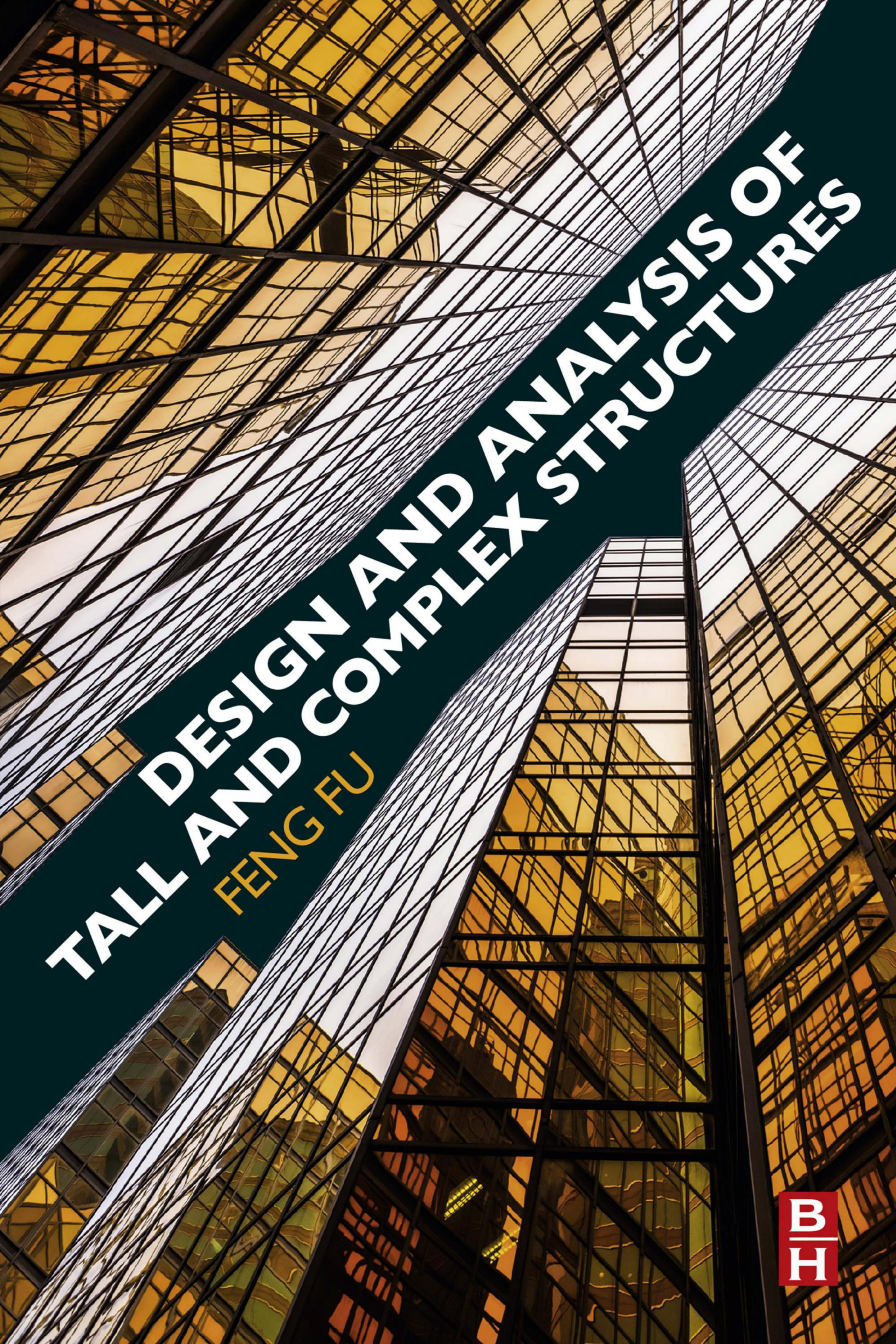
DESIGN AND ANALYSIS OF
TALL AND COMPLEX STRUCTURES
FENG FU

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CONTENTS

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He is currently a lecturer in Structural Engineering in City, University of London, following his work at the University of Bradford. Before his academic careers, he has worked for several world leading consultancy companies and was involved in the design of several prestigious construction projects worldwide. He worked in the advanced analysis team in the WSP Group Ltd., London, following his work as a structural engineer in Waterman Group Ltd., London. Before starting his PhD in the UK, he also worked as a structural engineer for one of the best design companies in China, Beijing Institute of Architectural Design (Group) Co., Ltd. In this book, the author takes up for discussion seven tall building projects designed by these three companies.

During his professional engineering experience, he worked with many world-class architects and designed and analyzed all kinds of complex and challenging structures around the world, such as tall buildings, long-span space structures, and bridges.

Dr. Fu has extensive research experience in the area of progressive collapse, structural fire analysis, and blast analysis of tall buildings and long-span structures. He also conducted his research in the area of tensegrity structures and steel and composite structures. His currently research interests also include fluid and structure interaction of offshore structures such as offshore wind turbine and oil platform. He is specialized in advanced numerical modeling and has developed several modeling programs using different programming languages. He is currently working on a €1,438,020.85 research project funded by the European Commission Research Fund for Coal and Steel (RFCS), project number STABFI 751583, "Steel cladding systems for stabilization of steel buildings in fire." He is a PhD supervisor. He is also a peer reviewer for more than 22 international journals and 4 books of internationally renowned publishers. He has published more than 30 technical papers and two textbooks:

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"Structural Analysis and Design to Prevent Disproportionate Collapse, March 10, 2016 Forthcoming by CRC Press, ISBN 9781498706797."

PREFACE

In the past decades, with the development of construction technology and computer analysis methods, extensively tall and complex structures, such as Burj Khalifa, Taipei 101, the Bird's Nest, etc., have been built. The demand from the construction market requires that a civil engineer has the ability to design and analyze these challenging structures. The author has been working in the industry and the academia for many years. As a professional engineer, I have been working on a variety of tall and complex structures, such as the Shard, the tallest building in West Europe, the scheme design of Nakheel Tower (1-km-tall building, designed to be the tallest building in the world, project is on hold), etc. I noticed that most practicing engineers as well as current college or university students lack the knowledge on the design and analysis of tall and complex structures. Therefore, a textbook conveying the knowledge of design and analysis of this type of structures is increasingly important.

The aim of this book is to provide engineers and students with knowledge on the design principles and analysis methods of tall and complex structures, the effective way to model these types of structures using the conventional commercial software, and the theories and design principles that underpin the relevant analysis. This book has been written to serve as a textbook for college and university students and also as a reference book for practicing engineers.

This book discuses almost all types of tall building systems and other complex structural forms such as long-span structures, tensile structures, tensegrity structures, offshore oil platforms, offshore wind turbine, etc. It covers the structural design problems, such as lateral stability analysis, earthquake analysis, wind engineering, foundation design for tall buildings, nonlinear geometric analysis and form finding method for tensile structures and tensegrity, multiphysics modeling for fire safety, fluid structure interaction for offshore structures, etc.

Another feature of this book is that most of the design principles and analysis methods are demonstrated using case studies and modeling examples of existing prestigious projects around the world, such as the tallest buildings in the world: the Jeddah Tower (the tallest building in the world), the Twin Towers (in New York, demolished by 9/11 attack), 432 Parke Avenue in New York (the tallest residential building in the world), Shanghai Tower

(632-m tall, the tallest mega-frame building in the world), Guangzhou International Finance Center (432-m tall, the tallest diagrid building in the world), China Zun Tower (528 m), Petronas Tower (378 m), Georgia Dome (the first tensegrity dome in the world), Allianz Tower in Milan, etc.

This book also introduces several projects designed by the author's previous employers, which include The Shard (the tallest building in Western Europe), One World Trade Center (541.3-m tall, the tallest building in the western hemisphere), 432 Park Avenue (425.5-m tall), Hearst Tower (182-m tall) by WSP group, NEO Bankside Tower by Waterman Group, Poly International Plaza, and China Zun Tower (528-m tall) by Beijing Institute of Architectural Design Group Co., Ltd.

The book also emphasizes on the features of major commercial programs used in the current design practice (such as SAP2000, ETABS, Abaqus, ANSYS, Rhino, Revit, and AutoCAD), helping the engineers to understand an effective way to model complex structures.

One of the highlights of this book is the introduction of the cutting edge technology used in the current construction industry, the Parametric Modeling and Building Information Modeling method.

In addition, this book also incorporates a vast number of project photographs across the world. Majority of them were taken by the author, which help the reader to have a better understanding of the topics introduced in this book.

Feng Fu

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Some of the models used in this book are built by me and some are based on the models set up by the MSc and final year students under my supervision. I am very appreciative of my final year and MSc students: Mrs. Hasine Rezaee, Mr. Ervin Duka, Mr. Shahzeb Khan, Mr. Elhashmi Galeisa, Mr. Jorge Caro Yika. Mr. Enammul Miah, Mr. Shariq Naqvi, Mr. Wing Sing Tsang, and Mr. Mauro Jorge Campos.

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CHAPTER ONE

Introduction

1.1 AIMS AND SCOPE

In the past decade, an increasing number of tall buildings and complex structures such as the Burj Khalifa in Dubai, the Bird's Nest Stadium in Beijing, and the London Aquatic Center were built. These projects demand that the civil and structural engineers have the ability to handle the increasing difficulty in designing even more complicated projects such as tall buildings or structures with complex geometries. The effective design of these types of structures is based on a clear understanding of the behavior of the structures, relevant analysis theory and method, knowledge of effective numerical modeling software, and of specific design principles. However, it has been noticed that, in the construction industry, most structural engineers find themselves lacking the relevant knowledge. In most universities, the basic design modules are taught to students. However, it is hard to find a module that has the systematic introduction on the design and analysis of tall and complex structures. Therefore, a book in this area is imperative.

The main purpose of this book is to provide students and design practitioners detailed knowledge of advanced design and analysis methods of tall and complex structures, such as tall buildings, long-span structures, tensile structures, tensegrity structures, offshore oil platforms, and offshore wind turbines. It also introduces major modeling programs (such as SAP2000 [1], ETABS [2], Abaqus [3], and ANSYS [4]) and the software to generate the structural geometry (Rhino [5], Revit [6], AutoCAD [7], and MicroStation [8]) used in structural design practice. It demonstrates the design knowledge through prestigious projects across the world, such as Jeddah Tower (1008-m tall), Shanghai Tower (632-m tall), etc. It also demonstrates the latest technologies used in design such as the BIM (building information modeling) and parametric modeling technique.

Design and Analysis of Tall and Complex Structures
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1.2 THE MAIN DESIGN ISSUES OF TALL AND COMPLEX STRUCTURES

The major design issues of tall buildings are the lateral stability system and the gravity system for the superstructure as well as for the foundation design. The primary design target is to provide sufficient stiffness to tall buildings to resist lateral or gravity loadings. It should also be ensured that a correct foundation be selected, which can securely transmit loads from the superstructure to the soil. Chapters 2–5 of the book cover such design issues.

For complex structures, one of the major design issues is how to set up a complex structural geometry. Due to the complexity, the innovative technology such as the BIM and parametric modeling techniques become the key solutions to it. Another design issue of complex structures is the capability to analyze complex structural problems such as fluid and structure interaction, geometrical nonlinear analysis, form finding, multiphysics modeling, etc. In addition, the designers should also understand the underpin analysis theories of some special types of structures, such as the offshore structures and tensile structures, as their design and analysis methods are different from the conventional structures. All the design issues for complex structures are covered in Chapters 6–8 of this book.

1.3 STRUCTURE OF THE BOOK

Chapter 1 is the introduction to the book.

Chapter 2 gives the fundamental principles of designing tall buildings. It begins with a history of world's tall buildings, then a brief description is given of the major loadings that need to be considered during the design stage. It is followed by a detailed introduction to the floor system and the vertical support system. The next section starts with a detailed discussion of the earthquake design, covering the structural analysis methods for evaluating earthquakes, energy dissipation, structural regularity, and measures to reduce earthquake response, such as using damping systems. The following section provides information on wind loading, enlightening the readers on the fundamentals of wind loadings, enlisting the major design requirements for the occupant comfort design. After this section, discussions on progressive collapse, fire safety design, and blast design are made. In the latter part of this chapter, the foundation design, construction method, as well as the shortening effect and cladding system are dealt with. Chapter 2 is the most important chapter for the topic of tall buildings, as it attempts to cover all the major design issues of tall buildings.

A detailed introduction of the lateral stability system is made in Chapters 3–5. Chapter 3 covers the shear wall and core wall system, the outrigger and belt truss system, and the buttressed core system. The case study of the tall building projects such as the Shard and the Jeddah Tower is discussed in this chapter. Chapter 4 introduces one of the widely used lateral stability systems, the Tube system. It gives a detailed description on the different tube systems, such as the tube-in-tube, framed tube, braced tube, bundle tube, and the hybrid tube system. The case studies of the Twin Towers and One World Trade Center (the replacement of the Twin Towers) are made. In the final part of Chapter 4, two super-slender buildings are considered as a case study: 432 Parke Avenue (425.5-m tall) and Allianz Tower. Chapter 5 focuses on the bracing system, diagrid system, and the mega-frame system. Different types of bracing systems, such as the concentric bracing and the eccentric bracing are introduced, followed by the three-dimensional (3D) truss system used in tall buildings. The diagrid system is described in great detail. A case study of Guangzhou International Finance Centre (432-m tall), the tallest diagrid structure in the world, is made. The latter part of the chapter deals with the moment resisting frames and mega-frame structures. The mega-frame structure is demonstrated using two case studies, the Shanghai Tower (632-m tall, the tallest mega-frame building in the world) and the China Zun Tower (528-m tall).

Chapter 6 deals with complex structures. It gives detailed examples of the existing complex structures in the world, which covers the opera house, train station, hotel, aquatic center, and cable car supports. This gives the readers a clear picture of the different types of complex structures. The major design considerations for complex structures, such as the design of space structure, arches, as well as support and connections are made. Real project examples are also given. It is followed by a brief introduction of the analysis methods for complex structures. The highlights of this chapter are the introduction of the BIM and parametric modeling, which are the cutting-edge technologies used in the construction industry. In the final part of this chapter, there is a brief description of the development of the API (application program interface) and GUI (graphical user interface).

Chapter 7 presents the design and analysis of two special types of structures: tensile structures and tensegrity structures. Detailed analysis theories for form finding and static analysis, such as the nonlinear finite element method, force density method, dynamic relaxation method are introduced, including some important design issues such as wrinkling of the membrane. The way to model membrane structure in ANSYS and Abaqus is also demonstrated. The analysis methods for tensegrity are demonstrated using the Georgia Dome as a prototype, which is modeled using the design-orientated program SAP2000.

Chapter 8 introduces the design and analysis of offshore structures, including offshore oil platforms and offshore wind turbines. The incidents of offshore structures are demonstrated. Detailed introduction of major loadings and design guidelines are made. The wave theories to model the fluid and structure interaction are also elaborated. In particular, the design guidelines on offshore structures are described. This chapter is endeavored to give the reader a fundamental understanding of how to design and analyze offshore structures.

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CHAPTER TWO

Fundamentals of Tall Building Design

Abstract

In this chapter, the fundamental of tall building design is explained. It starts with the introduction of the history of the tall buildings. Then it explains different loadings, floor systems, and vertical support systems. It also has detailed introduction on earthquake-resistant design, wind design, progressive collapse design, fire safety design, blast design, and foundation design. They are followed by further description on construction methods of tall buildings. It also covers special topics on shortening analysis and cladding systems for tall buildings.

Keywords: Earthquake, Fire, Blast, Progressive collapse, Fire, Foundation, Wind, Shortening analysis, Cladding

2.1 INTRODUCTION

In this chapter, the fundamental design for tall buildings is explained. In tall building design, the primary considerations are the effect of wind, seismic, and correspondent lateral stability system to resist wind or seismic loading. Owing to the height of the tall buildings, wind-induced vibration becomes another critical design issue, which affects the occupants' comfort level. Therefore, it also needs to be addressed in the design.

Foundation design is also another major consideration. Except that, considerations on gravity loading and correspondent flooring system are also essential.

After the incidents such as collapse of Twin Towers caused by the hijacked aircraft crashed into them on September 11, 2001 (NIST NCSTAR, [1]), collapse of building 7 World Trade Center (NIST NCSTAR 1A, [2]) due to fire set by the debris of Twin Towers on September 11, 2001, and blast-induced partial collapse of the Alfred P. Murrah Federal Building in Oklahoma City, 1995 (ODCM report, [3]), design a tall building to resist extreme loading such as fire and blast loading has become an increasingly important design task. How to design a tall building to prevent it from disproportionate collapse is one of the top priorities for a design engineer.

Design and Analysis of Tall and Complex Structures
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In this chapter, the above design issues will be explained in detail. It also covers other areas in tall building design, such as cladding design and shortening analysis. This is to give the readers all the important knowledge necessary for tall building design. The most important issue such as the lateral stability system of the tall buildings will be discussed systematically in the following chapters.

2.2 HISTORY OF TALL BUILDINGS

The definition of tall building is ambiguous in design practice. We normally define building below eight stories as low-rise buildings. For building with 8 and 20 stories it is called mediate-rise building. For building over 20 stories it is called tall building. However, with the fast development of modern construction technology, increasing number of supertall buildings was built and is going to be built. Therefore, the above definition is not accurate. The Council on Tall Buildings and Urban Habitat (CTBUH) has released a study on the world's 20 tallest buildings projected to be built by 2020 (Table 2.1). From this table, it can be seen that the definition of tall building is based on the comparisons to the height of other buildings, at present, people can hardly call a 20-story building as a tall building any more, if it is to be compared to the buildings listed in Table 2.1. However, from a structural engineer' point of view, the first priority in design consideration for a tall building would be the lateral stability system, as its design and structural analysis are mainly affected by the lateral loads such as wind or earthquake. Therefore, we can define a structure as a tall building when its lateral stability is the first priority in the design.

Before the skyscraper was built in Chicago in 1885, the world's tallest building was always a church or a cathedral. As shown in Fig. 2.1, the Ulm Minster was built in 1890 and it was the world's tallest building with a height of 161.53 m until 1901 when 167 m Philadelphia City Hall was built in 1901. Except Ulm Minster, the Eiffel Tower which was built in 1889 with a height of 324 m is the tallest structure in the world (Fig. 2.2).

In the 19th century, a new kind of structure was developed in Chicago using an iron or steel to bear the building's weight. The taller buildings of these kind are called skyscrapers. In 1885, Chicago's Home Insurance Building, is the world's first skyscraper with a height of 42 m. After that there were increasing numbers of tall building that were built, this includes the Empire State Building built in 1931 (Fig. 2.3), which has a 102-story skyscraper with

Table 2.1 List of World's 20 Tallest Buildings Projected to be Built by 2020 (The Global Tall Building Database of Council on Tall Buildings and Urban Habitat)

1.	Kingdom Tower, Jeddah	1000 m
2.	Burj Khalifa, Dubai	828 m
3.	Ping An Finance Center, Shenzhen	660 m
4.	Seoul Light DMC Tower, Seoul	640 m
5.	Signature Tower, Jakarta	638 m
6.	Shanghai Tower, Shanghai	632 m
7.	Wuhan Greenland Center, Wuhan	606 m
8.	Makkah Royal Clock Tower Hotel, Makkah	601 m
9.	Goldin Finance 117, Tian Jin	597 m
10.	Lotte World Tower, Seoul	555 m
11.	Doha Convention Center and Tower, Doha	551 m
12.	One World Trade Center, New York City	541 m
13.	Chow Tai Fook Guangzhou, Guangzhou	530 m
14.	Tianjin Chow Tai Fook Binhai Center, Tian Jin	530 m
15.	Dalian Greenland Center, Dalian	518 m
16.	Pentominium, Dubai	516 m
17.	Busan Lotte Town Tower, Busan	510 m
18.	Taipei 101, Taipei	508 m
19.	Kaisa Feng Long Centre, Kaisa	500 m
20.	Shanghai WFC, Shanghai	492 m

a roof height of 381 m. In 1974, Sear Tower was built in Chicago, which has a 108-story building with a roof height of 442 m. It was the tallest building in the world until it was surpassed by the Petronas Towers built in 1996, which are twin towers with the height of 451.9 m for both. After the millennium, with the fast development of computer modeling technique and new construction technologies, increasingly tall buildings were built around the world. In 2004, Taipei 101 with a height of 509.2 m became the tallest building in the world. However, it was surpassed by the Burj Khalifa in 2010, with an amazing height of 829.8 m. Table 2.2 has the list of current tall buildings projected to be built by 2020, given by CTBUH. It shows the height, the material, and the usage for the buildings.

2.3 THE LATERAL STABILITY SYSTEM FOR TALL BUILDINGS

As discussed in earlier sections, the key feature of the tall building from a structural engineer's point of view is its lateral stability structural system.

Fig. 2.1 Ulm Minster with height of 161.53 m, Ulm, Germany. *(Photo taken by the author.)*

The development of structural system for tall buildings can be divided into certain stages in the history. They are developed from rigid frame, bracing system, shear wall system to tube, core-outrigger, and diagrid structures.

Moment resisting frame (MRF) system (also be called as rigid frame) is a system where the beams and columns are rigidly connected to provide lateral resistance. Bracings and shear walls (primarily core walls) are used widely as the lateral resisting system in tall buildings, they are also widely used in different type of low-rise buildings. For tall buildings, purely rely on the core wall system to resist lateral loading is not sufficient, which will result in a very thick wall thickness. Therefore, it is quite common that firm horizontal members (such as outriggers) are used to connect the main core to the exterior columns. Outrigger structures have been used widely in tall building, the famous project examples are Shard in London (310-m tall) and Shanghai Tower (632-m tall).

Fig. 2.2 Eiffel Tower with a height of 324 m, Paris, France. *(Photo taken by the author.)*

Tube structures are one of the major lateral resisting systems in tall buildings. A building is designed to act like a hollow cylinder, cantilevered perpendicular to the ground in order to resist lateral loads. There are different types of tube structures, such as framed tubes, braced tubes, tube-in-tube and bundle tube. The famous examples for tube structures are John Hancock Centre (344-m tall) and Willis Tower (442-m tall).

Diagrid system is another lateral stability system. With their structural efficiency as a adapted version of the bracing systems, diagrid structures have been emerging as a new aesthetic trend for tall buildings in this era. The triangular geometry of diagrid structures can effectively prevent structural failure as a result of both horizontal and vertical loads resulted by the externally imposed lateral loads and gravity load. The famous examples are Gherkin in London and CCTV Tower in Beijing.

Fig. 2.3 Empire State Building in New York with height of 381 m, New York, U.S.A. *(Photo taken by the author.)*

Superframe (or mega frame) structures use mega columns and mega girders to work together as the primary lateral resisting structural systems. The famous example is Shanghai Tower in Shanghai and HSBC headquarters in Hong Kong.

Ali et al. [4] summarized the different structural system for tall buildings, their typical structural layout and efficient height limit, which is shown in Fig. 2.4. However, in the current design practice, few tall buildings only use one single structural system. For example, the structural system of Shanghai Tower is in the combination of mega frame, core wall system and outriggers. Some conventional combinations such as braced core, core and perimeter tube, braced core and outriggers, braced core and outriggers with ring truss, etc. can be found in the current design projects.

Table 2.2 List of Current Tall Buildings Projected to be Built by 2020 (The Global Tall Building Database of Council on Tall Buildings and Urban Habitat)

No.	Building Name	City	Height (m)	Floors	Complete	Material	Use
1.	Burj Khalifa	Dubai (AE)	828 m	163	2010	Steel/ concrete	Office/residential/hotel
2.	Shanghai Tower	Shanghai (CN)	632 m	128	2015	Composite	Hotel/office
3.	Makkah Royal Clock Tower	Mecca (SA)	601 m	120	2012	Steel/ concrete	Other/hotel
4.	One World Trade Center	New York City (US)	541.3 m	94	2014	Composite	Office
5.	Taipei 101	Taipei (TW)	508 m	101	2004	Composite	Office
6.	Shanghai World Financial Center	Shanghai (CN)	492 m	101	2008	Composite	Hotel/office
7.	International Commerce Center	Hong Kong (CN)	484 m	108	2010	Composite	Hotel/office
8.	Petronas Twin Tower 1	Kuala Lumpur (MY)	451.9 m	88	1998	Composite	Office
9	Petronas Twin Tower 2	Kuala Lumpur (MY)	451.9 m	88	1998	Composite	Office
10.	Zifeng Tower	Nanjing (CN)	450 m	66	2010	Composite	H otel/office
11.	Willis Tower	Chicago (US)	442.1 m	108	1974	Steel	Office
12.	KK100	Shenzhen (CN)	441.8 m	100	2011	Composite	Hotel/office
13.	Guangzhou International Finance Center	Guangzhou (CN)	438.6 m	103	2010	Composite	Hotel/office
14.	432 Park Avenue	New York City (US)	425.5 m	85	2015	Concrete	Residential
15.	Trump International Hotel and Tower	Chicago (US)	423.2 m	98	2009	Concrete	Residential/hotel

Continued

Table 2.2 List of Current Tall Buildings Projected to be Built by 2020 (The Global Tall Building Database of Council on Tall Buildings and Urban Habitat)—cont'd

No.	Building Name	City	Height (m)	Floors	Complete	Material	Use
16.	Jin Mao Tower	Shanghai (CN)	420.5 m	88	1999	Composite	Hotel/office
17.	Princess Tower	Dubai (AE)	413.4 m	101	2012	Steel/concrete	Residential
18.	Al Hamra Tower	Kuwait City (KW)	412.6 m	80	2011	Concrete	Office
19.	Two International Finance Centre	Hong Kong (CN)	412 m	88	2003	Composite	Office
20.	23 Marina	Dubai (AE)	392.4 m	88	2012	Concrete	Residential
21.	CITIC Plaza	Guangzhou (CN)	390.2 m	80	1996	Concrete	Office
22.	Shun Hing Square	Shenzhen (CN)	384 m	69	1996	Composite	Office
23.	Burj Mohammed Bin Rashid	Abu Dhabi (AE)	381.2 m	88	2014	Concrete	Residential
24.	Empire State Building	New York City (US)	381 m	102	1931	Steel	Office
25.	Elite Residence	Dubai (AE)	380.5 m	87	2012	Concrete	Residential
26.	Central Plaza	Hong Kong (CN)	373.9 m	78	1992	Concrete	Office
27.	Bank of China Tower	Hong Kong (CN)	367.4 m	72	1990	Composite	Office
28.	Bank of America Tower	New York City (US)	365.8 m	55	2009	Composite	Office
29.	Almas Tower	Dubai (AE)	360 m	68	2008	Concrete	Office
30.	JW Marriott Marquis Hotel Dubai Tower 1	Dubai (AE)	355.4 m	82	2012	Concrete	Hotel

31.	JW Marriott Marquis Hotel Dubai Tower 2	Dubai (AE)	355.4 m	82	2013	Concrete	Hotel
32.	Emirates Tower One	Dubai (AE)	354.6 m	54	2000	Composite	Office
33.	OKO—Residential Tower	Moscow (RU)	353.6 m	90	2015	Concrete	Residential/serviced apartments/hotel
34.	The Torch	Dubai (AE)	352 m	86	2011	Concrete	Residential
35.	Forum 66 Tower 1	Shenyang (CN)	350.6 m	68	2015	Composite	Hotel/office
36.	The Pinnacle	Guangzhou (CN)	350.3 m	60	2012	Concrete	Office
37.	T & C Tower	Kaohsiung (TW)	347.5 m	85	1997	Composite	Hotel/office/retail
38.	Aon Center	Chicago (US)	346.3 m	83	1973	Steel	Office
39.	The Center	Hong Kong (CN)	346 m	73	1998	Steel	Office
40.	John Hancock Center	Chicago (US)	343.7 m	100	1969	Steel	Residential/office
41.	ADNOC Headquarters	Abu Dhabi (AE)	342 m	65	2015	Concrete	Office
42.	Wuxi International Finance Square	Wuxi (CN)	339 m	68	2014	Composite	Hotel/office
43.	Chongqing World Financial Center	Chongqing (CN)	338.9 m	72	2015	Composite	Office
44.	Mercury City Tower	Moscow (RU)	338.8 m	75	2013	Concrete	Residential/office
45.	Tianjin World Financial Center	Tianjin (CN)	336.9 m	75	2011	Composite	Office
46	Shimao International Plaza	Shanghai (CN)	333.3 m	60	2006	Concrete	Hotel/office/retail
47.	Rose Rayhaan by Rotana	Dubai (AE)	333 m	71	2007	Composite	Hotel
48.	Minsheng Bank Building	Wuhan (CN)	331 m	68	2008	Steel	Office
49.	China World Tower	Beijing (CN)	330 m	74	2010	Composite	Hotel/office

Continued

Table 2.2 List of Current Tall Buildings Projected to be Built by 2020 (The Global Tall Building Database of Council on Tall Buildings and Urban Habitat)—cont'd

No.	Building Name	City	Height (m)	Floors	Complete	Material	Use
50.	Keangnam Hanoi Landmark Tower	Hanoi (VN)	328.6 m	72	2012	Concrete	Hotel/residential/office
51.	Longxi International Hotel	Jiangyin (CN)	328 m	72	2011	Composite	Residential/hotel
52.	Al Yaqoub Tower	Dubai (AE)	328 m	69	2013	Concrete	Hotel/office
53.	Wuxi Suning Plaza 1	Wuxi (CN)	328 m	67	2014	Composite	Hotel/serviced apartments/office
54.	The Index	Dubai (AE)	326 m	80	2010	Concrete	Residential/office
55.	The Landmark	Abu Dhabi (AE)	324 m	72	2013	Concrete	Residential/office
56.	Deji Plaza	Nanjing (CN)	324 m	62	2013	Composite	Hotel/office
57.	Q1 Tower	Gold Coast (AU)	322.5 m	78	2005	Concrete	Residential
58	Wenzhou Trade Center	Wenzhou (CN)	321.9 m	68	2011	Concrete	Hotel/office
59.	Burj Al Arab	Dubai (AE)	321 m	56	1999	Composite	Hotel
60.	Nina Tower	Hong Kong (CN)	320.4 m	80	2006	Concrete	Hotel/office
61.	Chrysler Building	New York City (US)	318.9 m	77	1930	Steel	Office
62.	New York Times Tower	New York City (US)	318.8 m	52	2007	Steel	Office
63.	HHHR Tower	Dubai (AE)	317.6 m	72	2010	Concrete	Residential
64.	Nanjing International Youth Cultural Centre T…	Nanjing (CN)	314.5 m	68	2015	Composite	Hotel/office
65.	Bank of America Plaza	Atlanta (US)	311.8 m	55	1992	Composite	Office

