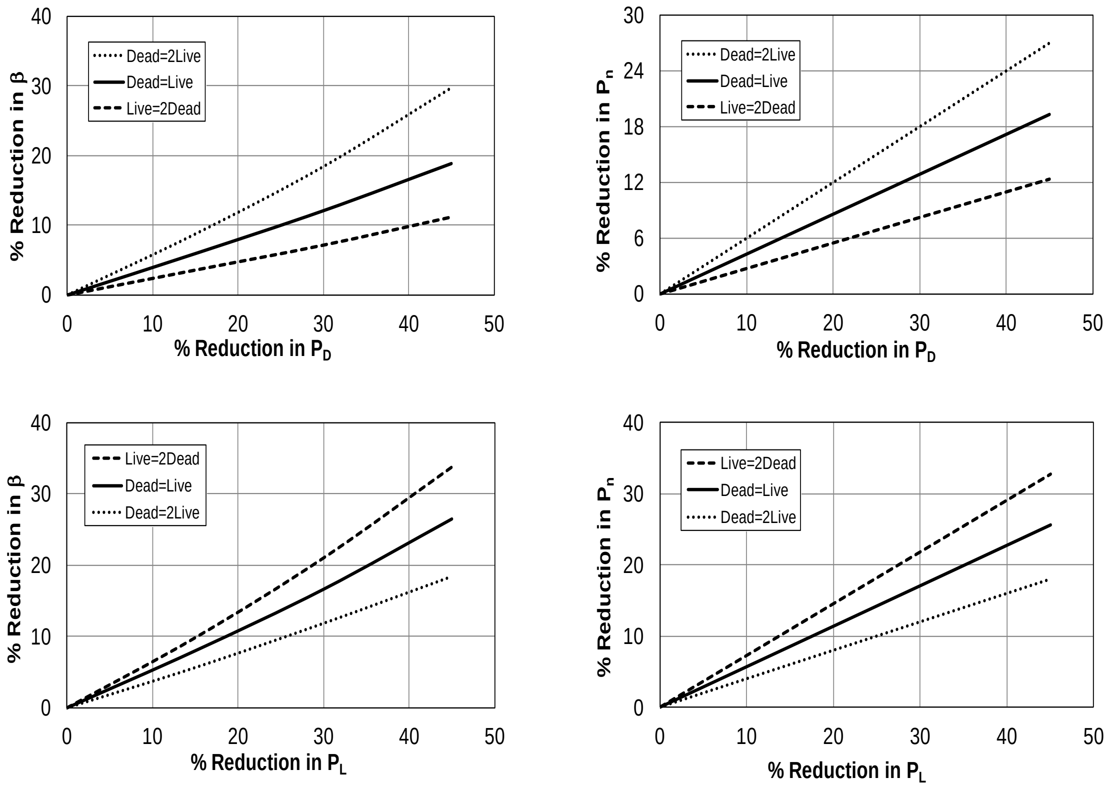

Figure 91: Effect of live-to-dead load ratio on reliability and nominal capacity of columns

However, when the reduction happened in dead load and live load at the same time, the two approaches gave different sensitivity behaviors, as demonstrated in Figure 92.

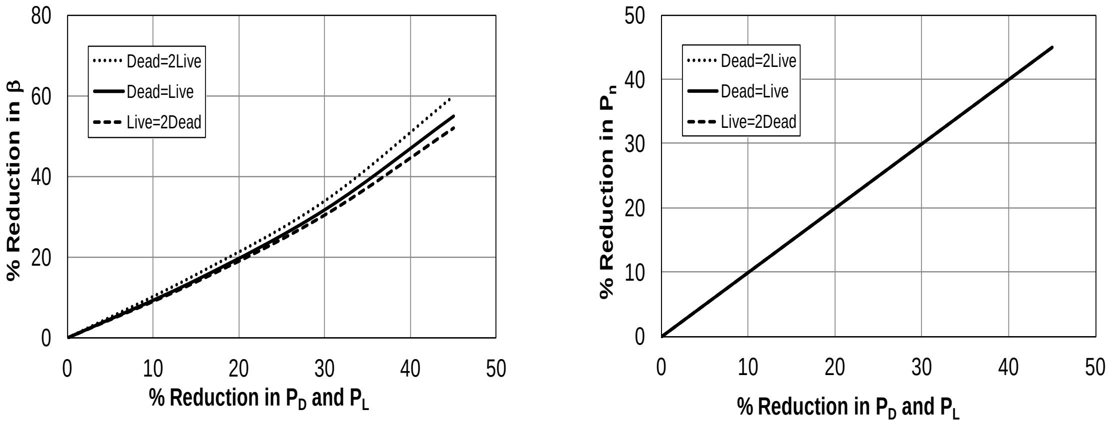

Figure 92: Effect of live and dead loads on reliability and nominal capacity of columns

Also, the two approaches gave different behavior for other scenarios of the loading cases, as shown in Figure 93.

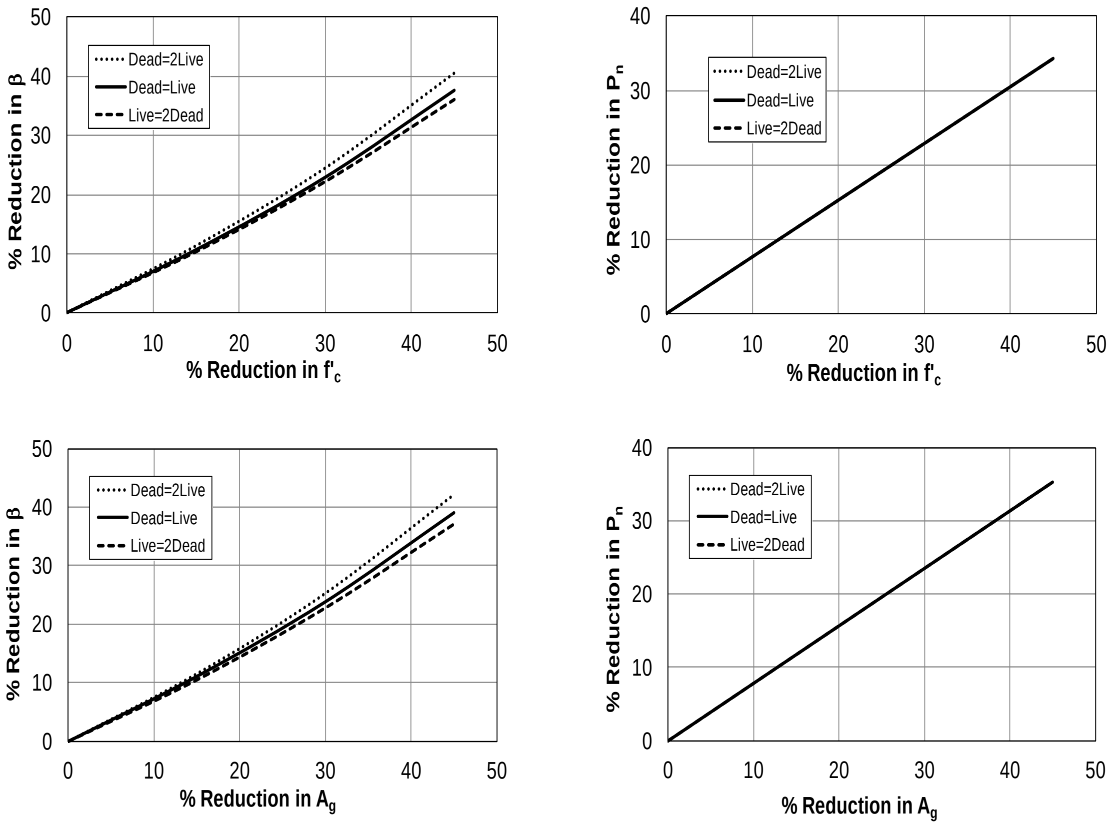

Figure 93: Reduction in reliability index and nominal capacity of columns

For the scenarios in which the reduction in certain design variables had high impact only on the most critical cross section of the reliability based approach, these scenarios are summarized below in Figure 94.

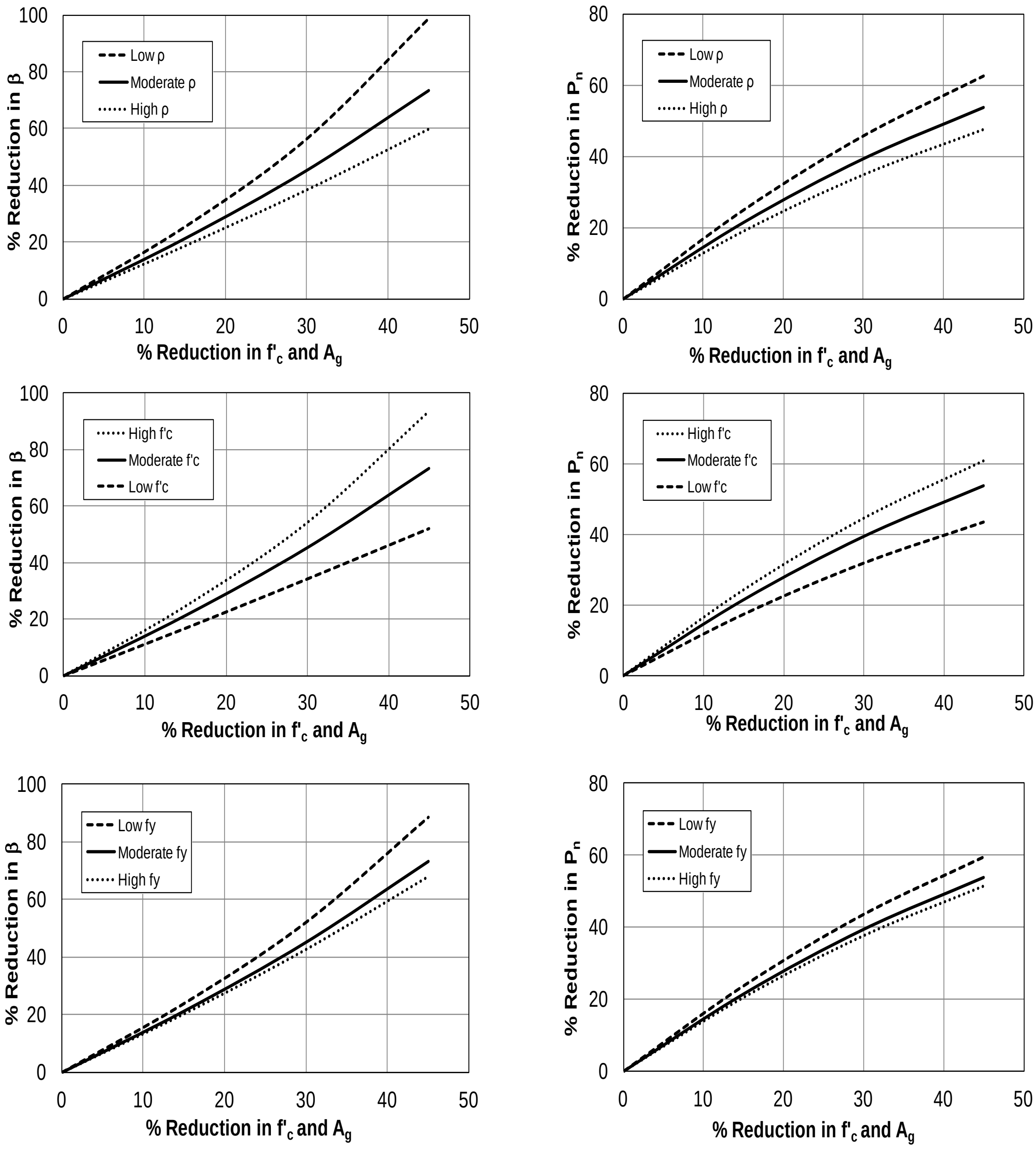

Figure 94: Significant reduction in reliability index and capacity of columns

Figure 94 indicates that for axially loaded columns designed with low reinforcement ratio, high compressive strength, or low reinforcement yield strength; both the deterministic and reliability-based approaches show large differences between the compromised section and intact section.

7.5 Interpretation of Results

7.5.1 Beams under Flexure

The results obtained from the deterministic and reliability based approaches can be interpreted using the equation of the nominal capacity:

$$\emptyset M_n = \emptyset A_s f_y (d - 0.5a) = 1.2M_D + 1.6M_L$$

where $$a = \frac{A_s f_y}{0.85 f'_c b}$$

Let's consider the case shown in Figure 63. In Figure 63.a, the cross section designed with higher area of steel is the most sensitive to the reduction in compressive strength of concrete, or the width of the beam. The reason is because the depth of Whitney block is larger in the cross section designed with higher area of steel. If the reduction occurs in the compressive strength of concrete or the width of the beam, then the increase in the depth of Whitney block in beams designed with higher area of steel will be the most. Consequently, the distance between the centroids of the compression and tension regions will be reduced. As a result, the reduction in the moment capacity will be largest in beams designed with higher area of steel. The same concept can be used to explain why beams designed with larger area of steel are the most sensitive to the reduction in the depth of the tension steel reinforcement, as illustrated in Figure 63.c.

For the case shown in Figure 63.b, it is shown that beams under flexure designed with the low area of steel are most sensitive to the reduction in the area of steel, although the difference amongst the three considered cross sections is small. When a 20% reduction in the area of steel is imposed, the reduction in flexural capacity is relatively more significant in lowly reinforced beams compared to highly reinforced ones.

When the live-to-dead load ratio is the controlling factor, as indicated in Figures 46 and 67, the sensitivity analysis has shown that the three cross sections will have different sensitivity functions in the reliability based approach, and similar sensitivity in the deterministic approach, except when the reduction happens in dead load or in live load. The interpretation of this is that cross sections with different loading scenarios have been designed to have the same nominal moment capacity, but with different ratio of dead to live load. Thus, any reduction in the nominal flexural capacity will be the same for the three considered cross sections. However, the reduction in reliability index for the three cross sections will not be the same because dead load and live load have different bias factors, and different coefficients of variation. As for the explanation of why the reduction in the nominal capacity is not constant for the three cross sections when the error is committed in live load or dead load, this is attributed to the contribution of dead load moment and live load moment to the moment capacity. As an illustration, when the dead load moment is as twice as the live load moment, the dead load moment in this case is contributing more to the nominal capacity, while the opposite applies when the live load moment is as twice as the dead moment. Hence, beams subjected to dead load moment larger than live load moment will be more sensitive to the reduction in the dead load moment. Notice that as per the equation$\emptyset M_n = 1.2M_D + 1.6M_L$, when the error is committed in dead load moment and live load moment at the same time, the error is taken as a common factor, leading to a constant reduction in the nominal capacity, as shown in Figure 67.h.

Although the sensitivity ordering for the three cross sections can be predicted using the nominal capacity equation, the exact difference in sensitivity among the three considered cross sections cannot be predicted, as there are scenarios in which the difference was very slight, and scenarios where the difference was significant.

7.5.2 Beams under Shear

The results obtained from the deterministic and reliability-based approaches can be also interpreted using the code’s equation of shear capacity:

$$\emptyset V_n = \emptyset(0.17\sqrt{f'_c}b_w d + A_v f_y d/s) = 1.2V_D + 1.6V_L$$

The above equation shows that both concrete and steel stirrups contribute to the shear capacity of the beam. Note that the depth of tension steel is the only common factor between the concrete and stirrups contributions to shear strength. If the depth of the tension steel is reduced because of a human error, the reduction in concrete and stirrups capacity will be the same. For this reason, all cross sections had the same reductions in the reliability index and nominal shear capacity when the reduction occurs in the depth of tension steel. The same reason explains why all cross sections had the same reductions in the reliability index and nominal shear capacity for all cases in which the cross sections differ only in the depth of tension steel.

For the case shown in Figure 73 in which the width of the beam is the controlling factor among the three considered cases, the cross section with the largest width is the most sensitive to reductions in the concrete compressive strength. On the contrary, cross sections with the smallest width are the most sensitive to the reductions in the reinforcement yield strength. The explanation to that is that for cross sections with the largest width, most of the shear capacity is coming from the concrete capacity, whereas most of the shear capacity is coming from the stirrups capacity for beams with the smallest width, provided that all other parameters are identical for the three cross sections. Therefore, when the error is committed in concrete compressive strength, the concrete capacity side of the equation will be only reduced, making cross sections with the largest width more susceptible to the reduction in the compressive strength of concrete. In contrast, when the error is committed in the reinforcement yield strength, the stirrups capacity side of the equation will be reduced, which will reduce the nominal shear capacity and the reliability index more for beams with the smallest width, since the stirrups side is contributing more to the shear capacity.

Using the same interpretation, one can deduce why beams with the smallest stirrups spacing, lowest concrete compressive strength, and higher stirrups are the most sensitive to the reductions in the area of stirrups, and are the least sensitive to the reductions in the width of the beam due to human errors.

As for why the three cross sections had similar sensitivity in the deterministic approach, but not in the reliability based approach when the live load fraction of the total load is the only difference among the three cross sections, the same interpretation made on the loading case for beams under flexure can be applied here.

As mentioned for beams under flexure, despite the nominal shear capacity equation can allow prediction of the behavior of the three cross sections when reductions in the design parameters take place, the difference in the reduction of nominal shear capacity and reliability index among the three cross sections cannot be obvious, unless sensitivity analysis is carried out.

7.5.3 Axially Loaded Columns

The conclusions made on the sensitivity analysis results can be verified using the column's nominal axial load capacity equation presented earlier in Chapter 5:

$$\emptyset P_n = 0.8[0.85f'_c(A_g - A_s) + A_s f_y] = 1.2P_D + 1.6P_L$$

The axial capacity of the column is coming from the concrete strength and the steel strength. Figure 76 is indicating that columns with higher reinforcement ratio are the least sensitive to the errors committed in concrete compressive strength, but are the most sensitive to the changes in the reinforcement yield strength. Since the three cross sections in these cases differ only in the reinforcement ratio, the steel side of the expression contributes more to the nominal capacity when the reinforcement ratio is high, whereas concrete side of the expression contributes more when the reinforcement ratio is low. Thus, when the error is committed in the reinforcement yield strength, the second side of the equation will be affected more, leading to a higher reduction in the cross sections with higher reinforcement ratio. On the opposite, the first side of the equation is affected when the reduction happens in concrete compressive strength. That is why the reduction in the nominal capacity and reliability index is lower for columns with lower reinforcement ratio when the error is committed in reinforcement yield strength. Similarly, the remaining scenarios and remaining cases can be explained using the same interpretation.

The reason why the three considered cross sections had the same reduction in the nominal capacity and reliability index when they differed only in the cross sectional area, is because the above equation can be rewritten after substituting in it $A_s = \rho A_g$. So, the nominal capacity equation will become:

$$\emptyset P_n = \emptyset 0.8[0.85f'_c(A_g - \rho A_g) + \rho A_g f_y] \quad (6.1)$$

Simplifying the equation will give:

$$\emptyset P_n = \emptyset 0.8A_g\left[0.85f'_c(1-\rho) + \rho f_y\right] \quad (6.2)$$

It is clear from the above that the cross sectional area is contributing to the both sides of the equation. Therefore, reducing the cross sectional area will reduce the nominal capacity of all cross sections by the same percentage.

Again, the interpretation of the inconsistency between deterministic and reliability based approaches for the loading cases is similar to what has been explained in beams under flexure and beams under shear.

CHAPTER 8

SENSITIVITY ANALYSIS FOR STRUCTURAL SYSTEMS

8.1 Introduction

In the previous chapter, the reduction in the nominal capacity and the reliability index due to changes in design variables (due to human errors) was examined at the local or element level. Yet, experience has shown that the structure works as a system, and local failures at one or two points within the system do not always result in structural collapse. Therefore, there is need to address the following questions:

- How will a change in any design variable affect the structure as a whole?
- Is the reduction related to the number of bays and stories within a given portal frame?
- Is the effect on the element as critical (or mild) as the effect on the system?

In order to answer to these questions, a static pushover analysis was applied on a large number of two-dimensional portal frames with different geometries, material properties and reinforcement characteristics using the ZUES-NL software so as to determine the maximum load at which the structure will exceed the maximum allowable lateral drift. The software ZEUS-NL is an analysis and simulation program that is freely available from the Mid-America Earthquake Center. The software was written in the FORTRAN language and is a state-of-the-art platform with static and dynamic analysis capabilities that is developed for structural engineering applications. It is efficient, validated, and user-friendly software that employs inelastic large displacement analysis of complex reinforced concrete frame structures using the fiber approach. It includes a collection of practical material models and elements to be used in the analysis [43].

Static pushover is “a static nonlinear analysis under permanent vertical loads and gradually increasing lateral loads”. When applying static pushover analysis on ZEUS-NL, “the applied lateral load (P) is kept proportional to the pattern of the defined nominal loads (P^0). The load factor (λ) is automatically increased by ZEUS-

NL until the frame reaches the maximum allowable drift" [44, 45]. This can be represented by the following equation:

$$P = \lambda . P^0 \tag{8.1}$$

From the above equation, computing the load factor (λ) will allow determining the load at which the structure will reach the maximum allowable drift, which can be thought of as the lateral load capacity of the frame structure. Based on this, the effect of reducing any of the design variables due to human errors on the nominal capacity of the structure as a whole can be investigated by studying the variation in the load factor (λ).

8.2 Analysis and Results

The static pushover analysis was applied on portal reinforced concrete frames with different number of stories and different number of bays to examine how the reduction in the design variables will affect the strength and stiffness of the considered frames. In the analysis, the load-controlled option has been adopted in the study, as opposed to the displacement-controlled option. Different results could have been obtained if the latter option was utilized. The frames that were investigated in the study are:

- 1 Story with 1 Bay Frame (1S1B)
- 1 Story with 3 Bays Frame (1S3B)
- 1 Story with 5 Bays Frame (1S5B)
- 3 Stories with 1 Bay Frame (3S1B)
- 3 Stories with 3 Bays Frame (3S3B)
- 3 Stories with 5 Bays Frame (3S5B)
- 5 Stories with 1 Bay Frame (5S1B)
- 5 Stories with 3 Bays Frame (5S3B)
- 5 Stories with 5 Bays Frame (5S5B)

Geometry of the above considered frame structures with their designation are shown in Figure 95.

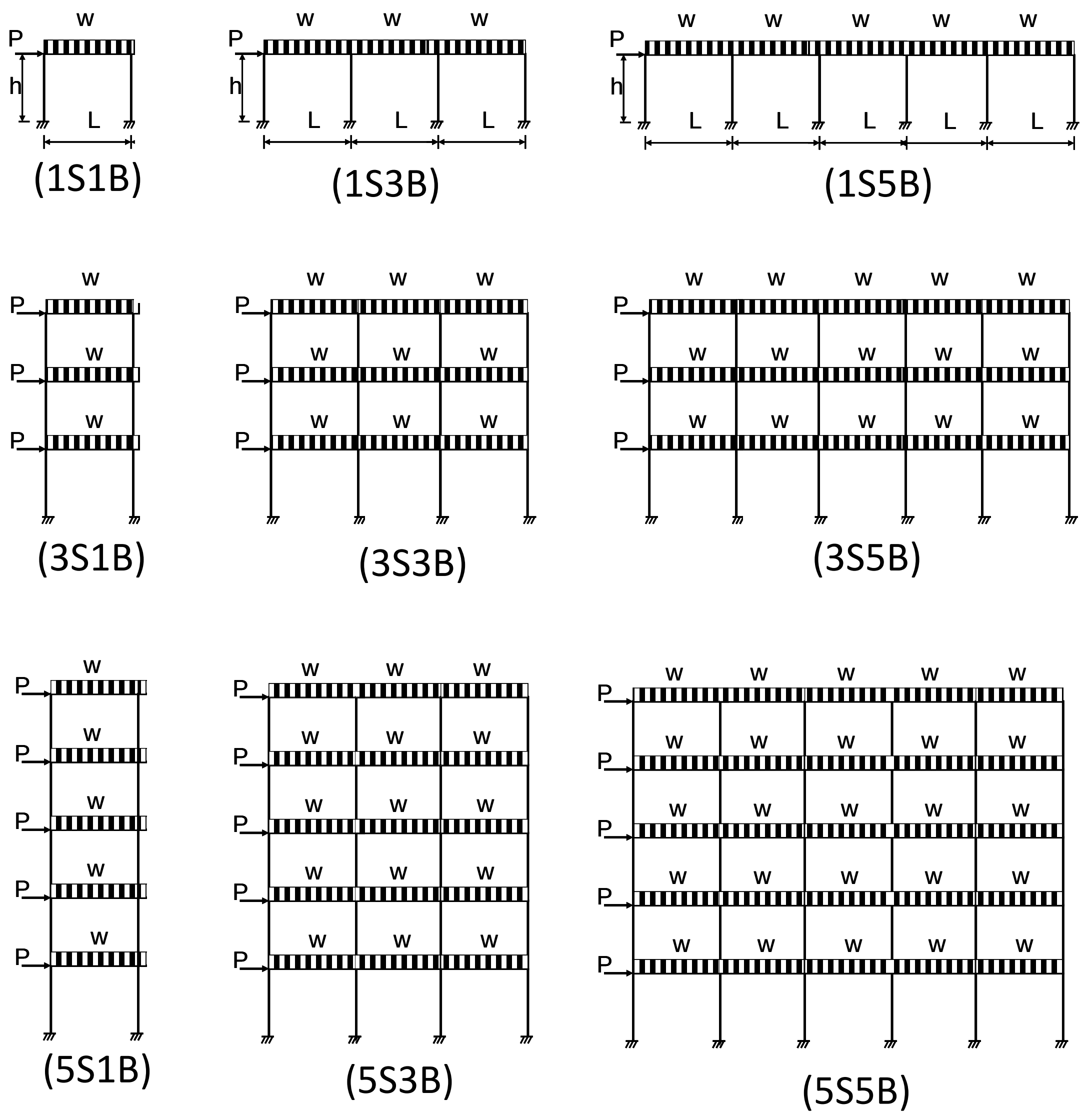

Figure 95: Frames considered in the pushover analysis

For simplicity, all frames were assumed to be fixed at their supports and constructed without shear-walls. Also, all frames were assumed to have the same story height, bay length, column size and reinforcement, beam size and reinforcement, as well as the same uniformly distributed gravity load along the horizontal members.

When the static pushover analysis was applied on the 1S1B Frame, assuming that the frame is intact (or error free), the load factor corresponding to a maximum allowable lateral displacement equal to 0.0025 times the building height was found to be 14.6, as shown in Figure 96.

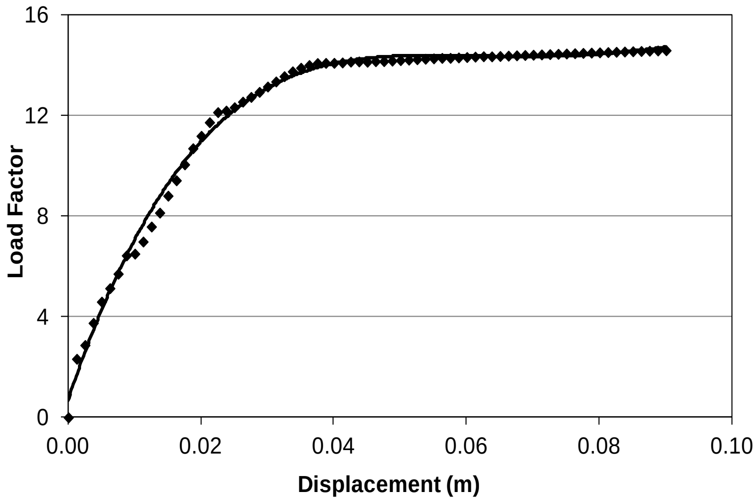

Figure 96: Static pushover analysis for 1S1B Frame

In order to verify the load factor, the static pushover analysis was applied to the same frame, but with stronger members (by enlarging the elements sizes and increasing the steel reinforcement). The load factor for the stronger frame was found to be larger than that of frame made with weaker members, as shown in Figure 97. This confirms the validity of the results.

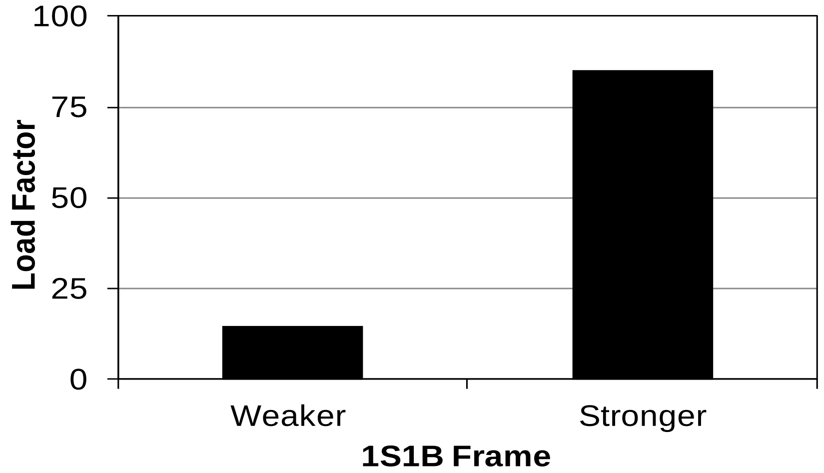

Figure 97: Comparison between small and large frames with respect to load factor

The change in the load factor from the pushover analysis due to reductions in some of the design parameters was investigated next for all the considered cases. Figure 98 shows the reduction in the load factor when the yield strength of the columns' reinforcement was reduced at different percentages. For the sake of simplicity, the reduction in steel yield strength was assumed to be constant in all columns at the same percentage. This is equivalent to using defective steel material in all the vertical members.

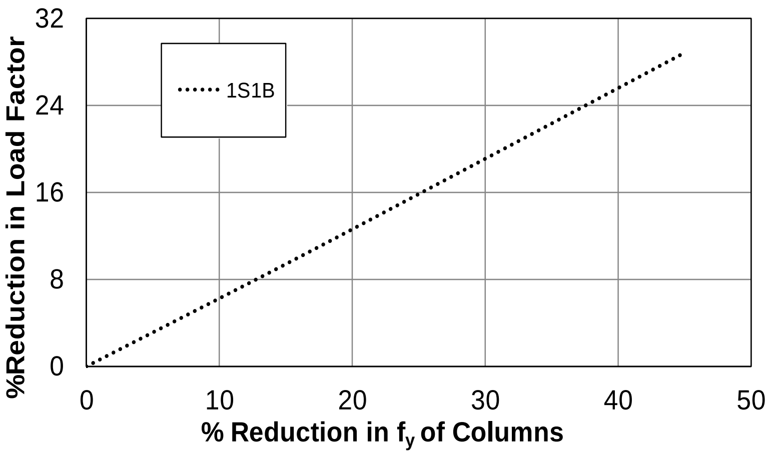

Figure 98: Effect of reinforcement yield strength on the load factor of 1S1B frames

When applying the same procedure to other design variables, the results are as presented in Figure 99. The considered design variables in the vertical columns were the cross-section width, cross-section depth, gross cross-sectional area, area of longitudinal steel reinforcement, steel yield strength and concrete compressive strength. Similarly, the considered design variables in the horizontal beams were the cross-section width, cross-section depth, gross cross-sectional area, area of longitudinal steel reinforcement, steel yield strength and concrete compressive strength.

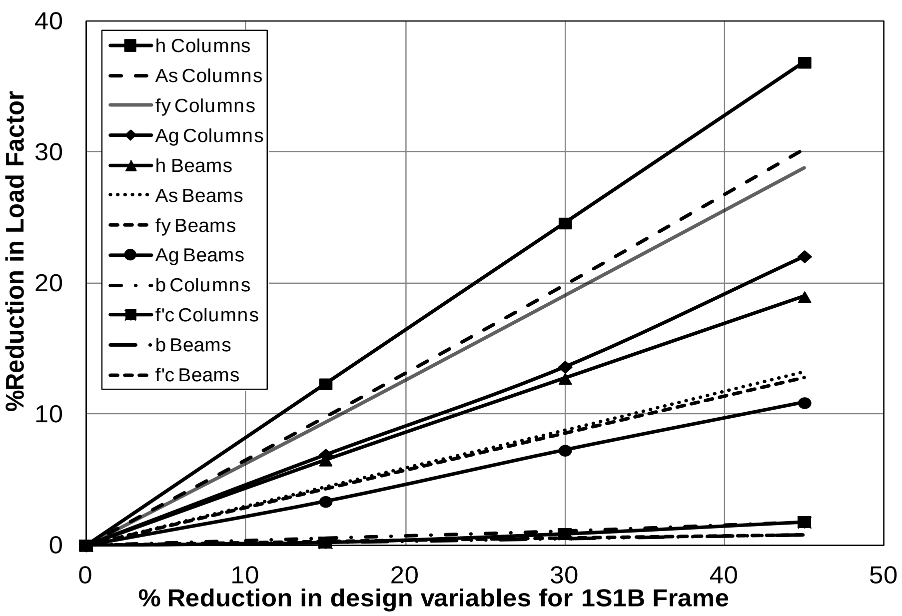

Figure 99: Effect of changing design variables on the load factor of 1S1B frames

Figure 99 indicates that the structure as a whole is very sensitive to changes in the thickness of columns (which resists bending), while it is least sensitive to the changes in concrete compressive strength of the beams. Notice that the reduction in the load factor for the structure as a whole due to the changes in other columns' parameters, except for concrete compressive strength and widths, is considered to be between moderate and high. On the contrary, the reduction in the load factor for the structure as a whole due to the changes in beams' parameters can be classified between low and moderate. This is an indication that columns are contributing more to the strength of 1S1B frames.

The next step was to check how the reduction in the load factor will differ among frames with different number of bays and number of stories. Static pushover analysis was applied on the 9 different reinforced concrete portal frames shown in Figure 95 for the case of reducing the yield strength of the reinforcement provided in columns. The sensitivity analysis results are presented in Figures 100 and 101.

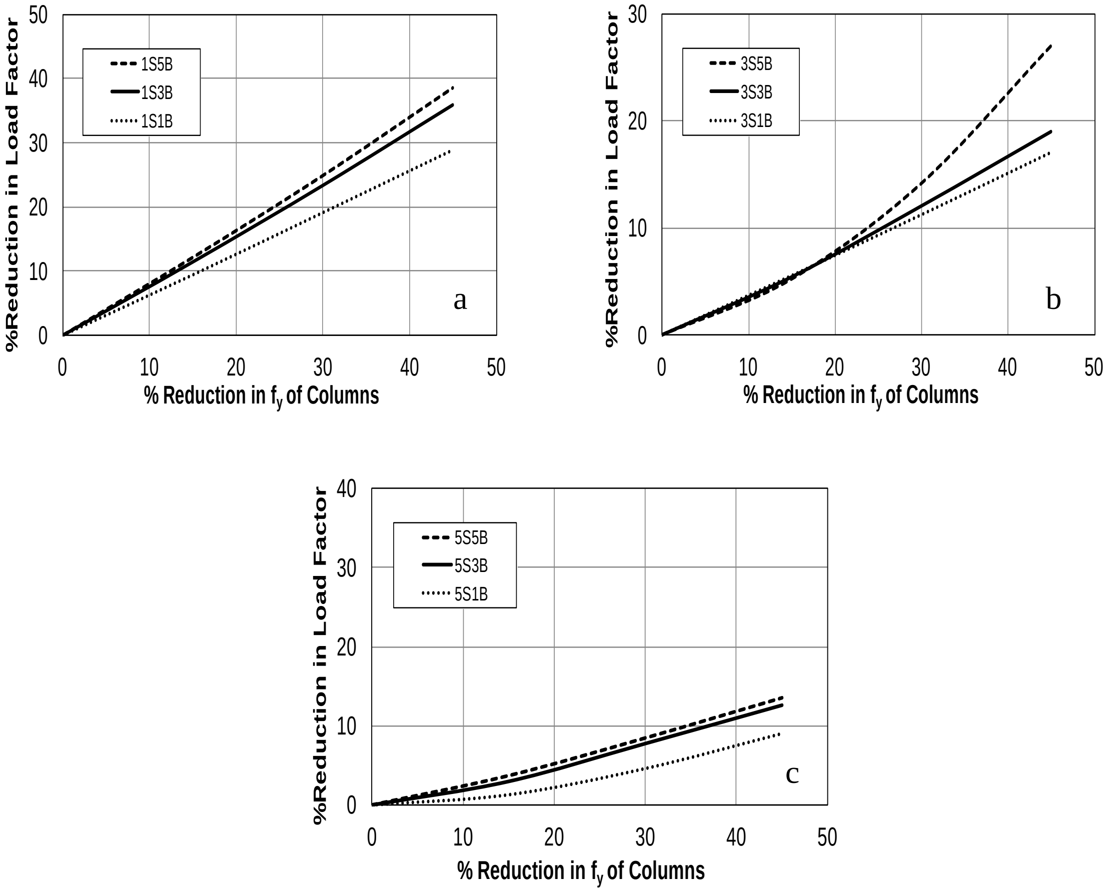

Figure 100: Effect of yield strength of reinforcement on the load factor for different bays

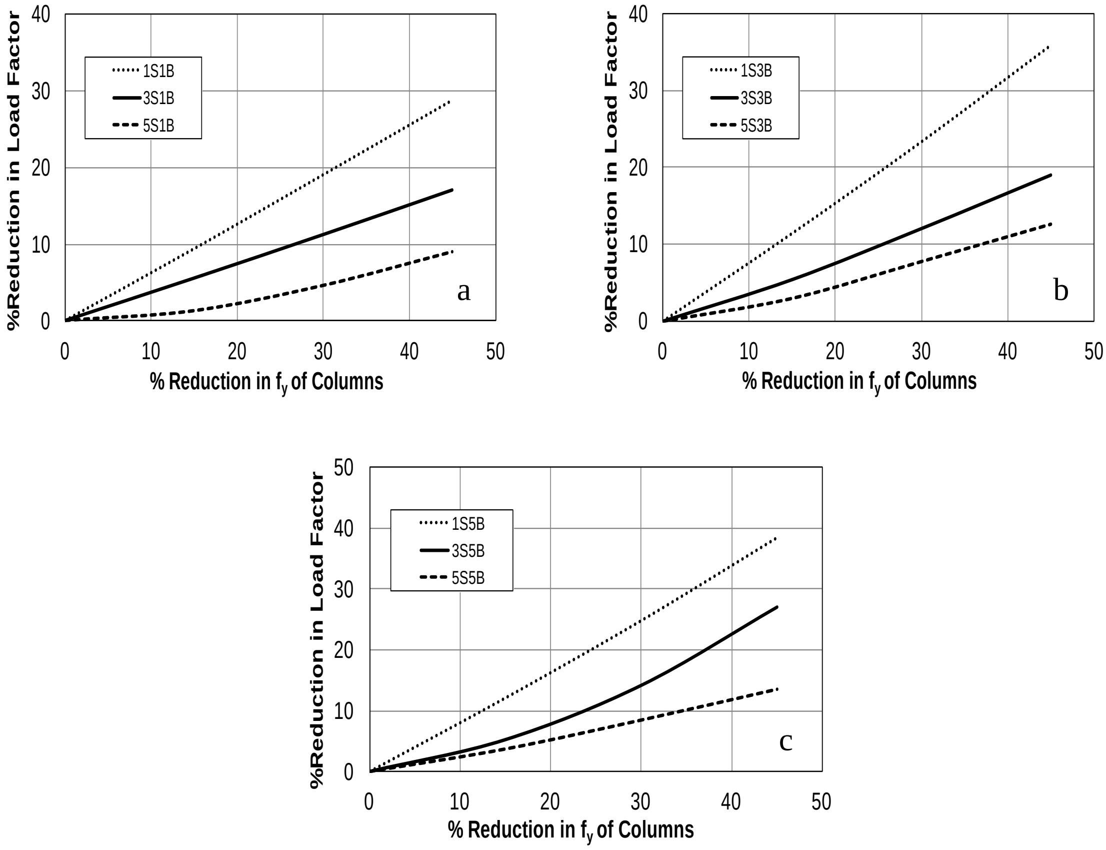

Figure 101: Effect of reinforcement yield strength on load factor for different stories

Figure 100.a shows a comparison between three frames with respect to the reduction in the load factor due to various reductions in the yield strength of reinforcement provided in columns. All three frames have one story, but different number of bays. The comparison is indicating that the reduction in the load factor increases as the number of bays increases. In Figure 100.b, the same comparison was made, except that the number of stories was fixed to be three for all three frames, instead of one story. The comparison is again showing that the reduction in the load factor increases with the increase in the number of bays. When fixing the number of stories to five, as illustrated in Figure 100.c, the comparison has also led to the same conclusion.

The opposite scenario is noticed in Figures 101.a, 101.b, and 102.c, as all frames are having the same number of bays, but different number of stories. The comparison in Figures 101.a, 101.b, and 101.c shows that the reduction in yield strength of columns' reinforcement has more impact on frames with small number of stories than on frames with large number of stories, provided that the number of bays is kept the same.

It is worth mentioning here that the effect of committing an error in the yield strength of columns' reinforcement on reducing the load factor varies significantly if the difference among frames is in the number of stories, as illustrated in Figures 101.a, 101.b, and 101.c. On the other hand, the effect is very minor when the frames differ only in the number of bays, as summarized in Figures 100.a, 100.b, and 100.c.

Since all comparisons shown in Figure 100 have brought up the same conclusion, and since the same applies for all comparisons shown in Figure 101, the study of committing errors in the remaining design parameters was carried out using two scenarios. In the first scenario, the number of bays was fixed to one for all three frames, whereas the three frames had only one story in the second scenario. The conclusion in both scenarios was generalized for different number of bays and stories respectively.

Figures 102-107 represent the effect of reducing some design variables on the load factor at which the allowable drift for the 9 above listed concrete portal frames is attained. The considered design variables were the cross-section width of columns and beams, cross-section depth of columns and beams, gross cross-sectional area of columns and beams, area of longitudinal steel reinforcement of columns and beams, steel yield strength of columns and beams, and concrete compressive strength of columns and beams.