66	Moi Center Tower A	Shenyang (CN)	311 m	75	2014	Composite	Hotel/office
67.	U.S. Bank Tower	Los Angeles (US)	310.3 m	73	1990	Steel	Office
68.	Ocean Heights	Dubai (AE)	310 m	83	2010	Concrete	Residential
69	Menara TM	Kuala Lumpur (MY)	310 m	55	2001	Concrete	Office
70.	Pearl River Tower	Guangzhou (CN)	309.4 m	71	2013	Composite	Office
71.	Fortune Center	Guangzhou (CN)	309.4 m	68	2015	Composite	Office
72.	Emirates Tower Two	Dubai (AE)	309 m	56	2000	Composite	Hotel
73.	Stalnaya Vershina	Moscow (RU)	308.9 m	72	2015	Composite	Residential/hotel/office
74.	Burj Rafal	Riyadh (SA)	307.9 m	68	2014	Concrete	Residential/hotel
75.	The Franklin—North Tower	Chicago (US)	306.9 m	60	1989	Composite	Office
76.	Cayan Tower	Dubai (AE)	306.4 m	73	2013	Concrete	Residential
77.	One57	New York City (US)	306.1 m	75	2014	Steel/ concrete	Residential/hotel
78.	East Pacific Center Tower A	Shenzhen (CN)	306 m	85	2013	Composite	Residential
79.	The Shard	London (GB)	306 m	73	2013	Composite	Residential/hotel/office
80.	JPMorgan Chase Tower	Houston (US)	305.4 m	75	1982	Composite	Office
81.	Etihad Towers T2	Abu Dhabi (AE)	305.3 m	80	2011	Concrete	Residential
82.	Northeast Asia Trade Tower	Incheon (KR)	305 m	68	2011	Composite	Residential/hotel/office
83.	Baiyoke Tower II	Bangkok (TH)	304 m	85	1997	Concrete	Hotel

Continued

Table 2.2 List of Current Tall Buildings Projected to be Built by 2020 (The Global Tall Building Database of Council on Tall Buildings and Urban Habitat)—cont'd

No.	Building Name	City	Height (m)	Floors	Complete	Material	Use
84.	Shenzhen CFC Changfu Centre	Shenzhen (CN)	303.8 m	68	2016	Composite	Office
85.	Wuxi Maoye City—Marriott Hotel	Wuxi (CN)	303.8 m	68	2014	Composite	Hotel
86.	Two Prudential Plaza	Chicago (US)	303.3 m	64	1990	Concrete	Office
87.	Diwang International Fortune Center	Liuzhou (CN)	303 m	75	2015	Composite	Residential/hotel/office
88.	Greenland Puli Center	Jinan (CN)	303 m	61	2014	Composite	Office
89.	Jiangxi Nanchang Greenland Central Plaza 1	Nanchang (CN)	303 m	59	2015	Composite	Office
90.	Jiangxi Nanchang Greenland Central Plaza 2	Nanchang (CN)	303 m	59	2015	Composite	Office
91.	Leatop Plaza	Guangzhou (CN)	302.7 m	64	2012	Composite	Office
92.	Wells Fargo Plaza	Houston (US)	302.4 m	71	1983	Steel	Office
93.	Kingdom Centre	Riyadh (SA)	302.3 m	41	2002	Steel/concrete	Residential/hotel/office
94.	The Address	Dubai (AE)	302.2 m	63	2008	Concrete	Residential/hotel
95.	Capital City Moscow Tower	Moscow (RU)	301.8 m	76	2010	Concrete	Residential
96.	Shenzhen Zhongzhou Holdings Financial Center…	Shenzhen (CN)	300.8 m	61	2014	Steel	Hotel/office
97.	Doosan Haeundae We've the Zenith Tower A	Busan (KR)	300 m	80	2011	Concrete	Residential
98.	Torre Costanera	Santiago (CL)	300 m	62	2014	Concrete	Office
99.	Abeno Harukas	Osaka (JP)	300 m	60	2014	Steel	Hotel/office/retail
100.	Arraya Tower	Kuwait City (KW)	300 m	60	2009	Concrete	Office

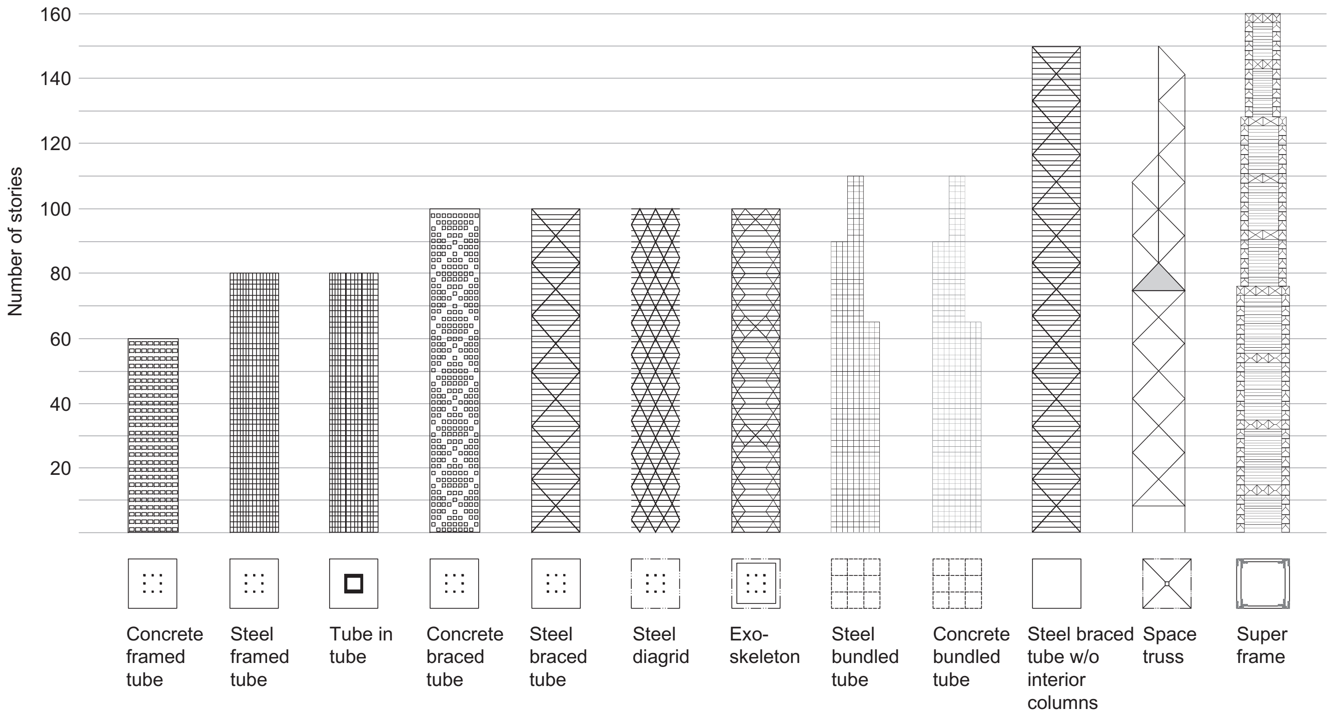


Fig. 2.4 List of different lateral stability system in tall buildings [4].

Except aforementioned lateral stability systems, the floor system also plays an important role in resisting the lateral loadings. In additional to supporting the gravity load, the floor system is also working as a floor diaphragm which is designed to transmit the lateral load from cladding to the lateral load-resisting systems at each level. Horizontal trusses or deep beams are also used where the floor diaphragm is interrupted by penetrations or openings. It is important that designers allow for the concentrations of horizontal stress within the floor diaphragm around openings and penetrations through the provision of diagonal corner steel within the flooring. Care should also be taken to ensure that the interconnection between the floor diaphragm and the lateral load-resisting system is sufficiently robust to transmit the required shear between elements.

As the lateral stability system is essential in designing a tall building, therefore in the following chapters (Chapters 3–5), it will be explained in detail.

2.4 LOADS ON TALL BUILDINGS

Design loading for a tall building is often specified in general building codes such as ASCE/SEI 2010 [5], EN 1991-1-1:2002 [6]. There are different types of loads that need to be considered which is going to be mentioned here.

2.4.1 Dead Loads

The dead loads are loads that permanently stay in a building, including self-weights of various structural members such as beams, columns, and core walls. It also includes weights of any objects that are attached to the structure such as finishes, ceilings, insulation materials, and partitions. Table 2.3 is an example of minimum density and design dead loads from Eurocode 1. The designers can check it for design purpose.

2.4.2 Live Loads

Live loads are loads that can vary in magnitude and location. An example of live loads can be people, furniture, and equipment. Depending on the function of the building, either commercial or residential buildings, the value of the live load varies. These loadings are generally tabulated in design codes. Eurocode 1 [6], gives the value of the live loads for different use of building and with different occupancy, as shown in Table 2.4.

Table 2.3 Construction Materials—Concrete and Mortar

Materials	**Density (kN/m^3)**
Concrete *(see EN 206)*	
Lightweight	
Density class LC 1.0	9.0–10.0[a,b]
Density class LC 1.2	10.0–12.0[a,b]
Density class LC 1.4	12.0–14.0[a,b]
Density class LC 1.6	14.0–16.0[a,b]
Density class LC 1.8	16.0–18.0[a,b]
Density class LC 2.0	18.0–20.0[a,b]
Normal weight	24.0[a,b]
Heavy weight	>[a,b]
Mortar	
Cement mortar	19.0–23.0
Gypsum mortar	12.0–18.0
Lime-cement mortar	18.0–20.0
Lime mortar	12.0–18.0

[a]Increase by 1 kN/m^3 for normal percentage of reinforcing and prestressing steel.
[b]Increase by 1 kN/m^3 for unhardened concrete.
Reproduced from Table A.1 of page 32 EN 1991-1-1:2002. Eurocode 1: Actions on structures—General actions–Part 1–1: Densities, self-weight, imposed loads for buildings, incorporating corrigenda. (Permission to reproduce and derive extracts from British and ISO standards is granted by BSI. British Standards can be obtained in PDF or hard copy formats from the BSI online shop: www.bsigroup.com/Shop or by contacting BSI Customer Services for hardcopies only: Tel: +44 (0) 20 8996 9001, Email: cservices@bsigroup.com.)

Table 2.4 Imposed Loads on Floors, Balconies, and Stairs in Buildings

Category A		
- Floors	1.5–2.0	2.0–3.0
- Stairs	2.0–4.0	2.0–4.0
- Balconies	2.5–4.0	2.0–3.0
Category B	2.0–3.0	1.5–4.5
Category C		
- C1	2.0–3.0	3.0–4.0
- C2	3.0–4.0	2.5–7.0 (4.0)
- C3	3.0–5.0	4.0–7.0
- C4	4.5–5.0	3.5–7.0
- C5	5.0–7.5	3.5–4.5
Category D		
- D1	4.0–5.0	3.5–7.0 (4.0)
- D2	4.0–5.0	3.5–7.0

Reproduced from Table 6.2 of page 22 EN 1991-1-1:2002. Eurocode 1: Actions on structures—General actions —Part 1–1: Densities, self-weight, imposed loads for buildings, incorporating corrigenda. (Permission to reproduce and derive extracts from British and ISO standards is granted by BSI. British Standards can be obtained in PDF or hard copy formats from the BSI online shop: www.bsigroup.com/Shop or by contacting BSI Customer Services for hardcopies only: Tel: +44 (0)20 8996 9001, Email: cservices@bsigroup.com).

2.4.3 Snow Loads

Snow load is actually one type of live load, however due to its unique feature, it becomes an independent load category in most of the design codes. In design code such as ASCE/SEI 7–10 [5], snow loads are determined from a zone map reporting 50-year recurrence interval. A designer can check it for the determination of the value of snow load.

2.4.4 Wind Loads

Effects of wind depend on density and flow of air, angle of incidence, shape and stiffness of the structure, and roughness of surface. Its load value also depends on roof geometry, wind exposure, location, and its importance.

In design, wind loadings can be treated as static or dynamic approach depends on the height of the buildings. EN 1991-1-4 [7] gives guidance on the determine action of natural wind actions for the structural design of building and civil engineering works for each of the loaded areas under consideration. This includes the whole structure or parts of the structure or elements attached to the structure, for example, components, cladding units and their fixings, and safety and noise barriers. However, EN1991-1-4 [7] is only applicable to buildings with maximum height of 200 m. For building over 200 m, dynamic approach would become dominant, and wind tunnel test is required for most of the projects. In tall building design, wind load is one of the essential design considerations, therefore, it will be discussed in detail in a later section.

2.4.5 Earthquake Loading

Earthquake is caused by either the rupturing of faults in the earth's crust or landslides and volcanic eruptions. During earthquake, the ground vibrates both horizontally and vertically. Horizontal acceleration produces shear forces working on the tall buildings. The magnitude depends on the amount and type of ground acceleration. The response of the building depends on the mass and stiffness of the structure. The major difference in earthquake loading to static loading is that the value and direction of the earthquake loading changes against time, and it is a dynamic problem. Therefore, in the analysis, it is required to solve the governing equation of motion. An analysis of the dynamic system can be simplified by assuming the mass structure to be concentrated at discrete points. Therefore, based on the feature of original structures, the correspondent analytical model can be simplified into single-degree-of-freedom (SDOF) system, or multi-degree-of-freedom

(MDOF) system. Eq. (2.1) is the governing equation of motion for the SDOF under earthquake loading:

$$\ddot{x}(t) + 2\xi\omega\dot{x}(t) + \omega x(t) = -\ddot{x}_g(t) \tag{2.1}$$

where $x(t)$ is the relative displacement, $\ddot{x}_g(t)$ is the ground acceleration, $\ddot{x}(t)$ is the relative acceleration of single mass, respectively, $\dot{x}(t)$ is the relative velocity, and ξ is the damping ratio.

A more complex MDOF system is essential in analyzing a tall building. As shown in Fig. 2.5, a tall building can be simplified as a model with lumped mass for each story. This may respond in as many shapes as its degrees of freedom.

Each mass in a MDOF system is generally subjected to inertia force, damping force, elastic force, and external dynamic forces, as has been demonstrated in SDOF systems. Therefore, the governing equation of motion is.

$$[m]\{\ddot{x}\} + [c]\{\dot{x}\} + [k]\{x\} = -\ddot{x}_g(t)[m]\{I\} \tag{2.2}$$

where $[m]$ is the mass matrix, $[k]$ is the stiffness matrix, $[c]$ is the damping matrix, $\{I\}$ is the unit vector, and $\ddot{x}_g(t)$ is the ground acceleration.

What we need to notice from Eqs. (2.1) and (2.2) is the introduction of the ground acceleration in these two equations. If the building is stiff with small masses, the period of vibration of the building will be short, the

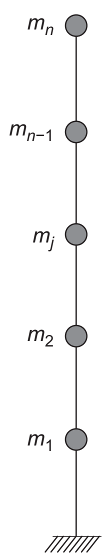


Fig. 2.5 Sketch of a MDOF system.

building will have acceleration with the similar motion as the ground and will undergo slight relative displacements. If the building is very flexible with large masses, induced motion will cause small accelerations of the building and large relative displacement.

2.5 FLOOR SYSTEM

In tall building design, similar floor structures to those in low-rise buildings are used.

However, there are certain aspects and properties that need to be considered in design floor system for a tall building. Firstly, the weight of the floor need to be minimized, due to its height, the lighter weight will result a more economical foundation. Secondly, it needs to consider the fire resistance, especially for composite floor system; thirdly, the floor should also resist the load during the construction; finally, due to its large column spacing inside most of the tall buildings, the slabs also need to have long distance span.

2.5.1 Concrete Floor System

Concrete floor system refers to a system with concrete slabs sitting on the concrete beams (Fig. 2.6). One-way or two-way slabs are used. The span of the slabs is normally between 3 and 8 m.

2.5.2 Flat Slab

Flat slab is another commonly used concrete floor system, where the slabs are made to directly sit on the column without using the beams (Fig. 2.7). However, punching shear of the slab near the column needs to be considered in

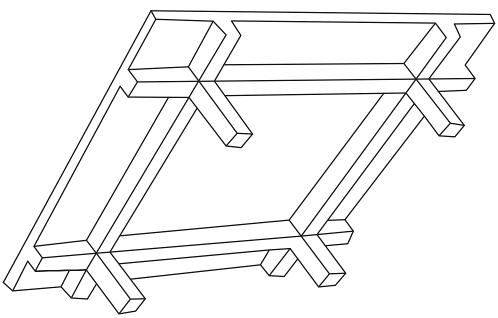


Fig. 2.6 Two-way concrete slab supported on beams and columns.

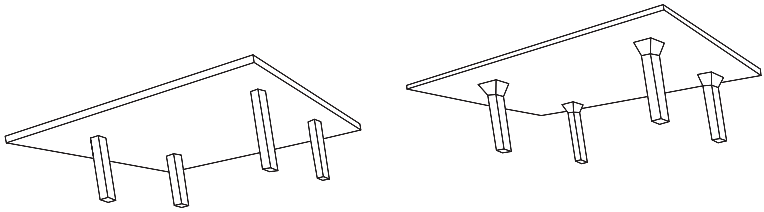


Fig. 2.7 Flat slab system: (A) without column head and (B) with column head.

certain projects, in some projects column head is designed to increase the shear capacity of the slab at columns, as shown in Fig. 2.7B. Flat slabs are popular for its fast track construction, allow easy service distribution underneath the slabs and are very economical for a span of 5–10 m.

2.5.3 Posttensioned Slab System

Posttensioned floor slabs are predominantly used in the multistory buildings, especially tall building construction across the world. This is because the slab sections can be about 30% thinner which enables the total weight carried by the foundations to be reduced.

The principle behind the posttensioned slab is that concrete has a low tensile strength but is strong in compression. By compressing a concrete element, when it is subjected to bending under gravity loads, it still remains in compression, a more efficient design for the structure can be achieved.

There are two methods of applying prestress to a concrete member.

- Posttensioning, the concrete is placed around ducts containing unstressed tendons. Once the concrete has gained sufficient strength the tendons are stressed against the concrete and locked off by special anchor grips, known as split wedges. In this system, all tendon forces are transmitted directly to the concrete. Since no stresses are applied to the formwork, conventional formwork may be used.
- Pretensioning, the concrete is placed around previously stressed tendons. As the concrete hardens, it grips the stressed tendons and when it obtained sufficient strength, the tendons are released, thus transferring the forces on to the concrete. Considerable force is required to stress the tendons, so pre-tensioning is principally used for precast concrete where the forces can be restrained by fixed abutments located at each end of the stressing bed, or carried by specially stiffened molds

Posttensioned slabs are more widely used in the tall building design due to its fast construction procedures. It can be used to span distances of up to 25 m between columns which reduces the number of columns [8]. It also gives the minimum structural thickness of the slab, therefore saves the total weight of the building, and significantly reduce the overall building height, however, providing the same number of stories. Therefore, it is widely used in tall buildings (such as the Shard).

2.5.4 Composite Floor System

Composite floor system is using the steel beams to be connected with the concrete floor slabs. It utilizes the advantage of both steel and concrete. Therefore, it is more cost-effective form of construction.

2.5.4.1 Solid R.C. or Profiled Metal Deck Floors

In composite floor system, the most common types of floor slabs used are solid R.C. or profiled metal deck floors with shear connectors to connect them to the steel beams. A typical shear connection is to use the shear stud as shown in Figs. 2.8 and 2.9, which is a typical metal decking composite floor system supported on the steel beams.

2.5.4.2 Precast Slab

As shown in Fig. 2.10, composite beams incorporating precast hollow core floor slabs is another composite floor system for buildings. The use of precast hollow-core slabs in steel composite construction was first developed in the 1990s. This type of floor system offer advantages where the use of a steel

Fig. 2.8 Steel beams with shear studs. *(Photo taken by the author.)*

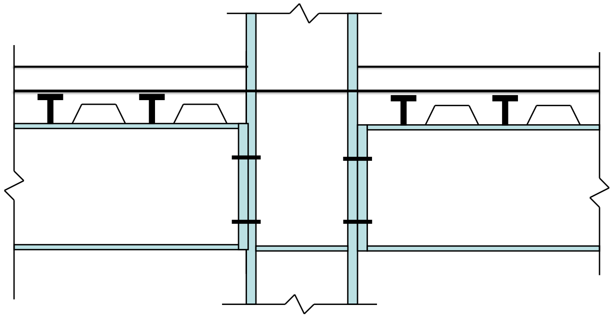


Fig. 2.9 A metal decking composite floor system supported on the steel beams.

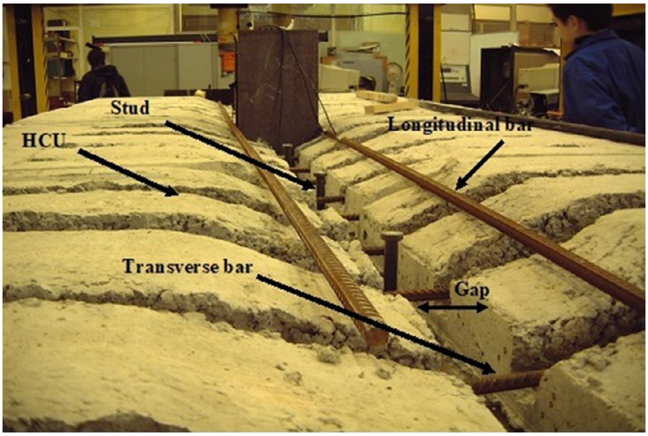


Fig. 2.10 Composite floor system using precast hollow-core slabs.

decking or solid slab system may be unsuitable. Precast concrete slabs or precast hollow-core concrete are also used with shear connectors to work together with steel beams.

Thickness of slabs is in the range of L/30 to L/15 of the span vary from 1.2 to 9 m. For steel beams, wide flange shapes are used. Welded plate girders, latticed girders, and vierendeel girders are also used. Castellated beams and stub girders are also used frequently in tall building. Fig. 2.11 shows an example of castellated beams used in tall buildings, which allow mechanical ductwork to be placed between short stubs, welded on top of these girders.

2.5.4.3 Slim Floor Construction

Slim floor use a special girder with a lower flange which is wider than the upper flange (Fig. 2.12). This arrangement makes it possible to fit the floor

Fig. 2.11 A tall building project using castellated beams at East Croydon, London, U.K. *(Photo taken by the author.)*

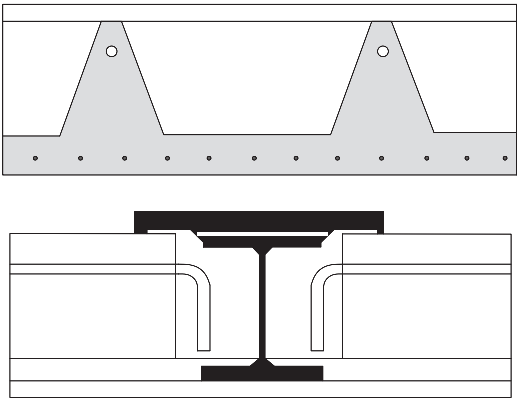


Fig. 2.12 Slim floor constructions.

slabs directly onto the lower flange plate of the beam. This product is developed around 1990s. In 1992, full-scale tests of slimfloor beam were carried out in my current department at the City, University of London. Compared to the other composite floor system, it has the following advantages:

1. Floor thickness reduction
2. Incorporating under-floor technical equipment

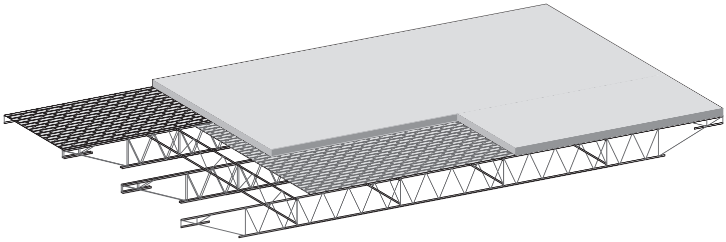


Fig. 2.13 Typical composite truss floor system of World Trade Center. *Permission to reproduce and derive from Fig. 1.6 of Federal Building and Fire Safety Investigation of the World Trade Center Disaster: Final Report of the National Construction Safety Team on the collapses of the World Trade Center Towers (NIST NCSTAR 1), 2005.*

3. Built-in fire resistance
4. Creating vertical movement space
5. Lighter structures

Therefore, it is widely used in multistory design and in some of the tall building designs.

2.5.5 Composite Truss Floor System

In tall buildings, composite trusses floor systems are frequently used for very long span supports. As shown in Fig. 2.13, it is the composite truss floor system used in World Trade Center before it collapsed. The floors system consists of 10-cm lightweight concrete slabs with steel deck supported on the bridging trusses and main trusses. This is one of the typical composite floor systems. The trusses are supported on the perimeter at alternate columns. The openings created in the truss braces can be used to accommodate large services.

2.6 VERTICAL SUPPORT SYSTEMS

The floor system is supported on the vertical support systems such as columns, walls, hangers, transfer girders, and roof truss to transfer the gravity load to the foundations. Structural steel, reinforced concrete, and composite columns are used. In addition, staggered trusses between floors, girders are used to bridge large openings at lower levels of a tall building. Suspended system is another vertical support system it has to support structure such as roof truss at the top, with floors suspended below by using cables or hangers.

Transfer structures are required where vertical load carrying elements cannot be taken through directly to foundations. It is widely used in tall building projects. Here, some examples are illustrated below.

2.6.1 Transfer Truss or Roof Truss

Transfer truss is a whole story height truss used to transfer column loads at offset (Fig. 2.14). As shown in Fig. 2.15, it is different to transfer truss, roof truss working together with hangers to support the suspension floor.

2.6.2 Inclined Columns

Inclined column is used when there is a change in the plan layout is required, this is mainly due to the requirement of the architect (Fig. 2.16).

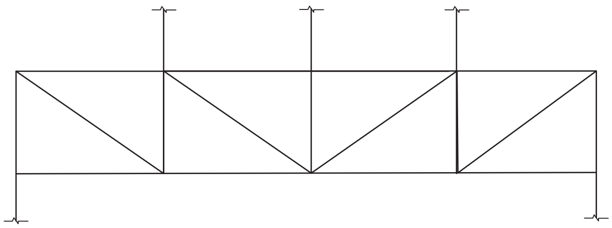


Fig. 2.14 Transfer truss.

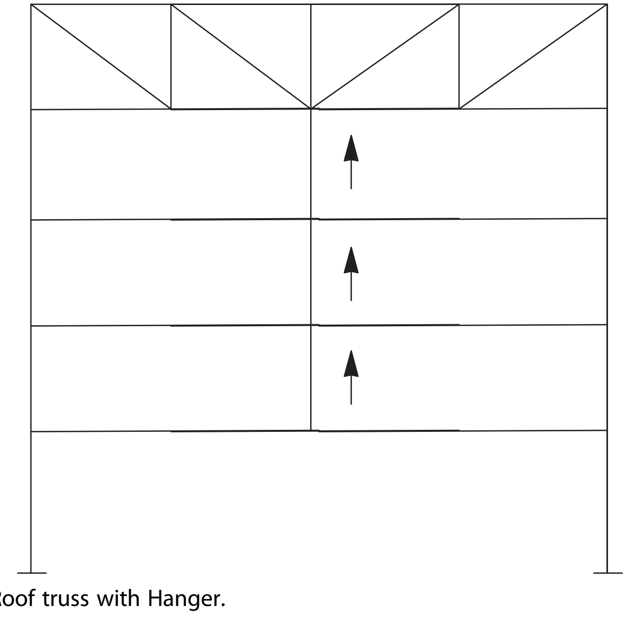


Fig. 2.15 Roof truss with Hanger.

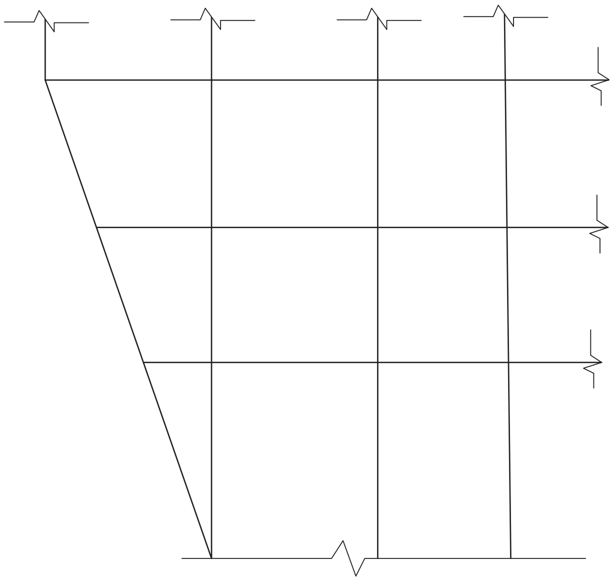


Fig. 2.16 Example of inclined columns in tall buildings.

2.6.3 Walking Column

Inclined columns or fin walls are sometimes used to offset column loads. In this particular case, the structure must be designed for the horizontal push-and-pull loads generated by the transfer. Walking column are often used one above the other to gradually transfer loads through multiple stories. One of the famous examples is Manchester Hilton Tower designed by my previous company WSP Group. At level 23, the building cantilevers are projecting out to roughly 4 m, as shown in Fig. 2.17. The cantilever stories are supported on two-story walking column working together with 2-m-long concrete cantilever beams projecting from the core (Fig. 2.18).

2.7 EARTHQUAKE DESIGN

As required by the design codes such as Eurocode 8 [9], ASCE/SEI 7–10 [5], the major objective of the seismic design is to ensure the safety of the public, especially reduces the loss of life and sometimes causes damage to the facilities. The structure must be designed to remain in a safe condition even after an earthquake, and to permit the safe evacuation of all its occupants. Although some structural and nonstructural damage will inevitably occur, it should be restricted by careful seismic detailing to ensure that demolition and rebuilding costs are less .

Fig. 2.17 Beetham Hilton Tower Manchester. *(This file is licensed under the Creative Commons Attribution-Share Alike 3.0 Free license, https://commons.wikimedia.org/wiki/File:Beetham_Tower_from_below.jpg.)*

Eurocode 8 [9] has two different requirements in the design based on the specific earthquake:

1. For a mean return period of 475 years of earthquake, which is the so-called "design seismic action," the performance requirement is to avoid collapse of structure, or so-called "no-collapse requirement."
2. For the so-called "serviceability seismic action," which corresponds to a mean return period of 95 years, the performance requirement is that the main structure is no need for any repair, for nonstructural members, it should be easy or economically repairable, this is also called "damage limitation requirement."

2.7.1 Horizontal and Vertical Seismic Actions

It is normal practice to design for horizontal seismic forces only, in most of the regions, vertical seismic forces are not significant. As a rule of thumb,

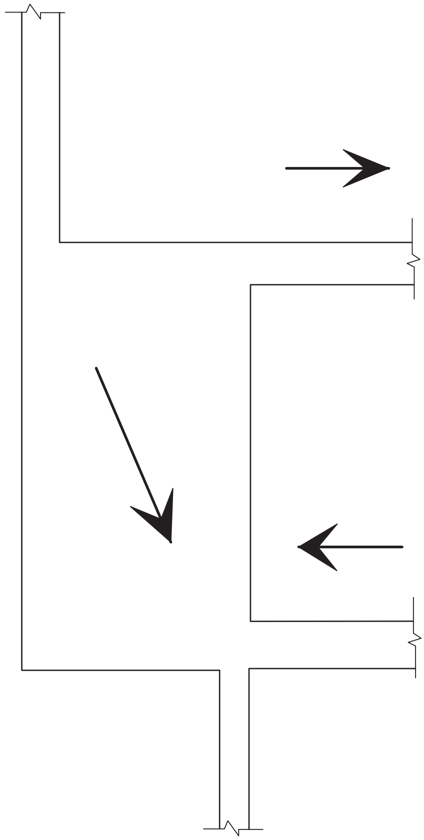


Fig. 2.18 Load transferring mechanism in the walking column.

horizontal seismic loading of 0.3 g are commonly used in design. However, exceptionally high vertical accelerations may result in damage to many of the buildings, or even collapse, in countries such as Japan and Turkey. During Kobe earthquake in Japan, 1995, vertical movement was greater than horizontal movement and that resulted in vertical forces >1.0 g.

ASCE/SEI 7–10 [5] gives the equation in both horizontal and vertical seismic load effect.

For horizontal seismic effect.

$$E_h = \rho Q_E \tag{2.3}$$

where E_h, is the horizontal seismic load effect, Q_E = effects of horizontal seismic forces from V or F_p, where required in ASCE/SEI 7–10 [5], such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other, and p = redundancy factor.

For Vertical seismic effect.

$$E_v = 0.2S_{DS}D \tag{2.4}$$

where E_v, is the vertical seismic load effect, S_{DS} = design spectral response acceleration parameter at short periods obtained from Section 11.4.4 of ASCE/SEI 7–10 [5], and D is the effect of dead load.

In Eurocode 8 [9], there are two different types of response spectrum, horizontal and vertical for designers to use in their project.

Both ASCE/SEI 7–10 [5] and Eurocode 8 [9] give an equation on the combination of the three seismic components (horizontal seismic load effect in the x direction x, y and vertical seismic load effect in z).

For ASCE/SEI 7–10 [5], following combinations can be used:

a. For use in load combination 5 in Section 2.3.2 or load combinations 5 and 6 in Section 2.4.1 of ASCE/SEI 7–10 [5]

$$E = E_h + E_v \tag{2.5}$$

b. For use in load combination 7 in Section 2.3.2 or load combination 8 in Section 2.4.1 of ASCE/SEI 7–10 [5]

$$E = E_h - E_v \tag{2.6}$$

where E is the seismic load effect, E_h is the effect of horizontal seismic forces as defined in Section 12.14.3.1.1 of ASCE/SEI 7–10 [5], and E_v is the effect of vertical seismic forces as defined in Section 12.14.3.1.2 of ASCE/SEI 7–10 [5]

For Eurocode 8 [9], following combinations can be used:

$$E_x + \lambda E_y + \lambda E_z \tag{2.7}$$

$$\lambda E_x + E_y + \lambda E_z \tag{2.8}$$

$$\lambda E_x + \lambda E_y + E_z \tag{2.9}$$

where $\lambda = 0.3$, E_x, is the horizontal seismic load effect in the x direction, E_y, is the horizontal seismic load effect in the y direction, and E_z, is the vertical seismic load effect.

However, the vertical component is used only exceptionally in countries such as Japan and Turkey, where, the vertical seismic action cannot be ignored.

2.7.2 Structural Analysis Method

There are three major earthquake analysis methods for tall buildings under earthquake loading, which are response spectrum analysis, push over analysis and time history analysis.

2.7.2.1 Response Spectrum Analysis

It is a linear-dynamic analysis, which defines the response (such as acceleration, velocity, or displacement) spectrum by enveloping and smoothing the spectra corresponding to different earthquake time histories. The maximum acceleration response of the structures is determined over the expected earthquake ground motion. It is the combination of each natural mode of vibration to indicate the likely maximum seismic response of an elastic structure. The elastic response spectrum can be defined according to Eurocode 8 [9] or ASCE/SEI 7–10 [5].

2.7.2.2 Pushover Analysis

It is a nonlinear static analysis rather than a dynamic analysis, which is carried out under conditions of constant gravity loads and monotonically increasing horizontal loads until a certain target displacement is obtained. The lateral load is predefined which is distributed along the building height. It estimates the expected plastic mechanisms and the distribution of damage.

As required by Eurocode 8 [9], the relation between base shear force and the control displacement (this is the so-called "capacity curve") should be determined by pushover analysis for values of the control displacement ranging between zero and the value corresponding to 150% of the target displacement.

2.7.2.3 Time History

It is a linear or nonlinear dynamic analysis. It is to analyze the building under a ground acceleration time history with the frequency content matches the design spectrum. The equation of motion in Eq. (2.2) is solved using direct integration methods. As it is required by Eurocode 8 [9] and ASCE/SEI 07 [5], the engineers are required to choose at least 3–7 accelerograms. Seismic motion may be made by using artificial accelerograms and recorded or simulated accelerograms. This analysis requires large computational cost. However, it can monitor the behavior of the structure more precisely.

2.7.3 Dissipative (Ductile) Structure Design

Ductility is the ability of the building or member to deform beyond the point of first yield while maintaining a substantial proportion of its initial maximum load carrying capacity and without brittle failure. This ductility can dissipate the energy input from earthquake, making the structure undergo the same displacements but use smaller sections compared to elastic design method, which will ensure that collapse does not take place and the energy produced is dissipated by ductile yielding. In addition, forces applied to the foundations will be reduced. Therefore, ductility is an important design requirement in earthquake-resistant design. It plays an important role in energy dissipation.

2.7.3.1 Design Spectrum (Behavior Factor q *or Force Reduction Factor* R*)*

The ability to deform plastically without loss of resistance is taken into account by using a behavior factor, q in Eurocode 8. This factor reduces the elastic spectrum into a design spectrum by dividing it using the behavior factor, q. The value of q ranges from a minimum 1.5 (low dissipation) up to 6 or more (high dissipation). Therefore, "to avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behavior of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a design spectrum" (Eurocode 8 [9]). In US design code ASCE/SEI 7–10 [5], similar requirement using a reduction factor R is imposed. To be able to use the design spectrum rather than elastic spetrum, one needs to gurantee that when designning the building, energy dissapation can be ensured. This will be achieved through selecting a proper ductility class and correct ductile detailing design.

2.7.3.2 Ductility Class for Design-Dissipative or Nondissipative Structure

According to Eurocode 8 [9] a designer can choose the ductility or dissipation class while designing, either dissipative (ductile) or nondissipative (elastic).Three dissipation classes are defined in Eurocode 8 [9] as follows:

Low, a nondissipative or low ductility class (DCL) structure is designed following the basic design codes, with checks for resistance to gravity, wind loads, etc.

Medium (DCM) higher level of plasticity are permitted, however, corresponding design and ductile detailing of structural members are imposed.

High (DCH) very high plasticity, more complex design and detailing requirement.

Ductility class DCM or DCH is designed for a seismic action which is lower than that used in a DCL design, because the behavior factor q is greater (in the range of 3–6). The weight of the structural elements can be substantially reduced. However, to achieve the design target, there are restrictions on the classes of sections, connections, and the control of the material properties.

2.7.3.3 Ductile Detailing

In order to design the structure based on the ductility, it is essential to ensure that the structure fails in a ductile manner. Rapid deterioration of structural member needs to be avoided. Therefore, ductile detailing is essential, which will be discussed later in this section.

2.7.3.3.1 Concrete Structures

Seismic building codes such as Eurocode 8 [9], imposes reinforcement details for all throughout the length of the members.

At beam/column interfaces, beam reinforcement is anchored into the column to develop full tensile strength with tension splices located away from the section of maximum tension. Web reinforcement in the form of closed links is spaced at close centers each side of the column face over the plastic hinge region of the beam as shown in Fig. 2.19.

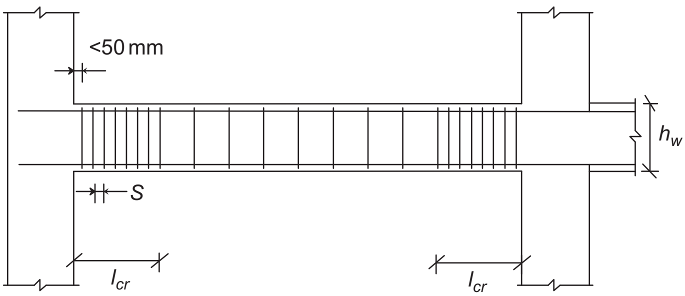


Fig. 2.19 Transverse reinforcement in critical regions of beams. *Reproduced according to Fig. 5.6 of BS EN 1998-1:2004: Eurocode 8: Design of structures for earthquake resistance—Part 1: General rules, seismic actions and rules for buildings, 2002. (Permission to reproduce and derive extracts from British and ISO standards is granted by BSI. British Standards can be obtained in PDF or hard copy formats from the BSI online shop: www.bsigroup.com/Shop or by contacting BSI Customer Services for hardcopies only: Tel: +44 (0)20 8996 9001, Email: cservices@bsigroup.com).*

For columns, confining reinforcement above and below the beam face in the form of closed links at close centers is required. This is to resist bursting stresses and consequent spalling of concrete (Fig. 2.20).

For shear walls, it also has special design equipment and detailing (Fig. 2.21) to be used on a wall in order to design a ductile wall.

2.7.3.3.2 Steel Structures

In steel moment frames design, typical requirements from design codes are to guarantee connections that are provided with a sufficient overstrength factor such that the dissipative zone (plastic hinges) occurs mainly in the beams. This can be achieved by special detailing:

1. Strengthen the beam to column connections, as shown in Fig. 2.22.
2. Locally reduce the beam section, such as trimming or perforating the flange, as it shown in Fig. 2.23.

2.7.3.4 Capacity Design

This design principle is to design the components adjacent to a dissipative mechanism with greater resistance than the dissipative mechanism. This will

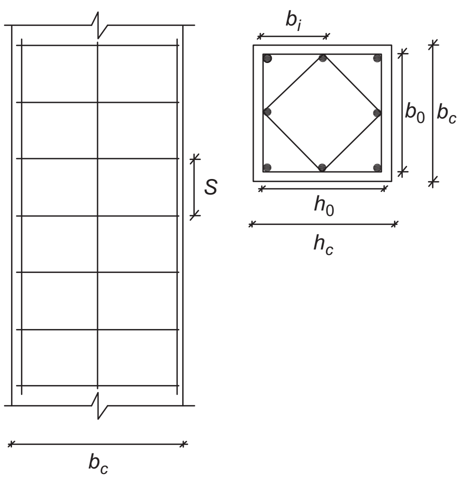


Fig. 2.20 Confinement of concrete core for columns. *Reproduced according to Fig. 5.7 of BS EN 1998-1:2004: Eurocode 8: Design of structures for earthquake resistance—Part 1: General rules, seismic actions and rules for buildings, 2002. (Permission to reproduce and derive extracts from British and ISO standards is granted by BSI. British Standards can be obtained in PDF or hard copy formats from the BSI online shop: www.bsigroup.com/Shop or by contacting BSI Customer Services for hardcopies only: Tel: +44 (0)20 8996 9001, Email: cservices@bsigroup.com).*

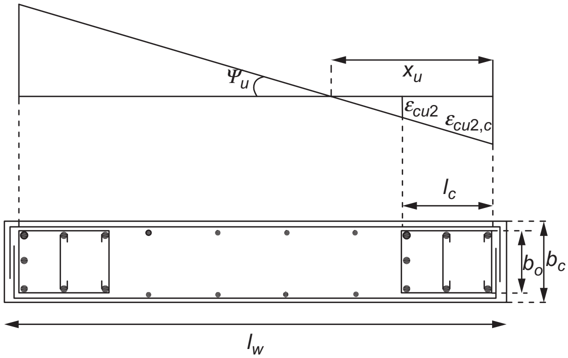


Fig. 2.21 Confined boundary element of free-edge wall end. *Reproduced according to Fig. 5.8 of BS EN 1998-1:2004: Eurocode 8: Design of structures for earthquake resistance—Part 1: General rules, seismic actions and rules for buildings, 2002. (Permission to reproduce and derive extracts from British and ISO standards is granted by BSI. British Standards can be obtained in PDF or hard copy formats from the BSI online shop: www.bsigroup.com/Shop or by contacting BSI Customer Services for hardcopies only: Tel: +44 (0)20 8996 9001, Email: cservices@bsigroup.com.)*

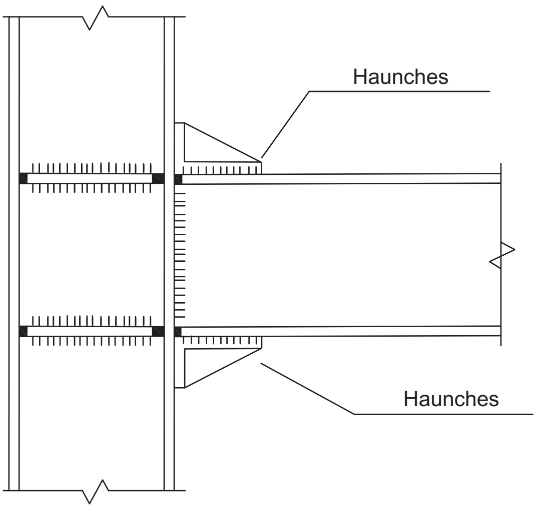


Fig. 2.22 Connection strengthening with haunches.

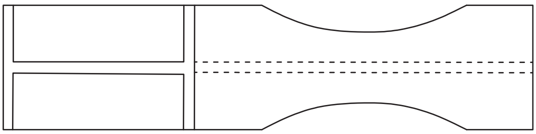


Fig. 2.23 Beam section reducing (a concept originally developed and patented by Arcelor Mittal [10]).

ensure that they remain elastic and stable when overall deformations are taking place.

2.7.4 Avoiding Soft Story-Strong Column Weak Beam Design

For structural frames, it is required to design ductile frames which are capable of dissipating seismic energy in a flexural mode at a significant number of plastic beam hinges. However, column hinge mechanisms are to be avoided. In the design it should make sure that the plastic hinges can form in the beam element (near the column interface).

Buildings are classified as having a "soft story" if that level is <70% as stiff as the floor immediately above it, or <80% as stiff as the average stiffness of the three floors above it [11]. Soft story buildings are vulnerable to collapse in a moderate-to-severe intensity earthquake is a phenomenon known as soft story collapse, as it will sometime results in the collapse of the entire building. Therefore, a soft-story should be definitely prevented in the earthquake design. The way to prevent the soft story is through the so-called strong column-weak beam capacity design, which is to force the plastic hinge out of column and into beams. It is shown in Eurocode 8 [9], this is achieved by making:

$$\sum M_{Rd,c} \geq 1.3 \sum M_{Rd,b} \tag{2.10}$$

where $M_{Rd,c}$ is the design value of column moment resistance at the face of the joint. $M_{Rd,b}$ is the design value of beam moment resistance at the face of the joint.

2.7.5 Structural Regularity

Structural regularity is particularly important in earthquake-resistant design, most of design standards such as Eurocode 8 [9], ASCE/SEI 7–10 [5] outline provisions of regularity in both plan and elevation. This is mainly to minimize the torsional effects.

Vertical regularity. The vertical regularity check is intended to avoid abrupt changes in overall strength or stiffness at any particular level therefore to avoid soft-story. Where such provisions are not met, then a more detailed analysis will be required to ensure that postelastic deformation capacity at each level can be met without unacceptable loss of strength or postelastic deformation demands in excess of their capacity.

It is wise to avoid abrupt curtailment of reinforcing steel at one level of a reinforced concrete frame or substantive changes in a column section. It is better to introduce such changes gradually, over several floors, thereby allowing a smooth transition between sections. Obviously, it is undesirable to curtail rebar in shear walls above their base as this also induces a very real potential for soft-story development.

Horizontal regularity. The horizontal motion comes from earthquake is resisted by the lateral resisting systems in tall buildings. Structures should possess adequate torsional resistance and stiffness in order to limit the development of torsional motions which tend to stress the different structural elements in a nonuniform way. To minimize the torsional effects, the floor plans should be regular and reasonably compact. Wide separation of horizontal lateral load-resisting systems is encouraged. Plan forms such as "L" and "T" plan layouts should be avoided, otherwise, seismic separation joints should be introduced between rectangular blocks.

Systems with structural irregularities. If the regularity has not achieved, both Eurocode 8 [9] and ASCE/SEI 7–10 [5] have the limitations and additional requirements in designing irregular buildings. Refer to them for further guidance.

2.7.6 Design for Structural Integrity

Integrity is important in the earthquake-resistant design. It is required by Eurocode 8 [9] clause 2.1 that *In no-collapse requirement, the structure shall be designed and constructed to withstand the design seismic action defined in Section 2.3 without local or global collapse, thus retaining its structural integrity and a residual load-bearing capacity after the seismic events*. The purpose of this requirement is to make the whole structural system working together.

2.7.6.1 Masonry Structures

For some types of structures, special measures are required. Unreinforced masonry tends to perform badly during earthquakes and rigorous restrictions with respect to height and location, together with special arrangement of ties and anchorages or loop beams or tie beams are imposed. It is essential to tie the walls and floors of a building together with these measures. In addition, concentrated mass at a height such as in chimneys, parapets, cantilever balconies, gables, flying buttresses, and water tanks needs to be carefully detailed, as it can contribute significantly to failure of the structure.

2.7.6.2 Connection Between Superstructure to Foundations

The interconnection of foundations is an important structural requirement, to ensure that the building will act as a unit in an earthquake. This is best achieved by connections in the form of tie beams or slabs at ground level, where the disturbance originates and where the connections provide a measure of safety if ground movements occur.

2.7.7 Measures to Reduce the Earthquake Response

It can be seen that, for tall buildings, earthquake loading is one of the major consideration. There are several ways to reduce the dynamic response. It is summarized in Fig. 2.24. Passive devices reduce earthquake response through dissipating the input energy of the excitation, therefore, allowing the real structure members to withstand less severe actions. An active control system utilizes the special devices to measure the external excitation or response variables, then work out the control force or change the stuffiness of the structure, therefore reduce the total response of the structures under earthquake. The hybrid system is the combination of both passive and active control methods. Some of these methods will be introduced in this chapter.

2.7.7.1 Base Isolation

Base isolation is one of the passive structural vibration control technologies, which consist of layers of steel and rubber inside the isolation to dissipate the energy (Fig. 2.25). They are widely used for high-rise buildings by decoupling them from their substructure.

Fig. 2.26 demonstrates the comparison between the building with or without base isolation. It can be seen that, the base isolation can effectively reduce the response of a tall building. This technology can be used for both new structural design and seismic retrofit.

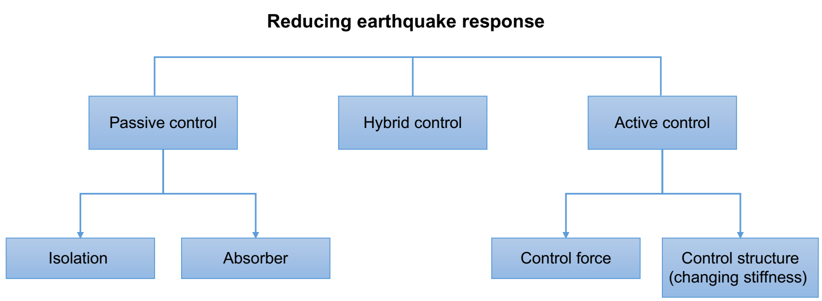


Fig. 2.24 Earthquake response reduction method.

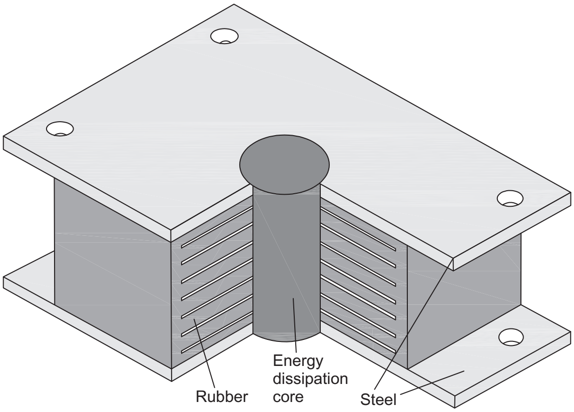


Fig. 2.25 A typical base isolation system.

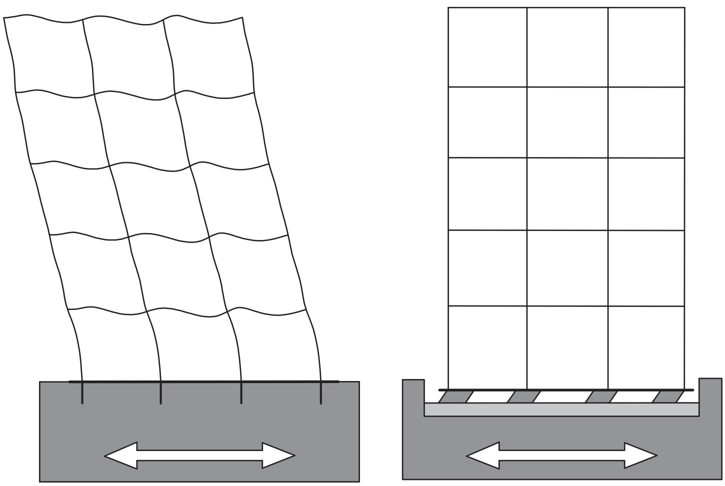


Fig. 2.26 Base isolation in response reduction: (A) no base isolation and (B) using base isolation.

2.7.7.2 Shock Absorber

Shock absorber is the method and concept that uses combination of steel and rubber or hydraulics dampers. It minimizes by protecting the amount of energy that building absorbs from an earthquake.

2.7.7.3 Damping Systems

The response of a tall building under the earthquake is also affected by its structural damping, it comes from the material of structural member of the buildings (either steel or concrete), it can also come from the friction at joints or movement of the secondary members.

For tall buildings, if the structural damping alone is not sufficient to limit the response of the building additional complex damping systems are required. There are three major damping devices that can be used in tall building: Tuned mass damper (TMD), viscoelastic damper, and tuned liquid damper (TLD).

TMD is one of the most popular damping devices. The vibration energy is absorbed through the motion of an auxiliary or secondary mass connected to the main system by viscous dampers. Two types of TMD are used in construction: Translation TMD or Pendulum TMD.

One of the Pendulum TMD examples is Taipei 101. Taipei 101 comprises of the largest TMD in the world, which is installed at the top floors this arrangement can lower displacement due to the forces significantly.

Viscoelastic dampers use the viscoelastic material which dissipates energy as heat through shear stresses in the material (Fig. 2.27). It relies on the operating temperature and heat transfer to adjacent structures.

TLD is a damping system which is similar to TMD, however, the mass, stiffness and damping component are provided by moving liquid (Fig. 2.28).

TLD dissipates energy through forcing fluid through an orifice.

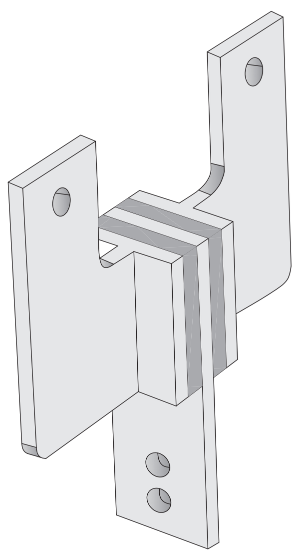


Fig. 2.27 Viscoelastic dampers.

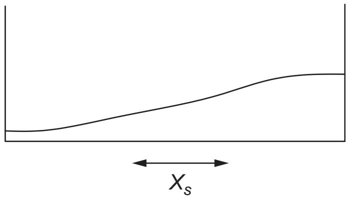


Fig. 2.28 Tuned liquid dampers.

Hysteretic dampers dissipates energy by cyclic yielding in tensions and compression. It is easy to install, but may need to be replaced after major event.

Shape memory alloy (SMA) damper, SMA is able to reach very large recoverable strains, this feature can be utilized in passive seismic structural control. They are usually used for historical, old, and masonry buildings.

2.8 WIND LOAD DESIGN

To understand wind action on tall buildings is important for an engineer. Compared to seismic loading, wind load is the governing load in tall building design in most instances for lateral stability system design. This is due to the longer natural period of the tall building, which results in a smaller earthquake response compared to low rise building. However, it depends on the location and importance of the building, sometimes a necessary check needs to be performed. While designing the Shanghai Tower [12], in order to compare the influences of wind load and earthquake action on the structural member design, the base reactions of 100-year wind load, frequent earthquake (50-year return period) and moderate earthquake (475-year return period) are analyzed. It is found that the structural responses under wind load is larger than those under frequent earthquake, but is smaller than those under moderate earthquake.

For low-rise buildings, such as building below eight stories, oscillation due to wind loading is rarely a problem. For building with 8 and 20 stories, the dynamic effect of wind loading becomes more important with height. The impact of the bracing system on core and elevation design is often greater than in low-rise buildings. For buildings over 20 stories where dynamic behavior under wind loading may govern and where 'whole structure' stability systems such as bundled tubes or external braced tubes may be appropriate.

For a tall building of certain height and slenderness, wind forces resulted motions in the upper levels become dominant factors in the structural design. This is primarily for the sake of the occupants' comfort. It is important to ensure first the buildings to meet strength and safety requirement under wind action for ultimate state design. Secondly, serviceability limit state design of a tall building during wind-induced motion is another critical design issue, as the occupant's comfort needs to be assessed, in addition, the excessive deflection of the structure may cause the fracture of the façade or glazing.

For tall buildings which are close together, the comfort of the pedestrian is also affected by the wind, therefore, in some projects, pedestrian wind environment studies should also be performed.

Therefore, engineers need to take all aforementioned design issues into consideration to deliver a high standard design for clients and future occupants. In most of the cases, the service limit design for occupants' comfort is a major control of the structural system and structural member sizes in tall building design.

In real design practice, tall buildings such as Taipei 101 and Burj Kalifa, extensive program of wind tunnel tests and other studies were normally undertaken by the wind tunnel consultant, to evaluate the effects of wind on building loading, behavior, and occupant comfort.

2.8.1 Fundamental of Wind Loading

Though we are not wind engineers, it is important for an structural engineer to understand the characteristic of wind around tall buildings. In tall buildings design, wind action needs to be considered at an early stage so that the size and structural form can be optimized to reduce wind load effects. The aspect ratio of the structure is a key factor when considering the wind loads effect. In addition, the effects of wind during construction stage should also be considered at the design stage. Therefore, minimize excessive wind loading on the structural elements of the building during construction. The flow of wind is further affected by the roughness of the Earth's surface. Therefore, it is difficult to be interpreted by structural engineers, especially for large structures and, hence, wind tunnel tests are the rational way of predicting unusual wind effects around buildings.

Fig. 2.29 shows the wind blows through a tall building, two wind responses can be observed. Along wind response and cross wind response needs to be considered in tall building.

As shown in Fig. 2.30, the along-wind caused the positive pressure on the windward faces as it is shown in region 1, and suction on the leeward

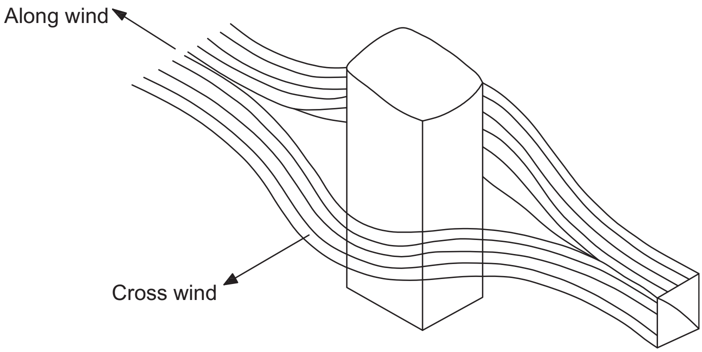


Fig. 2.29 Along wind and cross wind on tall buildings.