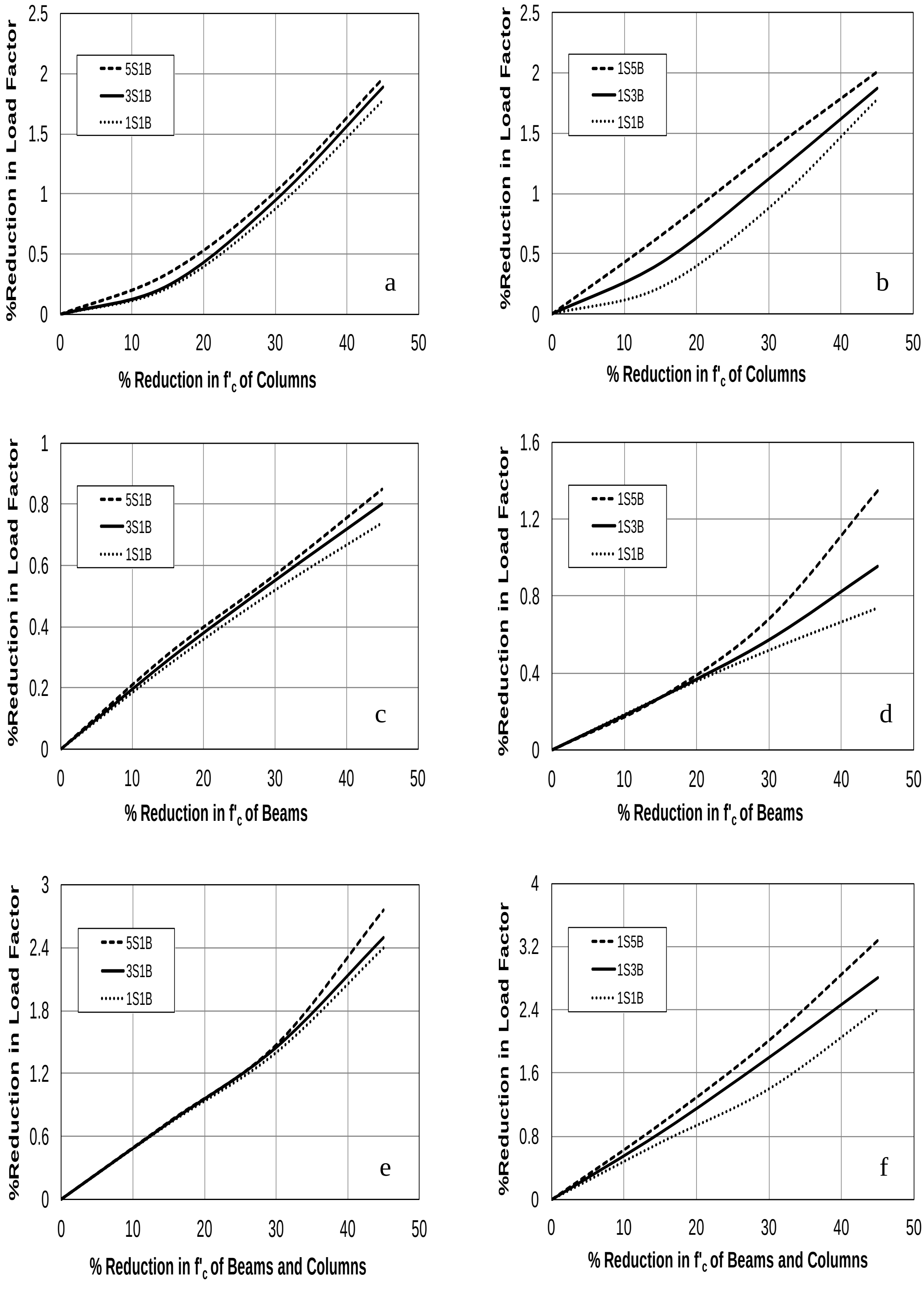

Figure 102: Effect of changing the concrete compressive strength on the load factor

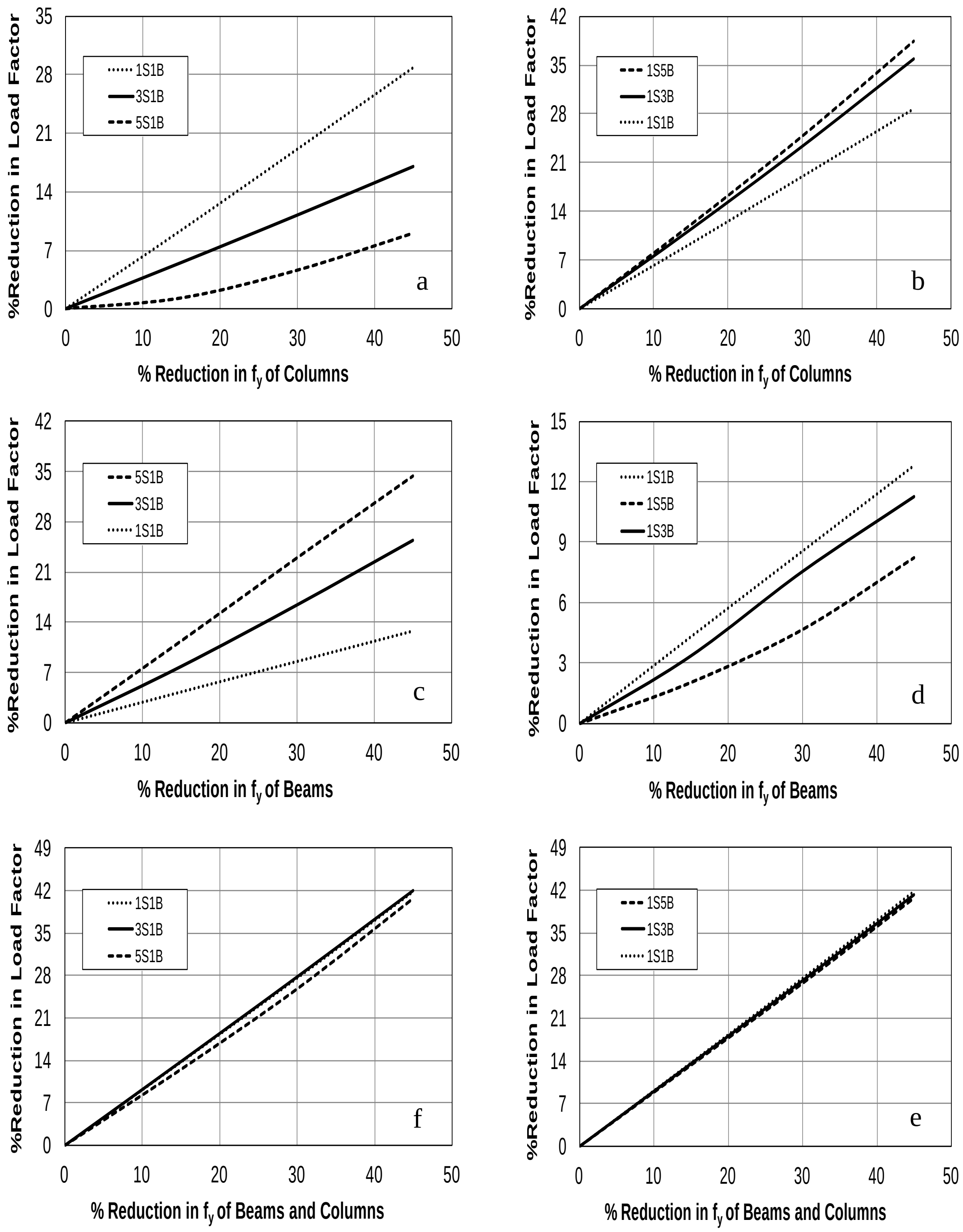

Figure 103: Effect of changing the yield strength of reinforcement on the load factor

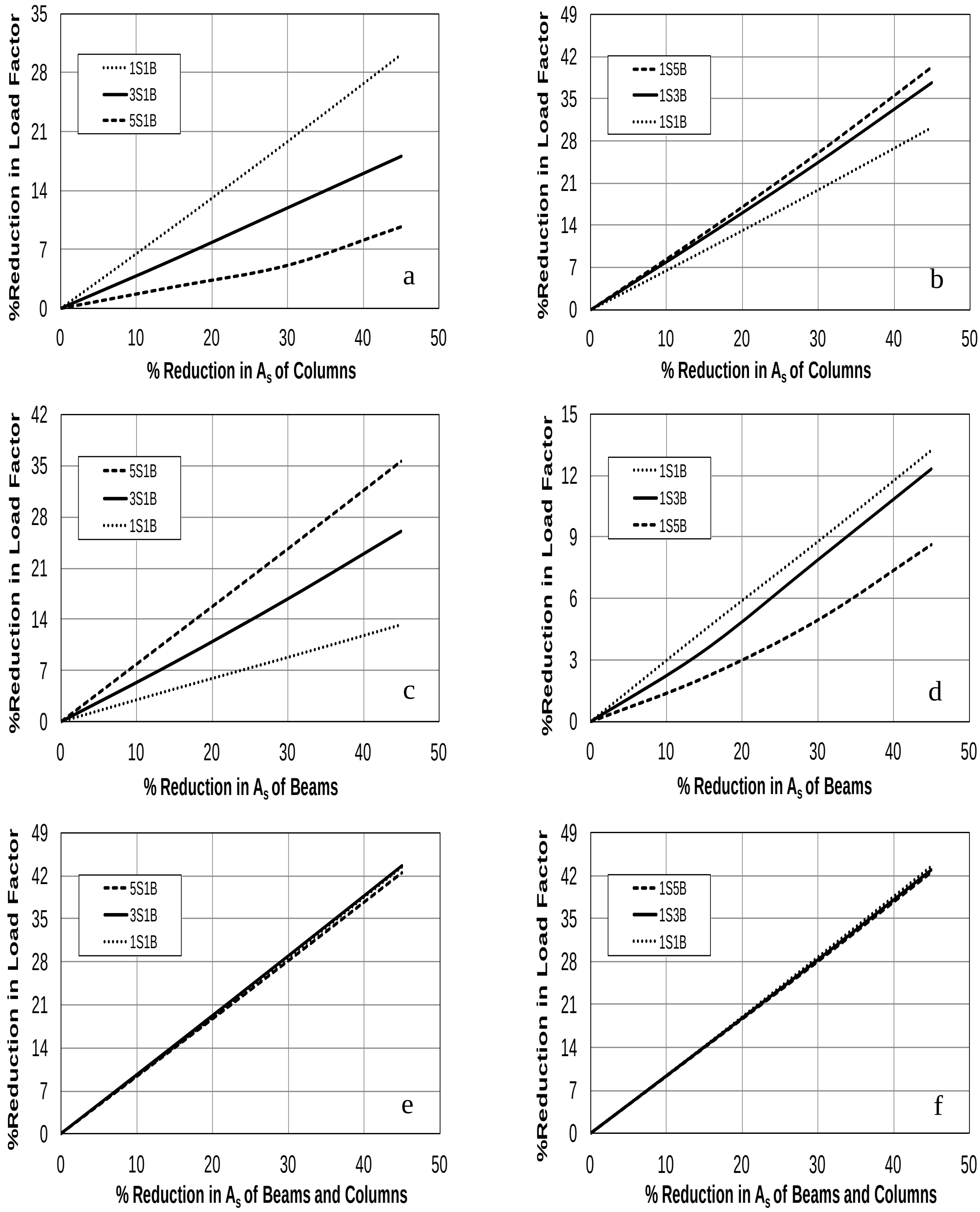

Figure 104: Effect of changing the area of steel reinforcement on the load factor

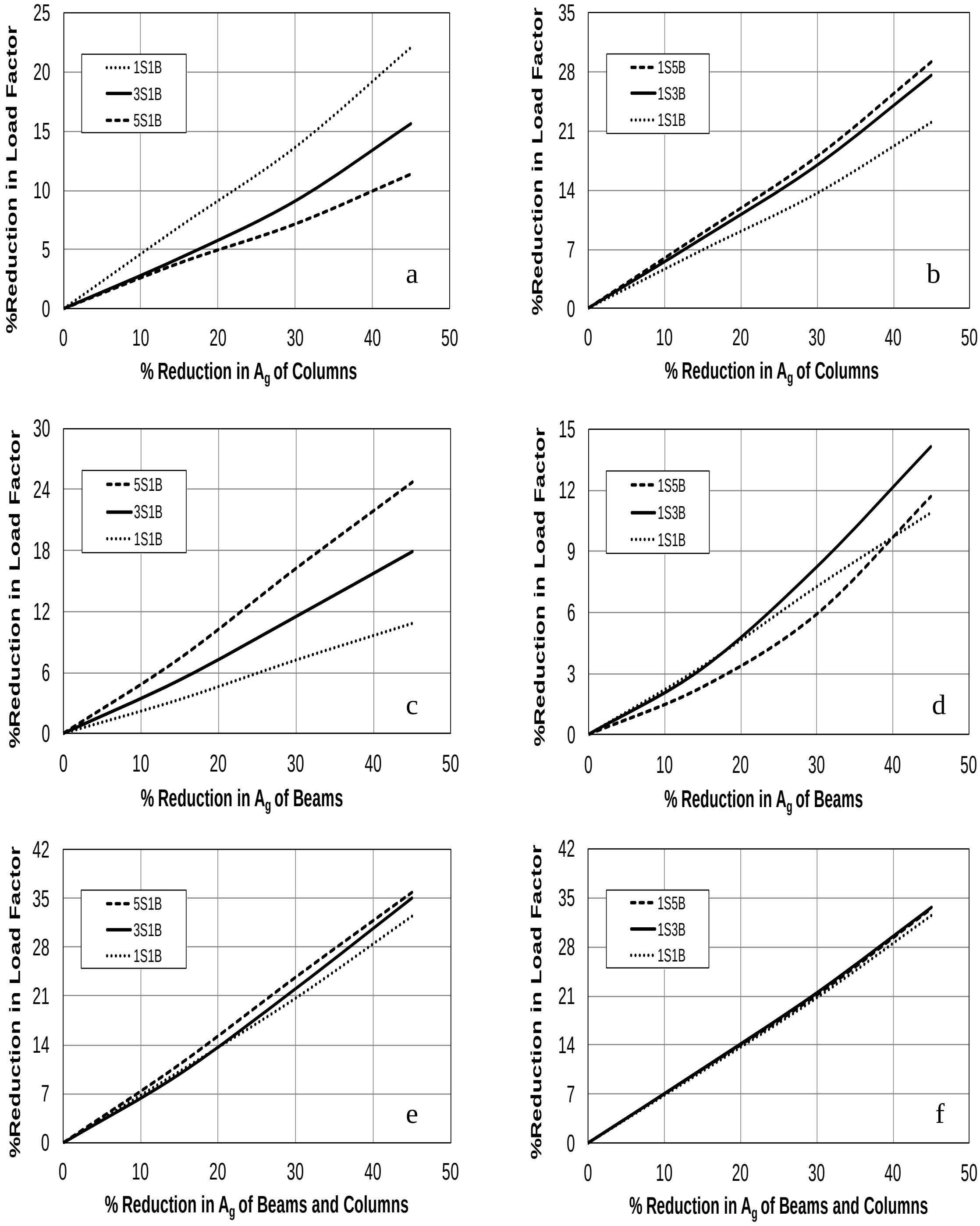

Figure 105: Effect of changing the gross sectional area of members on the load factor

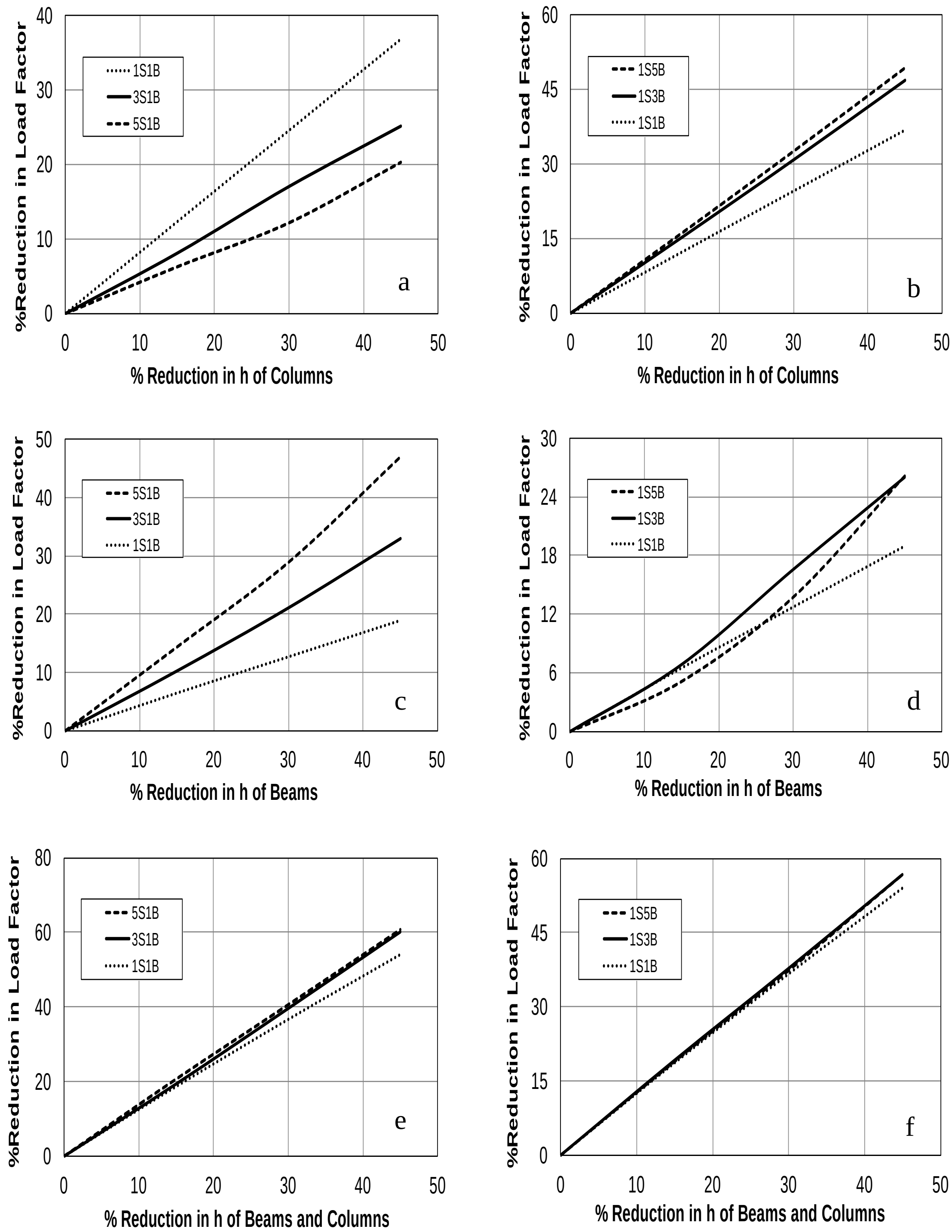

Figure 106: Effect of changing the thickness of members on the load factor

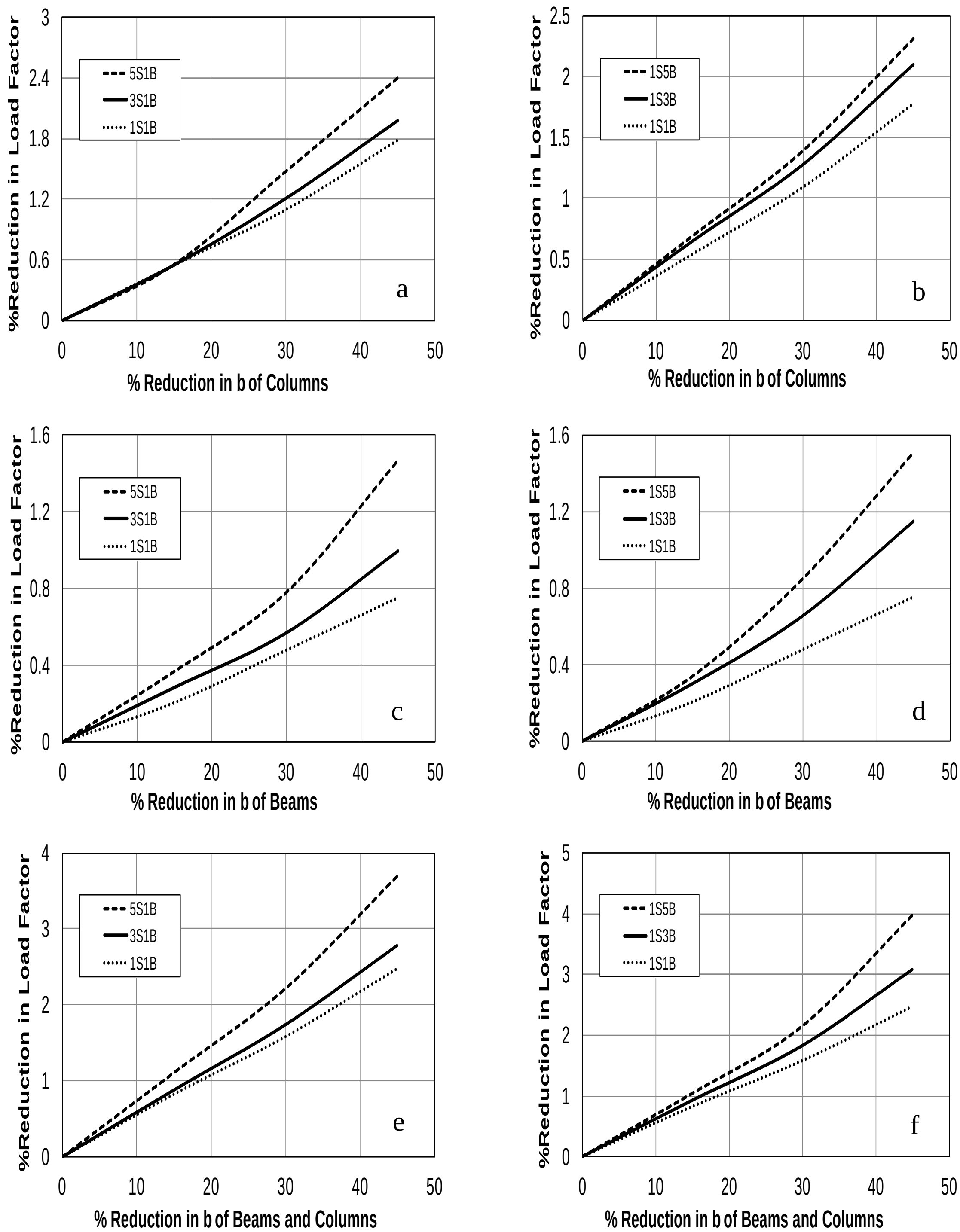

Figure 107: Effect of changing the width of members on the load factor

8.3 Discussion of Results

As shown in Figures 102-107, the reduction in the load factor varies from one frame to another, depending on the number of stories and number of bays the frame has. In Figure 102, where the error was assumed to occur in the concrete compressive strength, increasing the number of stories reduces the load factor for the frames with three bays by almost the same percentage. The same conclusion applies when the number of bays increases. Note that this conclusion is valid when the error is committed in either the concrete compressive strength of columns or the beams, or both. Note also that the maximum reduction in the load factor did not exceed 3%, which means that the reduction in the load factor for the structure as a whole due to the reduction in concrete compressive strength is always small no matter in which structural element the error is committed, and regardless of the number of stories and number of bays the frame is composed of.

The comparisons shown Figures 103 and 104 have given the same outcome. In graphs a, and b, reinforced concrete frames with large number of stories had lower reduction in the load factor when the reduction happened in the yield strength of columns reinforcement, or in the area of column reinforcement, when compared with corresponding frames having small number of stories. On the other hand, frames with large number of bays had larger reduction in the load factor for the same reductions. Still, it is important to note here that the reductions in the load factor for this case increased significantly as the number of less decreased. However, the reduction in the load factor did not vary substantially with the increase in the number of bays.