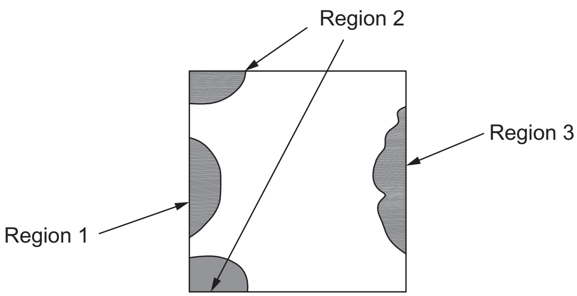


Fig. 2.30 Different wind pressure regions due to along-wind.

as it shown in region 3. There is also a drag effect on the surface parallel to the direction of the wind as it shown in region 2. The combination of these three forces will result a net force working on the structure, therefore, should be considered in the design.

For tall or slender structures, their dynamic effect can produce amplification force on the structure. For along-wind, the dynamic response comes from buffering effect caused by the turbulence. However, in tall building design, the crosswind motion perpendicular to the direction of wind is more critical than the along-wind motion. Due to high design wind speed and relatively low overall stiffness, tall buildings are susceptible to vortex-induced vibration. For supertall buildings, the across-wind load due to vortex shedding is even higher than the along-wind load. It is normal that cross-wind response dominates the design wind load.

2.8.1.1 Along Wind

There are two components in the along wind, a mean component due to the mean wind speed and a fluctuating component due to gust or eddies.

The separation of wind loading into mean and fluctuating components is based on gust-factor approach by many design codes. The mean load component is evaluated from the mean wind speed using pressure and load coefficients. The fluctuating loads are determined separately by a method which makes an allowance for the intensity of turbulence at the site, size reduction effects and dynamic amplification [13].

2.8.1.1.1 Wind Pressure

The pressure of the wind is correlated with wind velocity. There are two types of wind velocities acting on structures named as mean and gust wind velocities. Gust wind velocity is the maximum wind velocity applied on a building having the maximum and highest wind forces. Mean wind velocity is the average of all the wind velocities acting on the structure.

As the height of the structure increases, velocity increases proportionally until it reaches its maximum value. Moreover, the terrain of the area in the specific location of the structure relates to the maximum velocity that will be achieved. For instance, at an open area where there are no obstacles, maximum velocity will be achieved at a lower height compared to an area in urban cities where the skyscraper is surrounded by buildings that block the wind.

The speed of the wind varies with the return periods. In design practice, wind statistics played an important role in predicting levels of response to return period. Data such as ground-based wind data, balloon data and computer simulations employing Regional Atmospheric Modeling techniques are used to establish the wind regime at the upper levels. In the design codes, different return period of wind is chosen for different design purpose such as ultimate design and human comfort design.

The relationship between wind velocity and pressure is defined in design codes across the world. However, for buildings over 40 stories or height over 200 m (EN 1991-1-4:2005. Eurocode 1 [7]) wind tunnel test should be required for accurately assess the wind loading the structures.

Wind load is determined by a probabilistic-statistical method based on the concept of: "equivalent static wind load," on the assumption that structural frames and components/cladding behave elastically in wind.

SEI/ASCE 7–10 [5] gives below equations for wind pressure of along-wind:

$$P_z = \frac{1}{2}\rho(V_{33})^2 \times k_z \tag{2.11}$$

where P_z is the wind pressure at height z above mean ground level, V_{33} is the basic 3 s gust speed in exposure category C at reference height 33 ft. above mean ground level, ρ is the mass density of air, and k_z is the exposure coefficient.

According to Eurocode 1 (EN 1991-1-4, 2005) [14], the wind loading can be expressed as.

$$F_w(t) = \frac{1}{2}\rho C_p \int_0^H (v_m(z) + v(z,t))^2 dz \cong \frac{1}{2}\rho C_p \int_0^H (v_m(z) + 2v_m(z,t))dz$$

where ρ is the air density, H is the total height of the building from the ground, Cp is the pressure coefficient of the building, $vm\ (z)$ is the 10-min mean wind velocity at a height z from the ground, and $v(z,t)$ is the turbulent wind velocity.

$$v_m(z) = v_b k_R \ \ln\left(\frac{z}{z_0}\right), \quad z \geq z_{\min}$$
$$v_m(z) = v_b k_R \ \ln\left(\frac{z_{\min}}{z_0}\right), \quad z \leq z_{\min}$$

where v_b is the basic wind velocity, k_R is the terrain factor, $z_{\min}$ is the height below which the logarithmic profile loses its significance and the mean wind velocity is considered as constant, and z_0 is the roughness length of the site.

Different codes have different formulas. However, the relationships are quite similar. It is difficult to predict responses in the across-wind and torsional directions theoretically like along-wind responses. However, some prediction formula based on the fluctuating overturning moment in the across-wind direction and the fluctuating torsional moment for the first vibration mode in each direction can be developed.

2.8.1.2 Cross Wind

The cross-wind effect is mainly caused by the vortex shedding, which will be introduced in detail in the latter part. The study from Ref. [15] shows that: "Compared with along-wind response, across-wind response is more sensitive to wind speed. At lower wind speeds, the along-wind loads normally dominate but with increase of wind speed the across-wind loads take over." Therefore, the cross-wind acceleration is normally predominant as it is likely to exceed the along wind acceleration.

2.8.1.2.1 Vertex Shedding

Vortex shedding is a phenomenon, when the wind blows across a structural member, vortices are shed alternately from one side to the other, and where alternating low-pressure zones are generated on the downwind side of the structure giving rise to a fluctuating force acting at right angles to the wind direction (Fig. 2.31).

For slender (in tall building design, a structural member may be considered slender if the ratio of the height to its narrowest side exceeds 7 [16]) and lightly damped buildings, large amplitude vibrations in the plane normal to the wind may develop when the vortex shedding is in resonance with one of the natural frequencies of vibration. Vortex shedding induced effects for very flexible members may also be important for higher modes.

The character of the vortex shedding forces depends on the shape of the buildings. Therefore, for supertall buildings such as Burj Kalifa, wind tunnel test was used to shape the building to minimize wind effects through disturbance on vortex shedding over the height of the tower.

The frequency of vortex shedding can be determined by Strouhal number (*St*). The St number is dependent on the cross section of the building or object which flow passes over, and is also dependent on the velocity of the flow, or the Reynolds number. Eq. (2.12) shows how to work out the *St* number.

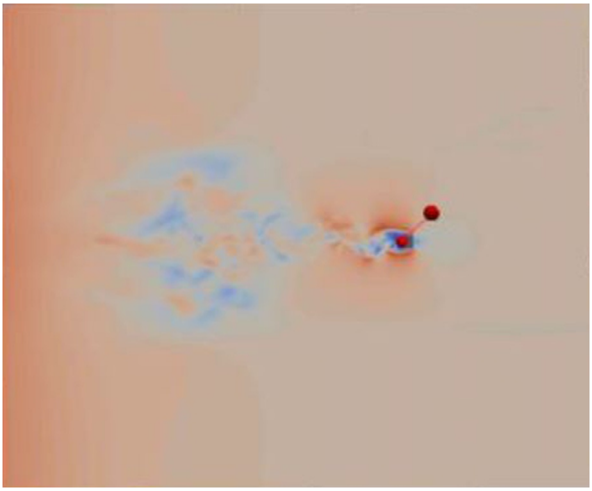


Fig. 2.31 Vortex shedding simulated by CFD software.

$$S_t = \frac{Df_v}{U} \tag{2.12}$$

where S is the St number, D is the width of the building or object over which the flow passes, f_v is the frequency of the vortex shedding, and U is the flow speed.

Once this frequency is close to the fundamental frequency of the building, cross-wind oscillations will be experienced inside the building.

2.8.2 Wind Drift Design

Limits for wind deflection or the relative deflection between adjacent floors in buildings are specified in many wind loading and design codes. In some cases, these limits are given as recommendations rather that as mandatory requirements. The main reasons for adopting wind drift deflection limits are to limit damage to the cladding (see Fig. 2.32) on the building facade and to partitions and interior finishes, reduce the effects of motion perceptibility and limit the P-Delta or secondary loading effects.

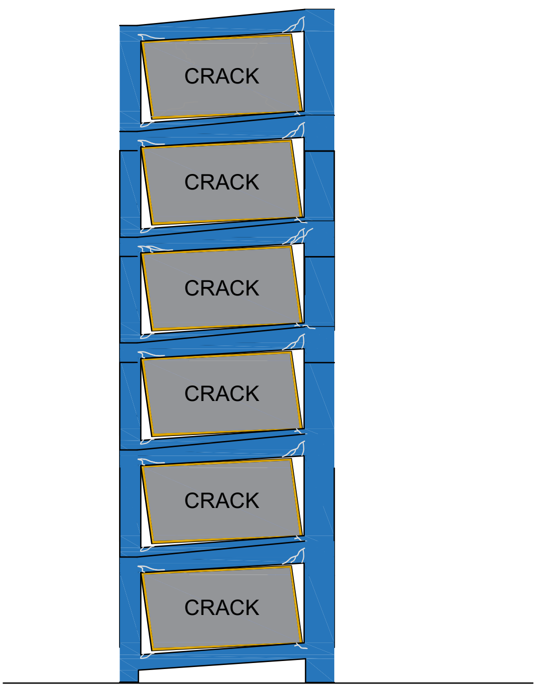


Fig. 2.32 Damage to the cladding due to drift.

2.8.3 Occupant Comfort and Criteria of Buildings to Wind Induced Vibrations

Most occupants in tall buildings will percept a certain degree of motion due to wind actions. With the increasing of the magnitude and frequency of occurrence, occupants' comfort may be affected when they are excessive comfort level. The occupants' comfort during wind-induced motion is generally assessed using the acceleration. Many wind codes and standards predict the acceptance criteria based on the acceleration.

2.8.3.1 Objective of Occupant Comfort Design

The main objective of occupant comfort design is to provide a comfortable environment. It needs to mitigate the fear and eliminating discomfort. People will tolerate discomfort with longer recurrence intervals and short time duration. Therefore, when evaluating the vibration of the building, 1-year recurrence interval becomes common.

2.8.3.2 Human Perception of Motion

The human response to the motion is a physiologically complex problem. In the research, motion simulator or real field investigation are conducted to determine the human perception to the wind induced motion, and the threshold acceleration on the basis of the natural frequency of the building can be determined.

2.8.3.3 Occupant Comfort Design Criteria

The international standards ISO 6897:1984 [17] predict a strong dependence between the threshold of comfort and the frequency of vibration. As shown in Fig. 2.33, The International Standard ISO 6897:1984 [17] specifies the maximum allowable RMS acceleration for a 10-min mean wind velocity with a 5-year return period, for a natural frequency range of 0.06–1 Hz. There are also other international standards such as ISO 2631-2 [18,19] provide essentially similar provisions, focusing on the interval from 1 to 80 Hz.

The recent ISO 10137:2007 standard [19] has the evaluation curve for wind-induced vibrations is given for peak acceleration for a 1-year return period. However, it provides guidelines for both tall and low buildings, the natural frequencies range from 0.06 to 5 Hz. Fig. 2.34 shows the serviceability criteria recommended by ISO 10137 [19] for human comfort against wind induced vibrations of buildings.

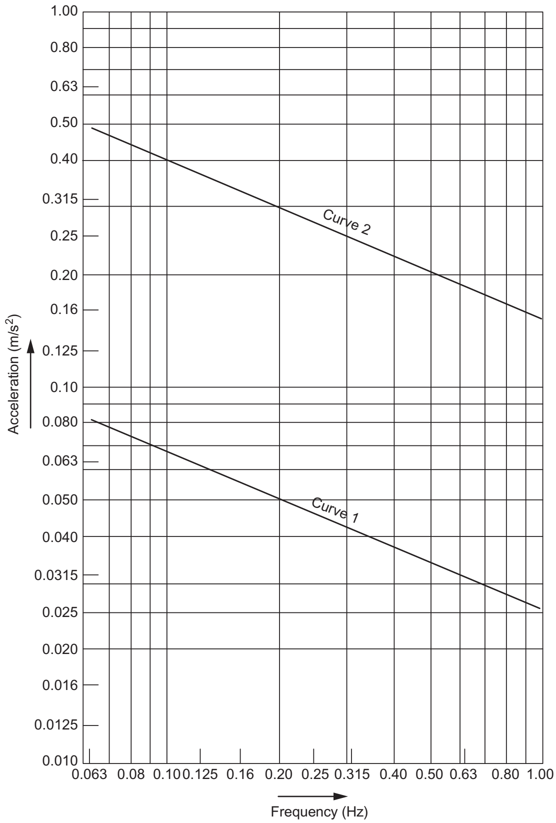


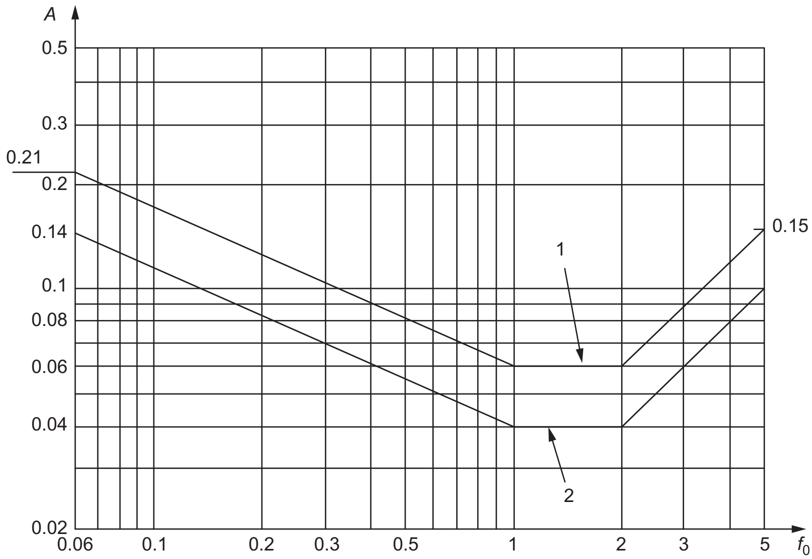
Fig. 2.33 Suggested satisfactory magnitude of horizontal motion in buildings used for general purposed. *Reproduced according to ISO 6897 [13]. Curve 1 is the lower threshold perception curve. Curve 2 is the average threshold perception curve. (Permission to reproduce and derive extracts from British and ISO standards is granted by BSI.)*

2.8.3.4 Mitigation of Building Motions in Design

To enhance the occupant comfort, the effective way is to reduce the motion produced by the wind excitation. The dynamic response of a tall building to wind loading is determined by several factors such as site conditions, shape and height, mass and structural systems.

There are several ways to reduce the motion:

1. Aerodynamic optimization: change the shape of the building, tapering, change corner shape, add spoilers.
2. Change the structural system, change mass, and stiffness.



A peak acceleration, m/s^2
f_0 first natural frequency in a structural direction of a building and in torsion, Hz
1 offices
2 residences

Fig. 2.34 Evaluation curves for wind-induced vibrations in building in horizontal direction for 1-year return period. *Reproduced according to ISO 10137 [19]. (Permission to reproduce and derive extracts from British and ISO standards is granted by BSI. British Standards can be obtained in PDF or hard copy formats from the BSI online shop: www.bsigroup.com/Shop or by contacting BSI Customer Services for hardcopies only: Tel: + 44 (0)20 8996 9001, Email: cservices@bsigroup.com.)*

3. Adding supplementary damping system, such as TMD. One of the famous examples is Taipei 101. It is can easily increase the damping ratio of the building and achieve 30%–40% reduction in the wind-induced vibration [20].

2.8.4 Effects of Neighboring Tall Buildings on Wind

When several tall buildings are constructed in proximity, there is a so-called Venturi effect which describes a wind created by air is squeezed through a narrow space known as "channeling." The fluid flow through the group may be significantly deformed and have a much more complex feature than isolated tall buildings, resulting in enhanced dynamic pressures and motions especially on neighboring structures. In addition, it will also affect the wind environment on the street level, and affecting the comfort for the pedestrians. The combination of these different effects is complicated. Therefore, study of mutual interference among closely located tall buildings is an important problem in tall buildings design. CFD analysis or wind tunnel test should be employed to make the wind environment analysis, to ensure there would be no damage to structures. But the potential effect on people is becoming more of a focus and no effective solutions have been found.

2.8.5 Outdoor Human Comfort Design for Pedestrian on the Street

As discussed in the earlier sections, in certain tall building projects, wind environment studies should be performed to satisfy the outdoor human comfort for pedestrian. This is due to the increasing incidents occurs to the pedestrians due to the strong gust outside the tall building. It is noticed that, high-rise buildings affect the surrounding pedestrian level wind environment. It is also observed that creation of low wind speed areas around tall buildings may lead to poor out-door air ventilation [21].

2.8.5.1 Effect of Wind on Outdoor Human Comfort

As it is reported by BBC News, a sales assistant almost got blown over when walking up the building outside the 20 Fenchurch Street Building, London, due to the strong wind. In Leeds, England, a 35-year-old person was crushed after strong winds toppled a lorry near the 32-story tall building, Bridgewater Place.

Owing to these reported incidents, some regulations are planned by the government. In UK, the City of London Corporation has promised a more rigorous assessment of developers' predictions of ground winds. In Toronto, Canada, a new law is planned to set to ensure planning for skyscrapers takes into account the risk of street winds. However, nothing has been put into enforcement.

The strong wind near tall buildings is caused by the "downdraught effect." As shown in Fig. 2.35, when the air hits a building it is pushed up, down and around the sides. The air forced downwards increases wind speed at street level. In addition, there is also an acceleration of wind around the side of the buildings if it has completely square corners. For corner-rounded buildings, downdraught effect is not severe, as the air does not accelerate around corners.

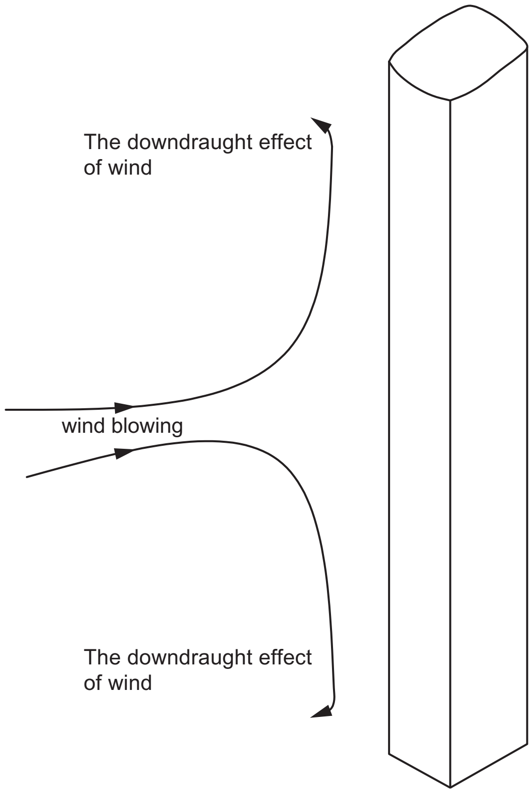


Fig. 2.35 Overall downdraught effect of wind hits buildings.

2.8.5.2 Measures to Improve Outdoor Human Comfort

There are several methods to reduce the effect of wind on the pedestrian level environment. The normal method is to rearrange the environment. In Leeds, the city council granted permission for angled shelters near the base of Bridgewater Place, known as "baffles."

The research of [21] shows:

1. A single-wider building created adverse effects on the natural air ventilation at pedestrian level.
2. A taller building improved the near-field air ventilation conditions.
3. An adverse effect on natural air ventilation at pedestrian level when the building separations are less than half the building width.
4. A wider building separation configuration improved the pedestrian comfort conditions by reducing the total area of the high wind speed zones.

5. Inclusion of a podium was also found to affect adversely the air movement around buildings. The podium resulted in potentially uncomfortable winds over a larger area, which has the potential to affect more people in the nearby surrounding environment.

2.8.6 Wind Tunnel Test

As discussed in the earlier sections, wind tunnel test is essential for most of the building designs. Fig. 2.36 shows a wind tunnel test facilities. The wind tunnel testing program normally includes rigid-model force balance tests, a full aeroelastic model study, and measurements of localized pressures, such as cladding pressure studies and pedestrian winds studies. A brief introduction for the wind tunnel test will be made here for reader's reference.

2.8.6.1 The Rigid Model Force Balance Tests

In wind tunnel studies, aeroelastic model will be accurate as it will provide detailed information of dynamic loads and motion of the prototype. However, it is more complicated and expensive and it cannot be carried out at the initial design stage when the structural system is not finally determined. However, a high-frequency force balance test is an alternative method which is faster and more cost efficient.

Force balance tests measure the mean and quasi-steady dynamic forces based on a rigid model [22]. The model is mounted on a stiff balance, the frequency of both the model and the balance are chosen to be sufficient high.

Fig. 2.36 A wind tunnel test facility.

The test results are combined with the dynamic properties of the building to determine the deflection, acceleration, the resonant dynamic component of the wind loads and responses. It can also determine the overall overturning moments acting on the building, estimate the equivalent static design load distribution over the height of the building, carry out parametric studies on the variation of dynamic properties such as natural frequency, mass, and damping.

2.8.6.2 Aeroelastic Model Study

Aeroelastic model tests are used for particularly complex geometries or structures such as slender or flexible buildings, or where the motion of the building itself may affect the aerodynamic forces. It is based on the dynamically scaled models of buildings. Aeroelastic tests are capable of providing overall mean and dynamic loads, displacement, rotation, and accelerations. Results from aeroelastic tests also include the basic scope as described in the force balance test, and it can provide more information than the force balance test.

2.8.6.3 Measurements of Localized Pressures

These measurements are mainly used for design of the claddings, which enable the structural engineer to understand the real pressure distribution on the glazing. These measurements use the scaled static models instrumented with pressure tap. It is used to test the mean and fluctuating local exterior pressures on the cladding or roofs. It can also estimate the interior pressure including fluctuation in the presence of significant openings [23].

2.8.6.4 Pedestrian Winds Studies

Based on the scaled static model of the buildings or bridges, this test can evaluate pedestrian level winds. It can study the characteristic of the flow around structure, the local wind speed and directions for wind environment studies as it mentioned in the earlier section.

2.8.7 Measures to Reduce the Wind Response

It can be seen that, for tall buildings, due to high wind speed at upper level, tall buildings are more susceptible to wind excitations, particularly to vortex-induced oscillations. There are several ways to reduce the wind response, which will be introduced here.

2.8.7.1 Aerodynamics Optimization

It is widely known that the response of the tall buildings under wind loading is greatly determined by building shapes. Aerodynamics optimization on the shape of supertall buildings in the design stage is an effective way to reduce the wind response. The fluid-based aerodynamic modification method can be used to optimize the shape of the tall buildings.

Research from Refs. [15,24] show that, tapering and stepping softened corners, tapering and setbacks, varying cross-sectional shape, adding spoilers and porosity, or openings in the building elevation of the tall buildings can reduce across-wind responses. However, for a low-return period response with low wind velocity, tapering or stepping may actually increase accelerations affecting occupant comfort. Twisting the shape of the buildings can considerably decrease the maximum across-wind responses; it can also lead to equalized response over wind directions. Although aerodynamic effectiveness of twisting generally increases with the increase of twisting level, the increment of effectiveness tends to decrease with the increase of twisting level and there exists a limit of maximum possible reduction. One of the examples is Ref. [12], in the design of Shanghai Tower, through aerodynamic optimization on the building shape, the wind load can be effectively reduced. Fig. 2.37 shows the possible aerodynamic optimization methods. There are several methods in optimizing the the plan layout, such as corner recession, corner cut, corner slot etc. in addition to that, optimization can be made in changing the elevation shape, such as curvilinear form, setback etc.

2.8.7.2 Damping Systems

The response of a tall building under the wind loading is also affected by its structural damping, it comes from the material of structural member of the buildings (either steel or concrete), it can also come from the friction at joints or movement of the secondary members.

For tall buildings, the structural damping alone is not sufficient to limit the response of the building to satisfy the serviceability design requirements, similar to earthquake design, additional complex damping systems are required to control oscillation due to wind loading. There are three major damping devices that can be used in tall building: TMDs, viscoelastic damper, and TLD; see Section 2.7 for earthquake-resistant design for further reference.

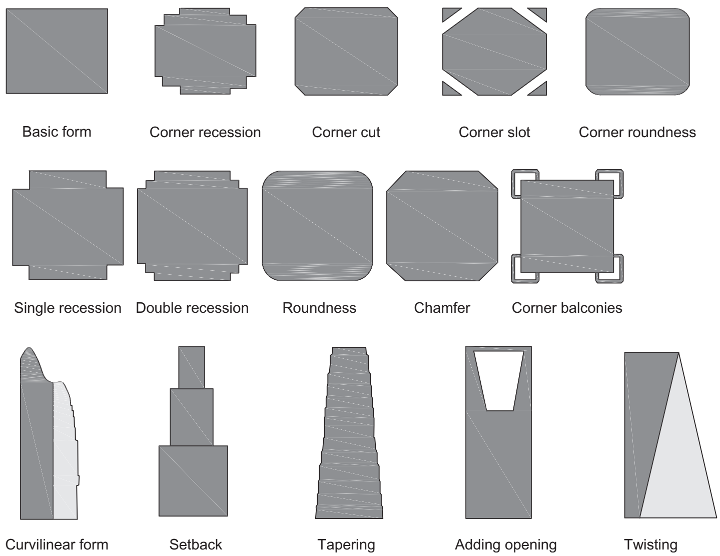


Fig. 2.37 Different methods of aerodynamics optimization. *(Based on references [49], M. Alaghmandan, P. Bahrami, M. Elnimeiri, The future trend of architectural form and structural system in high-rise buildings, Arch. Res. 4(3) (2014) 55–62. [15], J. Xie, Aerodynamic optimization of super-tall buildings and its effectiveness assessment, J. Wind Eng. Ind. Aerodyn. 130 (2014) 88–98.)*

2.9 DESIGN TO PREVENT PROGRESSIVE COLLAPSE

The event of September 11, 2001 (NIST NCSTAR [1]) sparkled the importance of designing a tall building under extreme loading conditions such as blast and fire. An engineer should understand the ways to prevent progressive collapse. Therefore, the major objective in the design is to provide cost-efficient design to minimize injuries and improve the probability of survival of people. In the following three sections, the relevant design and analysis knowledge will be introduced; detailed guidance can be referred to another book of the author Fu [25].

2.9.1 Design Method to Prevent Progressive Collapse

In the United States, Federal agencies such as DoD [26] and GSA [27] imposes standards and guidance documents to mitigate disproportionate collapse. In United Kingdom, guidance documents such as British Building regulation [28] and BS5950 [29] also stipulate design requirement in preventing disproportionate collapse. Eurocode BS EN 1990 [30] and

Eurocode BS 1991-1-7 [31] all have particular relevance to structural robustness.

These design requirements can be divided into two categories: indirect and direct design methods. These approaches are mentioned here.

Indirect design method requires of minimum levels of strength, continuity and ductility in the design.

Direct design method requires that the resistance to progressive collapse is considered directly during the design process. Alternate load path method is one of direct design method which is to be assessed on the structure with sudden removal of critical structural members such as columns to study the structures ability to redistribute the forces on the structure without progressively collapsing."

2.9.2 Progressive Collapse Analysis Method

To be able to assess the potential of progressive collapse, four main analysis procedures based on column removal were included in GSA [27]. They are:
- linear static,
- linear dynamic,
- nonlinear static, and
- nonlinear dynamic.

These analysis procedures can be implemented using the conventional analysis software in the current design market. They can be summarized as in Figs. 2.38 and 2.39.

2.9.3 Detailed Requirements in Tall Building Design

In both British building regulations [28] and DOD [26], the selection of the above design methods is primarily based on the occupancy category of the building. For tall building, based on the category Table A.1 in British building regulations [28], it falls into consequence classes above 2b. Based on the category Table 2-1 in DOD [26], it falls into occupancy category above III. From both design guidance, requirement on Tie Forces; alternate path for specified column and wall removal locations; enhanced local resistance are required. British building regulations [28] also required "systematic risk assessment of the building should be taken into account all critical situations, correspondent design measures should be taken" for class 3 buildings. The readers can make a selection based on these two tables according to the height and function of the tall building to be designed.

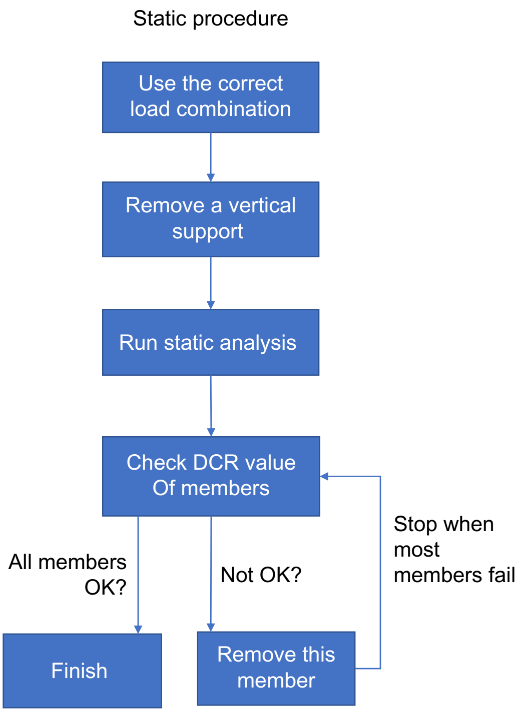


Fig. 2.38 Flow chart of static procedure for progressive collapse analysis.

In general, for steel structures, connections should be designed for the relevant tie forces. The strength of column splices (and some other connections) may need to specify tension as well as compression. Beams restrained by floors may need checking for twist about their web causing eccentric loading and failure. Important structural elements such as transfer beams, hangars should be designed as key element.