The opposite happened in graphs c and d of Figures 103, and 104, in which the error was committed in the yield strength of beams reinforcement, and area of beams reinforcement, respectively. The increase in the number of stories had a significant effect on reducing the load factor. Nevertheless, the increase in the number of bays had a slight effect on the reduction in the load factor.

When the error was assumed to be committed simultaneously in the yield strength of columns and beams reinforcement, or in the area of columns and beams reinforcement, the reduction in the load factor was constant for all frames, as shown in graphs e and f of Figures 103, and 104.

Moving to Figure 105 and 106, which show how the reduction in the load factor differs from one frame to another when the error is committed in the gross area, or in the thickness of the section. The conclusion reached here was somewhat similar to that of Figures 103 and 104. The only difference is that the pattern of sensitivity functions for beam gross area or thickness was not consistent as observed for other design parameters, as illustrated in Figures 103.d, and 104.d.

In order to verify this outcome, the same analysis was run on the considered frames, but with higher reinforcement ratio (i.e. stronger frames). The results for such cases are shown in Figure 108.

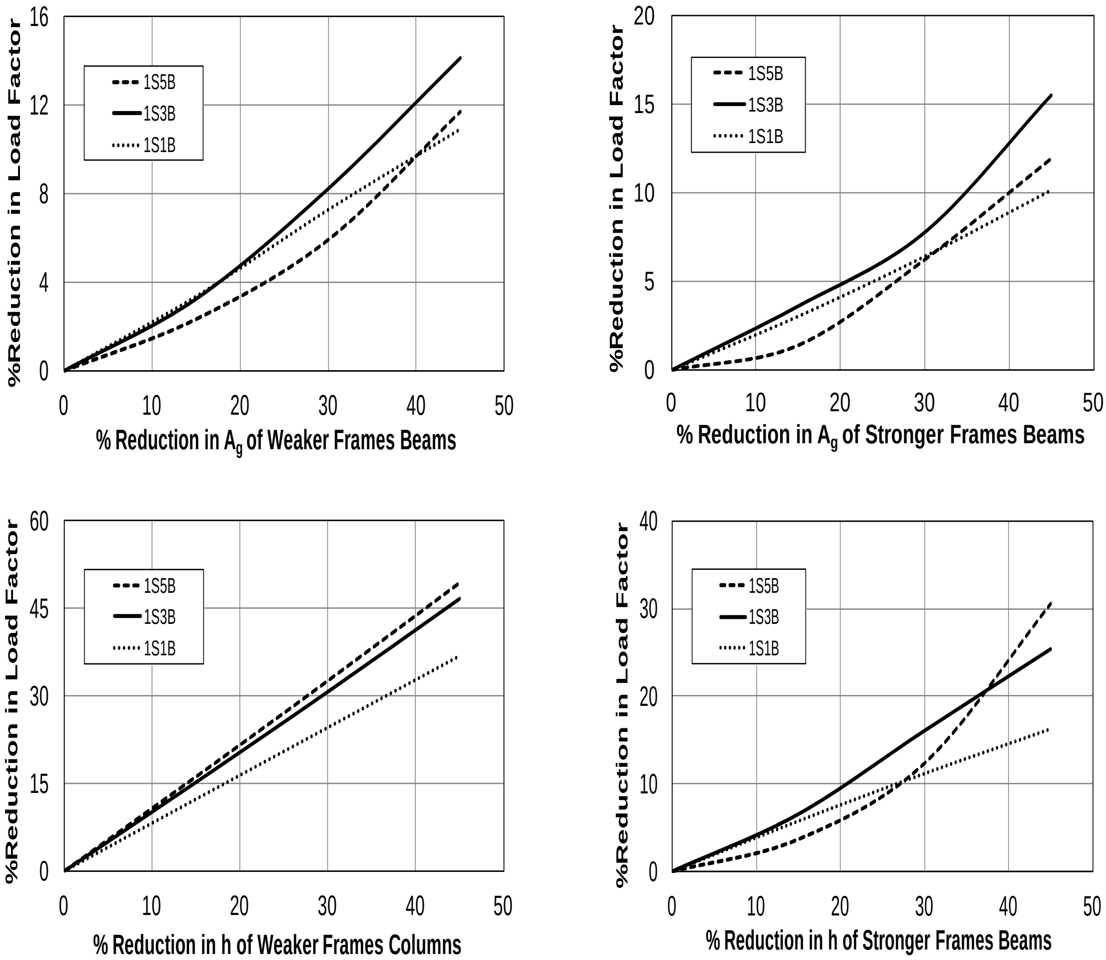

Figure 108: Effect of changing the thickness of members on the load factor

Figure 108 shows that increasing the size and reinforcement of the frame members also resulted in inconsistent sensitivity ordering in relation to the number of stories and bays.

Figure 107 is related to errors committed in the width of the members of the considered frames. The results show that reductions in the width of beams, width of columns, or both have minimal effect on the load factor, similar to the effect of concrete compressive strength case, presented in Figure 102.

Now, there is a need to determine whether the results obtained from Figures 102-107 are applicable for frames other than the ones considered in the Figures (i.e. with different number of bays and different number of stories). For example, the results of the sensitivity of 5S5B frames, with regard to reduction in yield strength of columns reinforcement, need to be compared with results for 1S1B frames.

Figures 103.a and 103.b show that the reduction in the load factor decreases significantly with the increase in the number of stories, whereas the reduction in the load factor slightly increases with the increase in the number of bays. So, when comparing 1S1B frames with 5S5B frames, the reduction in the load factor for the 1S is larger than that of 5S. On the contrary, the reduction in the load factor for 5B is slightly larger than that of 1B. Thus, the number of stories contributes more to the difference in the load factor between the two frames than the number of bays. Therefore, the reduction in the load factor for the 1S1B frames should be larger than that of 5S5B. In other words, 5S5B are less sensitive to errors committed in the yield strength of columns reinforcement than 1S1B frames.

To verify this outcome, static pushover analysis was applied to 1S1B and 5S5B frames, and the results are summarized in Figure 109 below.

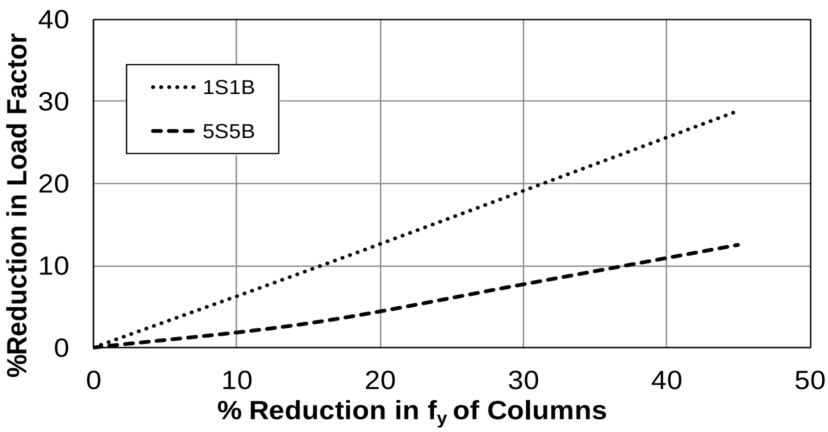

Figure 109: Comparison of 1S1B with 5S5B frames for steel yield strength of columns

Note that the findings of Figure 109 above, which shows that small frames are more sensitive to changes in yield strength than large frames, match well with the prediction reached earlier.

Graphs 104.c and 104.d have indicated that the reduction in the load factor increases as the number of stories increases for changes in the area of beams reinforcement, whereas it decreases as the number of bays increases. The reduction for the story case is significant, while it is very small for the bay case. When combining the two outcomes together, the reduction in the load factor due to errors committed in the area of beams reinforcement will be larger in 5S5B frames compared to that in 1S1B. When using the static pushover analysis to verify this, the outcome was exactly the same, as highlighted in the Figure 110.

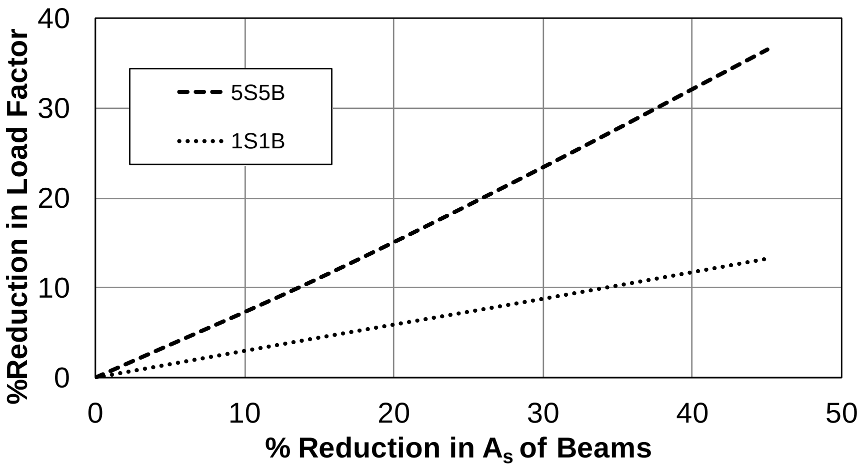

Figure 110: Comparison of 1S1B with 5S5B frames for the area of beams' steel

Based on this, the results obtained from the static pushover analysis can be used to compare frames with different number of stories and different number of bays, as a function of the reduction in the load factor due to human errors.

Figure 99 has indicated that load factor is more sensitive to changes in columns' parameters than changes in beams' parameters for 1S1B frames. Yet, how will this differ for other frames such as 5S1B frames, and 1S5B frames? This issue is highlighted in Figures 111 and 112.

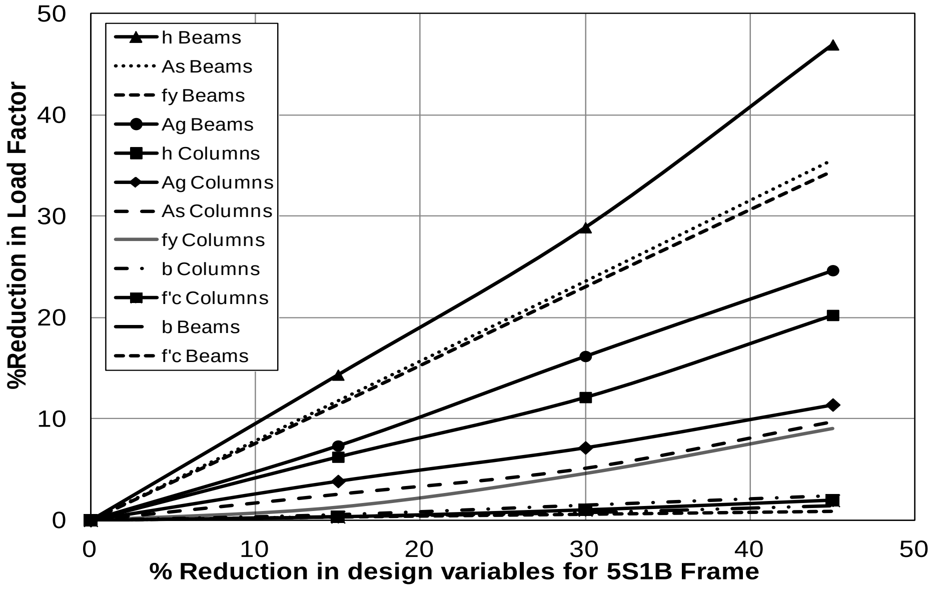

Figure 111: Effect of changing design variables on the load factor of 5S1B frames

For frames with large number of stories, the beams' design parameters govern over the columns' design parameters.

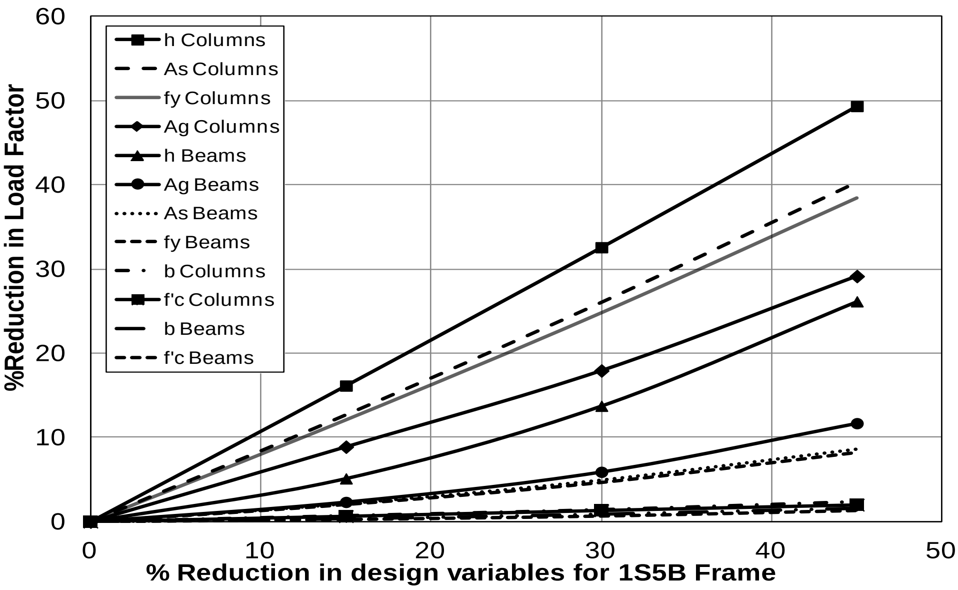

Figure 112: Effect of changing design variables on the load factor of 1S5B frames

On the other hand, for frames with large number of bays, the columns' design parameters govern over the beams' design parameters

8.4 Final Remarks

In conclusion, static pushover analysis was applied on frames with different number of bays and different number of stories to examine the reduction in the load factor of the structure as whole due to human errors committed in different design variables.

Static pushover analysis has shown that design variables have different effects on reducing the load factor of the structure as a whole when human error is committed in any of them. The analysis also has shown that the number of stories and bays in the frame affects the reduction in the load factor. The study has pointed out that as the number of stories increases, the reduction in the load factor due to errors committed in beams' parameters is larger than that of columns' parameters. Other than this, the reduction in the load factor because of errors committed in columns' parameters is larger.

Additionally, it was concluded that the reduction in the load factor when errors occur in beams width, columns width, compressive strength of beams concrete, or compressive strength of columns concrete is constant regardless of the number of bays, and number of stories. For other design parameters such as the area of steel, yield strength of reinforcement, gross area, and thickness, the reduction in the load factor differs, depending on the number of stories, number of bays, and whether the error is committed in beams, columns, or both.

Although static pushover analysis was performed on frames either with different number of stories, or different number of bays, it was verified that the results obtained can be used to make comparison between frames with different number of stories, and different number of bays at the same time.

CHAPTER 9

DESIGN AND CONSTRUCTION CHECKLISTS

9.1 Introduction

As seen in previous chapters, human errors in structural design and construction have resulted in structural failure and construction collapses in the UAE in the last decade. The filled in surveys have pointed out also that human errors are not committed in one design or construction activity, but rather in many activities. Furthermore, it was illustrated in the locally and globally sensitivity analysis that reducing some design parameters will reduce the capacity of structural elements and the structure substantially. For this reason, the purpose of this chapter is to develop several design and construction checklists that will help reduce the possibility of committing human errors in design and construction activities as much as possible. Still, it is important to understand that these checklists will not eliminate the occurrence of human errors during design and construction activities.

9.2 Structural Design Checklists

After consulting with engineering firms, the important design checklists that can be helpful in imposing quality control on design work are listed below:

- ETABS modeling and data entry
- Design of one-way slabs
- Design of two-way slabs
- Design of beams
- Design of columns
- Design of basement walls
- Design of raft
- Design of pile caps
- Design of Piles

ETABS is an approved software for the structural analysis and design of buildings that is commonly used in the UAE and around the world. The main checklist items that are related to computer modeling and data entry of the software include building plan, grid information, story data definition, units, design code, structural details, material properties, cross-sectional dimensions, gravity and lateral loads, errors, and interpretation of results.

The checklist of one-way and two-way slabs design includes items related to slab dimensions, thickness, unit consistency and fixed end conditions. In addition, it includes checklist items for moment used in the design, reinforcement provided in both directions, spacing provided, and transferring of results to structural drawings. The check for deflection is also included.

For the design of beams, the major checklist items involve the design against flexure, shear, and torsion. The checklist items include also material properties, concrete cover, and extraction of moment, shear and torsion from the model is also included. Besides, beams geometry and fixed end conditions are part of the checklist. Finally, RC schedule and detailing are also considered in the checklist.

Regarding the checklist of columns design, material properties, columns geometry, fixed end conditions, different loading scenarios have been taken into considerations. In addition, percentage of longitudinal steel provided, as well the size of stirrups are considered. Finally, RC schedule and detailing are included.

The checklist of basement wall design involves the design of the wall horizontally and vertically for the inner and outer faces. Material properties and wall geometry are included. The design of the basement wall is mainly against flexure, axial load and crack width control. RC schedule and detailing are shown in the checklist as well.

The last checklist section is related to foundation design. Checklists for the design of shallow (raft) and deep foundations (piles and pile caps) are provided. The checklists cover structural design (against bending and shear) and serviceability design (settlement and bearing capacity). RC schedule and detailing are also included.

Figure 113 is a sample example of the developed design checklists. This checklist is related to the design of one-way slabs. Checklists for all the above issues are included in Appendix B.

2. One-way Slabs **2.1. Design of one-way slabs**		**Slab Designation**			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Input Data**				
1.1	Slab has a designation, e.g. GS002				
1.2	Correct design code is used				
1.3	Compressive strength is as per the approved civil design basis				
1.4	Yield strength of steel is as per the approved design criteria				
1.5	Width of the panel considered is matching with drawings				
1.6	Length of the panel considered is matching with drawings				
1.7	Top concrete cover is as per the approved civil design basis				
1.8	Bottom concrete cover is as per the approved design criteria				
2	**Initial Calculations**				
2.1	Clear spans are correctly calculated				
2.2	Ratio of long span to short span is more than 2				
2.3	Minimum thickness of the slab is based on the correct table				
2.4	End conditions have been determined properly				
3	**Design**				
3.1	Unit consistency has been checked				
3.2	Ultimate shear is extracted correctly from the design software				
3.3	Ultimate moment is extracted correctly from the design software				
3.4	Slab thickness is adequate for shear				
3.5	Top and bottom reinforcement provided in the x direction is more than the minimum reinforcement limit				
3.6	Top and bottom reinforcement provided in the y direction is more than the minimum reinforcement limit				
3.7	Spacing between top steel in the x direction is less than or equal to the spacing required				
3.8	Spacing between bottom steel in the x direction is less than or equal to the spacing required				
3.9	Spacing between top steel in the y direction is less than or equal to the spacing required				
3.10	Spacing between bottom steel in the y direction is less than or equal to the spacing required				
3.11	Shrinkage and temperature reinforcement provided is more than the minimum limit				
3.12	Deflection in the slab is not exceeding the allowable limit				

Figure 113: Checklist for the design of one-way slabs

9.3 Construction Checklists

The major construction checklists that can be helpful in imposing quality control on design work are listed below:

- Soil Investigation work
- Surveying work
- Secant piling work
- Capping beam work
- Piling work
- Excavation work
- Backfilling work
- Blinding work
- Formwork and shuttering
- Rebar work
- Screeding work
- RC concreting work

Soil investigation is one of the most critical activities of any project because it defines the capacity of the soil on which the structure is constructed. The checklist for the soil investigation work includes the setting out work such as the demarcation of borehole locations as per the approved drawings, and trial pit excavation to locate existing services. Also, procedures to install piezometer and monitoring water table are included. Checklists for Standard Penetration Test (SPT), Plate Load Test, and Soil Permeability Test are provided. Procedures to store samples and monitoring cavities are included in the checklist as well.

The checklist for secant piling includes the procedures for setting out the levels and control points, dewatering, driving of temporary casing for secondary and primary piles, preparing of reinforcement cage, checking and casting of concrete, removing of temporary casing.

Checklist is also provided for the construction of capping beam that connects secant piles together. The major items included are related to formwork preparation, checking of steel and concrete properties, sequence of concreting and removal of formwork.

For the checklist of piling work, the levels and control points setting out procedures, dewatering, driving of temporary and permanent casings, preparation of steel cage and concrete casting sequence, removal of temporary casing are all included. The checklist contains also the pile head cutting and pile head treatment processes. Preliminary pile test, working pile test and integrity test are provided in the checklist as well.

Moreover, checklist is provided for excavation and backfilling work. For excavation checklist, it includes work prior to excavation (locating existing services, setting out levels) and work after excavation (dewatering, slope of excavation, and shoring).

Finally, checklists are developed for concreting and formwork work of blinding concrete, screed, raft, slabs, beams, columns and walls. The major items included are related to formwork preparation, rebar work, testing of concrete, testing of reinforcement, sequence of concrete casting, and removal of formwork.

Figure 114 is a sample of the developed construction checklists. This checklist is related to construction work on reinforcement steel cage in a bored pile. All checklists related to construction activities are enclosed in Appendix C.