Similar to steel, for in situ concrete structures, good detailing should provide sufficient reinforcement anchored to transmit the tie forces. Transfer beams will require special consideration and columns should be designed with minimum 1% reinforcement.

2.10 FIRE SAFETY DESIGN

Fire safety is particularly important for tall building design, especially after the collapse of the World Trade Centre 7 due to fire. In United Kingdom the Building Regulation 2010 "Fire Safety, Approved Document B" [32] has

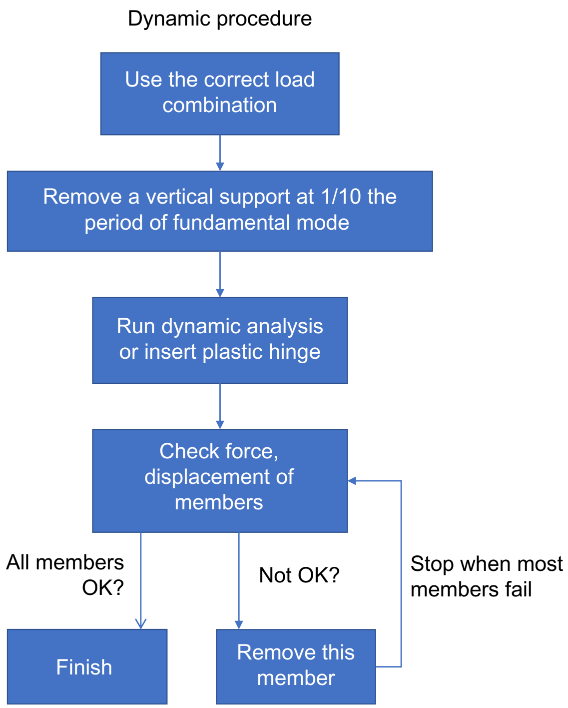


Fig. 2.39 Flow chart of dynamic procedure for progressive collapse analysis.

the detailed regulation of fire safety design, which includes the regulations on "Means of warning and escape, Internal fire spread (linings), Internal fire spread (structure), External fire spread, Access and facilities for the fire service." In the design of Burj Khalifa, currently the tallest building in the world, some special design measures for fire accidents are used, such as set up a separate refuge room, active fire protection method. It worth noting that, high-power fans are also installed inside the building, which is used to clean the smoke in the event of fire and to keep the evacuation route clear [33].

There are three key factors in fire safety design of tall buildings, they are:

- The evacuation route design
- The compartmentation design (fire spread mitigation)
- The structural fire design

In the fire safety design, these three factors are affecting to each other, for example, when designing the evacuation route, the time of evacuation is affected by the time of failure of structural members. Readers can refer to Ref. [34] for further details.

2.10.1 The Evacuation Route Design

Due to the height of tall buildings and limited number of vertical escape routes, it is important to design a safe zone such as refuge room to guarantee the safety of occupants once reached and allow safe transit to a place of refuge. There are different evacuation philosophies to be applied to tall buildings. The travel distances determine the evacuation times in consideration of the heating times and failure time of structural elements. "The route should allow occupants to escape the building as quickly as possible, while sheltering them from smoke and flames." As the vertical evacuation is a dominant design consideration, therefore, stairwells must remain smoke and heat free. So as mentioned, in Burj Kahlifa high-power fan is used.

2.10.2 The Compartmentation Design

"Another key strategy is to correctly design fire compartments to keep the fire from spreading quickly. This entails placing barriers in the building—such as fire-resistant doors and walls—to confine the fire to a local area, or at least slow the speed at which it can spread. These compartments are designed based on the function of the buildings by architects, so residential and commercial buildings will have different compartment design strategies" [35].

In order to ensure that occupants can remain safely, adequate compartmentation must be provided in support of the evacuation strategy.

It is also essential that the fire be prevented from spreading upwards or downwards from the floor of origin, endangering the lives of those waiting on more remote floors. In fire safety design, the designers normally focus on the interior of the building, however, the incident in June 14,2017 [35], the fire disaster in the Grenfell Tower, West London shows that the cladding are most likely to be the major cause that made the fire spread within 15 min from level 4 to above floors. Grenfell Tower is a 24-story tower built in 1974 by Kensington and Chelsea London Borough Council, London. In the middle of the night on June 14, 2017, while most residents were sleeping, a devastating fire started, there were at least 79 people believed to be dead. The tower was built primarily using the concrete which is good in term of fire resistance. The biggest question is over the use of insulation cladding on the outside of the building, installed in a renovation just last year. It caught fire and appeared to spread quickly the flames across the whole facade rather than keeping it confined. "The material used for the cladding was primarily aluminum, which is not good in fire resistant. What's more, aluminum has high conductivity, so the cladding itself could have heated up very quickly, failing to prevent the fire from traveling through the windows and up the exterior of the block from one story to another" [35].

In Approved Document B [32], the Building Research Establishment, recommends that fire-proof barriers should be installed with cladding to prevent fire traveling up insulation. Therefore, the cladding is also one of the important design considerations.

2.10.3 The Structural Fire Design

The structural fire design determines the performance design of the buildings under fire. As it is required in Ref. [36], the main objective for structural fire design is to make sure that when fire occurs, the main stability structural system of the tall buildings will continue to function until all occupants have escaped, or been assisted to escape, from the building.

There are major structural fire design codes, such as Eurocode 2 Part 1–2 [37]. Eurocode 3, Part 1–2 [38], and Eurocode 4, Part 1–2 [39]. Design codes for fire safety in buildings can be either a prescriptive type or performance-based type. The main differences between them are the following.

2.10.3.1 Prescriptive-Based Design

This method is to set up safety factors by constraining design output to pre-established bounds, in other words, it is to design the structure based on fire rating of materials which is compliance with a code specified value. If a designer follows these rules, they will fall within the bounds.

2.10.3.2 Performance-Based Design

A designer needs to first understand the level of the performance is expected, and then satisfy these levels in the design, which includes evaluating the strength and stiffness of the structural members for a particular design fire therefore to achieve the robustness of the structure.

2.10.3.3 Structural Fire Analysis

The performance-based design requires accurate assessment of the fire resistance of the structure. Therefore, an accurate fire numerical simulation method is important. In current design practice, there are three major fire numerical methods:

- Zone model, this is a computer model which divides the fire compartment into separate zones, inside each zone, the condition is assumed to

be the same, and is applied with the fire temperature. One of the most used software is called OZONE. However, this method is ideal to model local fires rather than the global fire response for large-scale buildings.
- Computational fluid dynamics (CFD), which is directly using the conservation law of physics to perform the heat transferring analysis, it is very expensive in term of computational cost.
- Finite element modeling (FEM).

For structural engineers, the most frequently used modeling method is FEM. General purpose programs such as Abaqus and ANSYS can be used for analyzing tall buildings exposed to fires.

The analysis procedure based on the FEM can be summarized as:
- Identify the compartment layout, size, and opening (this is mainly based on the architectural drawings), then determine the atmosphere temperature of the compartment.
- Determine thermal response of structural members.
- Determine the structural response of the building under the thermal response.

In finite element analysis, either standard fire temperature curve or parametric fire temperature curve (Fig. 2.40) is used to represent the fire development (atmosphere temperature) inside a compartment. Standard fire curve (Fig. 2.40) is used mainly for determination of the properties of structural element under fire; however, it is still used by most engineers for structural fire analysis. The parametric temperature-time curves is given by EN1991-1-2:Eurocode 1; Part 1.2 [40], it is more close to the real fire development, therefore is used more widely by the engineers.

There are two types FEM method for fire analysis, which are:
1. **Multiphysics fire analysis** (coupled stress-thermal analysis)
   In this method, the thermal analysis (heat transferring analysis) such as convection, radiation, conduction is performed, as well as the stress analysis. In the thermal analysis procedure, the engineers need to define the certain thermal coefficient such as Stefan-Boltzmann constant, the software will work out the temperature increase for each structural member. The available software can be used is ANSYS. This method is more applicable in analyzing the detailed behavior of each individual structural members.
2. **Codified analysis method**
   When running the 3D FE structural fire analysis, the temperature increase of the protected or unprotected structural members are determined based on equations from design guidance such as Eurocode 3,

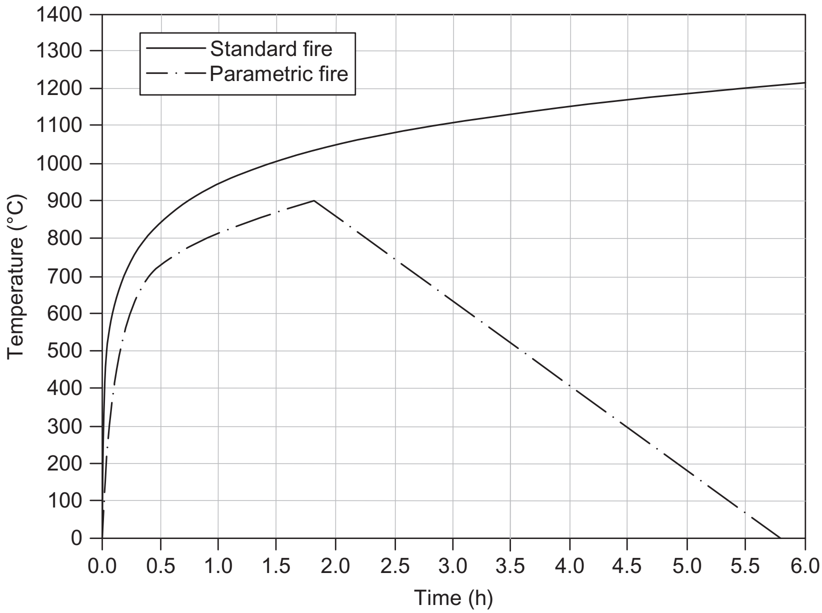


Fig. 2.40 Standard and parametric fire temperature. *The standard fire curve is reproduced from Figure 2 of Page 33 BS 476-20: 1987 Incorporating Amendment No. 1. Fire tests on building materials and structures—Part 20: Method for determination of the fire resistance of elements of construction (general principles); The parametric fire curve is reproduced according to BS EN 1991-1-2 (2002): Eurocode 1: Actions on Structures-Part 1-2: General actions—Actions on structures exposed to fire, British Standards Institution, 2002. (Permission to reproduce and derive extracts from British and ISO standards is granted by BSI. British Standards can be obtained in PDF or hard copy formats from the BSI online shop: www.bsigroup.com/Shop or by contacting BSI Customer Services for hardcopies only: Tel: +44 (0)20 8996 9001, Email: cservices@bsigroup.com.)*

Part 1-2 [37] and Eurocode 4, Part 1-2 [38]. They are used to determine the temperature increase of steel and concrete members, no actual heat transferring analysis will be performed to determine the temperature increase of the structural remembers. This method is more applicable in analyzing the global behavior of the building structures. The available software can be used is Abaqus; refer to another book of the author Fu [25] for further details.

2.10.4 Summary

The fire safety design is an essential design consideration, as a key member of advanced analysis team in WSP group, the author has been working on the structural fire design of the Shard, the tallest building in Western Europe.

I particularly understand the importance of the fire safety design. When the accident of Grenfell Tower occurred, I was invited by the website *The Conversation* to write an article about the fire safety design [35], I expressed my concerns about the old buildings such as the Grenfell Tower. Most of the current guidelines across the world contain detailed design requirements for fire safety such as evacuation routes, compartmentation, and structural fire design. However, most old buildings do not conform to the latest guidelines for fire safety design, they become particularly vulnerable when fire happens. In Grenfell Tower, only one staircase was designed, and no sprinklers were installed. They are also the reasons to cause the casualties. So, it is imperative to update the old buildings by installing sprinklers, fire alarms, and extra fire evacuation staircases.

2.11 DESIGN OF TALL BUILDING UNDER BLAST LOADING

The protective objective for a building under blast loading is related to its type and function. As the primary asset of a tall building is its occupants, therefore, to save life is the main objective of designing a tall building under blast loading. Therefore, while designing, we need to minimize the damage of the structures and prevent the collapse of the building. To achieve this target, designing blast-resistant structural elements and providing hazard mitigating measures are essential in the design.

2.11.1 Fundamental of Blast Loading

An explosion is a large-scale, rapid, and sudden release of energy. Explosive materials can be classified as solids, liquids, or gases. It can also be identified as physical, nuclear, or chemical explosions. According to their causes and delivery methods, there are different types of explosion scenarios, such as vehicle bombs, package bombs, mortar bombs, culvert bombs, airplane crash-induced blast, gas-release-induced blast. The effect of them would be determined using the TNT explosive as a basis. In the design itself, the blast overpressure-time profile is frequently used. The blast loadings are evaluated using empirical relationships based on principle of the scaling law which are used by the design guidance such as SCI [41] and UFC 3-340-02 [42].

2.11.2 Hazard Mitigating Measures

It would be ideal to prevent or delay a potential terrorist attack to happen, although this is hard to achieve. This can only be achieved in certain extent by some preventive measures which will prevent an attack by making it more difficult to implement or delay the attack.

1. Access control by making security checks at entrance, this will exclude the possibility to bring the explosive such as backpacker bomb or package bomb into the building.
2. Maximize the stand-off from a vehicle-borne device, such as preventing close parking of vehicle to the building or installation of bollards, provide a physical barrier (a fence or wall). Constructing a building not close to public road.
3. Providing shelters which are fully enclosed structures and are used to protect personnel from injury, prevent damage to valuable equipment, and prevent detonation of sensitive explosives. One of the examples is to provide blast doors at the entrance.

2.11.3 Design a Blast Resistant Structure Members

If a blast attack has been delivered, whether the structural member can resist or partially resist the impart loading, so that providing enough time for occupants to evacuation, becomes important. Therefore, designing a structural member such as beam, column, slab, wall, and cladding to resist blast loading is another way to achieve design objective.

Owing to the high rates of strain, the dynamic yield strength increase and it can be much greater than the static yield strength, therefore a dynamic increasing factor (DIF) is used in the design, which a designer can use, as shown in Eq. (2.13).

$$\mathrm{DIF} = \frac{\sigma_{\mathrm{dyn}}}{\sigma_{\gamma}} \tag{2.13}$$

Different material such as steel, stainless steel, and concrete correspondent to different DIF values, check SCI [41] and UFC 3-340-02 [42] for these values.

In design code such as UFC 3-340-02 [42], when designing a structural member such as beam, column, slab, or wall under blast loading, the SDOF analysis method is used to determine the response of structures and load configurations by transforming the structure into a SDOF equivalent lumped-mass system. When using this method, the transformation factors which have been readily worked out by the design code can be used for the transformations. Then based on the energy method, the response of the structural members can be worked out and the member size can be designed. The codified method has it is limitation, it can only be used to determine the response of structural members under certain support conditions, the most accurate and fast design and analysis method is to use the commercial software.

2.11.4 Design and Analysis for Overall Response of Tall Buildings Under Blast Loading

The ENV 1991-1-7 [30] proposes three approaches to design for accidental actions, each assigned to an accidental design situation.

Category 1: defined as having "limited consequences," requires no specific considerations for accidental actions.

Category 2: with "medium consequences" requires either a simplified analysis by static equivalent models or the application of prescriptive design/detailing rules, depending on the specific circumstances of the structure in question.

Category 3: which relates to "large consequences" recommends a more extensive study, using dynamic analyses, nonlinear models and load-structure interaction if considered appropriate.

Most tall buildings will fall into Category 3, therefore, detailed analysis would be required. The most accurate and fast design and analysis method is to use the commercial software such as ABAQUS, ANSYS, AUTODYN, ATBLAST, CONWEP, and NASTRAN. (see Fu [25] for further details).

2.11.5 Analysis of Building Response Using Pressure-Impulse (Iso-Damage) Diagrams

The behavior of the building under blast loading can also be represented diagrammatically by pressure-impulse diagrams (it is also called Iso-damage curves). These diagrams are developed based on analytical, experimental, or real-life events. The pressure-impulse diagrams can be made for evaluating the damage level of structure or structure members, such as beam, slab, and wall, it can also evaluate the level of human injuries. They are commonly used in preliminary blast-resistant design to assess the maximum of damage-related parameters. It is a simple but effective way to study the various response regimes that may occur. It can easily tell the designers the damage level for different explosion scenarios. Fig. 2.41 is an example of pressure-impulse curves for a laminated glass pane. It can be seen that the pressure-impulse diagrams are used together with blast parameter such as

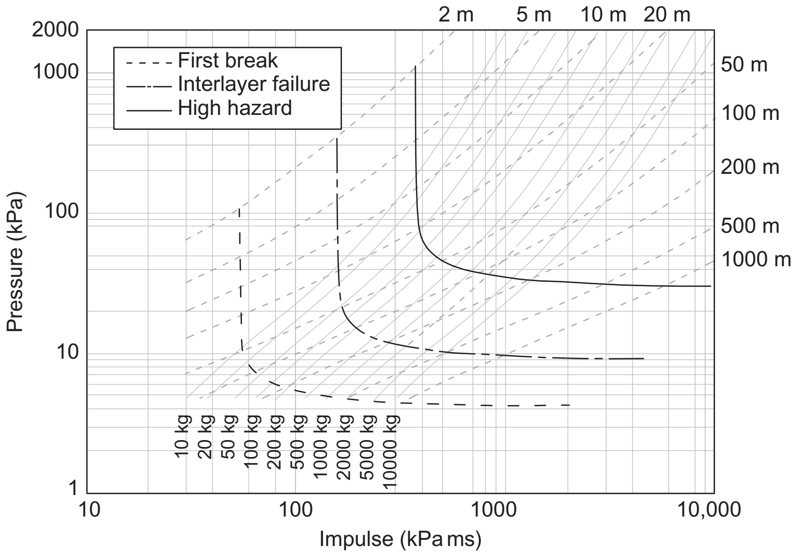


Fig. 2.41 Schematic pressure-impulse curves for a laminated glass pane.

TNT weight and scaled distance graphs by simply overlaying them together into one diagram. It gives level of damages. The three different lines are corresponding to different level of damages, starting from first break, to interlayer failure until total damage. When the parameters of the blast are known, a designer can easily check the level of the damage. For example for a 20 kg of TNT at 5 m range, the damage level falls above high hazard, which indicates almost complete damage of the panel.

2.12 FOUNDATION DESIGN FOR TALL BUILDINGS

Foundation design for tall buildings is an important design task. Owing to the feature of tall building, some special design issues need to be noticed by the design engineers.

2.12.1 Major Design Issues

There are several issues that need to be addressed:

- The foundation should be able to accommodate the extra heavy gravity load.
- The lateral load such as wind and earthquake will produce consequent additional moments and lateral force on the foundation system which will also cause huge vertical loads on the foundation.

- Owing to the dynamic effect of wind and earthquake, for pile foundations, full mobilization of shaft friction along the pile may happen. It may degrade the foundation capacity, give rise to resonance, and cause increased settlement.
- As the tall buildings are frequently built together with low-rise buildings such as podium, the differential settlements between them also need to be controlled.

2.12.2 Foundation Types

Depends on the location of the tall building and the soil profile of site, piles foundation is the most frequently used foundation type. For supertall buildings such as Jeddah Tower, pile foundations probably will be the only solution. As shown in Fig. 2.42, raft foundation is another major type of foundation for tall buildings. The combined pile-raft foundations are used widely as a particularly effective form of foundation system for tall buildings. This is because the raft is able to provide a reasonable measure of both stiffness and load resistance. As explained in Ref. [43], it has benefits as mentioned below:
- Raft can provide most of the stiffness at serviceability loads.
- Raft can provide additional capacity at ultimate loading.

Fig. 2.42 Raft foundation for a tall building in Manchester, U.K. *(Photo taken by the author.)*

- Raft can reduce the required number of piles when the raft provides this additional capacity.
- Raft can provide redundancy to the piles.

2.12.3 Effect of Soil to the Foundations Under Earthquake Loading

Under earthquake, piles may be subjected to significant additional lateral forces as a result of the soil mass attempting to move past the pile group. Settlement due to earthquake excitation may also occur in loose dry noncohesive soils. The possibility of liquefaction under earthquake motion should be investigated for noncohesive soils underlie such as loose saturated sands. This is because this type of soil may lose their strength due to liquefaction under the cyclic effect of earthquake motions. It will result in serious foundation failure. Friction piles may lose their bearing capacity and slender end bearing piles may become laterally unstable. There are some measures such as dynamic compaction or grouting, ground replacement may be used.

2.12.4 Soil-Structure Interaction

Under earthquakes, the structural displacements and the ground displacements are not independent of each other. The response of the soil influences the motion of the structure and the motion of the structure influences the response of the soil. This is called as soil-structure interaction (SSI).

SSI is a complicated problems, it requires the use of commercial software. NIST GCR12-917-21 [44] summarized two analysis methods:

1. **Direct method**, as shown in Fig. 2.43, the soil is modeled using finite elements along the boundary of the foundation, interface elements are defined between the foundation and the soil. However, this complicate mode will cause a computational difficulty, especially when the system is geometrically complex or contains significant nonlinearities in the soil or structural materials.
2. **Substructure approach**.
   This method is to dissemble the soil, foundation and superstructures into separate models, then it will:

- Evaluate free-field soil motions and corresponding soil material properties.
- Evaluate transfer functions to convert free-field motions to foundation input motions.

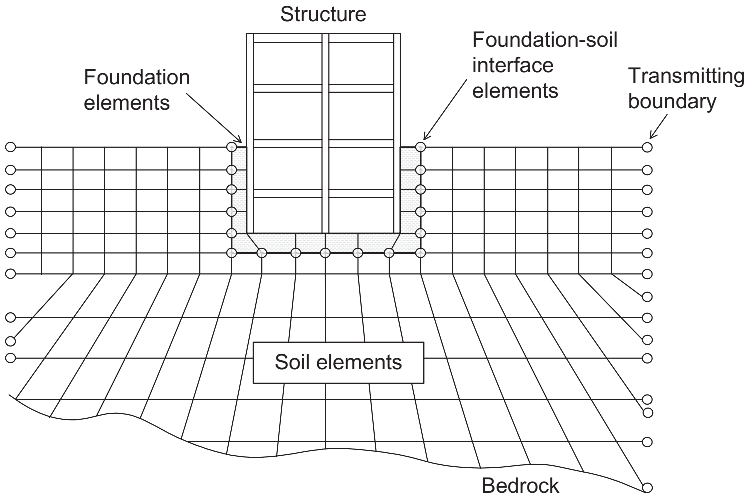


Fig. 2.43 Schematic illustration of a direct analysis of soil–structure interaction using continuum modeling by finite elements. *(Permission to reproduce and derive from Figure 1.1 of National Institute of Standard and Technology: Soil-Structure Interaction for Building Structures, NIST GCR12-917-21 [44].)*

- Represent the stiffness and damping at the soil foundation interface, using springs and dashpots (or more complex nonlinear elements).
- Response analysis of the combined structure spring/dashpot system with the foundation input motion applied.

2.13 CONSTRUCTION METHODS AND TECHNOLOGIES

Owing to its unique features, there are some different construction methods and technologies used in tall building constructions, which is to be introduced here. One of the notable example is the construction of the Shard, the tallest building in Western Europe. Many innovative construction methods have been used in this project, such as the top-down method and the plunge columns method. I was fortunately to be one of the design team members.

2.13.1 Top-Down Construction

In current construction practice, a top-down construction method has been gradually adopted for most high-rise projects. It is used to reduce the duration of the overall construction schedule. This new construction sequence allows the construction of the building superstructure together with the ground excavation and basement construction. This method excavates and

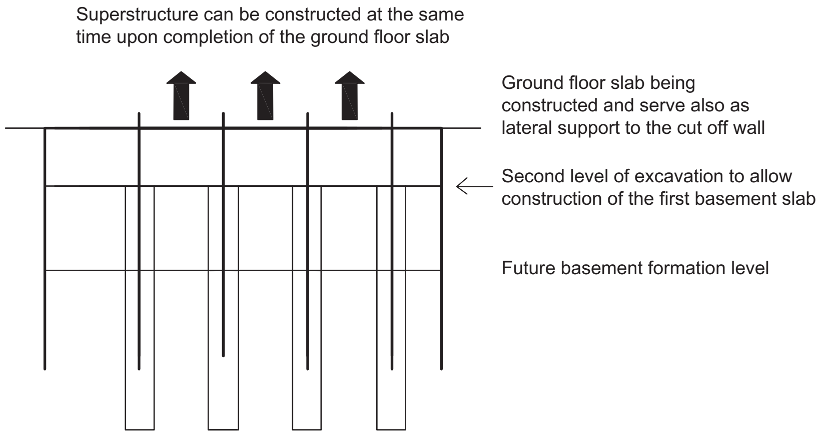


Fig. 2.44 Top-down construction sequence.

constructs an underground structure from top to bottom. The frame and slabs of the substructure are constructed during excavation and work as the supporting structure. In the construction, the pile foundation is imbedded, and then the ground floor slab is constructed as the top slab of excavation, which allows the excavation of the substructure and construction of the superstructure at the same time, as shown in Fig. 2.44.

Advantages of top-down construction:

1. Reduce the total construction time.
2. The deformation of the surrounding ground and adjacent buildings and facilities attributable to the excavation is also decreased because of the high stiffness of the supporting structure. Can control deformation attributable to deep excavation protect the surrounding buildings and facilities is a question?

Disadvantages of top-down construction

1. As earthwork is carried out below the substructure, the excavated soil must be hauled to the outlets on the slab and then taken out, which affects the efficiency of the excavation.
2. The time required for construction and curing of underground reinforcement concrete (RC) slabs is longer.
3. Conflict between substructure construction and earthwork also results in a discontinuous excavation process.

2.13.2 Plunge Columns

In the project of 306-m tall the Shard, a new construction techniques plunge column is used. A plunge column is a structural steel or concrete section embedded in a freshly poured concrete pile, eliminating the need for baseplates and holding-down bolt which allows for a top-down construction sequence. In the construction of Shard, a three-story basement are supported by BBGE Plunge Column foundations.

However, the construction process of plunge columns is quite complicated, a temporary frame needs to be set for the positioning and installation of the plunge columns. Plunge columns typically need to be supported inside the excavation after the frame has been removed. To achieve this support, gravel is typically added around the column. It is imperative that the gravel exerts an even more pressure around the column so it does not push the column out of position. Therefore, poor workmanship may cause the different support condition and extra lateral load and bending may be produced in the construction stage. Therefore, the strength and stability of the plunge column need to be further investigated.

2.13.3 Construction Technology in Burj Khalifa Tower

The Buji Khalifa is currently the tallest building in the world. As it is introduced in Abdelrazaq [45], several different types of latest construction technologies are used. It worthwhile to be mentioned here:

- The walls are constructed using Doka's SKE 100 automatic self-climbing formwork system.
- The floor slabs are poured on MevaDec panel formwork.
- Three primary self-climbing Favco Tower cranes are located adjacent to the central core, with each continuing to various heights as required.
- A specialized GPS monitoring system has been developed to monitor the verticality of the structure.
- Two of the largest concrete pumps in the world were used to deliver concrete to heights over 600 m in a single stage.
- A horizontal pumping trial was conducted prior to the start of the superstructure construction to ensure pumpability of the concrete mixes.

2.14 CREEP, SHRINKAGE, AND COLUMN SHORTENING EFFECT

It is widely known that concrete has creep and shrinkage. Depending on the structure system and construction sequence, this feature would affect the differential vertical displacement in tall buildings. Especially in hybrid structures [46], which are the combination of steel and concrete structural systems, as the steel structure does not have creep and shrinkage as it does for concrete structures, therefore, some countermeasures need to be used to avoid the different vertical displacement.

In reinforced concrete tall buildings, creep and shrinkage will cause long-term column shortening, which is highly correlated to the axial stress level in the column and its development in time. The column shortening will also make the floor undergo differential displacements from their supports, which induce stress redistributions and gravity load redistribution. Therefore, it must be carefully investigated. In addition, it will also cause excessive deformation to nonstructure elements such as facades, pipes, rails and partitions. In addition, differential shortening between columns and core wall causes additional stress on the structural members. Therefore, the column shortening effects need to be taken into consideration in the designing stage which is normally checked after the design structure such as wind and gravity loadings.

2.14.1 Shortening Analysis

When predicting the column shortening, the construction method and phases must be taken into account as a key factor. Different construction phases should be taken into account, together with the loading history. Most of it is based on the latest computer modeling software and it can help the designers to evaluate long-term effects of column shortening, also keeping track of the different construction phases. The construction stage analysis is quite important when considering the column shortening. This is because the dead load is applied sequentially due to the construction sequence.

2.14.2 Mitigating Column Shortening, Shortening Compensation

To tackle differential shortening problem, it is important to adjust the stress ratio of vertical members in the design stage. Based on the computer analysis, stress compensation using special construction method is a solution.

Kim et al. [47] introduced three different methods to control the column shortening:

- Raise a column during the construction phases; however, it requires high workmanship and extra cost.
- Place additional reinforcement in the columns.
- Connecting the columns and core wall with rigid joined horizontal members such as stiff beam or outrigger, however, these horizontal

members need to be designed to accommodate the extra shear stress caused by the shortening.

Kim et al. [48] introduced another method to release the stress caused by differential shortening between the core wall and the columns, the postinstallation of the belt wall joint can minimize gravity load transfer between core and adjacent columns, therefore minimizing the stress redistribution caused by the shortening effect.

2.15 CLADDING

Cladding is prefabricated panels that are attached to the structural frame of the building. The main function of cladding is to prevent the transmission of sound, provide thermal insulation, create an external façade, and prevent the spread of fire. There are different cladding systems, such as curtain wall, metal curtain, stone cladding, brick claddings, precast concrete, and timber cladding.

Cladding systems are nonstructural elements. However, cladding can play a structural role in transferring wind loads, impact loads, and self-weight back to the structural framework. In particular, wind causes positive and negative pressure on the surface of buildings, so cladding must be designed to have adequate strength and stiffness to resist this load, both in terms of the type of cladding selected and its connections back to the structure. Particularly in tall buildings, the wind pressure on the glazing is one of the important design considerations, this is because if one glazing fails, when it is fallings off, it will also hit the glazing on the below floors, which will cause a continues failure of the glazing.