5. Piling Work **5.1 Reinforcement Cage**		**Pile Designation**			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Checking of the Reinforcement Cage**				
1.1	Yield Strength of reinforcement is as per the approved design				
1.2	Diameter of steel bars in the reinforcement cage is as per the approved design				
1.3	Number of steel bars in the reinforcement cage is as per the approved design				
1.4	Diameter of spirals is as per the approved design				
1.5	Mill Certificate has been checked				
2	**Preparation of the Reinforcement Cage**				
2.1	Reinforcement cage has been made at the location nearest possible to the pile shaft				
2.2	The length of the reinforcement cage has been checked				
2.3	Spirals have been spaced as per the approved design				
2.4	Spacers have been provided with assured capacity of holding the steel cage in position				
2.5	More spacers have been provided at cut off level to ensure the centralization				
2.6	PVC pipes as per the approved length have been provided to protect the reinforcement extending above the pile cut off level				
2.7	PVC pipes have been stopped 50mm above the cut off level to avoid inclusions into concrete				
2.8	Earthing steel to be routed through the pile has been separated from pile reinforcement				
3	**Protection of Reinforcement Cage**				
3.1	Reinforcement Cage has been stored above the ground level				
3.2	Reinforcement Cage has been covered properly with polythene sheets				
4	**Installation of Reinforcement Cage**				
4.1	The reinforcement cage has been placed immediately after boring				
4.2	The reinforcement cage has been raised vertically by a crane from its horizontal laying position				

Figure 114: Checklist for reinforcement steel cage in a bored pile

CHAPTER 10

SUMMARY AND CONCLUSION

10.1 Summary

The UAE economy has grown a lot in the past few years due to efforts by the UAE government to diversify its economy from oil-based to other industries. This has resulted in significant developments in the construction sector in all emirates. Although there has been slow demand for real estate in the past couple of years due to the global financial crises, the UAE is still the biggest construction market in the Gulf region, with billions worth of private and public construction projects currently either in the planning stage or under construction.

The fast growth in the construction sector in the past decade had caused an increase in the number of structural failures and construction collapses in a number of projects that have led to fatalities and/or injuries due to human errors. One of the reasons behind this fact could be due to the fact that the civil engineering and construction industries in the UAE are currently dominated by foreign consultants and contractors who lack the in-depth knowledge of the local design codes, work practices, and construction environment.

Although municipalities and departments of public work have quality control procedures to review the engineers' work, they hardly have enough time to thoroughly review design calculations, reports and shop drawings. Consequently, higher rate than expected human errors that are committed during the design and construction phases are inevitable in this fast-growing environment. Based on the above, there is a need to find an approach that efficiently measures the effects of human errors on structural reliability in the UAE, and assist in providing guidelines to control and improve the structural safety of constructed facilities.

This research aims to reduce cases of structural failures and construction collapses due to unintended human errors in structural design and construction in the country. The results of the study serve as a basis for developing an error control strategy and decision making scheme that can be used in the design and construction of structures.

The study surveyed engineers in the country working for local practicing construction companies, local structural design companies, and municipalities on the common human errors committed during the design and constructions stages. It perform deterministic and reliability based sensitivity analysis approaches on different structural elements to determine the most critical design parameters affecting the nominal capacity and reliability index of these structural elements. Specifically, it considered reinforced concrete beams subjected to flexure, beams subjected to shear, and columns subjected axial compression. It addressed geometric properties, material mechanical properties, steel reinforcement characteristics and load components. In addition, the research includes sensitivity analysis on the reinforced concrete frame structure as a whole using a static pushover analysis to determine the most critical design parameters affecting the capacity of the structural system under lateral loads. Finally, checklists have been developed for different design and construction activities related to the substructure and superstructure so as to help reduce the occurrence of human errors during design and construction stages of buildings. In structural design, checklists on ETABS modeling and data entry, slabs, beams, columns, basement walls, raft, pile caps and piles were developed. On the other hand, the construction checklists were related to soil investigation, surveying, secant piling, beam capping, pile driving, excavation, backfilling, blinding work, formwork and shuttering, steel rebar work, screeding, and concreting work.

10.2 Conclusions

The various parts of the study lead to the following conclusions:

1. The surveys' results from structural and construction engineers working in the UAE have shown that the range of errors differs from one engineer/designer to another, depending on the level of experience of the surveyed engineer, type of work (structural design versus construction), and the nature of the activity. Although the surveyed engineers perceived the frequently of committed errors to be less than 5% for the majority of cases, the surveys have shown that some engineers/designers encounter errors at a higher rate. The surveys have also

pointed out that contractors are more reluctant to report human errors than structural designers.

2. While both deterministic and reliability-based sensitivity analysis are helpful in identifying the critical design parameters to structural capacity and structural reliability, respectively, the reliability-based approach provides more insight into the issue by quantifying the effect of variations in a design parameter on structural safety (or increase in probability of failure). Both approaches, however, can provide a basis behind the development of a quality control mechanism to be used structural design scenarios or construction situations.
3. Reductions in concrete compressive strength have minor effect on the reliability of beams in flexure, moderate effect on the reliability of beams in shear and severe effect on the reliability of columns in axial compression. Changes in steel yield strength for both longitudinal reinforcement and stirrups/ties have great effect on the reliability of beams in flexure, moderate effect on the reliability of beams in shear and mild effect on the reliability of column in axial compression. Beam width has a negligible effect on the flexural capacity and moderate effect on the shear strength. Cross-sectional dimensions of a column are very important factors to the axial compressive capacity. Effective steel reinforcement depth from the extreme compression fibers is critical to both flexural strength and shear strength. The live-to-dead load ratio in a structural member affects the sensitivity analysis.
4. Static pushover analysis is a useful tool for judging the strength of reinforced concrete frames under the effect of gravity and lateral loads. Static pushover analysis has shown that the number of stories and number of bays in a frame greatly affect the sensitivity analysis for the considered design parameters. As the number of stories increases, the sensitivity of the lateral load capacity of a given frame become more dependent on the beams' design parameters than on the columns' design parameters. Furthermore, reduction in the lateral load capacity of a frame when errors occur in beams width, columns width, compressive strength of beams concrete, or compressive strength of columns concrete is constant regardless of the number of bays, and number of stories. On the other hand, the area of longitudinal steel reinforcement, yield strength of reinforcement, gross area of members, and thickness of beams, the

reduction in the lateral load capacity becomes more dependent on the number of stories, number of bays, and whether the error is committed in beams, columns, or both.

5. Structural design checklists are helpful in providing an additional layer of quality control in consulting offices when structural members are designed and detailed with the use of hand calculations or software. Likewise, construction work checklists can aid the contractor in focusing attention on the critical parameters in a given construction activity and reducing the occurrence of human errors.

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Appendix A:

Surveys of Human Errors in the UAE

Appendix A.1: Survey in English

Survey of Errors in Structural Design and Construction in the UAE

A. Background Information:

- Title (e.g. senior structural engineer, project manager, project engineer, etc.): ……………………
- Nature of work: □ Structural Design □ Construction □ Other (please Specify) …………......
- Field of application: □ Buildings □ Infrastructure □ Other (please Specify) ……………...
- Number of years of experience: In the UAE: ……..… Outside of the UAE: …………

B. Questions related to construction:

How many times did you encounter unexpected problems in construction due to:

Q1: Improper soil investigation:

Frequency: ………………… per 100 cases

Q2: Improper sub-grade work, such as dewatering or water proofing:

Frequency: ………………… per 100 cases

Q3: Improper foundation work (e.g. wrong pile location, errors in drilled shaft construction, improper backfilling & soil compaction without following the specifications):

Frequency: ………………… per 100 cases

Q4: Using poor quality construction materials not complying with the specification:

Frequency: ………………… per 100 cases

Q5: Modifying details shown on drawings without referring to consultant/designer:

Frequency: ………………… per 100 cases

Q6: Improper formwork (e.g. shuttering and scaffolding):

Frequency: ………………… per 100 cases

Q7: Errors in designing/constructing temporary shoring and bracing during excavation:

Frequency: ………………… per 100 cases

Q8: Errors in details such as expansion/cold/construction joints:

Frequency: ………………… per 100 cases

Q9: Errors in reinforcement details (e.g. inadequate lap slices and end hooks):

Frequency: ………………… per 100 cases

Q10: Inadequate concrete cover and/or member sizes:

Frequency: ………………… per 100 cases

Q11: Improper concreting (e.g. inadequate concrete compaction or concrete placement):

Frequency: ………………… per 100 cases

Q12: Overloading the structure during construction or premature removal of formwork/scaffolding:

Frequency: ………………… per 100 cases

C. Questions related to structural design:

How many times did you encounter unexpected errors in structural design due to:

Q1: Conceptual mistakes (e.g. load transfer, support boundary conditions, connection, etc.)

Frequency: …………………. per 100 cases of design

Q2: Unit related errors (e.g. using cm for m, using inches instead of cm):

Frequency: …………………. per 100 pages of calculations

Q3: Calculation mistakes:

Frequency: …………………. per 100 pages of calculations

Q4: Wrong extraction of information from tables, architectural drawings or charts:

Frequency: …………………. per 100 tables/drawings/charts

Q5: Neglecting water table, buoyancy, soil weight or live load surcharge in calculations:

Frequency: …………………. per 100 cases of design

Q6: Mixing equations from different codes inappropriately (e.g. ACI code with BS standard):

Frequency: …………………. per 100 calculation steps

Q7: Wrong selection of factors of safety, load combinations, or load factors:

Frequency: …………………. per 100 cases of design

Q8: Wrong assumptions of wind load or seismic load (Not checking the governing case, wrong wind speed and wind factor, or wrong selection of seismic factors and accelerations):

Frequency: …………………. per 100 cases of design

Q9: Neglecting load eccentricity on a column, torsion, punching shear or uplift force:

Frequency: …………………. per 100 cases of design

Q10: Lack of knowledge with regard to use of software (e.g. wrong input or wrong interpretation of the output from the software):

Frequency: …………………. per 100 cases of design

Q11: Not checking the reinforcement limits according to the code (e.g. minimum & maximum reinforcement, rebar/stirrup spacing, etc.):

Frequency: …………………. per 100 cases of design

Q12: Not checking the required serviceability limits (e.g. minimum member thickness, crack width, temperature and shrinkage reinforcement, etc.):

Frequency: …………………. per 100 pages of calculations

Q13: Wrong transferring of results from design calculations to drawings:

Frequency: …………………. per 100 cases of transferring results

Q14: Wrong reinforcement around typical details (e.g. around openings, connections, etc.):

Frequency: …………………. per 100 cases of design

- Please provide additional information, if you wish, on errors committed during the design and/or construction stages that were not addressed in Parts B and C. Indicate their frequencies as well.

…………………………………………………………………………………………

…………………………………………………………………………………………

Appendix A.2: Survey in Arabic

استبيان عن تأثير الأخطاء البشرية على سلامة المنشآت والمباني في دولة الإمارات العربية المتحدة

أولاً: المعلومات الشخصية:

- المسمى الوظيفي (مثال: مهندس تصميم أول، مدير مشروع، مهندس مشروع، الخ):
  ....
- طبيعة العمل: ☐ التصميم الهندسي ☐ تنفيذ الإنشاءات ☐ مجال آخر(يرجى ذكره):
  ....
- مجال العمل: ☐ المنشآت ☐ البنية التحتية ☐ مجال آخر(يرجى ذكره):
  ....
- عدد سنوات الخبرة: ☐ داخل دولة الإمارات: .... ☐خارج دولة الإمارات: ....

ثانياً: الأسئلة المتعلقة بأخطاء الإنشاءات

كم عدد المرات التي واجهت فيها مشاكل غير متوقعة أثناء تنفيذ الإنشاءات نتيجة:

1. أخطاء في فحص التربة

   التكرار .... في 100 حالة

2. أخطاء في أعمال التربة كأعمال سحب الماء الجوفي أو العزل المائي

   التكرار .... في 100 حالة

3. أخطاء في أعمال الأساسات (أخطاء في تحديد مكان الركائز "الخوازيق الأوتاد"، أخطاء في حفر الركائز "الخوازيق" أو في ردم ودك التربة دون مراعاة المواصفات)

   التكرار .... في 100 حالة

4. استخدام مواد بناء غير متوافقة مع المواصفات

   التكرار .... في 100 حالة

5. تعديل تفاصيل على المخططات دون الرجوع إلى الاستشاري/المهندس

   التكرار .... في 100 حالة

6. أخطاء في شدات الطوبار (مثال القوالب والسقائل)

   التكرار .... في 100 حالة

7. أخطاء في التصميم/التنفيذ للإسناد الجانبي المؤقت للتربة والتدعيم خلال الحفريات

   التكرار .... في 100 حالة

8. أخطاء في بعض التفاصيل مثل فواصل التمدد/على البارد/ فواصل التنفيذ

التكرار .... في 100 حالة

9. أخطاء في تفاصيل حديد التسليح (عدم كفاية طول التثبيت "التشريك" أو طول الخطاف)

التكرار .... في 100 حالة

10. عدم كفاية الغطاء الخرساني حول حديد التسليح و/أو قياسات العنصر الإنشائي

التكرار .... في 100 حالة

11. أخطاء في صب الخرسانة (عدم دك الخرسانة بشكل جيد أو التوزيع السيء)

التكرار .... في 100 حالة

12. زيادة تحميل المبنى أثناء عمليات الإنشاء أو إزالة القوالب / السقائل قبل وقتها اللازم

التكرار .... في 100 حالة

ثالثاً: أسئلة متعلقة بأخطاء التصميم الهندسي

كم عدد المرات التي واجهت فيها مشاكل غير متوقعة أثناء مراحل التصميم الهندسي نتيجة:

1. أخطاء في فكرة التصميم (طريقة انتقال الأحمال، سند الموقع المحيط، وصلات الربط)

التكرار .... في 100 حالة تصميم

2. أخطاء في وحدات القياس (استخدام السنتمتر بدل المتر، أو الإنش بدل السنتمتر)

التكرار .... في 100 صفحة حسابات

3. أخطاء في الحسابات (جمع، طرح، جذر تربيعي، الخ)

التكرار .... في 100 صفحة حسابات

4. أخطاء في استخراج المعلومات من الجداول، المخططات المعمارية أو المخططات البيانية

التكرار .... في 100 جدول/مخطط معماري/مخطط بياني

5. إهمال منسوب المياه الجوفية، وزن التربة، أو وزن المركبات المتحركة والحمولة الحية في الحسابات

التكرار .... في 100 حالة تصميم

6. استخدام معادلات من أكواد مختلفة بصورة غير ملائمة (استخدام الكود الأمريكي والبريطاني في الوقت نفسه)

التكرار .... في 100 خطوة حسابية

7. اختيار خاطئ لعوامل الأمان، حالات التحميل أو عوامل الحمولات

التكرار .... في 100 حالة تصميم

8. أخطاء في حساب حمولات الرياح والزلازل (عدم التأكد من الحالة المسيطرة، افتراض خاطئ لسرعة الرياح ومعامل الرياح، اختيار خاطئ لمعامل الزلازل والتسارع)

التكرار .... في 100 حالة تصميم

9. إهمال القوى اللامركزية على العمود، الفتل، الثقب، أو القوى الرافعة

التكرار .... في 100 حالة تصميم

10. قلة خبرة في استخدام برامج التصميم الهندسية (إدخالات خاطئة أو تفسيرات خاطئة لنتائج البرامج الهندسية)

التكرار .... في 100 حالة تصميم

11. عدم التأكد من موافقة نسب التسليح للكود (الحد الأدنى والأقصى من حديد التسليح، المسافات بين الوصلات /الكانات "الأساور")

التكرار .... في 100 حالة تصميم

12. عدم التأكد من حدود التشغيل (سماكة العنصر الإنشائي، سُمك الشق، درجات الحرارة وحديد التقلص)

التكرار .... في 100 صفحة حسابات

13. أخطاء في نقل النتائج من حسابات التصميم إلى المخططات

التكرار .... في 100 حالة نقل النتائج

14. أخطاء في حديد التسليح حول التفاصيل الخاصة (حول الفتحات والوصلات، الخ)

التكرار .... في 100 حالة تصميم

- في حال كان لديكم معلومات إضافية عن الأخطاء الشائعة أثناء عمليات التصميم والإنشاءات والتي لم يُتطرق إليها في الاستبيان، فيرجى ذكرها إضافة إلى عدد مرات تكرارها

....

....

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Appendix B:

Design Checklists

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	1. ETABS Modeling and Load Entry				
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Building Plan Grid System and Story Data Definition**				
1.1	Unit convention is correct				
1.2	Number of grids in the x direction is consistent with architectural drawings				
1.3	Number of grids in the y direction is consistent with architectural drawings				
1.4	Spacing between grids is consistent with architectural drawings				
1.5	Number of floors is correct				
1.6	Floor heights are correct				
1.7	Correct design code has been chosen				
1.8	All openings have been located correctly				
1.9	All expansion joints have been located correctly				
2	**Defining Concrete and Reinforcement Properties**				
2.1	Compressive strength of concrete is as per the approved civil design basis				
2.2	Yield strength of reinforcement is as per the approved civil design basis				
3	**Defining and Assigning Columns**				
3.1	Columns have been assigned the correct construction material				
3.2	Each group of columns has been named with correct designation				
3.3	Dimensions of each column group are correct				
3.4	Property modifiers are correct				
3.5	All columns are drawn in the correct location				
4	**Defining and Assigning Beams**				
4.1	Beams have been assigned the correct construction material				
4.2	Each group of beams has been named with correct designation				
4.3	Dimensions of each beam group are correct				
4.4	Property modifiers are correct				
3.5	All beams are drawn in the correct location				
5	**Defining and Assigning Slabs**				
5.1	Slabs have been assigned the correct construction material				
5.2	Each group of slabs has been named with correct designation				

5.3	Thickness all slab groups is correct				
5.4	Property modifiers are correct				
5.5	All slabs are drawn in the correct location				
6	**Defining and Assigning Shear Walls**				
6.1	Shear walls have been assigned the correct construction material				
6.2	Each group of shear walls has been named with correct designation				
6.3	Thickness all shear wall groups is correct				
6.4	The length of the shear wall is correct				
6.5	Property modifiers are correct				
6.6	All shear walls are drawn in the correct location				
7	**Assigning Loads**				
7.1	Superimposed dead load value, units and location are correct				
7.2	Live load value, units and location are correct				
7.3	Roof live load value and units are correct				
7.4	Cladding weight and units are correct				
7.5	Partition weight, units and location are correct				
7.6	Lateral earth pressure value and units are correct				
7.7	Surcharge unit load considered is correct				
7.8	Impact factor has been considered for the surcharge load				
7.9	Correct code has been selected for wind and seismic loads				
7.10	All wind load parameters are as per the approved civil design basis				
7.11	All seismic load parameters are as per the approved civil design basis				
7.12	Orthogonal effect of seismic load has been considered				
7.13	The governing between wind and seismic loads has been considered in the design				
7.14	Governing load has been applied from both directions				
8	**Define Load Combinations**				
8.1	All ultimate load combinations are considered				
8.2	All service load combinations are considered				
9	**Checks Before Running the Model**				
9.1	Diaphragm action has been applied				
9.2	Fixed end conditions are correct				
9.3	Material properties have been rechecked				
9.4	Mass source has been correctly defined				

9.5	Correct analysis option has been selected				
9.6	Correct frame sway has been selected				
9.7	Model is Error-free				
10	**Checks After Running the Model**				
10.1	Analysis Log has been checked for global errors				
10.2	Masses obtained from ETABS are similar to that calculated manually				
10.3	Story shears obtained from ETABS are similar to that calculated manually				

2. One-way Slabs **2.1. Design of one-way slabs**		**Slab Designation**			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Input Data**				
1.1	Slab has a designation, e.g. GS002				
1.2	Correct design code is used				
1.3	Compressive strength is as per the approved civil design basis				
1.4	Yield strength of reinforcement is as per the approved civil design basis				
1.5	Width of the panel considered is matching with drawings				
1.6	Length of the panel considered is matching with drawings				
1.7	Top concrete cover is as per the approved civil design basis				
1.8	Bottom concrete cover is as per the approved civil design basis				
2	**Initial Calculations**				
2.1	Clear spans are correctly calculated				
2.2	Ratio of long span to short span is more than 2				
2.3	Minimum thickness of the slab has been calculated using the correct table				
2.4	End conditions have been determined properly				
3	**Design**				
3.1	Unit consistency has been checked				
3.2	Ultimate shear is extracted correctly from the design software				
3.3	Ultimate moment is extracted correctly from the design software				
3.4	Slab thickness is adequate for shear				
3.5	Top and bottom reinforcement provided in the x direction is more than the minimum reinforcement limit				
3.6	Top and bottom reinforcement provided in the y direction is more than the minimum reinforcement limit				
3.7	Spacing between top steel in the x direction is less than or equal to the spacing required				

3.8	Spacing between bottom steel in the x direction is less than or equal to the spacing required				
3.9	Spacing between top steel in the y direction is less than or equal to the spacing required				
3.10	Spacing between bottom steel in the y direction is less than or equal to the spacing required				
3.11	Shrinkage and temperature reinforcement provided is more than the minimum limit				
3.12	Deflection in the slab is not exceeding the allowable limit				

2. One-way Slabs 2.2. RC Detailing		Slab Designation			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**RC schedule**				
1.1	The designation of the slab is correct				
1.2	Thickness of the wall is correct				
1.3	Top reinforcement size and spacing in the x direction have been properly transferred to the correct designation				
1.4	Bottom reinforcement size and spacing in the x direction have been properly transferred to the correct designation				
1.5	Top reinforcement size and spacing in the y direction have been properly transferred to the correct designation				
1.6	Bottom reinforcement size and spacing in the y direction have been properly transferred to the correct designation				
2	**Structural Drawings**				
2.1	Slab reinforcement is matching with the RC schedule				
2.2	Tension splice length is as per the code requirements				
2.3	Location of tension splice is correct				
2.4	Compression splice length is as per the code requirements				
2.5	Location of compression splice is correct				
2.6	Anchorage length is as per the code requirements				
2.7	Reinforcements around openings have been provided correctly				

3. Two-way Slabs **3.1. Design of two-way slabs**		**Slab Designation**			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Input Data**				
1.1	Slab has a designation, e.g. GS002				
1.2	Correct design code is used				
1.3	Compressive strength is as per the approved civil design basis				
1.4	Yield strength of reinforcement is as per the approved civil design basis				
1.5	Width of the panel considered is matching with drawings				
1.6	Length of the panel considered is matching with drawings				
1.7	Top concrete cover is as per the approved civil design basis				
1.8	Bottom concrete cover is as per the approved civil design basis				
2	**Initial Calculations**				
2.1	Clear spans are correctly calculated				
2.2	Ratio of long span to short span is less than 2				
2.3	Minimum thickness of the slab has been calculated using the correct table				
2.4	Distribution factors have been selected correctly				
2.5	End conditions have been determined properly				
2.6	Relative stiffness of the beam and slab has been calculated correctly				
2.7	Relative restraint provided by the torsion resistance of the effective transverse edge beam has been calculated correctly				
3	**Design**				
3.1	Unit consistency has been checked				
3.2	Ultimate shear is extracted correctly from the design software				
3.3	Ultimate moment is extracted correctly from the design software				
3.4	Slab thickness is adequate for shear				
3.5	Top and bottom reinforcement provided in the x direction is more than the minimum reinforcement limit				

3.6	Top and bottom reinforcement provided in the y direction is more than the minimum reinforcement limit				
3.7	Spacing between top steel in the x direction is less than or equal to the spacing required				
3.8	Spacing between bottom steel in the x direction is less than or equal to the spacing required				
3.9	Spacing between top steel in the y direction is less than or equal to the spacing required				
3.10	Spacing between bottom steel in the y direction is less than or equal to the spacing required				
3.11	Deflection in the slab is not exceeding the allowable limit				

3. Two-way Slabs 3.2. RC Detailing		Slab Designation			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**RC schedule**				
1.1	The designation of the slab is correct				
1.2	Thickness of the wall is correct				
1.3	Top reinforcement size and spacing in the x direction have been properly transferred to the correct designation				
1.4	Bottom reinforcement size and spacing in the x direction have been properly transferred to the correct designation				
1.5	Top reinforcement size and spacing in the y direction have been properly transferred to the correct designation				
1.6	Bottom reinforcement size and spacing in the y direction have been properly transferred to the correct designation				
2	**Structural Drawings**				
2.1	Slab reinforcement is matching with the RC schedule				
2.2	Tension splice length is as per the code requirements				
2.3	Location of tension splice is correct				
2.4	Compression splice length is as per the code requirements				
2.5	Location of compression splice is correct				
2.6	Anchorage length is as per the code requirements				
2.7	Reinforcements around openings have been provided correctly				

4. Beams 4.1. Design against Bending		Beam Designation			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Input Data**				
1.1	The beam has a proper designation, e.g. GB001				
1.2	Correct design code is used				
1.3	Ultimate bending moment at right support for top steel has been extracted properly from the design software				
1.4	Ultimate bending moment at right support for bottom steel has been extracted properly from the design software				
1.5	Ultimate bending moment at left support for top steel has been extracted properly from the design software				
1.6	Ultimate bending moment at left support for bottom steel has been extracted properly from the design software				
1.7	Ultimate bending moment at mid span support for top steel has been extracted properly from the design software				
1.8	Bottom concrete cover is as per the approved civil design basis				
1.9	Ultimate bending moment at mid span for bottom steel has been extracted properly from the design software				
1.10	Compressive strength is as per the approved civil design basis				
1.11	Yield strength of reinforcement is as per the approved civil design basis				
1.12	Top concrete cover is as per the approved civil design basis				
1.13	Bottom concrete cover is as per the approved civil design basis				
2	**Design for Moment**				
2.1	Unit consistency has been checked				
2.2	The selected reinforcement ratio is larger than the minimum limit				
2.3	The selected reinforcement ratio is less than the reinforcement ration that results in steel strain of 0.005				

2.4	Effective depth to top reinforcement is correct				
2.5	Effective depth to bottom reinforcement is correct				
2.6	Beam height is larger than the minimum requirement				
2.7	Beam span to height ratio is not exceeding the limit				
2.8	Area of top steel provided at right support is more than the required area of reinforcement				
2.9	Area of bottom steel provided at right support is more than the required area of reinforcement				
2.10	Area of top steel provided at left support is more than the required area of reinforcement				
2.11	Area of bottom steel provided at left support is more than the required area of reinforcement				
2.12	Area of top steel provided at mid span is more than the required area of reinforcement				
2.13	Area of bottom steel provided at mid span is more than the required area of reinforcement				
2.14	The width of the beam is adequate for placing the reinforcement				
2.15	The moment capacity of the beam is more than the ultimate moment				
3	**Deflection Check**				
3.1	The deflection of the beam is not exceeding the allowable limit				

4. Beams 4.2. Design against Shear and Torsion		Beam Designation			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Input Data**				
1.1	Ultimate shear has been extracted properly from the design software				
1.2	Ultimate torsion has been extracted properly from the design software				
1.3	Width of the beam considered is correct				
1.4	Height of the beam considered is correct				
1.5	Compressive strength is as per the approved civil design basis				
1.6	Yield strength of reinforcement is as per the approved civil design basis				
2	**Design for Shear**				
2.1	Unit consistency has been checked				
2.2	Effective depth to top reinforcement is correct				
2.3	Effective depth to bottom reinforcement is correct				
2.4	The cross section is adequate for shear				
2.5	Area of stirrups provided is more than the required area				
2.6	Top concrete cover is as per the approved civil design basis				
3	**Design for Torsion**				
3.1	Unit consistency has been checked				
3.2	Minimum torsion resistance has been calculated correctly				
3.3	Area of stirrups due to torsion provided is more than the required area				
3.4	Longitudinal reinforcement due to torsion has been calculated				
4	**Final Longitudinal Reinforcement**				
4.1	Longitudinal reinforcement due to torsion has been added to the longitudinal reinforcement due to moment				
4.2	Longitudinal reinforcement provided is more than required				
4.3	Side bars have been calculated properly				

5	**Final Stirrups Spacing**				
5.1	Area of stirrups due to torsion has been added to the area of stirrups due to shear				
5.2	Area of stirrups provided is more than the required				
5.3	Stirrups spacing is as per the code requirements				

4. Beams 4.3. RC Detailing		Beam Designation			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**RC Schedule**				
1.1	Beam dimensions have been correctly transferred to the correct beam designation				
1.2	Top and bottom reinforcement at right support of the beam have been correctly transferred				
1.3	Top and bottom reinforcement at left support of the beam have been correctly transferred				
1.4	Top and bottom reinforcement at mid span of the beam have been correctly transferred				
1.5	Side bars have been provided as calculated				
1.6	Stirrups size has been correctly transferred				
1.7	Spacing of stirrups has been correctly transferred				
2	**Detailing**				
2.1	Longitudinal reinforcement is matching with the RC schedule				
2.2	Stirrups size and spacing are matching with the RC schedule				
2.3	Tension splice length is as per the code requirement				
2.4	Location of tension splice is correct				
2.5	Compressive splice length is as per the code requirement				
2.6	Location of compression splice is correct				
2.7	Anchorage and development lengths is as per the code requirement				

5. Columns 5.1. Design of Columns		Column Designation			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Input Data**				
1.1	Column has proper designation, e.g. GC1A				
1.2	Correct structural design code is used				
1.3	Compressive strength of concrete is as per the approved civil design basis				
1.4	Yield strength of steel reinforcement is as per the approved civil design basis				
1.5	Effective length of the column is correct				
1.6	Column end conditions considered are correct				
1.7	Maximum ultimate axial load and corresponding moments about 2 directions have been extracted properly from the design software				
1.8	Maximum ultimate moment about the x direction and corresponding axial force and moment about the y direction have been extracted properly from the design software				
1.9	Maximum ultimate moment about the y direction and corresponding axial force and moment about the x direction have been extracted properly from the design software				
1.10	Concrete clear cover is as per the approved design criteria				
2	**Design of Column**				
2.1	Unit consistency has been checked				
2.2	All design parameters have been inserted correctly to the column design computer program				
2.3	The point representing factored load and moment is inside the interaction diagram				
2.4	The percentage of steel in the column is above the minimum limit				
2.5	The percentage of steel in the column is less than maximum limit				
2.6	Size of lateral steel ties is correct				
2.7	Spacing of lateral ties is as per the code requirement				

5. Columns 5.2. RC Detailing		Column Designation			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**RC Schedule**				
1.1	Column size has been correctly transferred to the correct column designation				
1.2	Number of bars in the x-direction is correct				
1.3	Number of bars in the y-direction is correct				
1.4	Size and spacing of ties are correct				
2	**Detailing**				
2.1	Steel distribution in the x and y directions is correct				
2.2	Concrete cover is correct				
2.3	Splice length is as per the code requirement				
2.4	Anchorage length is as per the code requirement				

6. Basement Wall 6.1. Design of Basement Wall		Wall Designation			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Input Data**				
1.1	The wall has a designation, e.g. W002				
1.2	Correct design code is used				
1.3	Unit consistency has been checked				
1.4	Thickness of the wall is correct				
1.5	Height of the wall is correct				
1.6	Compressive strength of concrete is as per the approved civil design basis				
1.7	Yield strength of reinforcement is as per the approved civil design basis				
1.8	Side concrete cover is as per the approved civil design basis				
2	**Vertical Design of the Wall (Inner and Outer Faces)**				
2.1	Factored design moment is taken correctly from the design software				
2.2	Axial stress is taken correctly from the software output				
2.3	Effective Depth is correct				
2.4	Steel has been calculated for bending and axial stress				
2.5	Spacing between reinforcement is as per the code requirement				
2.6	Crack width criteria is not exceeding the allowable limit				
3	**Horizontal Design of the Wall (Inner and Outer Faces)**				
3.1	Factored design moment is taken correctly from the design software				
3.2	Axial stress is taken correctly from the software output				
3.3	Effective Depth is correct				
3.4	Steel has been calculated for bending and axial stress				
3.5	Spacing between reinforcement is as per the code requirement				
3.6	Crack width criteria is not exceeding the allowable limit				

6. Basement Wall **6.2. RC Detailing**		**Wall Designation**			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**RC schedule**				
1.1	The designation of the wall is correct				
1.2	Thickness of the wall is correct				
1.3	Vertical reinforcement of inner face is correct				
1.4	Spacing between vertical reinforcement of inner face is correct				
1.5	Vertical reinforcements of outer face are correct				
1.6	Spacing between vertical reinforcement of outer face is correct				
1.7	Horizontal reinforcement of inner face is correct				
1.8	Spacing between horizontal reinforcement of inner face is correct				
1.9	Horizontal reinforcement of outer face is correct				
1.10	Spacing between horizontal reinforcement of outer face is correct				
1.11	Hidden beams around openings are provided				
2	**Detailing**				
2.1	The designation of the wall is correct				
2.2	Thickness of the wall is correct				
2.3	Vertical reinforcement of inner face is correct				
2.4	Spacing between vertical reinforcement of inner face is correct				
2.5	Vertical reinforcements of outer face are correct				
2.6	Spacing between vertical reinforcement of outer face is correct				
2.7	Horizontal reinforcement of inner face is correct				
2.8	Spacing between horizontal reinforcement of inner face is correct				
2.9	Horizontal reinforcement of outer face is correct				
2.10	Spacing between horizontal reinforcement of outer face is correct				
2.11	Hidden beams around openings are provided				

7. Raft Foundation 7.1. Design for Bending					
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Input Data**				
1.1	Correct design code is used				
1.2	Dimensions of the raft are correct				
1.3	Columns have been located properly on the raft				
1.4	Column sizes have been inserted properly				
1.5	Columns reaction from superstructure has been properly extracted				
1.6	Compressive strength is as per the approved civil design basis				
1.7	Yield strength of reinforcement is as per the approved civil design basis				
1.8	Expansion joints location is located correctly				
1.9	Ultimate load combinations have been considered				
1.10	Allowable load combinations have been considered				
1.11	Top concrete cover is as per the approved civil design basis				
1.12	Bottom concrete cover is as per the approved civil design basis				
1.13	Side concrete cover is as per the approved civil design basis				
2	**Design for Moment in the X Direction**				
2.1	Unit consistency has been checked				
2.2	Maximum moment at the top has been extracted correctly from the software				
2.3	Maximum moment at the bottom has been extracted correctly from the software				
2.4	Uplift water pressure is considered				
2.5	Extra bars are required for tension at the top				
2.6	Extra bars are required for tension at the bottom				
2.7	Effective depth to top reinforcement is correct				
2.8	Effective depth to bottom reinforcement is correct				
2.9	Top reinforcements provided is more than required				
2.10	Bottom reinforcements provided is more than required				

2.11	Top extra bars provided is more than required				
2.12	Bottom extra bars provided is more than required				
2.13	Spacing provided for top reinforcements is as per the code				
2.14	Spacing provided for bottom reinforcements is as per the code				
2.15	Spacing provided for top extra bars is correct				
2.16	Spacing provided for bottom extra bars is correct				
3	**Design for Moment in the Y Direction**				
3.1	Unit consistency has been checked				
3.2	Maximum moment at the top has been extracted correctly from the software				
3.3	Maximum moment at the bottom has been extracted correctly from the software				
3.4	Uplift water pressure is considered				
3.5	Extra bars are required for tension at the top				
3.6	Extra bars are required for tension at the bottom				
3.7	Effective depth to top reinforcement is correct				
3.8	Effective depth to bottom reinforcement is correct				
3.9	Top reinforcements provided is more than required				
3.10	Bottom reinforcements provided is more than required				
3.11	Top extra bars provided is more than required				
3.12	Bottom extra bars provided is more than required				
3.13	Spacing provided for top reinforcements is as per the code				
3.14	Spacing provided for bottom reinforcements is as per the code				
3.15	Spacing provided for top extra bars is correct				
3.16	Spacing provided for bottom extra bars is correct				

7. Raft Foundation 7.2. Checking the Design					
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Checking Shear in the X Direction**				
1.1	Maximum shear has been extracted correctly from the software				
1.2	Diameter of bending reinforcement used in the check is correct				
1.3	Thickness of the raft used in the check is correct				
1.4	Shear capacity at the face of the support is more than the maximum shear				
1.5	Shear check at 1.5d from the face of the column is safe				
2	**Checking Shear in the Y Direction**				
2.1	Maximum shear has been extracted correctly from the software				
2.2	Diameter of bending reinforcement used in the check is correct				
2.3	Thickness of the raft used in the check is correct				
2.4	Shear capacity at the face of the support is more than the maximum shear				
2.5	Shear check at 1.5d from the face of the column is safe				
3	**Checking Punching Shear**				
3.1	Diameter of bending reinforcement used in the check is correct				
3.2	Effective depth used in the check is correct				
3.3	Punching shear check for corner columns is ok				
3.4	Punching shear check for exterior columns is ok				
3.5	Punching shear check for interior columns is ok				
3.6	Drop panels are provided				
4	**Other Checks**				
4.1	Uniform settlement is less than the allowable uniform settlement				
4.2	Differential settlement is less than the allowable differential settlement				
4.3	Pressure at all corners is less than the allowable soil bearing capacity				
4.4	Pressure at all corners is less than the allowable soil bearing capacity considering seismic				
4.5	Pressure at all corners is less than the allowable soil bearing capacity considering overlapping of stresses.				

7. Raft Foundation 7.3. RC Detailing					
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Structural Drawings and RC Details**				
1.1	Thickness of the raft is correct				
1.2	Dimensions of the raft are correct				
1.3	Top reinforcement size and spacing in the x direction are correct				
1.4	Bottom reinforcement size and spacing in the x direction are correct				
1.5	Top reinforcement size and spacing in the y direction are correct				
1.6	Bottom reinforcement size and spacing in the y are correct				
1.7	Tension splice length is as per the code requirements				
1.8	Location of tension splice is correct				
1.9	Compression splice length is as per the code requirements				
1.10	Location of compression splice is correct				

8. Pile Cap 8.1. Design of Pile Cap		**Pile Cap Designation**			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Input Data**				
1.1	Correct design code is used				
1.2	Pile cap has a proper designation, e.g. PC1				
1.3	Total ultimate load on columns has been extracted properly from the design software				
1.4	Column dimensions are correct				
1.5	Pile diameter is correct				
1.6	Pile location has been assigned properly on the pile cap				
1.7	Distance between piles in the x direction within the pile cap is correct				
1.8	Distance between piles in the y direction within the pile cap is correct				
1.9	Compressive strength is as per the approved civil design basis				
1.10	Yield strength of reinforcement is as per the approved civil design basis				
1.11	Top concrete cover is as per the approved civil design basis				
1.12	Bottom concrete cover is as per the approved civil design basis				
1.13	Side concrete cover is as per the approved civil design basis				
2	**Design for Moment in the X Direction**				
2.1	Unit consistency has been checked				
2.2	Maximum moment at the top has been extracted correctly from the software				
2.3	Maximum moment at the bottom has been extracted correctly from the software				
2.4	Uplift water pressure is considered				
2.5	Extra bars are required for tension at the top				
2.6	Extra bars are required for tension at the bottom				
2.7	Effective depth to top reinforcement is correct				
2.8	Effective depth to bottom reinforcement is correct				
2.9	Top reinforcements provided is more than required				

2.10	Bottom reinforcements provided is more than required				
2.11	Top extra bars provided is more than required				
2.12	Bottom extra bars provided is more than required				
2.13	Spacing provided for top reinforcements is as per the code				
2.14	Spacing provided for bottom reinforcements is as per the code				
2.15	Spacing provided for top extra bars is correct				
2.16	Spacing provided for bottom extra bars is correct				
3	**Design for Moment in the Y Direction**				
3.1	Unit consistency has been checked				
3.2	Maximum moment at the top has been extracted correctly from the software				
3.3	Maximum moment at the bottom has been extracted correctly from the software				
3.4	Uplift water pressure is considered				
3.5	Extra bars are required for tension at the top				
3.6	Extra bars are required for tension at the bottom				
3.7	Effective depth to top reinforcement is correct				
3.8	Effective depth to bottom reinforcement is correct				
3.9	Top reinforcements provided is more than required				
3.10	Bottom reinforcements provided is more than required				
3.11	Top extra bars provided is more than required				
3.12	Bottom extra bars provided is more than required				
3.13	Spacing provided for top reinforcements is as per the code				
3.14	Spacing provided for bottom reinforcements is as per the code				
3.15	Spacing provided for top extra bars is correct				
3.16	Spacing provided for bottom extra bars is correct				

8. Pile Cap 8.2. Checking the Design		Pile Cap Designation			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Checking Shear in the X Direction**				
1.1	Maximum shear has been extracted correctly from the software				
1.2	Diameter of bending reinforcement used in the check is correct				
1.3	Thickness of the raft used in the check is correct				
1.4	Shear capacity at the face of the support is more than the maximum shear				
1.5	Shear check at 1.5d from the face of the column is safe				
2	**Checking Shear in the Y Direction**				
2.1	Maximum shear has been extracted correctly from the software				
2.2	Diameter of bending reinforcement used in the check is correct				
2.3	Thickness of the raft used in the check is correct				
2.4	Shear capacity at the face of the support is more than the maximum shear				
2.5	Shear check at 1.5d from the face of the column is safe				
3	**Checking Punching Shear**				
3.1	Diameter of bending reinforcement used in the check is correct				
3.2	Effective depth used in the check is correct				
3.3	Punching shear check is ok				
4	**Other Checks**				
4.1	Uniform settlement is less than the allowable uniform settlement				
4.2	Differential settlement is less than the allowable differential settlement				

8. Pile Cap **8.3. RC Detailing**		**Pile Cap Designation**			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Structural Drawings and RC Details**				
1.1	Thickness of the raft is correct				
1.2	Dimensions of the pile cap are correct				
1.3	Top reinforcement size and spacing in the x direction are correct				
1.4	Bottom reinforcement size and spacing in the x direction are correct				
1.5	Top reinforcement size and spacing in the y direction are correct				
1.6	Bottom reinforcement size and spacing in the y are correct				
1.7	Tension splice length is as per the code requirements				
1.8	Location of tension splice is correct				
1.9	Compression splice length is as per the code requirements				
1.10	Location of compression splice is correct				

9. Pile Foundation 9.1. Design of Piles		Pile Designation			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Extracts from soil report**				
1.1	The pile is designed for the worst boreholes' parameters				
1.2	The pile head surface has been made clean and dust free				
1.3	Pile depth is as per the soil investigation report recommendations				
1.4	Pile diameter has been extracted from the soil investigation report				
1.5	Corresponding working load, uplift force, lateral load and settlement have been extracted correctly				
1.6	Soil friction angle at base is as per the soil investigation report				
1.7	Average soil friction angle is as per the soil investigation report				
1.8	Minimum UCS of rock at pile toe has been extracted correctly from the soil investigation report				
1.9	Minimum UCS in rock socket has been extracted correctly from the soil investigation report				
1.10	Rock socket length has been taken as per the soil investigation report recommendations				
2	**Other Inputs**				
2.1	The pile has a proper designation, e.g. PILE001				
2.2	Correct design code is used				
2.3	Concrete compressive strength is as per the approved civil design basis				
2.4	Yield strength of reinforcement is as per the approved civil design basis				
2.5	Concrete cover is as per the approved civil design basis				
2.6	Factor of safety considered is as per the approved civil design basis				
2.7	Capacity of the pile has been reduced if the distance between it and next pile is less than 1.5xPile Diameter				
2.8	Unit consistency has been checked				

3	**Structural Design of the Pile**				
3.1	The ultimate working load recommended in the soil investigation report (considering a load factor) is not exceeding the value of N for the assumed reinforcement to be used in piles				
3.2	The recommended horizontal load in the soil investigation report has been increased to account for construction allowances				
3.3	Ultimate moment due to eccentricity has been correctly calculated				
3.4	Moment due to rotation has been correctly calculated				
3.5	Moment due to horizontal working load (considering construction allowances) has been correctly calculated				
3.6	The moment considered for the reinforcement design is the summation of the eccentricity moment, rotational moment and horizontal load moment				
3.7	Main reinforcement has been calculated as per the code requirements				
3.8	Reinforcement percentage is as per the code requirements				
3.9	Spirals have been calculated as per the code requirements				
4	**Pile Capacity in Rock**				
4.1	End bearing capacity of pile has been calculated correctly				
4.2	Shaft frictional resistance of pile has been calculated correctly				
4.3	Total pile capacity in rock has been calculated considering end bearing capacity and shaft frictional resistance of pile				
4.4	Pile capacity in rock is more than the applied load				