Fig. 2.45 shows the typical connection between the façade to structural members. On tall buildings, access systems must be provided to cladding system allowing regular inspection, maintenance, cleaning, and replacement (in particular, replacement of external seals).

Curtain wall are used widely for tall buildings. Typically, curtain wall systems comprise a lightweight frame onto which glazed or opaque infill panels can be fixed. These infill panels are often described as 'glazing' whether or not they are made of glass as shown in Figs. 2.46 and 2.47.

The frames play an important role in transferring loads back to the primary structure of the building and accommodating differential movement and deflection. Therefore, needs to be designed in detail. In some companies, therefore special façade design team to handle it.

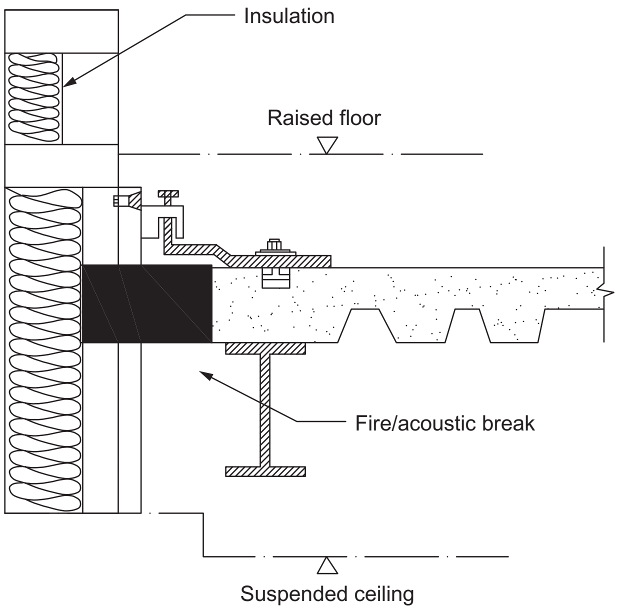


Fig. 2.45 Connection details of cladding to structural members.

Fig. 2.46 A typical cladding of a tall building in Hague, Netherland. *(Photo taken by the author.)*

Fig. 2.47 A glass curtain wall example of a building in Delft, Netherland. *(Photo taken by the author.)*

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CHAPTER THREE

Shear Wall, Core, Outrigger, Belt Truss, and Buttress Core System for Tall Buildings

Abstract

In this chapter, the shear wall and the core system will be introduced. They are followed by outrigger and belt truss system, a case study of the tallest building in the Western Europe the Shard is made to demonstrate the outrigger system. The buttressed core system is also introduced. A case study of the Jeddah Tower is also made to demonstrate the buttress core system in the last section of this chapter.

Keywords: Outrigger, Belt truss, Ring truss, Buttress core

3.1 INTRODUCTION

In Chapters 4 and 5, tube system, diagrid system, and mega-frame system will be introduced in detail. From the introduction of these systems, it will be seen that for supertall buildings, core is a key element in those lateral stability systems. Although structural systems such as diagrid system and mega-frame can work alone without any assistance of the core for certain height of tall buildings, it is very rare that core is not used in any of these systems when the height increases to above 200 m. When the height increases to above 400 m, hybrid systems with core wall (especially concrete core) as one of the key stability systems is dominant in the design of serval supertall buildings such as Jeddah Tower (1000-m tall), Burj Khalifa (828-m tall), Shanghai Tower (632-m tall), China Zun Tower (528-m tall), and Guangzhou International Financial Center (432-m tall). Therefore, in this chapter, the core wall system will be discussed in detail. For buildings with height between 200 and 500 m, purely relying on core wall to resist the lateral load is also very rare, some supplementary systems such as outriggers, belt truss, or buttress wall are needed, and therefore these types of structural systems are dealt with in detail in this chapter. In the last section of this chapter, the buttress core system will also be discussed.

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3.2 SHEAR WALL AND CORE SYSTEM

Shear walls are widely used for both tall buildings and low-rise buildings. They are important structural members used in the lateral resisting system. They work as a deep vertical cantilevered beam supported at the ground. They also carry vertical load together with columns. Some structures may require coupled shear walls, where girders and the floor system join the two or more walls together as a coupled system to provide more stiffness.

In tall buildings, shear walls are generally located at the center of the building, normally in the form of core wall system to accommodate the vertical translation system such as lifts for the tall building. It is a very common form of lateral load support system in tall buildings. Therefore, the core wall systems are dealt in detail in this chapter.

3.2.1 Type of Cores

There are two major types of cores: concrete core and steel framed cores. Concrete walls are used widely in the tall building design; on certain occasions, steel core can be found in buildings built before 9/11, they being much lighter, can save the cost of the foundation. However, they are gradually abandoned after 9/11 attack, the reason for this will be explained in this chapter and the next chapter.

3.2.1.1 Concrete Core

Fig. 3.1 shows a concrete core for a tall building under construction. Reinforced concrete cores are a more standard option for tall buildings in general, as seen from the history, concrete structure is dominant in the market because they provide more stiffness than steel cores, and it is relatively cheaper to use a concrete core in certain countries such as China. In certain countries such as China, the steel production was not sufficient in the past; therefore, most of the tall buildings were built in concrete. In addition, some codes require that the core of the building be constructed using reinforced concrete in case of fire and for emergency safety. When designing a concrete core, there are several issues that must be considered:

- Constructability and construction sequencing: The erection sequence for building with a concrete core will see the casting of the core proceed ahead of the steel framing. The core and elevator shafts can therefore allow for the use of a climbing crane rather than having to use separate tower cranes.

Fig. 3.1 Concrete core of a tall building under construction. *(Photo taken by the author at Manchester, UK.)*

- It is also quite common to install the steel embedment such as steel plate in the core to further strengthen the core. One of the project example is the China Zun Tower in Beijing, which will be introduced in Chapter 5.
- As it is discussed in Chapter 2, due to the shortening effect, differential movement may incur especially in the structural system of the building, which is a combination of steel and concrete, the tendency of concrete to creep or shrink over time and thermal expansion of steel need to be considered.

3.2.1.2 Steel-Framed Cores

Steel core was quite common for tall building design before 9/11 attack. Most tall buildings in the United States at that time were predominantly using the structural steel for the cores. The twin towers in World Trade Center is one of the examples. It used the steel core at the center of the building. Another example is Swiss Re Tower in London, also called as 30

St Mary Axe or Gherkin. Both buildings will be explained in detail in the latter part of this chapter. The main reason for using steel core is that it provides a lightweight structure solution. The total weight of the structure is quite important for tall building design, as it will directly affect the foundation design. Therefore, a lightweight solution will make a cost-effective foundation design possible. However, the investigation of NIST NCSTAR [1] "Final Report of the National Construction Safety Team on the Collapses of the World Trade Center Towers," shows that fire was the major cause of the collapse of the World Trade Center as majority of its structural members were steel and this will be explained in detail in the next section. So for supertall buildings designed after 9/11, steel core is rarely used.

As summarized in Ref. [2], when designing the steel-framed cores, one need to check the lateral loads, fire protection, constructability, and erection sequencing. Among them, the most important issue is the fire protections, due to the lessons learnt from the twin tower. Different fire protection strategies can be considered, such as intumescent paint, board, spray, etc.

3.2.2 The Importance of Core Design

As discussed in the next chapter, the structural system of Twin Towers is the so-called framed tube system. An internal steel core was used for Twin Towers, the floors were made of the steel composite truss floor system, under the fire, the truss starts sagging, which pulled inward on the perimeter columns: "This led to the inward bowing of the perimeter columns and failure of the south face of WTC 1 and the east face of WTC 2, initiating the collapse of each of the towers" [1].

As a result of 9/11 attack, more and more design engineers began to focus on how to design a tall building to be able to resist a similar attack. Therefore, a concrete core became one of the major choices for consideration.

As discussed, core is also part of evacuation route when hazards happen, therefore, concrete core would also be a good option.

3.3 INTRODUCTION OF OUTRIGGER, BELT TRUSS, AND BUTTRESS CORE SYSTEM

In the previous sections, the role of core or shear walls in designing the lateral stability for the tall buildings have been discussed. The core is a very important component, which has been used heavily by structural engineers to stabilize the tall buildings. However, when the height of tall building increases, it will be very uneconomical to rely purely on the core wall system to resist lateral loading, as this will result in massive core walls or much thicker walls. One of the measures to complement the core wall system is to use stiff horizontal members (such as outriggers, belt truss, buttress wall) to connect the main core to the exterior columns. The efficiency of the building structure may be improved by about 30% through the use of the horizontal belt trusses that tie the frame to the core [3].

If we look at the latest built tall buildings such as Shanghai Tower in Shanghai, Burj Khalifa in Dubai, and the Shard in London, ancillary systems such as outriggers, belt truss, or buttress wall were used to further brace the core walls. These lateral resistance systems are discussed in detail here.

3.4 OUTRIGGER STRUCTURES

The concept of outrigger dates back to 50 years, it origined in deep beams. It has been derived from deep beam into concrete walls, and now in the form of one or several story outrigger trusses. Outriggers are one of the most widely used systems for relative regular floor plan. It is constructed using steel trusses, girders, concrete walls, or deep beams to connect the core and the columns at the perimeter. The outrigger trusses are normally one-story high, some even occupy several storys. The cores are normally located at the center of the building, whereas the outriggers extend out to the outer columns (as it is shown in Fig. 3.2).

Therefore, the outriggers and the outer columns work together as a further restrain to the core wall. Under lateral load, the belt trusses act as lever arms that directly transfer axial stresses to the perimeter columns. The bending, axial tension, and compression of the outer columns connected to the outriggers help resist the external moments of the structure. This resistance enhances the overall stiffness of the core, helps in reducing the lateral deflections, and overturning moments. The outrigger columns work together particularly helping to restrain the rotation of the core. Overall, major advantage of using the outrigger is to resist the rotation of the core and significantly reduce the lateral deflection and overturning moment.

One of the famous examples of this system is Shard, London Bridge Tower (Fig. 3.6). It has core wall at the center and outrigger truss at high levels, inside the plant room, to connect the central core and outer raking columns. The project Shard will be introduced in detail as a case study in the latter part of this study.

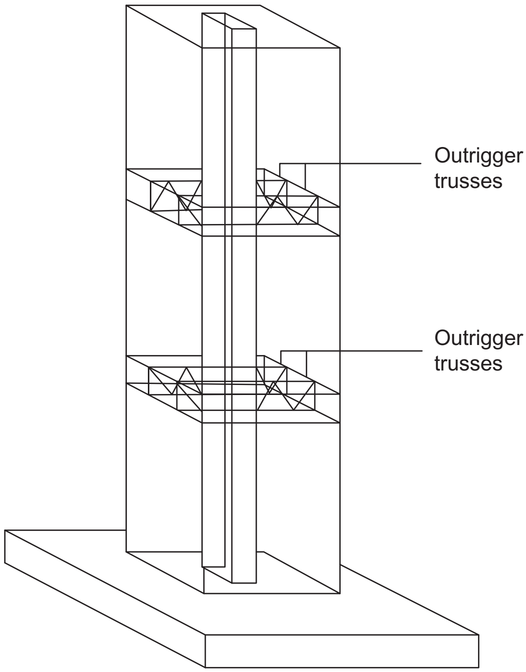

Fig. 3.2 Outrigger structures.

If the outrigger is used together with external tube systems (will be discussed in Chapter 4), it can more evenly distribute the large vertical forces applied by outriggers across the multiple columns.

Analysis and design of a core-and-outrigger system requires the use of computer program. This is because the distribution of forces between the core and the outrigger system are determined by the relative stiffness of each element: the core, the outrigger, and the columns. Therefore, it is difficult to calculate manually.

3.4.1 Types of Outriggers

There are several different types of outrigger system, such as steel outriggers, concrete outriggers, and hybrid outrigger (using both concrete and steel material). Among them, steel outriggers are most conventional type outriggers. The famous examples are: Twin Tower (collapsed in the 9/11 attack) and the Shard in London. Concrete outriggers are used in some tall buildings. One of the famous examples is 432 Park Avenue building in New York. With the development of the construction technology, new types of outriggers such as hybrid outriggers and damped outriggers have emerged in the construction projects.

3.4.1.1 Steel Outriggers

Steel outrigger systems are extensively used in a lot of tall buildings as most of tall buildings are either steel or composite structural system. In the conventional design, the outrigger is designed to be a story height truss.

3.4.1.2 Concrete Outriggers

The benefit of concrete outrigger system verses steel is high stiffness and low cost. Under wind load cases, the outrigger system needs to be of stiff concrete deep beam or of concrete wall which can be easily achieved by this. Fig. 3.3 shows a typical outrigger using concrete wall. This type of system is more common in a concrete structure rather than in a steel frame structure.

3.4.1.3 Hybrid Outriggers

The steel outrigger is not as stiff as concrete outrigger. However, a pure concrete outrigger system is very brittle. In Ref. [4], an innovative type of steel-concrete hybrid outrigger truss was developed in two 370-m tall mega-high-rise towers in Raffles City Chongqing, in which the steel truss is embedded into the reinforced concrete outrigger wall as shown in Fig. 3.4. Both the steel truss and the concrete outrigger wall work compositely to enhance the overall structural performance of the tower structures under extreme loads.

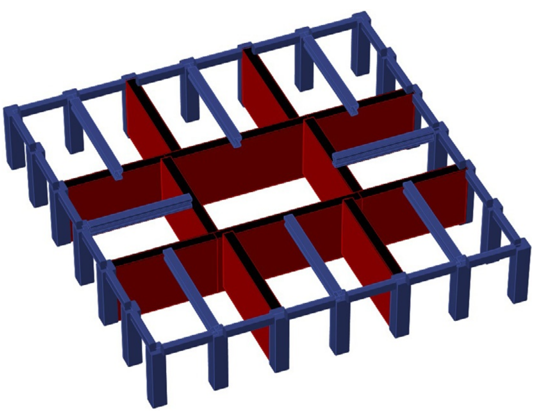

Fig. 3.3 A typical outrigger using concrete wall modeled using ETABS. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

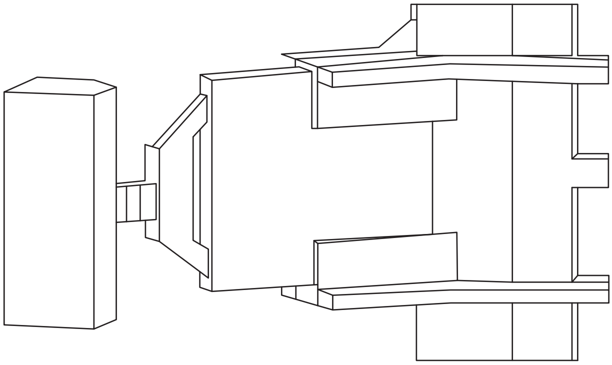

Fig. 3.4 Fused outriggers (a concept originally developed by Arup).

3.4.1.4 Damped Outrigger

In the event of severe earthquake, the overall structural system should be able to dissipate energy and maintain its robustness against the collapse.

As reported in Ref. [5] additional viscous dampers can be installed in the outrigger for a nonlinear response and tuned to meet multilevel performance objectives. In case the dampers fail, the outriggers which is designed to yield in a ductile manner will remain intact. Thus, it can reduce wind-induced vibration and can also be used as fuse to protect the building under a severe earthquake condition [5].

This type of outriggers is also used in Raffles City Chongqing [4]. As shown in Fig. 3.4, low-yield steel dampers were also adopted as a "fuse" device between the hybrid outrigger and the mega-column. The dampers were designed to protect the structural integrity of important structural components of the hybrid outrigger during moderate-to-severe earthquakes.

Another type of damped outrigger is designed by Ying [6]. The diagonal web member of the outrigger uses the so-called buckling restrained brace (BRB) (Fig. 3.5). The parameter of the BRBs will be selected according to the seismic performance under different design levels.

It can be seen that the outrigger works as an energy dissipation device, therefore, effectively resist earthquake loadings and hence the material cost. Although there are some additional costs for dampers, testing, and installation of this system, the saving of material will offset these costs. However, the disadvantage is that the stiffness of the outrigger system reduces a lot because of the damper.

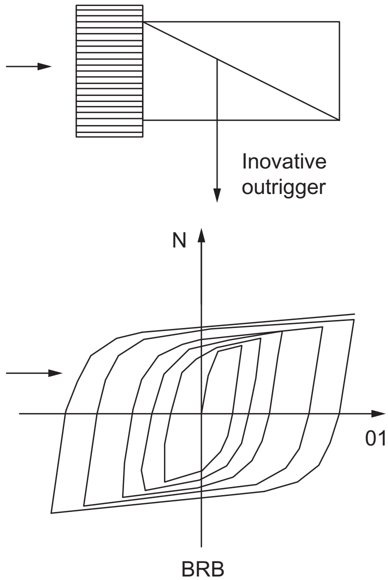

Fig. 3.5 BRB outrigger [6].

3.4.2 Disadvantage of Outriggers

1. As most outriggers occupy at least a full story height, they constraint the use of the floors at which the outriggers are located. Though they are normally placed in mechanical equipment floors, the presence of outrigger truss members can be a major problem as they will also restrain the places to set equipment. In addition, due to the constraints from the architect, there is possibly no place to set large outrigger columns and diagonals.
2. Although outriggers can reduce overturning moment of core, it could not reduce horizontal story shear forces on the core. In fact, shear force in the core can increase (and change direction) at outrigger stories due to the coupled horizontal force inside the outrigger acting on it.
3. The connections between the outrigger trusses to the core are normally very complicated, especially when a concrete core is used. As the connection need to accommodate huge shear force and axial force both come from the core.
4. As in Chapter 2, the effect of shortening is an important design consideration. During the construction stage, the core and the outrigger columns will not shorten equally under gravity load. The outrigger trusses, which need to be very stiff to be effective as outriggers, can be severely

stressed as they try to restrain the differential shortening between the core and the outrigger columns. In practice, engineer can delay the completion of certain truss connections until after the building has been topped out, this can alleviate the problems caused by differential shortening. In the construction of the Shard, the engineers just used this strategy (this will be discussed in the case study of the project—The Shard).

3.4.3 Case Study of the Shard

As shown in Fig. 3.6, the Shard is currently the tallest building in Western Europe. The total height of the building is 306 m with 84 floors, which was completed in 2012. I was luckily to be part of the design team members for

Fig. 3.6 The Shard. *(Adapted and reuse with the permission of Asset bank, City, University of London, ID24899 https://photos.city.ac.uk/asset-bank/action/viewHome.)*

engineering design company WSP on this project. WSP provided the services of structural engineering, fire, geotechnical, acoustics, traffic and transport, and drainage services. The lateral stability of the Shard is primarily provided through the central concrete core (see Fig. 3.7) which was constructed using slip-form techniques. As discussed in Chapter 2, for tall buildings, combined piled raft dominant the selection of the foundation system, therefore, raft and pile foundation were selected for the tower. As in Chapter 2, the top-down construction sequence and the new technique called plunge columns were also used for the Shard. The client is Sellar Property, the architect is Renzo Piano Building Workshop, and the construction company for Shard is MACE.

3.4.3.1 Structural System of Shard

For better demonstrating the structural system of Shard, a three-dimensional (3D) ETABS model was built in ETABS as shown in Fig. 3.8.

Fig. 3.7 The Shard under construction. (This file is licensed under the Creative Commons Attribution 3.0 Unported license https://commons.wikimedia.org/wiki/File:Shard_London_Bridge_January_2011.jpg.)

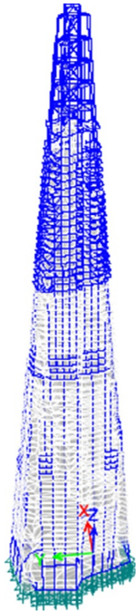

Fig. 3.8 3D Model of Kingdom Tower in ETABS. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

The Shard is designed as steel composite structure. It used steel composite slab as the flooring system, with steel columns and beams. As mentioned earlier, the central reinforced concrete core (see Fig. 3.7) is the major lateral stability system. The push-and-pull forces resulted from the lateral load primarily from wind are transferred to the piles and the raft.

In designing the lateral stability for Shard, in addition to the central core, a number of strategies were adopted for maximizing its inertia and therefore its stiffness by engineers in WSP [7]: The cross-walls are extended outside the main perimeter at the location of each side of the service riser in order to form buttresses. Similar to the design of the collapsed Twin Towers in New York (see Chapter 4 for details), in the high level plant rooms at levels 66–68, a "hat truss" (outriggers) of large steel members transfer forces between the core and the perimeter columns (Fig. 3.9). Through these measures, the engineers from WSP ensured that peak lateral accelerations in the apartments remain below the 0.015 m/s^2 limit recommended by the Council on Tall Buildings and Urban Habitat.

As in Ref. [7], during the construction, the hat truss members were not connected until the major construction was complete, thereby avoiding large axial forces being induced by differences in the axial shortening of

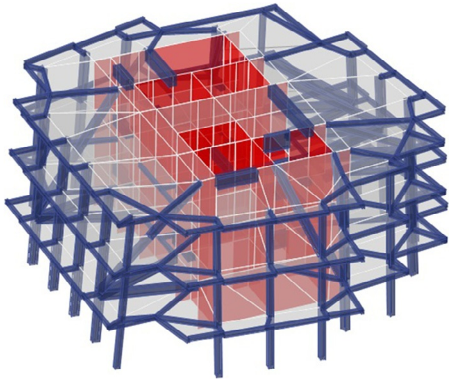

Fig. 3.9 Hat truss (outrigger) 3D ETABS model (storys 66–68). *(ETABS screenshot reprinted with permission of Computer and Structures.)*

the core and the perimeter columns (see Chapter 2 for relevant knowledge about shortening effect in tall building design) as it is shown.

3.4.3.2 Structural Analysis Result

The model is analyzed based on the gravity load and wind. As the Eurocode 1 Part 1–4 [8] is only accurate for building under 200 m and there is no wind tunnel report available, so ASCE 7–10 [9] code is used for analysis the basic wind speed chosen for the exercise is 85 mph. A rigid diaphragm was defined for each floor. ETABS can work out the wind exposure based on the extents of the diaphragms. The other loads such as dead load and live load are also selected based on Eurocode 1 Part 1–1 [10]. It cannot 100% represent the original design, however, it is accurate enough for case study purpose.

Fig. 3.10 shows the result of the maximum story displacement under wind loading. It can be seen that the maximum displacement of 316 mm was achieved which is satisfactory to relevant design rule. As stated in Ref. [11], an empirical formula to work out the allowable maximum lateral displacement of the tall building is the height of the building/500, for Shard, that is equal to $306/500=0.6$ m, so 0.316 m is satisfactory. Therefore, the design team did an excellent job in designing this challenge building.

Fig. 3.11 shows the overturning moment of the building under wind and gravity load combination. It can be seen that the maximum overturning moment is 12,825,126 kNm.

Fig. 3.12 shows the distribution of stress S11 in the core wall under wind loading. It clearly shows the push-and-pull effect caused by lateral loading. The windward side of the wall is in tension and the leeward side of the wall is in compression.

Fig. 3.10 Maximum story displacement under wind loading.

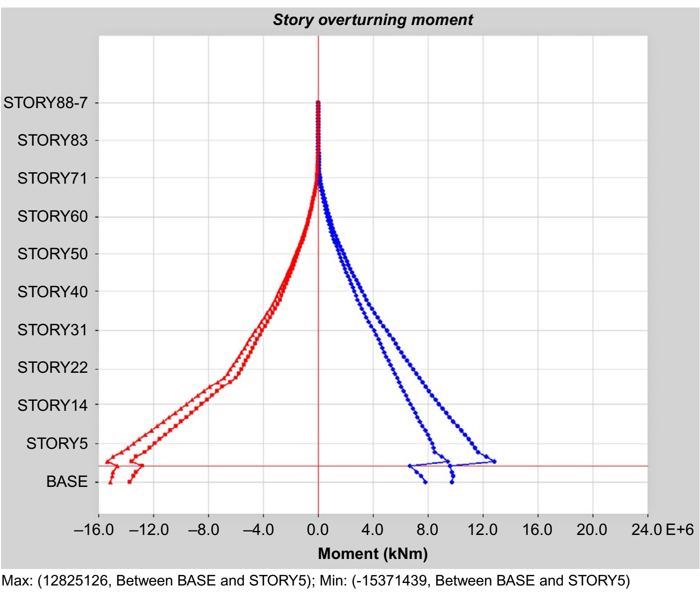

Fig. 3.11 Overturning moment under wind and gravity load combination.

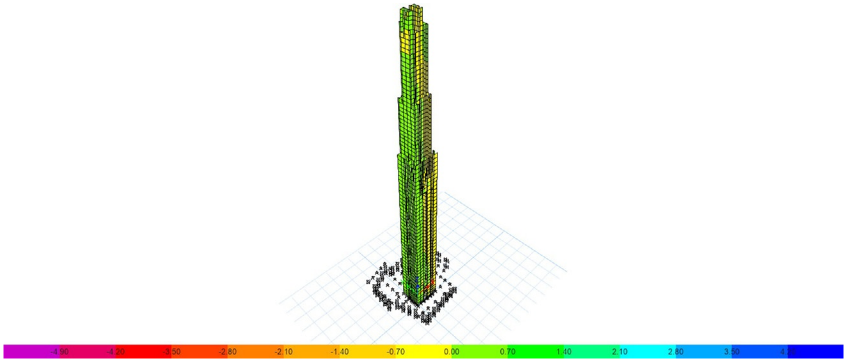

Fig. 3.12 Resultant of S11 stress in core wall under wind loading.

Fig. 3.13 shows the axial force distribution in the steel columns. It can be seen that the windward side is in tension and the leeward side is in compression.

Figs. 3.12–3.14 are good demonstration of the mechanism of the lateral stability system using core wall and outrigger. Owing to the contribution of the outrigger system, it can be seen that the stress in the core wall is in the allowable level, although the thickness of the wall is in the range of 450 mm (at the top) to 800 mm (at the bottom level). For a building with the height

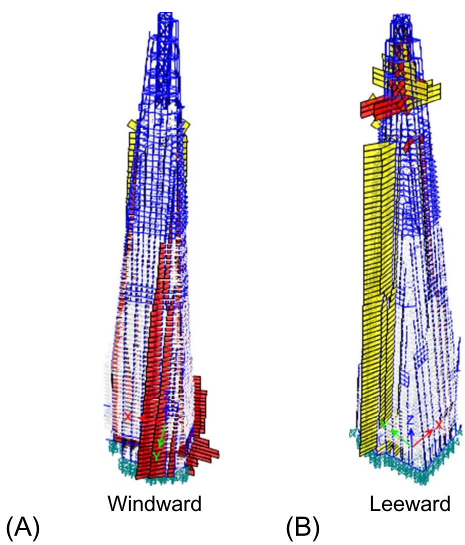

Fig. 3.13 Axial force distribution in the steel columns under wind loading: (A) windward and (B) leeward.

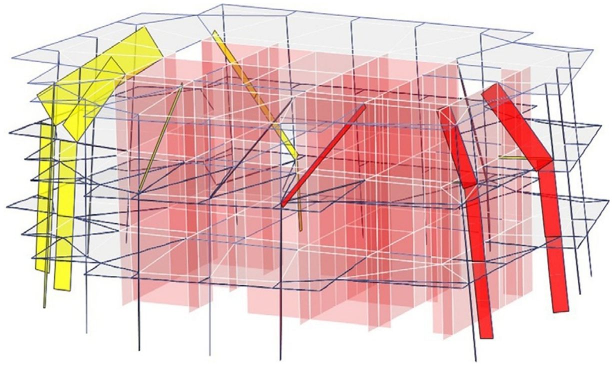

Fig. 3.14 Axial force distribution in the outriggers under wind loading.

of 300 m, the thickness of the wall is fairly economical. The only disadvantage is the loss of office space at the outrigger level.

Fig. 3.14 shows the axial force distribution in the outriggers. It can be seen that the outriggers help to transfer the load to the perimeter columns.

It is noted that, under wind loading, some columns are in tension, however, due to the gravity load, the resultant force in these columns are still in compression, as shown in Fig. 3.15. Fig. 3.15 shows the column load under load combination of wind, dead, and live. The columns at the bottom level are all in compression.

3.5 BELT TRUSS AND RING TRUSS SYSTEM

Fig. 3.16 shows a tall building under construction, which uses the so-called belt truss system as one of its lateral stability system. Belt truss system is another effective structural system to control the excessive drift due to lateral load. As shown in Fig. 3.17, belt truss connects all perimeter columns of the building by trusses (or walls in certain occasions).

Belts can improve lateral system efficiency. Different from outriggers, belt truss is located at the perimeter of the floor; hence, if outriggers are not used, it will transfer forces and moment through the floor diaphragm from outer columns to the core. Accordingly, when designing this type of structure, the floor diaphragm needs to be stiff enough to enable the forces transferring. As the floor slabs will be subjected to huge in-plane shear, so, they should be reinforced appropriately. In many applications, thicker slabs are used.

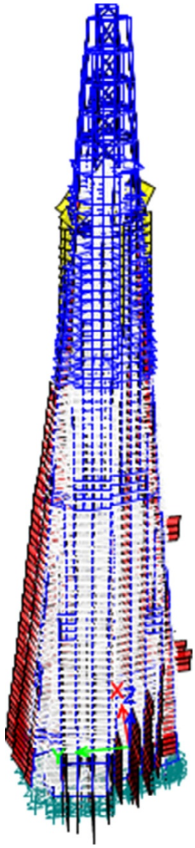

Fig. 3.15 Axial force distribution in steel columns under wind and gravity load combination.

Belt trusses can avoid most of the problems associated with conventional outriggers, such as

1. connection difficulty between outrigger and core is eliminated,
2. free interior space of the building, and
3. no need to consider differential shortening of core and outer column.