9. Pile Foundation 9.2. Checking the Design		Pile Designation			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Checks**				
1.1	Working stress in concrete is not exceeding the allowable limit of the code				
1.2	Ultimate stress in concrete is not exceeding the allowable limit of the code				
1.3	Tension stress capacity of the pile is more than the uplift force provided in the soil investigation report				
1.4	Clear spacing of reinforcement has been checked				
1.5	Anchorage length has been checked				
1.6	Working shear stress in pile has been checked				
2	**Settlement Check**				
2.1	Settlement due to elastic shortening of the pile has been considered				
2.2	Settlement due to actual pile load has been considered				
2.3	Settlement due to load transfer through pile shaft has been considered				
2.4	Total settlement is not exceeding the allowable settlement				
3	**Checking Pile Capacity against Applied Load**				
3.1	The vertical load acting on the pile from the superstructure is less than the vertical capacity of the pile				
3.2	The horizontal load acting on the pile from the superstructure is less than the horizontal capacity of the pile				

9. Pile Foundation 9.3. RC Detailing		Pile Designation			
Item	**Parameter**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Drawings and RC Details**				
1.1	Pile Designation is correct				
1.2	Diameter of the pile is correct				
1.3	Concrete cover is shown correctly				
1.4	Diameter of steel bars is correct				
1.5	Number of steel bars is correct				
1.6	Spiral diameter is correct				
1.7	Spacing of spirals is correct				
1.8	Total cage length is correct				
1.9	Rock socket levels are correct				
1.10	Top pile level is correct				
1.11	Toe pile level is correct				
1.12	Pile cut off level is correct				

Appendix C:

Construction Checklists

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1. Soil Investigation 1.1.Preparatory Work		Borehole Number			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Setting out Work**				
1.1	Drawing showing the location of approved boreholes is endorsed and signed by the geotechnical agency				
1.2	Approved method of statement is available at site				
1.3	Locations of all boreholes and trial pits at site have been established by the geotechnical expert according to approved drawing and relative to existing bench mark at site				
1.4	No existing services have been found at boreholes locations				
1.5	Ground level of all boreholes and trial pits has been measured and recorded in the borehole log				
2	**Installation of Piezometer**				
2.1	The standpipe has been installed in the boreholes at each of the shaft boreholes				
2.2	The installation procedure has been performed as per the approved drawing				
2.3	The borehole has been backfilled with clean sand until the approved depth				
2.4	Gravel of approved thickness has been placed on top of the sand fill				
2.5	The porous end of the piezometer has been sealed into the gravel bed to prevent the blockage of the filter				
2.6	The borehole has been backfilled again till the top of piezometer/borehole				
2.7	A protective mounted flash cover has been provided				
3	**Observation of Groundwater Level**				
3.1	Ground water level has been measured and recorded once equilibrium was reached				
3.2	The development of all wells has been monitored properly				
3.3	All wells have been monitored properly as per the approved duration				
3.4	The fluctuation of water table has been monitored carefully				

4	**Preparing the Boreholes**				
4.1	The instrument has been installed properly in the soil				
4.2	The bottom of the borehole has been cleaned of loose materials				
4.3	The depth of all boreholes is as per the approved depth				
4.4	Water has been added to maintain the water level in the borehole at ground level, and minimize disturbance of the soil at the base of the borehole				
4.5	The instrument has been withdrawn slowly from the borehole to avoid deterioration of the soil sample taken				
4.6	Boring shell has not been driven below the approved level				

1. Soil Investigation 1.2.SPT		Borehole Number			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**General Guidelines:**				
1.1	The interval of the test has been carried out as approved interval or whenever there is a change in the soil strata				
1.2	Drilling fluid has been used to cool the bit and to remove the debris				
1.3	The rate of flow has been adjusted to ensure the core cuttings are washed up				
1.4	Casing or drilling mud has been used to support the borehole in weak or broken rock formations				
1.5	The geotechnical expert has clarified the type of each soil strata and recorded it in the borehole log				
1.6	The borehole log has been signed by all required parties				
2	**Excavation below Water table**				
2.1	Dewatering has been performed in order to lower the water table and keep the excavation dry				
2.2	Loose sand has been encountered below water table				
2.3	Excavation has only extended to the level of the water table only				
3	**Cavity Monitoring**				
3.1	Fast Drilling of the borehole has been carefully monitored and recorded in the borehole log				
3.2	Loss of water in the borehole has been carefully monitored and recorded in the borehole log				
3.3	Drop of tools has been carefully monitored and recorded in the borehole log				
4	**Common Test Procedures in all Types of Soils**				
4.1	The sampler assembly has been lowered to the approved bottom level of the borehole				
4.2	The initial penetration under the dead weight of the assembly has been recorded				
4.3	Whenever there has been a change in the soil strata or the color of revealed sand, the driller has measured and recorded the exact depth of the strata				

5	**Test Procedures in Sand**				
5.1	The N-value has been recorded as Zero when the penetration exceeded 450mm				
5.2	The sample has been driven through either 150mm or 25 blows (whatever reached first) and the result has been recorded				
5.3	The number of blows (N-value) required for an additional 300mm penetration has been recorded				
5.4	The SPT of each interval has been recorded in the borehole log				
5.5	Cavities have been carefully monitored during drilling				
5.6	The SPT of each interval has been recorded in the borehole log				
6	**Test Procedures in Rock**				
6.1	The number of blows (N-value) to achieve each 75mm penetration has been recorded				
6.2	The SPT of each interval has been recorded in the borehole log				
6.3	The test has been terminated after 100 blows if a penetration of 300mm is not achieved, and the depth has been recorded				
7	**Backfilling of Trial Pits and Boreholes**				
7.1	Boreholes have been backfilled using compacted soil or a cement based grout				
7.2	Trial Pits have been backfilled with weak concrete mix or with soil compacted by an excavator bucket				

1. Soil Investigation 1.3.Plate Load Test		Location			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Excavation and Lowering down the Plate**				
1.1	Excavation to the test level has been done as quickly as possible to minimize the effects of stress relief				
1.2	The plate has been bedded onto a leveled layer of clean dry sand to fill in any hollows in the ground				
1.3	No extraneous loose materials have been not introduced when the plate was lowered into position				
2	**Applying Load**				
2.1	The load has been applied in equal increments up to the approved bearing load				
3	**Prior to Releasing the Load on the Test Plate**				
3.1	The reference datum level and ambient air temperature have been recorded				
4	**After Releasing Load**				
4.1	The applied incremental loads have been recorded				
4.2	Settlement against time has been recorded				
4.3	The overall duration of the test including the delays as well				

1. Soil Investigation 1.4.Soil Permeability Test		Location			
Item	Activity	Compliance			Remarks
		Yes	No	NA	
1	**Test Procedures**				
1.1	Clear water has been used in the test				
1.2	Pipe casing has been carefully cleaned till the bottom of the casing				
1.3	Water has been added inside the open ended pipe casing till the approved depth				
1.4	Constant rate of gravity flow into casing of pipe has been measured				
1.5	The variation of flow rate with time has been monitored and recorded				

1. Soil Investigation 1.5.Storage of Samples		Borehole Number			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Disturbed Samples**				
1.1	Samples have been placed in polythene bags immediately after being taken				
1.2	If more than one type of soil has been discovered, each soil has been placed in a separate bag and marked accordingly for depth of sample				
1.3	The polythene bags have been labeled in accordance with the requirements of Quality System Procedure				
1.4	Samples for moisture content tests have been stored in cooler aluminum tins				
1.5	Sample for moisture content have been transferred quickly to the lab				
2	**Undisturbed Samples**				
2.1	Samples have been immediately placed in air tight containers to preserve their natural moisture content				
2.2	The containers have been labeled and wax sealed in accordance with the requirements of Quality System Procedure				
2.3	Samples have been stored in a cool place				
2.4	Samples have been quickly transferred to the lab				
3	**Rock Core Samples**				
3.1	The core has been extruded horizontally into a half section of round plastic tube				
3.2	Wooden depth spacers have been inserted into the core box between cores from successive runs				
3.3	Wooden stoppers have been fixed in the core slots to prevent damage due to movement in transit				
3.4	Samples have been labeled in accordance with the requirements of Quality System Procedure				
3.5	Samples have been covered in polythene sheet before the core box is sealed				
3.6	Core samples which will not be tested within 48 hours have been coated with paraffin wax, wrapped in polythene sheet and covered in aluminum foil				

4	**Ground Water Samples**				
4.1	Samples have been taken by bailer in one and one half liter plastic bottles				
4.2	Samples have been suitably identified and labeled in accordance with the requirements of Quality System Procedure				
4.3	Special care to avoid sample contamination has been taken				
4.4	Laboratory tests on ground water samples have been carried out within 48 hours of being received				
5	**Prior to Laboratory Testing**				
5.1	Samples have been stored at temperatures not exceeding 45ºC				
5.2	Variation of daily temperature of samples has not exceeded 20ºC				
6	**After Submitting the Soil Investigation Report**				
6.1	Samples have been stored in a temperature controlled environment for a period specified by the client				

2. **Surveying Work**					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Procedures**				
1.1	The surveying work has been carried out by a qualified surveyor				
1.2	Instruments are calibrated and suitable to start the surveying work				
1.3	Unique reference number to each survey station has been assigned by the surveyor				
1.4	Reference points have been established at site (temporary bench mark, control points, coordinates)				
1.5	Checking has been done between two survey stations to avoid errors				
1.6	Site plot has been graded approximately to the required levels				
1.7	Grids have been established as per the approved construction drawings and with reference to the datum level				
1.8	Data and calculations of the survey have been recorded and maintained neatly in the field book of the report				
1.9	Accuracy has been maintained throughout the surveying work and all errors have been rectified before proceeding with work				
1.10	Drawings have been produced to an appropriate scale with contour intervals as per specifications				

3. Secant Piling 3.1.Preparatory Work					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Preliminary Work**				
1.1	Approved method of statement is available at site				
1.2	All approved drawings are available at site				
1.3	All required equipment and machineries are available at site				
1.4	All existing services have been allocated, diverted and protected				
1.5	Safe working environment is provided at site				
2	**Working Platform for Secant Piles**				
2.1	The existing ground level has been taken as per the approved drawing				
2.2	A pre-excavation to reach the approved working platform level has been carried out				
2.3	The working platform has been made firm and dry				
2.4	The working platform has been leveled properly				
2.5	The working platform is free of storage materials				
3	**Surveying Work**				
3.1	Reference points have been established at site (temporary bench mark, control points, coordinates and platform level)				
3.2	The corners of the site plot have been marked as per the approved affection plan				
3.3	The plot and building lines have been established properly				
3.4	The shoring line has been established properly				

3. Secant Piling 3.2.Guide Beam					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Guide Beam Construction**				
1.1	Excavation has been carried out along the secant pile wall centerline until reaching the approved depth				
1.2	Formwork has been fabricated as per the approved drawing				
1.3	Steel bar diameter is as per the approved design				
1.4	Reinforcement cage has been prepared as per the approved drawings				
1.5	Concreting of the guide beam has been carried out as per the approved sequence				
2	**Guide Beam after Construction**				
2.1	The guide wall has been backfilled properly with soil				
2.2	The location of primary and secondary piles has been marked correctly as per the approved design				

3. Secant Piling 3.3.Casing and Boring					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Before Driving the Temporary Casing**				
1.1	The temporary casing used is free from significant distortion				
1.2	The temporary casing used has uniform cross section throughout the length				
1.3	The inner diameter of the temporary casing is similar to the approved pile diameter				
2	**After Driving the Temporary Casing**				
2.1	The casing has been driven in the pile center through the guide wall slot				
2.2	The casing has been driven up to the approved level				
2.3	Top level is correct				
2.4	Coordinates of pile are correct				
2.5	The center of the casing top is within the approved tolerance				
2.6	The axis of the casing is not deviating from the approved tolerance				
3	**Boring**				
3.1	Boring has been carried out by an approved calibrated drilling rig				
3.2	Boring diameter is correct				
3.3	While the casing was pushed down, excavation within the casing was simultaneously progressing				
3.4	The bottom of the pile shaft has been cleaned properly before lowering the steel cage				
3.5	The depth of drilling has been constantly monitored				
3.6	Cavities have been carefully monitored during drilling				

3. Secant Piling 3.4.Reinforcement Cage					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Checking of Reinforcement Cage**				
1.1	Yield Strength of reinforcement is as per the approved design				
1.2	Diameter of steel bars in the reinforcement cage is as per the approved design				
1.3	Number of steel bars in the reinforcement cage is as per the approved design				
1.4	Diameter of spirals is as per the approved design				
1.5	Mill Certificate has been checked				
2	**Preparation of Reinforcement Cage**				
2.1	Reinforcement cage has been made at the location nearest possible to the pile shaft				
2.2	The length of reinforcement cage has been checked				
2.3	Spirals have been spaced as per the approved design				
2.4	Spacers have been provided with assured capacity of holding the steel cage in position				
3	**Protection of Reinforcement Cage**				
3.1	Reinforcement cage has been stored above ground level				
3.2	Reinforcement cage has been covered properly with polythene sheets				
4	**Installation of Reinforcement Cage (Primary Only, Secondary Only or Both)**				
4.1	The reinforcement cage has been placed as soon as boring is completed				
4.2	The reinforcement cage has been raised vertically by a crane from its horizontal lying position				

3. Secant Piling 3.5.Concreting					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Prior to Concreting**				
1.1	Delivery note has been checked				
1.2	The transportation duration for concrete has not exceeded the approved duration				
1.3	Concrete of approved compressive strength has been used				
1.4	Workability of concrete is as per specifications				
1.5	Slump of concrete is as per specifications				
1.6	Temperature of concrete is as per specifications				
1.7	Concrete cubes for compressive strength test have been made, cured, stored and tested for all tests as per specifications				
1.8	Concrete cover has been provided as per specifications				
2	**During Concreting**				
2.1	Concreting of primary and secondary piles has been carried out alternatively as per the approved concreting sequence				
2.2	Concreting has started as soon as boring is completed and reinforcement cage is lowered				
2.3	Concrete has been poured via tremie pipe				
2.4	Concrete has been poured without any interruption				
2.5	Tremie pipe has penetrated into the placed concrete till completion of concreting				
2.6	Concrete has been placed without any interruption to avoid the formation of cold joints				
2.7	The tremie pipe has been gradually withdrawn as the concreting proceeds ensuring that the bottom 2-3 meters of the tremie pipe always remains within the previously placed concrete				
2.8	Concrete top level is at least 0.8m higher than the pile cut off level to assure sound concrete with no any contamination in the vicinity of the pile cut off level				
3	**Extracting Temporary Casing**				
3.1	The temporary casing has been extracted back from the soil as per the approved technique				

3.2	The casing has been extracted about 5-10 minutes after concreting				
3.3	Sufficient quantity of concrete has been maintained in the casing while extracting it to ensure that the pile is neither reduced in section nor contaminated				
4	**Testing of Concrete Samples**				
4.1	Concrete durability test has been conducted as per specifications				
4.2	The compressive strength of secondary piles concrete after 28 days is as per the approved strength				
4.3	The compressive strength of primary piles concrete after 28 days is as per the approved strength				

3. Secant Piling 3.6.Excavation and Anchoring				
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Excavation**				
1.1	Excavation has only started after shoring is completed				
1.2	Dewatering has been carried out to keep the excavation platform level free of water				
1.3	Easy and safe access is available				
1.4	The excavation has been carried out in stages as per the approved sequence				
1.5	Excavation in every stage has been carried out till the approved level				
2	**Working Platform for Anchors Installation**				
2.1	Existing utility lines has been located, diverted and relocated				
2.2	The working platform for the installation of the anchors has been established at the required level				
2.3	The working platform has been made firm and dry				
2.4	The working platform has been leveled properly				
2.5	The working platform is free of storage materials				
2.6	Water level has been maintained below the platform level				
3	**Installation of Anchors**				
3.1	The location of the anchor on the secant pile wall has been marked				
3.2	The boom of the anchor drilling rig has been adjusted to the required anchor inclination for the drilling process				
3.3	Enough water is available to support the drilling process				
3.4	The concrete face of primary piles at anchor location within secant pile wall has been cored				
3.5	Permanent casing of an approved length has been pushed into the core				
3.6	The anchor borehole has been drilled as per the approved technique				
3.7	Drilling tools, flushing technique and parameters of anchorage have been adjusted in case the soil is different than that mentioned in the soil report				
3.8	The anchor has been fitted with two grout pipes				

3.9	The prefabricated anchor has been installed into the core				
3.10	Primary grouting has been filled as per the approved method of statement				
3.11	A rubber squeeze packer has been pushed into the anchor hole to seal it against water leakage				
3.12	Secondary (Pressure) grouting has started 8 hours after primary grouting				
3.13	Secondary (Pressure) grouting has continued until the pressure reached the approved limit				
3.14	Post tensioning of anchors has been carried out after the grout has gained the required strength				
3.15	Anchors have been post tensioned up to the approved load				
3.16	The anchor head has been covered				
3.17	The number of anchor layers is as per the approved design				
3.18	The spacing between anchors is as per the approved design				
4	**Testing and Stressing of Anchors**				
4.1	Testing has started 7 days after the grout has gained the required strength				
4.2	Anchors have been stressed as per the approved method of statement				
4.3	The strength of anchors after 28 days is as per the approved strength				

3. Checklist for Soil Investigation 3.7.Dewatering				
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**General**				
1.1	Approved number of permanent well has been installed in the secant wall				
1.2	Approved number of temporary wells has been installed inside the secant wall area				
1.3	Well pumps have been installed inside temporary wells and switched on one week at least before the start of excavation				
1.4	Deep wells are not coinciding with piles and columns				
1.5	The temporary wells have been shortened in height as excavation proceeded to match the lowering ground level condition				
1.6	Diesel pumps and temporary channels to the nearest wells have been used to remove “standing water”				
1.7	Non return and flow control valves have been connected to each pump				
1.8	Pumps have been connected to a standby generator				
1.9	Pumps have been connected to control panels fitted with audio and visual warning alarms				
1.10	Control panels have been connected to a distribution board				
1.11	The distribution board has been connected to the generator				
2	**Controlling Ground water at the Completion of Excavation**				
2.1	French drains have been installed around the inside perimeter of the pit				
2.2	French drains have connected to the permanent wells				
2.3	Deeper French drains have been installed from the nearest permanent walls well where the perimeter drain is not effective				
2.4	When the French drain is completed, the pump has been transferred to the respective permanent wells, and temporary wells have been decommissioned				
3	**Discharge of Water**				
3.1	All water from wells and diesel pumps has been connected to a slit collection tank and to the designated discharge point				

4. Capping Beam 4.1.Preparatory Work		Pile Designation			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Preliminary Work**				
1.1	The excessive concrete above the cut off level has been chipped				
1.2	The leveling instrument is Calibrated				
2	**Formwork Fabrication**				
2.1	The flexural stress of formwork timber is as specified				
2.2	Formwork in contact with concrete has been coated with approved bond release agent				
2.3	Release agent has not been allowed to come into contact with reinforcement, anchor bolts, or existing concrete surfaces which will be poured against				
2.4	For water tight concrete structures, the formwork fixing devices have not left any holes after removing the shuttering				
2.5	Formwork has been adequately braced and tied in position to retain its shape and position before, during and after concreting				
2.6	Formwork top levels and dimensions are correct				
3	**Steel Properties**				
3.1	Only approved drawings and RC details are used at site				
3.2	Mill certificate has been verified for each delivery of steel to site				
3.3	Testing of reinforcement has been carried out as specified				
4	**Protection of Steel**				
4.1	Reinforcements have been inspected on arrival				
4.2	Reinforcements have been inspected periodically during storage				
4.3	Reinforcements have been daily covered with polythene to protect the bars from oxidation.				
4.4	No welding of reinforcement at site has been allowed				
4.5	No walking on reinforcement was allowed				

5	**Placing of Reinforcements**				
5.1	Number of bars provided is as per the approved drawings and design				
5.2	Diameter of bars provided is as per the approved drawings and design				
5.3	Overlapping, anchorage, and development length have been provided as per the code requirements				
5.4	Diameter of stirrups provided is as per the approved drawings and design				
5.5	Spacing between stirrups is as per the approved drawings and design				
5.6	Concrete cover has been provided for concrete element as per specifications				

4. Capping Beam 4.2.Before and After Concreting		Pile Designation			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Before Concreting**				
1.1	All reinforcement bars are free from oil grease and other harmful material				
1.2	The formwork and reinforcement have been sprayed with a small amount of water				
2	**At Time of Concreting**				
2.1	Delivery note has been checked				
2.2	No admixture has been added to the concrete mix at site				
2.3	Concrete slump is matching with specifications				
2.4	Temperature of concrete is not exceeding the approved limit				
2.5	Concrete samples for quality tests have been taken as per the approved method of statement				
3	**During Concreting**				
3.1	Concrete has been placed and compacted in its final position within 90 minutes of the water being added to the mix				
3.2	Concrete has not been placed in adverse weather conditions such as dust storm or heavy rain				
3.3	Concrete has been poured using a tremie pipe				
3.4	Concrete has been placed in horizontal layers of approved thickness				
3.5	Each layer of concrete has been compacted properly via mechanical vibrators				
4	**After Concreting**				
4.1	Concrete has been leveled and finished as per the levels				
4.2	Polythene sheets have been placed over the hardened fresh concrete				
4.3	Curing has been carried out as per specifications				
4.4	Surplus and waste materials have been removed after completion of work				
4.5	Repair of honeycombs and cracks have been carried out using approved materials				
5	**Testing of Concrete Samples**				
5.1	Compressive strength test has been carried out after 7 days and 28 days				

5.2	Concrete durability test has been conducted as per specifications				
6	**Removal of Formwork**				
6.1	The formwork has been removed as per the specifications				
6.2	Water proofing system has been applied as per specifications				

5. Piling Work 5.1.Preparatory Work		Pile Designation			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Preliminary Work**				
1.1	Approved drawings are available at site				
1.2	Approved method of statement is available at site				
2	**Assigning the location of the pile at site**				
2.1	Reference points have been established at site (temporary bench mark, control points, coordinates and pile platform level)				
2.2	The location of the pile has marked as per the approved drawing by the surveyor				

5. **Piling Work** **5.2.Casing and Boring**		**Pile Designation**			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Permanent Casing Properties**				
1.1	Material of the permanent casing is made of mild steel				
1.2	Permanent casing has been cleaned as per the approved method				
1.3	After cleaning the permanent casing, coal tar epoxy paint has been applied in two coats as per manufacturer specifications				
1.4	Permanent casing cut portion has been welded to the temporary casing using electric welding machine				
2	**Before Driving the Temporary/Permanent Casing Combination**				
2.1	The temporary casing used is free from significant distortion				
2.2	The temporary casing used has uniform cross section throughout the length				
2.3	The inner diameter of the temporary casing is similar to the approved pile diameter				
2.4	Thickness of permanent casing is as the approved design				
2.5	Diameter of permanent casing is as the approved design				
3	**Checking the Temporary/Permanent Casing Combination after it is driven**				
3.1	The casing has been driven up to the consolidated soil such that the permanent casing is at the bottom side as per the approved technique				
3.2	Top level of pile has been checked				
3.3	Coordinates of pile have been checked				
3.4	The center of the casing top is within the approved tolerance				
3.5	The axis of the casing is not deviating from the approved tolerance				
4	**Boring**				
4.1	Boring has been carried out by an approved calibrated drilling rig				
4.2	The bottom of the pile shaft has been cleaned using a cleaning bucket before lowering the steel cage.				