If the belt trusses work with outriggers, belt system can direct more gravity load to the columns and to minimize net uplift, reinforcement or the column splices are required to resist tension and stiffness reduction associated with concrete in net tension. In general belts can further enhance overall stiffness.

Fig. 3.16 A building with belt truss at Stratford, London, U.K. *(Photo taken by the author.)*

3.6 BUTTRESSED CORE SYSTEM

Buttressed core system is an evolution of the buttress structure which has been used in most of the ancient structures such as churches and bridges. As shown in Fig. 3.18, a buttress is an architectural structure built against major structure to support or reinforce it. They provide support to act against the lateral (sideways) forces arising from the structures. The term counterfort is synonymous with buttress, however, if used in dams, retaining walls, and other structures holding back earth.

3.6.1 Buttress Core System

Buttressed core structural stability system is developed based on the concept of the buttress structural system. It was first developed by Skidmore, Owings & Merrill LLP (SOM) for project Burj Khalifa (Fig. 3.19). As in

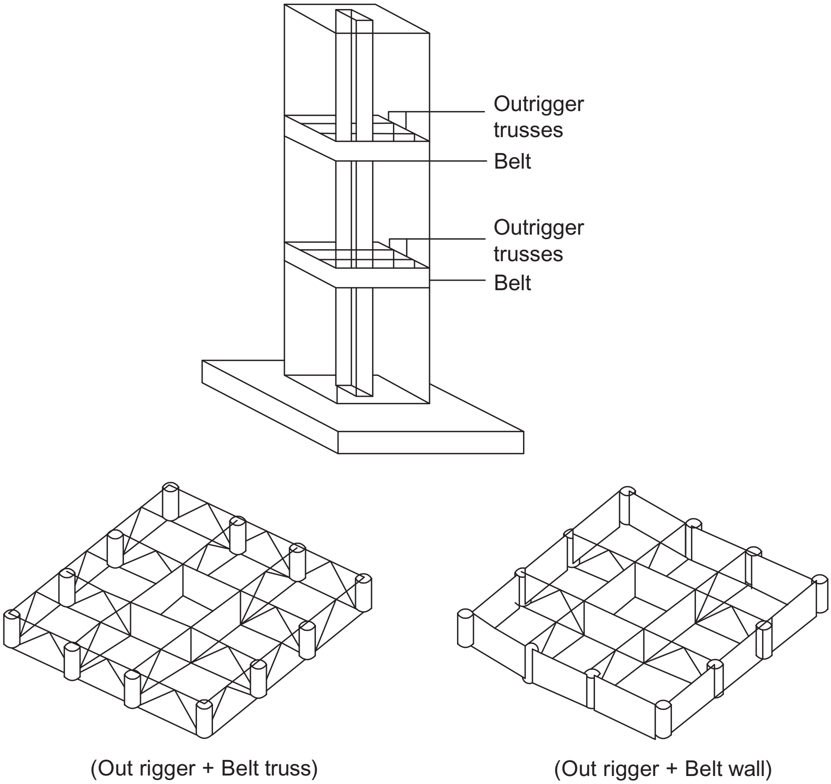

Fig. 3.17 Demonstration of belt truss system.

Ref. [12], the system is a tripod-shaped structure in which a strong central core is connected to three building wings. It is an inherently stable system. This is because each wing is buttressed by the other two. The central core provides the torsional resistance for the building, while the wings provide the shear resistance and increased moment of inertia (Fig. 3.19).

Buttressed core system made a major development in the stability design of the supertall buildings by dramatically increasing the height of the tall buildings to a world record of 828 m without using any special materials, although conventional construction techniques are still used. As also in Ref. [12], another strategy of the Burj Khalifa's structural design was "managing gravity." This meant moving the gravity loads to where they would be most useful in resisting the lateral loads. Structural engineers manipulated the tower's setbacks in such a way that the nose of the tier above sat on the cross-walls of the tier below, yielding great benefits for both tower

Fig. 3.18 Ancient buttress structure—Notre Dame de Paris Church. *(Photo taken by the author at Paris, France.)*

strength and economy. Engineers also employed a series of "rules" to simplify load paths and construction.

3.6.2 Case Study of Jeddah Tower (Also Known as Kingdom Tower)

As in Fig. 3.20, Jeddah Tower, previously known as Kingdom Tower, is a skyscraper under construction in Jeddah, Saudi Arabia. On its completion, it will become the tallest building in the world, as it is planned to be more than 1000-m tall with 170 floors. The triangular shape of the building can be considered as a direct descendant of Burj Khalifa, particularly, it also uses the buttress core as its major structural stability system. As buttress core system is proved to be successful for extremely tall buildings following the completion of the Burj Khalifa. Similar to the Shard, a system of combined piled raft has been selected for the tower.

The architect of this building is Adrian Smith and Gordon Gill Architecture. The structural engineer is Thornton Tomasetti. The wind tunnel consultant is Rowan, Williams, Davies & Irwin, Inc. (RWDI).

3.6.2.1 Aerodynamic Optimization

Following Burj Khalifa, the tower also has a similar triangular footprint and tapering form to reduce the effect of the wind and enhance the stability.

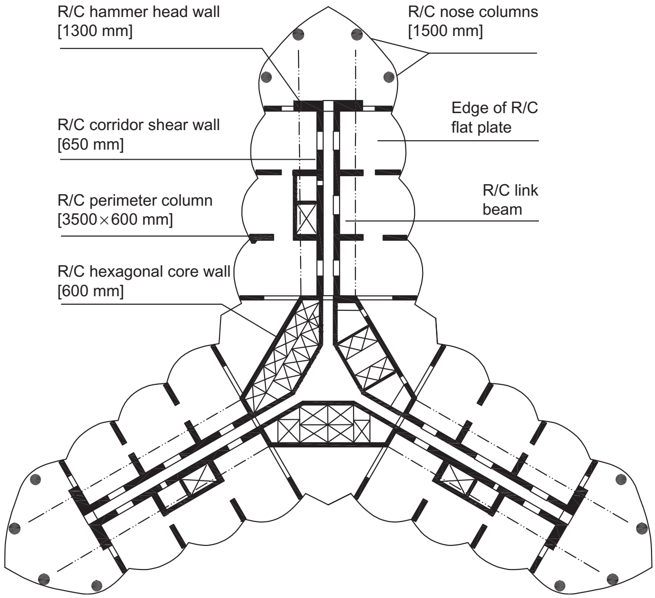

Fig. 3.19 Typical floor plan of Burj Khalifa [13].

Owing to it superheight, wind is one of the biggest structural design challenge. Aerodynamic optimization based on the wind tunnel test has been performed to create a stable structure.

3.6.2.2 Structural System

To be able to demonstrate the structural system, a 3D model was built in ETABS (Fig. 3.21). The model is set up based on the literatures and drawings found from the web. Owing to the limited access to the real design documents, the dimensions were estimated based on the available documents and pictures found online. It cannot 100% represent the original design, however, it is accurate enough for case study purpose.

The structural materials used in this project are primarily concrete. It uses concrete walls with concrete slabs. High-strength concrete is used. The strength of the concrete used in the tower is 85 MPa from the base to level 95, 75 MPa up to the spire, and 65 MPa in the spire. The yield strength of the reinforcing bars are 420 and 520 MPa with diameters of up to 40 mm [14].

Fig. 3.22 shows a typical floor plan of the tower, a buttress core system was used as the major structural system for this supertall tower. It consists of

Fig. 3.20 Jeddah Tower under construction (this file is licensed under the Creative Commons Attribution-Share Alike 4.0 International license, https://en.wikipedia.org/wiki/Jeddah_Tower#/media/File:Jeddah_Tower_Building_Progress_as_of_13-Jul-2016_004.jpg).

series high-strength reinforced concrete walls: a strong central core is connected to three building wings. Similar to Burj Khalifa, as each wing is buttressed by the other two, therefore, the stability of the structure is greatly enhanced. Each wing is also braced by a series of buttress walls which are connected by coupling beams and radiated from a central closed prismatic tube.

Figs. 3.23–3.25 show the floor plan at different levels. There are three groups of walls: end walls, corridor walls, and fin walls. The central core plays an essential role in torsional strength of the tower. The wall thicknesses are between 1200 mm at the base and 600 mm in the spire and the coupling beams' depth are 1500 or 1600 mm [14].

3.6.2.3 Result Analysis

For buildings with such height, wind load is determined by the wind tunnel test. As no wind tunnel test report is available, ASCE 7–10 code [9] were

Fig. 3.21 3D model of Jeddah Tower in ETABS. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

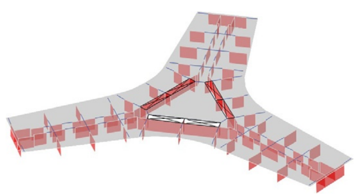

Fig. 3.22 Typical floor plan of Jeddah Tower. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

used to determine the wind load. Other loading such as live and dead were calculated based on ASCE 7–10 [9] as well.

Fig. 3.26 shows the overturning moment of the tower under wind and gravity load combination. It can be seen that moment of 60×10^6 kNm was obtained, which is about five times larger compared with 12,825,126 kNm. The height of the building is 1000 vs. 300 m of the Shard.

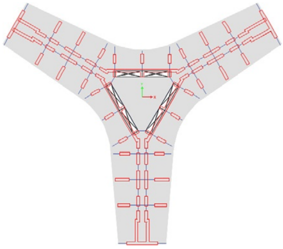

Fig. 3.23 Typical floor plan of Jeddah Tower. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

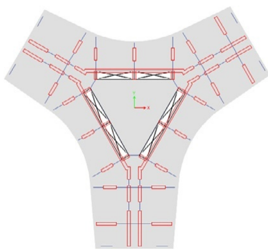

Fig. 3.24 Level 100 floor plan of Jeddah Tower. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

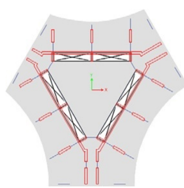

Fig. 3.25 Level 164 floor plan of Jeddah Tower. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

Further check was also made based on the result of the analysis. As required by ASCE 7–10 [9], the Commentary to Appendix C that the following load combination should be used in order to check the serviceability: D + 0.5 L + 0.7 W. The maximum story drift is normally limited to 1%, the analysis results show that 0.0015 drift is obtained, and, it is satisfactory. The maximum compressive stress in the wall is also checked which is about 72 MPa, as the 85 MPa concrete was used, therefore it is also satisfactory.

Based on the analysis results for such a tall building, buttress core system is an effective system for both lateral and gravity load resistance.

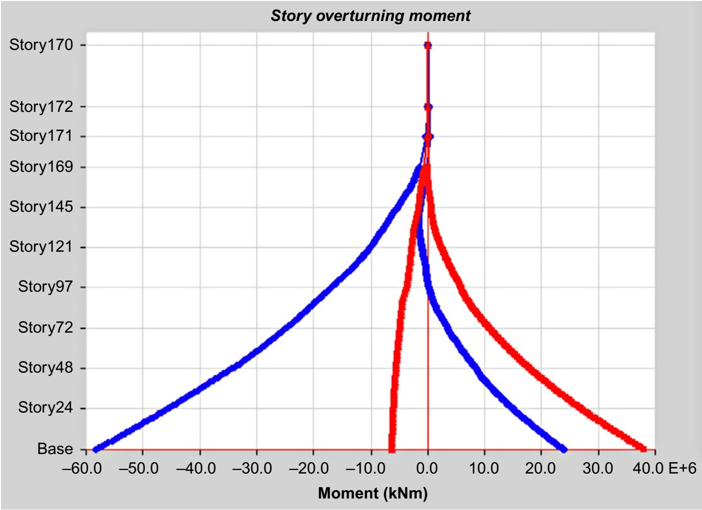

Fig. 3.26 Overturning moment under wind and gravity load combination. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

3.6.2.4 Analysis Model Set Up Method

This is a very challenge model, as the whole building is tapered from the bottom to the top, which means each floor plate is different. Therefore, it is worth introducing the effective way to set up this complicated model. Based on this feature, the method discussed in Chapter 3 of Ref. [11] was used in the modeling process. A 3D model was first set up in AutoCAD. At the beginning, the floor plate with the location for all the walls (represented using a polyline for each wall) for each floor was drawn, and translated to the correct levels, as shown in Fig. 3.27. Based on this wireframe model, a 3D model was fully developed in AutoCAD, then it is imported into ETABS. For further modeling technique, see Ref. [11].

3.7 SUMMARY

In this chapter, we introduced the most important stability system—cores. The core can be constructed using either reinforced concrete or steel-framed core. However, for tall buildings, it is unlikely to rely purely on the core wall system to resist the lateral loading. Outriggers, belt truss, and buttress wall are common auxiliary system, which help to strengthen further the lateral resisting system. Owing to its relative larger stiffness and better fire protection performance, concrete core is the first choice in selecting the core

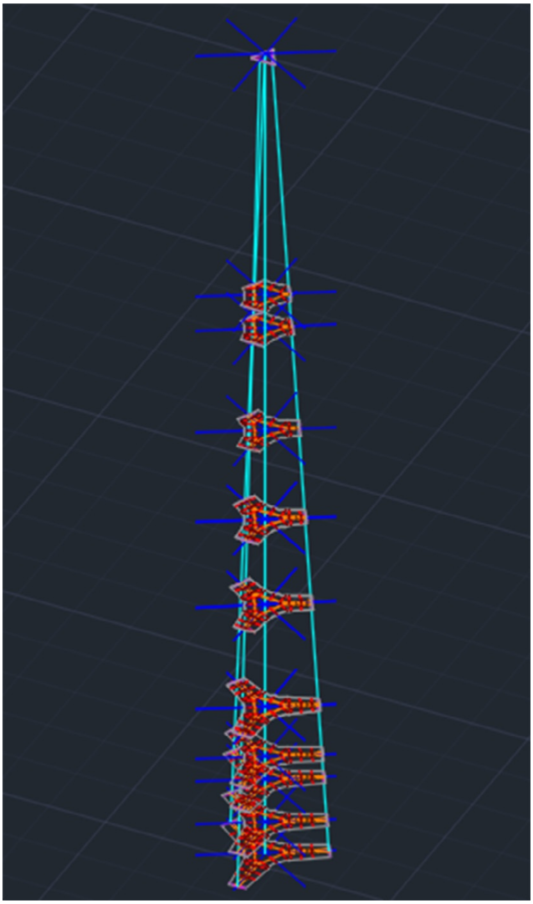

Fig. 3.27 3D AutoCAD wire frame model.

system. For extreme tall buildings such as Burj Khalifa and Jeddah Tower, concrete core with the buttress is an effective structure system. It also increases the robustness due to its better fire-resistant feature than steel structures.

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CHAPTER FOUR

Tube System in Tall Building

Abstract

In this chapter, one of the widely used lateral stability system tube system is introduced. It gives detailed description on different types of tube systems, such as tube-in-tube, framed tube, braced tube, bundle tube, and hybrid tube system. The case studies of the Twin Towers and One World Trade Center (the replacement of the Twin Towers) are made. In the final section, two superslender buildings 432 Parke Avenue and Allianz Tower are introduced.

Keywords: Tube, Tube-in-tubeframed, Tubebracedtube, Bundle tube, Hybrid tube system

4.1 INTRODUCTION OF TUBE STRUCTURES

In the tall building design, the tube system is one of the common lateral stability systems. It is designed to act as a vertical cantilevered hollow shell cylinder. This allows to create an indefinite stiff "shell" around the building exterior. This system was introduced by Fazlur Rahman Khan from the firm Skidmore, Owings & Merrill (SOM) in 1970s [1]. It was an innovative lateral stability system for designing a taller, more efficient building at that time. It has dramatic difference compared to the traditional structural system for multistory buildings, such as portal frame system, core wall strengthened by outrigger, etc.

The first example of the tube's use is the 43-story DeWitt-Chestnut Apartment Building, Chicago, Illinois wascompleted in 1966 [2]. Steel, concrete, or composite construction can be used for this type of system. For this particular type of system, the exterior framing should be designed sufficiently strong (normally in terms of rigid beam to column connections). The perimeter of the exterior consists of closely spaced columns that are tied together with deep spandrel beams through moment connections to resist all lateral loads [3]. The distance between the exterior and the core frames is spanned with beams or trusses. This can maximize the effectiveness of the perimeter tube by transferring some of the gravity loads within the structure to it and increases its ability to resist overturning due to lateral loads.

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Tube structures are categorized into several different types: tube-in-tube, framed tubes, braced tube, bundle tube, hybrids tube system, etc.

In this section, different types of tube system will be discussed in detail, several modeling examples based on the existing prestigious tall buildings projects such as Petronas Tower, Twin Towers, and One World Trade Center will be demonstrated.

4.2 TUBE-IN-TUBE SYSTEM

4.2.1 Introduction

This structure system is a coupled structural system by outer tube around the exterior and the inner core (can be a concrete core wall or a steel-framed tube) is called a tube-in-tube structure. The inner braced core and the outer tube are normally connected through the floor diaphragm; in certain occasions, they are connected through outriggers. They work together to resist lateral loads such as the earthquake and wind. They are also part of the gravity-resistance system. Most of the lateral loads are normally taken by the outer tube because the external tube holds much greater structural significance in comparison to the internal core due to the structural depth. Fig. 4.1 shows the Petronas towers, which is one of the typical tube-in-tube structures which we will introduce in the next section.

There are some confusion between the concept of tube-in-tube and framed tube systems which will be introduced in latter part of this chapter. In tube-in-tube system, the inner tube and the outer tube are a pair of soft tubes, especially the outer tube is not that stiffer. In framed tube structure, the outer tube is stiffer, as they are composed of close spaced columns connected by the spandrel beams which make a very stiff outer tube. However, with the fast development of the modern construction technology, many tall buildings are using hybrid lateral stability system, so it is not that important to distinguish these two terminologies.

4.2.2 Case Study of Petronas Tower (Tube-in-Tube)

The Petronas Towers used to be the tallest building in the world. It is also known as the Petronas Twin Towers. They are twin skyscrapers in Kuala Lumpur. The architect of Petronas Tower was Cesar Pelli and Associates and the structural engineering company was Thornton-Tomasetti Engineers. The height to the antenna spire is 451.9 m and the height to the roof is 378.6 m. The total floor area of the tower is 395,000m^2.

Fig. 4.1 Petronas Towers, Kuala Lumpur, Malaysia. *(Photo taken by the author.)*

Thornton-Tomasetti used the tube-in-tube system design for this ironic building in Malaysia.

As shown in Fig. 4.2, to enable the case study, a three-dimensional (3D) model was built in ETABS base using the available drawings and introduction documents on the web. Owing to the limited access to the design documents, the model is not 100% representation of the real structure; however, it is accurate enough for the case study purpose.

4.2.2.1 Structural System

The building is built primarily in concrete. Most of the structural members are made with high-strength concrete. High-strength concrete was used in the central core, perimeter columns, perimeter ring beams, and outrigger beams. The two towers are connected through a sky bridge. The foundation of the tower was constructed using 104 concrete piles; the towers sit on a large concrete raft.

The structural system of each tower comprises a 25 m × 25 m central core and an outer ring of widely spaced 16 cylindrical supercolumns. Fig. 4.3 demonstrates the structural system of each tower. These 16 cylindrical columns are

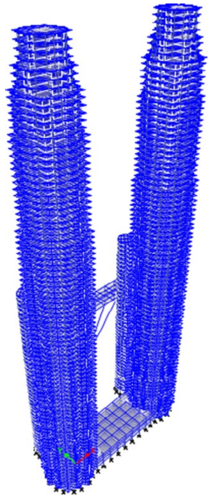

Fig. 4.2 3D model of Petronas Towers. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

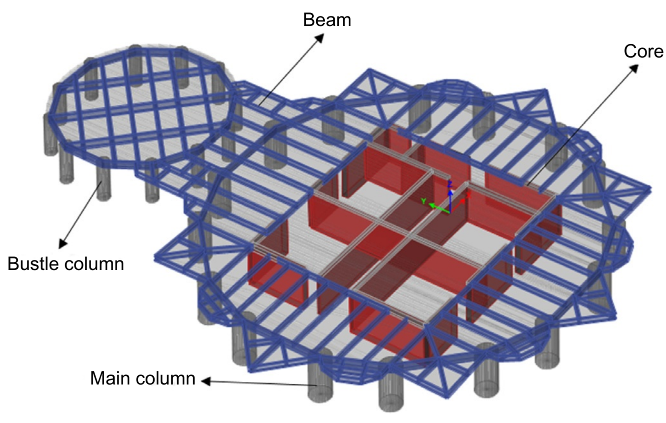

Fig. 4.3 Demonstration of the structural system for Petronas Towers. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

constructed using high-strength reinforced concrete. These columns are linked by ring beams to build a moment frame outer tube. This is one of the good examples of tube-in-tube system, as there is a pair of "soft tubes." Fig. 4.4 represents the tube-in-tube structure system for Petronas Towers, with the inner core and the outer tube shown separately.

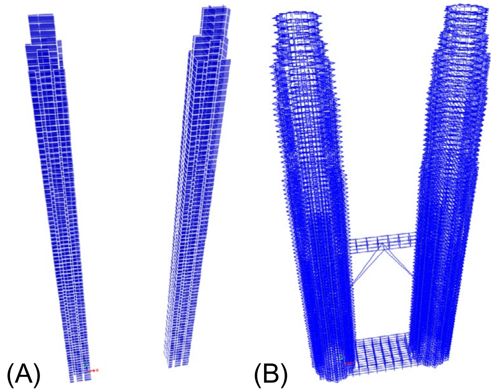

Fig. 4.4 Demonstration of the center tube and outer tube for Petronas Towers. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

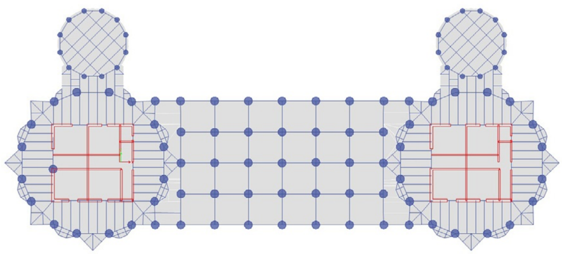

Fig. 4.5 Ground floor plan layout of Petronas Towers. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

Figs. 4.5–4.7 further demonstrate the structural arrangement of the Petronas Towers. In between the outer tube and the inner tube, concrete beams are also used to connect them. In addition, steel beams are also used, however, they are primarily used to support the floor slabs.

As shown in Fig. 4.6, there is a sky bridge between the two. The bridge is supported on the two towers, which is pin connected to the two towers, so allows them to move freely. This avoids the damage to the bridge when large movements occur between the two towers.

4.2.2.2 Result Analysis

The building was analyzed under the different load cases. Figs. 4.8–4.10 show the result of the tower under the earthquake loading. The response of the structure from the earthquake.

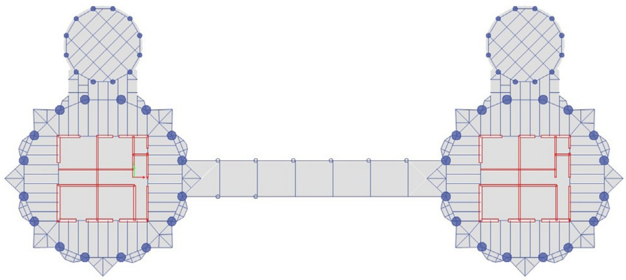

Fig. 4.6 Story 42 plan layout of Petronas Towers. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

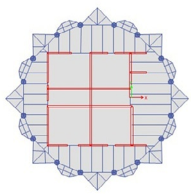

Fig. 4.7 Story 80 plan layout of Petronas Towers. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

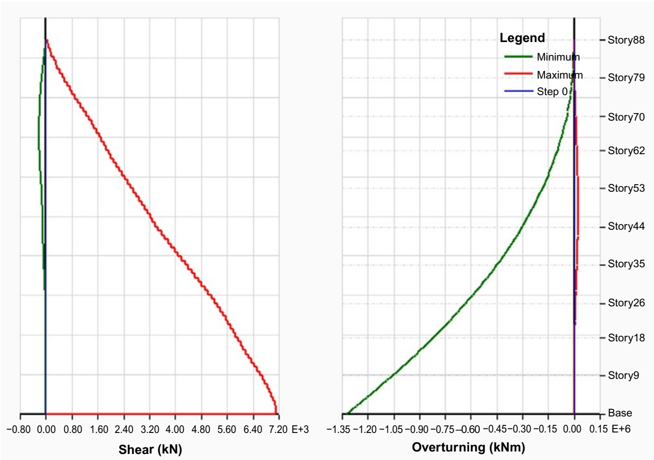

Fig. 4.8 Shear force and overturning moment distribution during earthquake (*y* direction). *(ETABS screenshot reprinted with permission of Computer and Structures.)*

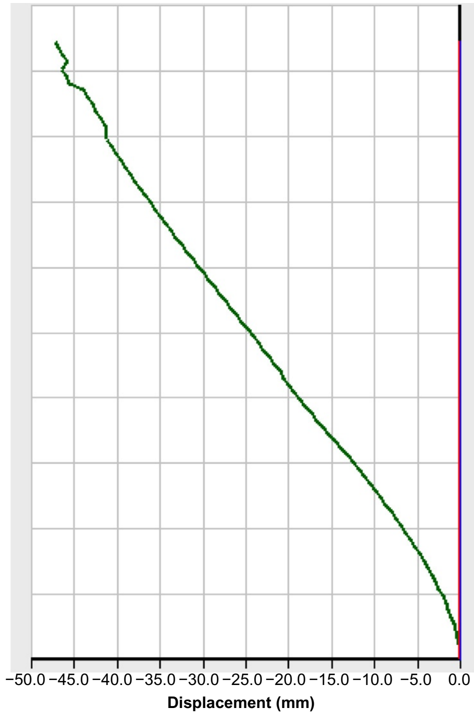

Fig. 4.9 Lateral displacement (*y* direction). *(ETABS screenshot reprinted with permission of Computer and Structures.)*

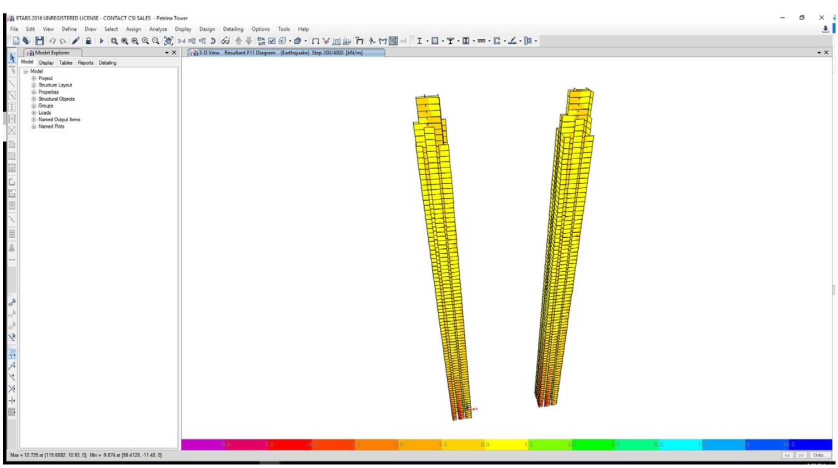

Fig. 4.10 Stress of the core wall under earthquake loading. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

4.2.2.3 Model setup methods

In this section, the way to set up this complicated model will be briefly introduced. A detailed modeling technique can be found in Ref. [4]. The building features two towers linked by a podium as well as a sky bridge. As the layout of the building is relatively regular, the model is first built in ETABS by importing the AutoCAD drawings of floor plan into ETABS as the templates. As ETABS is a story-based program, when all the typical storys have been defined, it is easier to setup a 3D model through duplications of typical floors generated based on floor plan AutoCAD drawings. However, it is difficult to model the sky bridge directly in ETABS, as it occupies several storys. Therefore, after the 3D model of the two towers is set up, a 3D model of sky bridge is first setup in AutoCAD, then it is imported into ETABS. When it is first imported, by default, all the members imported are selected by ETABS, therefore, it is ideal to name these members as a new group. Then select these members by the group name and move them to the correct location of the towers in the ETABS model.

4.3 FRAMED TUBES

4.3.1 Introduction

Framed tube system is one of the most widely used tube systems. Compared to tube-in-tube system, it featured a much stiffer exterior tube in this type of system. The stiff tube was achieved through closely spaced columns connected by deep spandrel beams which are firmly joined together to make the stiff exterior shell. Depending on the structure, the spacing of the column is quite close, generally 1.5–4.5 m spacing. Spandrel beam depths can range from 0.5 to 1.2 m.

As shown in Fig. 4.11, the Twin Towers in New York were one of the first structures to use a framed tube design. As shown in Fig. 4.12, which is the typical floor of One World Trade Center (WTC1), numerous columns with tubular sections can be seen around the exterior of this plan layout. It can also be seen that, the towers had a steel core at the center which had 47 columns spaced relatively evenly.

The disadvantage of this type of structure is the huge cost. To be able to ensure the rigidity of the connection in the outer tube, high level workmanship is needed for welding and high-strength bolt connections are required. The erection and the fabrication are also more expensive in terms of the working hours. Another drawback is the so-called shear lag which will be dealt in Section 4.3.2.