4.3	The depth of the bore has been measured after cleaning				
4.4	Boring diameter has been checked.				
4.5	Cavities have been carefully monitored during drilling				
4.6	Spoil from drilling of boreholes has been stored on site to dry out and carted away at regular intervals to the designated location				
5	**In case of Underground Water Vicinity**				
5.1	Additional water has been filled up during the drilling				
5.2	The hydro static head pressure has been at least 1m above the highest ground water table when filled boring tools are extracted				

5. Piling Work 5.3.Reinforcement Cage		Pile Designation			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Checking of the Reinforcement Cage**				
1.1	Yield Strength of reinforcement is as per the approved design				
1.2	Diameter of steel bars in the reinforcement cage is as per the approved design				
1.3	Number of steel bars in the reinforcement cage is as per the approved design				
1.4	Diameter of spirals is as per the approved design				
1.5	Mill Certificate has been checked				
2	**Preparation of the Reinforcement Cage**				
2.1	Reinforcement cage has been made at the location nearest possible to the pile shaft				
2.2	The length of the reinforcement cage has been checked				
2.3	Spirals have been spaced as per the approved design				
2.4	Spacers have been provided with assured capacity of holding the steel cage in position				
2.5	More spacers have been provided at cut off level to ensure the centralization				
2.6	PVC pipes as per the approved length have been provided to protect the reinforcement bars extending above the pile cut off level				
2.7	PVC pipes have been stopped 50mm above the cut off level to avoid inclusions into concrete				
2.8	Earthing steel to be routed through the pile has been separated from pile reinforcement				
3	**Protection of Reinforcement Cage**				
3.1	Reinforcement Cage has been stored above the ground level				
3.2	Reinforcement Cage has been covered properly with polythene sheets				
4	**Installation of Reinforcement Cage**				
4.1	The reinforcement cage has been placed immediately after boring				
4.2	The reinforcement cage has been raised vertically by a crane from its horizontal laying position				

5. Piling Work 5.4. Concreting Work		Pile Designation			
Item	Activity	Compliance			Remarks
		Yes	No	NA	
1	**Before concreting**				
1.1	Delivery note has been checked				
1.2	The transportation duration for concrete has not exceeded the approved duration				
1.3	Concrete of approved compressive strength has been used				
1.4	Workability of concrete is as per specifications				
1.5	Slump of concrete is as per specifications				
1.6	Temperature of concrete is as per specifications				
1.7	Concrete cubes for compressive strength test have been made, cured, stored and tested for all tests as per specifications				
1.8	Concrete cover has been provided as per specifications				
2	**During Concreting**				
2.1	Concreting has started as soon as possible after boring and lowering the reinforcement cage				
2.2	Concrete has been poured via concrete pump				
2.3	Concrete has been poured without any interruption to avid the formation of cold joints				
2.4	Tremie pipe has penetrated into the placed concrete till completion of concreting				
2.5	The tremie pipe has been gradually withdrawn as the concreting proceeds ensuring that the bottom 2-3 meters of the tremie pipe always remains within the previously placed concrete				
2.6	Concrete top level is at least 0.8m higher than the pile cut off level to assure sound concrete with no any contamination in the vicinity of the pile cut off level				
2.7	The reinforcement cage has been kept at required level hanging to the steel casing				
2.8	No pile has been produced within 3 x Pile Diameter center to center distance from a previously cast pile within 24 hours				
2.9	The method of construction for all piles have been the same				

3	**After Concreting**				
3.1	The empty upper portion above the cutoff level has been backfilled with bored material				
3.2	Pile head has been cured properly				
4	**Extracting Temporary Casing**				
4.1	The temporary casing has been extracted back from the soil as per the approved technique				
4.2	The casing has been extracted about 5-10 minutes after concreting				
4.3	The joint between temporary and permanent casing has been cut at the pile platform level				
4.4	The joint between temporary and permanent casing has been cut as per the approved technique				
4.5	Sufficient quantity of concrete has been maintained in the casing while extracting it to ensure that the pile is neither reduced in section nor contaminated				
5	**Pile Construction Report**				
5.1	Pile construction report has been kept for each pile				
5.2	Pile records have been filled in with the data of casting, pile reference numbers, coordinates, toe level, cut off level, top level of concrete, the volume of the concrete				
6	**Testing of concrete samples**				
6.1	Compressive strength test has been carried out after 7 days and 28 days				
6.2	Concrete durability test will be conducted as per specifications				

5. Piling Work 5.5. Testing of Piles		Pile Designation			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Pile Testing**				
1.1	Preliminary Pile Test has been carried out as per specifications				
1.2	Working Pile Test has been carried out as per specifications				
1.3	Integrity Test has been carried out for all piles as per specifications				
1.4	Enough concrete cores have been randomly taken to evaluate possible contamination within the concrete				

5. Piling Work 5.6. Pile Head Breaking		Pile Designation			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Pile Head Breaking**				
1.1	Curing of the pile has been completed				
1.2	The pile is at least 7 days old before head breaking				
1.3	Cut off level for all piles has been marked correctly				
1.4	The lower part has been broken using a jack hammer				
1.5	No direct hitting to rebar by machine has occurred				
1.6	The top small part has been broken up to the approved cut off level				
1.7	The exposed pile head has been protected with suitable covering sheet				
1.8	The exposed pile head rebar has been protected with suitable covering sheet				
1.9	The final position of reinforcement cage has been checked following the pile head breaking activity				

5. Piling Work 5.6. Pile Head Treatment					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Blinding**				
1.1	Blinding of approved thickness has been provided after cutting the pile head				
1.2	The pile head surface has been made clean and dust free				
2	**Vinyl Easter Based Gasket**				
2.1	Approved swelling vinyl easter based gasket has been stretched until tout and then wrapped around the pile head				
2.2	The two ends of the swelling vinyl easter based gasket been overlapped and stapled				
2.3	The swelling vinyl easter based gasket has been positioned as per the approved sequence				
3	**Protection Screed**				
3.1	Protection screed of approved thickness has been provided above blinding				
3.2	Protection screed has been cured for 7 days at least				
3.3	Screed fillet has been provided to protect the swelling vinyl easter based gasket				
4	**Grouting**				
4.1	The substrate surface has been made clean and dust free				
4.2	Presoaking for a minimum of 2 hours prior to grouting has been done				
4.3	Free water has been removed before grouting				
4.4	Formwork has been prepared to achieve the required grout thickness				
4.5	Formwork has been made leak proof using foam rubber strip or mastic sealant beneath the constructed formwork and between joints				
4.6	Grout has been mixed as per the approved method of statement				
4.7	Approved grout has been used				
4.8	Sufficient curing for 7 days at least has been carried out for the pile head				

6. Excavation					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Prior to Excavation**				
1.1	Approved shop drawings showing the foundation parameters are all available at site				
1.2	Reference points have been established at site (temporary bench mark, control points and coordinates)				
1.3	All existing services have been allocated, diverted and protected				
1.4	Existing ground level has been checked and recorded				
1.5	Dewatering requirements have been checked				
1.6	Adequate barriers, warning tapes, sufficient lightning and safe working environment are provided at site				
1.7	All rubbish and unacceptable materials within the site plot of have been cleared except the area to be excavated				
2	**During Excavation**				
2.1	Dewatering has been carried out properly when water is encountered				
2.2	Excavation has not exceeded the established level				
2.3	Lean concrete has been used to fill the excavation beyond the excavation level				
2.4	The slope of excavation has been constructed as per the soil investigation report recommendations				
2.5	The last portion of excavation has been carried out manually up to the required level to minimize disturbance of the soil below the required sub grade level				
2.6	Excavated material has been shift and stockpiled in approved locations				
2.7	Necessary temporary supports (stepping, sheeting, shoring, bracing) have been provided as per the recommendation of the soil investigation report				

7. Backfilling					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Procedures**				
1.1	Compaction of excavation bottom has been carried out as per specifications				
1.2	Field density test has been carried out upon the completion of excavation bottom compaction				
1.3	Only approved materials are used for backfilling				
1.4	Backfilling has been carried out in layers of approved thickness until reaching the approved level				
1.5	Backfilled layers have been compacted up to the approved degree of compaction				
1.6	Confined spaces have been compacted manually				
1.7	The level of top backfilled layer is correct				
1.8	The top backfilled layer is smooth and properly sloped				
1.9	Compaction test has been carried out as per the approved sequence				
1.10	Moisture content has been checked and found to be acceptable				

8. Blinding Work					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Preliminary Work**				
1.1	The leveling instrument is calibrated				
1.2	Bench marks and control points have been established at site				
1.3	Grid lines have been established as per the approved drawings				
1.4	Formation level has been checked				
1.5	Compaction test below foundations has been approved				
1.6	Grid lines have been established as per the approved drawings				
2	**Before Concreting**				
2.1	Formwork has been constructed properly				
2.2	Formwork top level and dimensions are as per the approved level				
2.3	The area has been wetted with water				
2.4	Polyethylene sheet has been laid on earth surface before placing concrete				
3	**At time of Concreting**				
3.1	Delivery note has been checked				
3.2	Concrete slump is matching with specifications				
3.3	Temperature of concrete is not exceeding the approved limit				
3.3	Concrete samples for quality tests have been taken as per the approved method of statement				
4	**During Concreting**				
4.1	Concrete has been placed and compacted in its final position within 90 minutes from adding water to the mix				
4.2	Blinding concrete has been spread using rakes and shovels, and leveled with an aluminum straight edge				
4.3	Concrete has been placed in horizontal layers of approved thickness				
4.4	Concrete has not fallen freely from a height more than 1.5 meters				
4.5	Concrete has been compacted properly with mechanical vibrators				

5	**After Concreting**				
5.1	Polythene sheets have been placed over the hardened blinding				
5.2	Curing has been carried as per specifications				
5.3	Water proofing system has been applied as per specifications and after curing is completed				
6	**Testing of Concrete Samples**				
6.1	Compressive strength test has been carried out after 7 days and 28 days				
6.2	Concrete durability test will be conducted as per specifications				

9. Formwork and Shuttering 9.1. Fabrication					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Procedures**				
1.1	Flexural stress of formwork timber (σ_b) is as specified				
1.2	Formwork in contact with concrete has been coated with approved bond release agent				
1.3	Release agent has not been allowed to come into contact with reinforcement, anchor bolts, or existing concrete surfaces which will be poured against				
1.4	All forms for reuse have been cleaned, repaired and have been stored carefully				
1.5	All holes left from fixing devices have been closed with plastic plugs and epoxy mortar				

9. Formwork and Shuttering 9.2. Raft Formwork					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Procedures**				
1.1	Foundation layout has been marked over the protection screed				
1.2	Formwork has been placed adjacent to each other and assembled properly to match with layout marking				
1.3	Proper supports have been provided to ensure verticality, dimensions and alignment of the formwork				
1.4	Formwork has been made tight and all joints have been sealed with joint tapes				
1.5	Permanent shuttering has been provided as per the approved sequence				
1.6	Top level, dimensions and opening locations are correct				

9. Formwork and Shuttering 9.3. Slab Formwork					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Procedures**				
1.1	Inspection for reinforcement, cover blocks and inserts has been carried before the formwork activity has started				
1.2	Supporting system for roof slab has been constructed as per the approved drawings				
1.3	Formwork in contact with concrete has been coated with approved bond release agent				
1.4	Release agent has not been allowed to come into contact with reinforcement, anchor bolts, or existing concrete surfaces which will be poured against				
1.5	Proper supports have been provided to ensure verticality, dimensions and alignment of the formwork				
1.6	Slab openings have been located as per approved drawings				
1.7	Construction joints have been made with properly constructed stop boards that are firmly fixed and holed where necessary				
1.8	Levels, dimensions and opening locations are correct				

9. Formwork and Shuttering 9.4. Wall Formwork					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Procedures**				
1.1	Inspection for reinforcement, cover blocks and inserts has been carried before the formwork activity has started				
1.2	Installation of formwork has only started after completion of reinforcement fixing				
1.3	Proper working platform with access and hand railing has been provided for safe working environment				
1.4	All conduits and inserts have been provided as per the approved drawings				
1.5	Timber and wailers have been placed in position				
1.6	Timber and wailers have been connected with flange claw assemble				
1.7	Plywood and timber beams have been fixed with nails				
1.8	Enough screws with appropriate length have been used to fix the plywood to timber beams				
1.9	Enough screws with appropriate length have been used to fix the plywood to timber beams				
1.10	Wooden planks have been fixed at top and bottom sides of shutters				
1.11	Slots have been provided in the top plank for lifting bracket location				
1.12	Tie rods with wing nut and waler plate arrangement with PVC pipe sleeve plastic cones have been used to keep the forms in correct apart distance				
1.13	Construction joints have been made with properly constructed stop boards that are firmly fixed and holed where necessary				
1.14	Complete fixing of reinforcement, cover blocks fixing, inserts, pipe sleeves, and installation of water stops have been ensured before closing the other side of formwork				
1.15	Top level of formwork is correct				
1.16	Levels, dimensions and opening locations are correct				

9. Formwork and Shuttering 9.5. Column Formwork					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Procedures**				
1.1	Inspection for reinforcement, cover blocks and inserts has been carried before the formwork activity has started				
1.2	Proper working platforms with access and hand railing have been provided for safe working environment				
1.3	Kickers have been provided as per approved drawings				
1.4	Clamps have been fixed at appropriate distance to maintain the column size				
1.5	Props have been supported to the clamps on all sides				
1.6	Props have been adjusted till the achievement of proper plumb to the columns				
1.7	Proper supports have been provided to ensure verticality, dimensions and alignment of the formwork				
1.8	Plumbing has been adjusted using a string line				
1.9	Levels, dimensions and opening locations are correct				

9. Formwork and Shuttering **9.6. Formwork Removal**					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Procedures**				
1.1	Vertical forms have remained in place for a minimum of 24 hours after placing of concrete prior to removal				
1.2	Soffit of slab formwork has been removed after 7 days				
1.3	Props to slab formwork has been removed after 14 days				
1.4	Soffit of beam formwork has been removed after 14 days				
1.5	Prop formwork under beam has been removed after 21 days				

10. Rebar Work					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Steel Properties**				
1.1	Only approved drawings and RC details are used				
1.2	Mill certificate has been verified for each delivery of steel to site				
1.3	Testing of reinforcement has been carried out as specified				
2	**Protection of Steel**				
2.1	Reinforcements have been inspected on arrival				
2.2	Reinforcements have been inspected periodically during storage				
2.3	Reinforcements have been daily covered with polythene to protect the bars from oxidation				
2.4	No welding of reinforcement at site has been allowed				
2.5	No walking on reinforcement was allowed				
3	**Placing Reinforcement**				
3.1	Number of bars provided is as per the approved drawings and design				
3.2	Diameter of bars provided is as per the approved drawings and design				
3.3	Overlapping, anchorage, and development length have been provided as per the code requirements				
3.4	Reinforcement bars have not been connected to earthing system				
3.5	Separate earthing mesh has been provided				
3.6	Diameter of stirrups provided is as per the approved drawings and design				
3.7	Spacing between stirrups is as per the approved drawings and design				
3.8	Concrete cover has been provided for concrete element as per specification				
3.9	Reinforcement around openings has been provided as per the approved RC details				
4	**Before Concreting**				
4.1	All reinforcement bars are free from oil grease and other harmful material				
4.2	Reinforcement steel has been cleared off all concrete chips				

11. Screeding Work					
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Preliminary Work**				
1.1	The leveling instrument is calibrated				
1.2	Bench marks and control points have been established at site				
1.3	Grid lines have been established as per the approved drawings				
1.4	Sub panels not exceeding $12m^2$ and strips not exceeding 4.5m in width have been prepared for screed pouring				
1.5	Locations of expansion and construction joints have been marked as per the approved drawings				
1.6	The surface has been chipped out and cleaned from loose concrete or dry mortar				
1.7	The surface has been chipped out and cleaned from loose concrete or dry mortar				
2	**Before Screeding**				
2.1	Formwork has been provided all around openings at floor slab				
2.2	Cement slurry with bonding agent has been applied over the sub panel floor area just before placing the screed				
3	**At time of Screeding**				
3.1	Delivery note has been checked				
3.2	Temperature of screed is not exceeding the approved limit				
3.3	Screed samples for quality tests have been taken as per the approved method of statement				
4	**During Screeding**				
4.1	Screed has been placed and compacted in its final position within 90 minutes from adding water to the mix				
4.2	Screed concrete has been poured in alternate sub panels				
4.3	Screed has been vibrated properly				
4.4	Final screed level is matching with approved drawings				
4.5	Finishing of top surface has been done as per specifications				

5	**After Screeding**				
5.1	Polythene sheets have been placed over the screed				
5.2	Curing has been carried as per specifications				
6	**Testing of Screed Samples**				
6.1	Compressive strength test has been carried out after 7 days and 28 days				

12. RC Concreting Work 12.1. Preparatory Work		Pile Designation			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Preparatory Work**				
1.1	Trial mix prepared at the batching plant is approved				
2	**Equipment and Hand Tools at Site**				
2.1	Sufficient hand tools to place concrete are available				
2.2	Concrete pump is available				
2.3	Stand pump is available before concreting the raft and slabs				
2.4	Stand by vibrator is available				
2.5	Fuel for vibrator is available				
2.6	Equipment to screed and finish concrete is available				
3	**Preliminary Work**				
1.1	The leveling instrument is calibrated				
1.2	Bench marks and control points have been established at site				
1.3	Locations of expansion and construction joints have been made as per the approved drawings				
1.4	Construction joints have been made at locations away from maximum moment and where the shear is low				
1.5	Approved PVC water bars have been installed at different construction joints as per the approved drawings				
1.6	MEP clearance has been obtained				

12. RC Concreting Work 12.2. During and after Concreting		Pile Designation			
Item	**Activity**	**Compliance**			**Remarks**
		Yes	**No**	**NA**	
1	**Before Concreting**				
1.1	The form and reinforcement have been sprayed with small amount of water				
1.2	For slabs thicker than 900mm, thermocouples have been provided to control and record the variation of concrete temperature during curing				
2	**At time of Concreting**				
2.1	Delivery note has been checked				
2.2	No admixture has been added to the concrete mix at site				
2.3	Concrete slump is matching with specifications				
2.4	Temperature of concrete is not exceeding the approved limit				
2.5	Concrete samples for quality tests have been taken as per the approved method of statement				
3	**During Concreting**				
3.1	Concrete has been placed and compacted in its final position within 90 minutes from adding water to the mix				
3.2	Concrete has not been placed in adverse weather conditions such as dust storm or heavy rain				
3.3	Concrete has been poured via a tremie pipe				
3.4	Concrete has been placed in horizontal layers of approved thickness				
3.5	No cold joint has been formed				
3.6	Concrete has not fallen freely from a height more than 1.5 meters				
3.7	Each layer of concrete has been compacted properly via mechanical vibrators				
4	**Concreting at the Interface between Old and New Concrete at Construction Joints**				
4.1	Surface of cast concrete has been cleaned from all defective concrete and dirt				
4.2	Surface of cast concrete has been roughened by chipping, hammering or other techniques				

4.3	Surface of cast concrete has been wetted and saturated with water				
4.4	Excess water has been removed from the surface of horizontal joints before concrete sets				
4.5	Bonding agent has been spread over the surface of cast concrete				
4.5	Concrete has been cast continuously up to the construction or expansion joints				
5	**After Concreting**				
5.1	Concrete has been leveled and finished as per the approved level				
5.2	Polythene sheets have been placed over the hardened fresh concrete				
5.3	Curing has been carried out as per specifications				
5.4	Surplus and waste materials have been removed after completion of work				
5.5	Repair of honeycombs and cracks have been carried out using approved materials				
6	**Observations**				
6.1	Honeycomb has occurred				
6.2	Segregation has occurred				
7	**Testing of Screed Samples**				
7.1	Compressive strength test has been carried out after 7 days and 28 days				
7.2	Concrete durability test will be conducted as per specifications				

Vita

Ali Yousef Al Amouri was born on February, 1986, in Kuwait, Kuwait. He is from Jordan. He studied in Abu Dhabi during high school, and achieved the second ranking over all UAE schools in 2004 High School exams. Accordingly, he was sponsored by Abu Dhabi Water and Electricity Authority (ADWEA) to study Civil Engineering in the American University of Sharjah, and in return work in ADWEA for 8 years after graduation. Ali received his Bachelor degree in 2008 with a 4.0GPA, and Summa Cum Laude distinction.

Ali joined Abu Dhabi Transmission and Despatch Company (TRANSCO) in 2008 as a civil engineer in Projects Division. Ali started his master degree at the American University of Sharjah in 2010, and was awarded a Master degree in Civil Engineering Science in 2012 with a 4.0GPA.