Fig. 4.11 World Trade Center, New York City. *(This file is licensed under the Creative Commons Attribution-Share Alike 3.0 Unported license, https://commons.wikimedia.org/wiki/File:World_Trade_Center,_New_York_City_-_aerial_view_(March_2001).jpg).*

4.3.2 Shear Lag Effect in Framed Tube System

Shear lag effect in a tall building can be demonstrated as shown in Fig. 4.13. It shows a plan cross section of a building with a moment induced because of lateral load. It shows the theoretical and the real distribution of axial stresses in peripheral columns. Under lateral load such as wind or earthquake, the whole structure works as a vertical cantilever beam. Therefore, the stress distribution in the cross section should follow the theory of bending (see Fig. 4.13A). However, the real distribution (see Fig. 4.13B) of these stresses is not linear. In the cross section, magnitude of stress at the corner side is higher in comparison to the columns of the middle location. Therefore, in the flange of the cross section, the stress in the middle columns is less than that in the corner columns. The same kind of nonlinear distribution of axial stress can also be observed in the web of the cross section as shown in Fig. 4.13. This

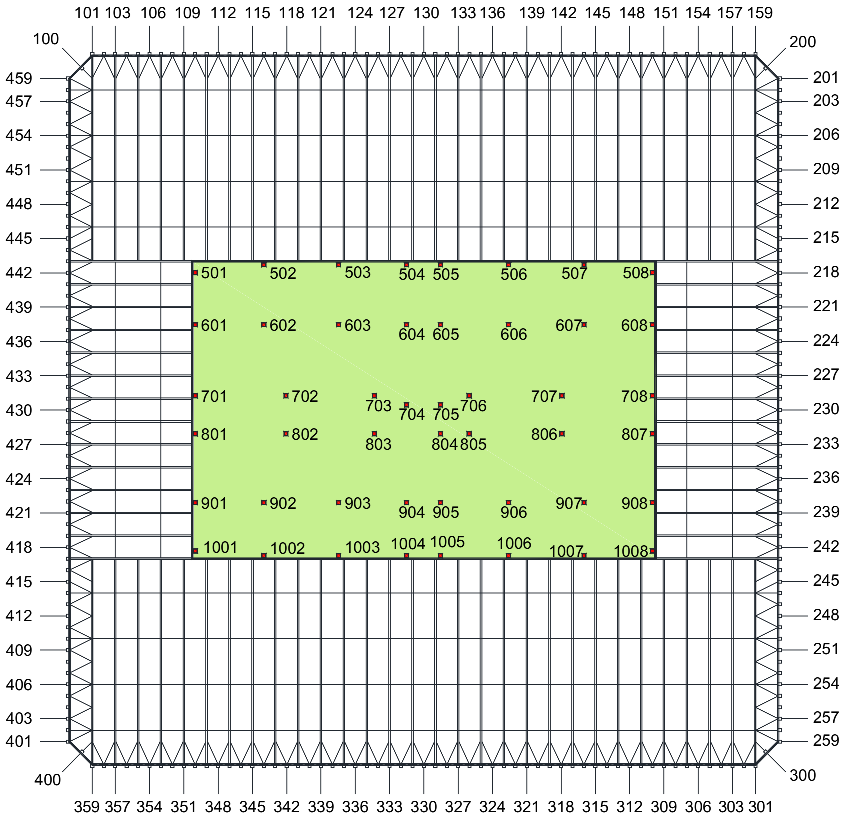

Fig. 4.12 Typical floor layout for WTC1. *(Permission to reproduce and derive from Figs. 1–3 of Federal Building and Fire Safety Investigation of the World Trade Center Disaster: Final Report of the National Construction Safety Team on the Collapses of the World Trade Center Towers (NIST NCSTAR 1, 2005).).*

kind of nonlinear distribution of axial stress along the flange and the web of the cross section of the tall building plan is called the shear lag effect.

In theory, if the exterior tube is a perfect tube in the tall building with framed tube system, then the building should behave like a true cantilever with all the lateral forces being resisted by the exterior tube. However, due to the flexibility of spandrel girders and columns, there is a shear lag effect, with a hyperbolic-type stress distribution in the plan cross section. Therefore, when the exterior framed tube is subjected to wind loading, columns situated near the corners of the tube experience the greatest axial forces and it spreads nonlinearly for the web frame and flange frame of the plan, whereas the middle columns experience a reduction in the axial load and hence

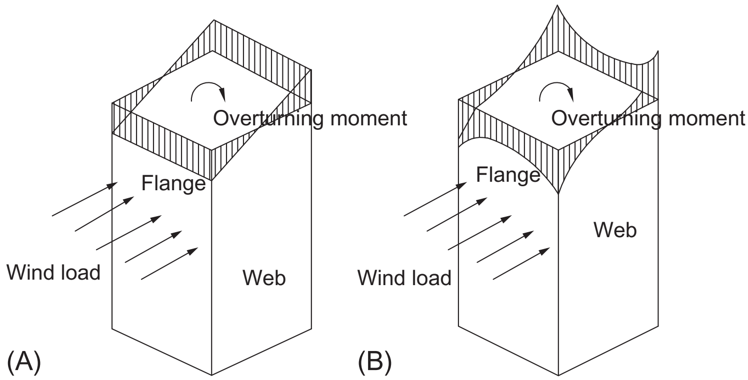

Fig. 4.13 Axial stress distribution in the columns of the building in web as well as in flange panels. *(Source Patil and Kalwane, Shear Lag in Tube Structures, IJISET—International Journal of Innovative Science, Engineering & Technology, Vol. 2, Issue 3, March 2015. www.ijiset.com.).*

stiffness. The stresses in the inner columns lag behind due to the bending of the spandrel beams.

The shear lag effect greatly reduces the effectiveness of the framed tube. In the design of this type of building, an engineer should limit the level of shear lag and aim structural behavior similar to that of cantilevers. As stated in [5] by Taranath, for building taller than 50–60 stories the window opening should be made relatively narrow to reduce the shear lag.

4.3.3 Case study of Twin Towers (also called as World Trade Center), New York (Tube Frame Structure)

As shown in Fig. 4.11, the Twin Towers was a large complex of seven buildings in Lower Manhattan, New York City, United States. It featured landmark twin towers, which was opened on April 4, 1973, and was destroyed in the September 11 attacks. The structural system of twin tower is a typical framed tube as shown in Fig. 4.8. It used the steel core and perimeter columns to create a relatively lightweight structure to resist the lateral loadings. Therefore, it was primarily a steel structure, as most of the major structural elements were steel members. For the sake of fire resistance, fire protection measures such as sprayed fire protection material and gypsum wallboard were used to protect some structural steel elements in the towers, including core columns. However, it is found by "*Final Report on the Collapse of the World Trade Center Towers*" [6] that fire was the major cause of the collapse of the Twin Towers. Therefore, after 9/11, more and more designers are prone to use concrete core rather than steel core systems, the main reason is to avoid fire-induced collapse.

4.3.3.1 Internal Core

Rather than using a concrete core, a steel internal core was used in the World Trade Center. The core columns were connected to each other at each floor by large square girders and I-beams about 2-ft. deep. As it has been explained in the previous chapters, the main purpose for using a steel core is to save the total weight of the building. This, in turn, also saves the cost of the foundation. The core contained 47 steel columns from the bedrock to the top of each tower [6].The size of the core columns at lower levels are 558 mm by 1371 mm steel box columns which are reduced to 406 mm × 914 mm box columns at the upper third levels. The columns were tapered after the 66th floor, and consisted of welded box sections at lower floors and rolled wide-flange sections at upper floors.

4.3.3.2 Exterior Tube

As shown in Fig. 4.14, the exterior tube was built using 60 high-strength perimeter steel columns spaced closely together which are connected by spandrel plates to form a strong rigid exterior tube which were 64-m long

Fig. 4.14 South tower and slurry wall "bathtub" under construction in 1969. *(Usrlman, the copyright holder of this work, release this work into the public domain, https://commons.wikimedia.org/wiki/File:WTC_bathtub_east.JPG.)*

on each side. The perimeter columns were designed to resist lateral loads as well as gravity loads.

As also shown in Fig. 4.14, the spandrel plates were welded to the columns to create the modular pieces off-site at the fabrication shop. The spandrel plates were located at each floor, transmitting shear stress between columns, to allow them to work together in resisting lateral loads. The joints between modules were staggered vertically, so the column splices between adjacent modules were not at the same floor [6].

4.3.3.3 Connection Between Exterior Tube and the Central Core

As shown in Fig. 4.15, the perimeter tube and central core was linked by composite trusses floor system. The floors consisted of 10 cm thick lightweight concrete slabs on a steel deck with shear connections for composite action. The slabs were supported with trusses apart from resisting the gravity load, the floor system provides lateral stability to the exterior tube. The trusses are connected to the perimeter at alternate columns. The top chords of the trusses were bolted to seats welded to the spandrels on the exterior side and a channel welded to the core columns on the interior side.

The floors were connected to the perimeter spandrel plates with viscoelastic dampers, which helped to reduce the amount of sway felt by building occupants [6].

Besides the composite truss floor, hat trusses (outrigger trusses) are also used from the 107th floor to the top of the buildings. The outrigger truss system consisted of six trusses along the long axis of the core and four along the short axis.

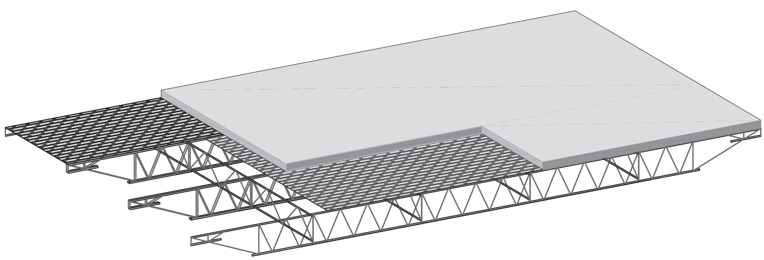

Fig. 4.15 Composite floor truss system of World Trade Center. *(Permission to reproduce and derive from Figs. 1–6 of Federal Building and Fire Safety Investigation of the World Trade Center Disaster: Final Report of the National Construction Safety Team on the Collapses of the World Trade Center Towers (NIST NCSTAR 1), 2005.)*

4.3.3.4 Finite Element Analysis of World Trade Center

To be able to fully understand the structural behavior of the World Trade Center, the full-scale model of WTC1 was modeled in ETABS. The analysis of result will be presented here to help the readers to understand fully the structural behavior of the tall building using the framed tube system as a lateral resistance system. The model was built based on the available literature on the web. It represents the real structural system of the WTC1, such as the same column size and beams sizes, however, 100% replication is not possible, due to the lack of the design information; however, it can accurately represent the true behavior of WTC1. The model is shown in Figs. 4.16 and 4.17.

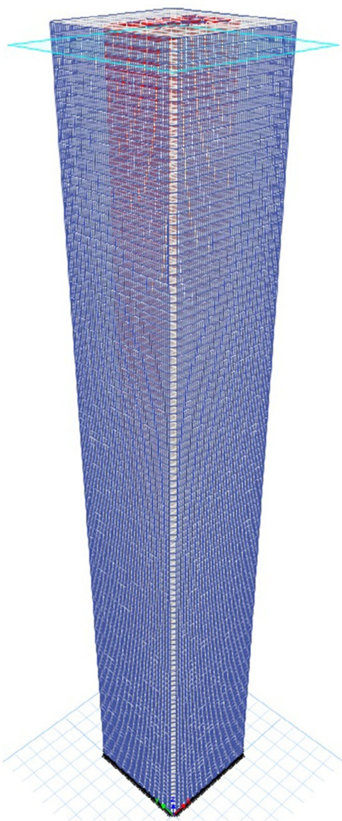

Fig. 4.16 Framed tubes structure model of World Trade Center Tower 1 (WTC1). *(ETABS screenshot reprinted with permission of Computer and Structures.)*

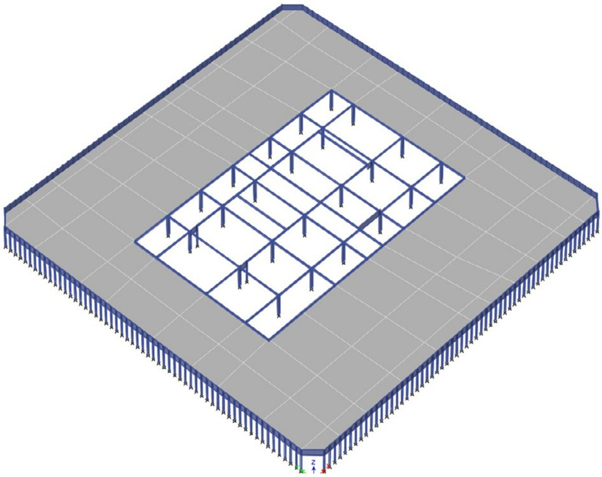

Fig. 4.17 Ground floor of World Trade Center Tower 1 (WTC1). *(ETABS screenshot reprinted with permission of Computer and Structures.)*

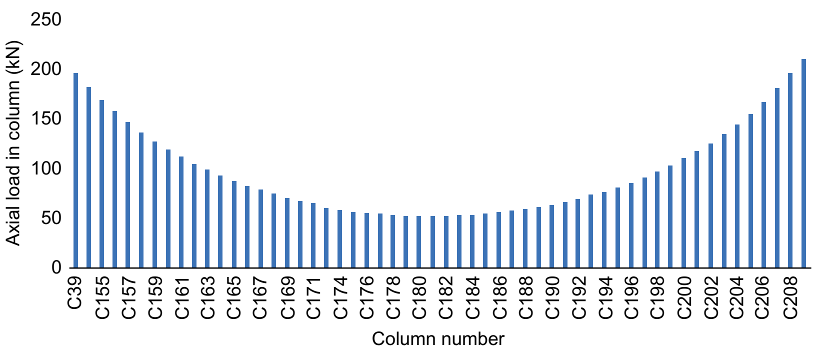

Fig. 4.18 Axial force distribution of columns at one edge of the exterior tube of Twin Towers.

Fig. 4.18 shows the distribution of the column load at one edge of the exterior tube for WTC1under the lateral wind load, it clearly shows the shear lag effect.

4.4 BRACED TUBES STRUCTURE (TRUSSED TUBE STRUCTURE)

The braced tube (also known as trussed tube) is similar to the tube-in-tube structure but with comparatively fewer exterior columns. In most of

Fig. 4.19 John Hancock Center in Chicago. *(The copyright holder of this file, E. Kvelland, allows anyone to use it for any purpose, provided that the copyright holder is properly attributed. Redistribution, derivative work, commercial use, and all other use is permitted. https://commons.wikimedia.org/wiki/File:John_Hancock_Center.jpg.)*

the cases, steel bracings are used to compensate for the fewer columns by tying them together. By this arrangement, the overall cost of the building dropped dramatically.

The advantage of braced tube is that the diagonal brace can take the lateral load in axial action, thus reducing the shear lag. However, there are also some disadvantages such as the large brace blocked some windows. In addition, braced tubes are only used for structures with less than 60 stories, due to the fact that the external shell is not as stiff as the framed tube. John Hancock Center in Chicago (as it is shown in Fig. 4.19) is one of the famous examples.

4.5 BUNDLED TUBE

When the height of the building increases, one tube is not sufficient to resist the huge lateral load that occurs from either earthquake or wind. Bundle tube is a structural system which consists of several tubes tied together to

Fig. 4.20 Willis Towers, Chicago, Illinois, U.S.A. *(Photo taken by the author.)*

resist lateral forces. Such buildings have interior columns along the perimeters of the tubes when they fall within the building envelope. One of the famous examples is the Willis Towers (Fig. 4.20). It consists of a set of singular tubes which are joined together to form a multicell-tubed structure as individual towers are connected by belt trusses. It is especially suitable for very tall structures. The bundled tube structure has the ability to reduce shear lag, hence provides a relatively lighter structure.

Fig. 4.21 shows a detailed plan and layout of the tower at each level. It can be seen that it consists of nine bundled tubes at the base, each of approximately 23 m × 23 m. These tubes rise to different heights and are terminated at certain level. As the structure increases in height at several locations, two or more tubes are dismissed with two tubes continuing to the top of the building from the 90th story.

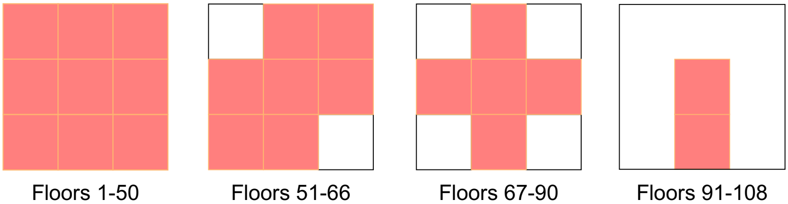

Fig. 4.21 Bundled tube system within Willis Towers. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

4.6 HYBRIDS TUBE SYSTEM

4.6.1 Introduction

As mentioned in Chapter 2, with the fast development in tall building design, most of the tall buildings consist of more than one type of structural systems. Therefore, these building systems should be called as hybrid systems. Hybrids Tube system include structures where the basic concept of tube is used, and supplemented by other system which is not introduced in the previous section of this chapter. This method is used where a building is supertall or subjected to extreme loading such as typhoon, that tube system itself cannot provide adequate strength or stiffness.

4.6.2 Case Study of One World Trade Center (Moment Steel Frame + Concrete Core)

As in Fig. 4.22, One World Trade Center (also known Freedom Tower) is the main building of the rebuilt World Trade Center complex in Lower Manhattan, New York City. It is 541.3 m tall, currently it is the sixth tallest building in the world. The building is to replace the Twin Towers which was destructed during the 9/11 attack. The structural engineer of this building is also my previous employer of WSP group. It was designed by the colleagues in the New York office. The architect of this building is Skidmore, Owings, and Merrill (SOM).

4.6.2.1 Structural System

One World Trade Center is a hybrid steel and concrete structure comprising a high-strength concrete core surrounded by a perimeter steel moment frame. They are paired together to provide extra rigidity and structural redundancy. Based on wind tunnel, aerodynamic optimization was made in the shape of the building to reduce its exposure to wind loads while simultaneously reducing the amount of structural steel needed. That is why the corners were tapered. As reported in Ref. [7], the floor of this building is concrete slab up to 90 cm thick and supported by composite beams. The exterior moment frame and core are connected by beams. High-strength concrete and steel are used and superhigh-strength 100 MPa concrete mix was used for this project.

A 3D ETABS model was built as shown in Fig. 4.23, the typical floor layout are shown in Figs. 4.24–4.26.

Fig. 4.22 One World Trade Center. *(This file is licensed under the Creative Commons Attribution-Share Alike 2.0 Generic license. Attribution: Joe Mabel, https://commons.wikimedia.org/wiki/File:OneWorldTradeCenter.jpg.)*

4.6.2.2 Blast Protection Wall

It is worth noting that owing to the lessons learnt from the collapse of the Twin Towers, at the ground level of the One World Trade Center two 56-m tall windowless concrete walls were built, designed to protect it from truck bombs and other ground-level attacks [8]. These wall (Fig. 4.24) is shown in the ground floor plan in the ETABS model.

4.7 SUPERSLENDER TALL BUILDING DESIGN

Construction of high-rise buildings is primarily taking place in large cities such as Beijing, New York, and London. In these cities, the design of tall building is heavily driven by the demand for space in densely

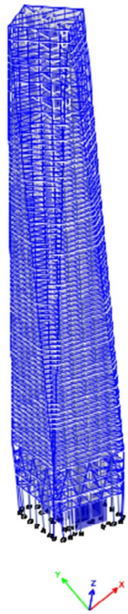

Fig. 4.23 3D model of One World Trade Center. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

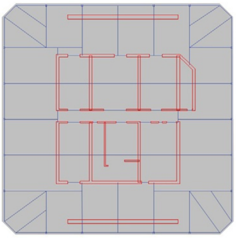

Fig. 4.24 Ground floor plan of One World Trade Center. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

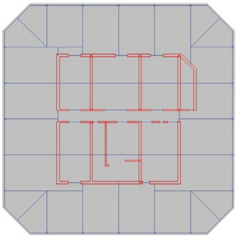

Fig. 4.25 Level 25 floor plan of One World Trade Center. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

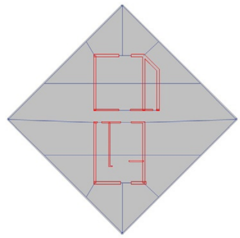

Fig. 4.26 Top floor plan of One World Trade Center. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

populated land areas. With the increasing shortage of lands, growth of population, and high demand for commercial office space in big cities such as New York and London, tall and slender buildings are becoming increasingly popular among property developers. This is because these kinds of buildings require smaller footprint; however, they can provide more living or commercial space. In tall building design, slenderness is measured by the so-called slenderness ratio which is the ratio of the height to the width of the base of the building. Structural engineers generally consider buildings with a slenderness ratio greater than 10:1 or 12:1 to be slender. As explained in Ref. [9], a tower is considered to be slender if the slender ratio is greater than 7:1, by the New York building codes. The newly built tall building, 432 Park Avenue which has a slenderness ratio of 15:1, is currently the most slender residential building in the world. Table 4.1 illustrates the list of the slender tall buildings in New York. It can be seen that in accordance with the requirement of the property developers, a structural engineer should be able to design a slender tall building. However, it is more challenging to achieve than the normal tall building designs. It is imperative to understand the physical constraints from engineering prospective as well as economic ones. One of the design goals for this type of building is to maximize the useable space under those special constraints compared to normal tall buildings. Therefore, in this section, the relevant design issues for slender tall buildings will be demonstrated.

Table 4.1 Slenderness Ratio for Tall Buildings in New York [10]

Building Name	111 West 57th Street	432 Park Avenue	53 W53	30 Park Place	56 Leonard	One 57
Slenderness ratio	24:1	15:1	13:1	10:5	10:5	8:1

4.7.1 Key Design Consideration for Superslender Tall Buildings

In designing a superslender tall building, the stability becomes one of the key design considerations. In addition, the selection of the material for the structural member is also critical. As explained in Ref. [9], "You will never see a slender building with a steel structure-it will vibrate like a tuning fork, as it does not have enough weight". To increase the mass of the buildings, concrete material becomes primary selections. As introduced in Chapter 2, the human comfort is one of the key design considerations for tall buildings. For superslender tall buildings, this becomes more challenging.

To illustrate further the key design considerations, in the following section, a case study on a real superslender tall building project—432 Parke Avenue in New York will be presented.

4.7.2 Case Study on Superslender Tower 432 Parke Avenue in New York (Tube-in-Tube)

The 432 Park Avenue is a residential skyscraper in Manhattan, New York City that overlooks Central Park. It is the skinniest supertall skyscraper in the world. As shown in Fig. 4.27, 432 Parke Avenue is the third tallest building in the United States and the tallest residential building in the western hemisphere. The height of the building is 425 m. The story height of 432 Park Avenue is 4.75 m with floor area of 38,335 m^2. The structural engineer of this building is also my previous employer of WSP group. It was designed by the colleagues in the New York office. However, as has been mentioned earlier, the slenderness ratio is only 15:1. It has occupied the footprint of only 28.5 m in each direction [9]. This is a very challenging slenderness, particularly for a residential building, the serviceability requirement is more stringent. To understand its behavior fully, the tower was modeled in ETABS, as shown in Fig. 4.27.

4.7.2.1 Structural Material

Owing to its slenderness, it is designed using concrete rather than steel members, which allows the designer to maximize the floor area of the apartment. The structure of the tower is composed of a very small, 9.1 m^2, reinforced concrete core. High-strength concrete material was used to construct 432 Park Avenue. The strength of concrete used is 100 MPa. Mixing fly ash with cement changes the color of the concrete. Moreover, clay mineral kaolinite was also added, which increased the strength of concrete and made its color lighter color. Reinforcement inside the concrete also has a very high strength, which was from grade 60 (280 MPa) to grade 97 (690 MPa), respectively.

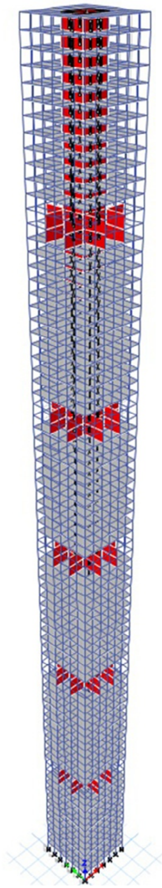

Fig. 4.27 Model of 432 Park Avenue in ETABS. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

The thickness of the slab is 250 mm. The shear wall and columns are connected through prestressed beams. Columns dimensions are 1.16×1.16 m with a shear wall thickness of 760 mm.

4.7.2.2 Lateral Stability

The lateral stability of the building was achieved through the tube-in-tube system. There is a $9 \text{ m} \times 9 \text{ m}$ internal concrete core working together with the perimeter tube formed by the concrete frame system sitting on the perimeter of the building as shown in Fig. 4.28. The core wall is 750-mm thick, which enhances its stiffness. In addition, the outer tube frame is connected to the central core every 12th floor using large stiffening beams, as shown in Fig. 4.27, which coincides with the double-story plant rooms.

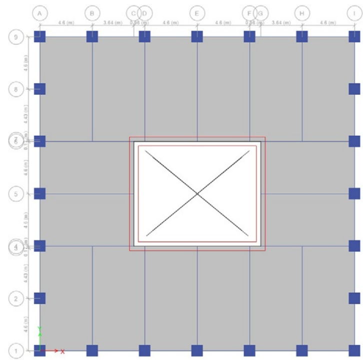

Fig. 4.28 Typical plan of 432 Park Avenue. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

4.7.2.3 Extra Measure to Limit the Acceleration

Owing to its slenderness, the building is more susceptible to wind-induced vibrations, which will cause human discomfort; practically this is a residential building which has more stringent requirements for vibration than commercial buildings. To solve this problem, WSP has comes up with several complimentary design solutions [9]. As shown in Fig. 4.29, the double-story plant rooms are not glazed, which allow the wind to pass through the buildings. This solution is developed by WSP through the wind tunnel testing. In addition, two tuned mass damper is also installed at the top of the building for further controlling the acceleration of the buildings.

To further limit the acceleration, when the structure complete, the engineers also add 1300 tones extra weight on top of the building.

4.7.3 Case Study on Slender Tower Allianz Tower, Millan

It is worth introducing another new built slender tower, Allianz Tower here due to the uniqueness of its special structural system. The Allianz Tower is primarily a concrete building. As discussed in Ref. [11], it has a rectangular 24×61 m footprint. The height of the tower is 242 m. It is the tallest building in Italy. The slenderness ratio of this building is 18.9:1.

As shown in Fig. 4.30, it has two concrete core located at the end of longer sides of the building, six columns at each longer side, and four mega columns at the center. To ensure the coupling of the two shear cores, perimeter

Fig. 4.29 432 Park Avenue. *(This file is licensed under the Creative Commons Attribution 3.0 Unported license. https://commons.wikimedia.org/wiki/File:432ParkAvenueJuly2015.JPG.)*

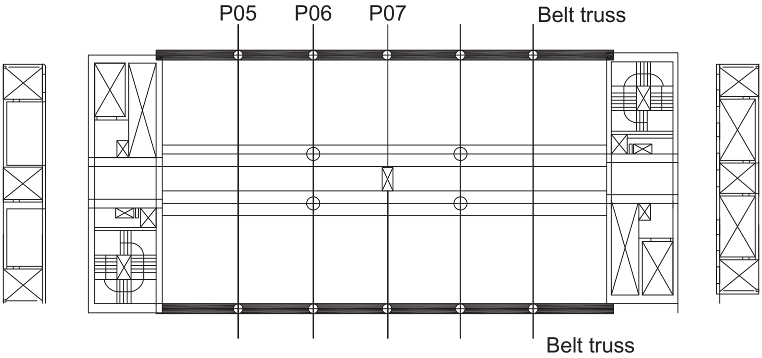

Fig. 4.30 Typical floor plan of Allianz Tower [11].

belt trusses are placed at the two longer side of the building. These perimeter trusses connect the two cores, making them work together to resist the lateral loading such as wind and earthquake.

However, the designer still wants to safeguard further the building due to its slenderness.

4.7.4 Summary

From above discussion, it can be seen that wind-induced vibration is the essential design considerations for the slender tall buildings. Owing to its larger slenderness, special design measures need to be considered. Enhancing the lateral stiffness, increasing the overall mass are the key design methods. Thus, it can be seen from the two projects introduced, some special arrangements have been made for the lateral stability systems.

In addition, tuned mass damper and aerodynamic optimization of the shape of the buildings are the auxiliary methods for designing these kinds of structures. Particularly, for Allianz Tower, four external struts support with viscous damper at their base were used for the first time. Although from the aesthetic point of view, it is not agreed with these arrangements, it opens a door for us as a structural engineer to use some auxiliary devices outside the building.

4.8 CONCLUSION

In this chapter, the tube system as one of the common lateral stability systems is introduced. Serval types of tube systems such tube-in-tube, framed tube, braced tube, bundle tube, and hybrid tube are discussed. Finally, the superslender tall building as a new structural form of tall building is also introduced. It can be seen that tube system is one of the effective systems for tall buildings. The selection of the type of the tube system will also rely on the height of the building, the budgets, and the requirement of the architect. Readers can make a flexible selection based on the introduced systems in this chapter.

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CHAPTER FIVE

Bracing, Diagrid, 3D Space Frame, and Mega Frame Structural Systems in Tall Buildings

Abstract

In this chapter, several different structural systems for tall buildings are introduced, which include the bracing system, moment fame system, diagrid system, three-dimensional (3D) space frame, and the mega-frame systems. Three case studies were also made: Guangzhou International Finance Centre (the tallest diagrid structure in the world), HSBC Tower in Hong Kong (a typical mega frame structure), and the Shanghai Tower (the tallest mega frame structure in the world).

Keywords: Bracing, Diagrid, Moment frame, Mega frame, 3D space frame

5.1 INTRODUCTION

Bracing systems are widely used in tall building designs. Primarily, they provide lateral stability for tall buildings. They are also part of the major structural units for diagrid and mega frame structures. Therefore, in this chapter the bracing system is introduced first. It is followed by a three-dimensional (3D) space structure, diagrid structure, and mega frame structures. In addition, an introduction to moment frame structures will also made.

5.2 BRACING SYSTEMS

As shown in Fig. 5.1, bracing system is used widely as lateral stability system in multistory buildings. For aesthetic purposes, braced frames are usually positioned in the cavity of the walls and in the lift-shaft core area. However, with the changing aesthetic views of people's, the bracing is exposed to the public in most of the newly built buildings. Bracing is also one of the major structural systems for providing stiffness and strength to resist lateral load. It is a highly efficient and economical method for resisting

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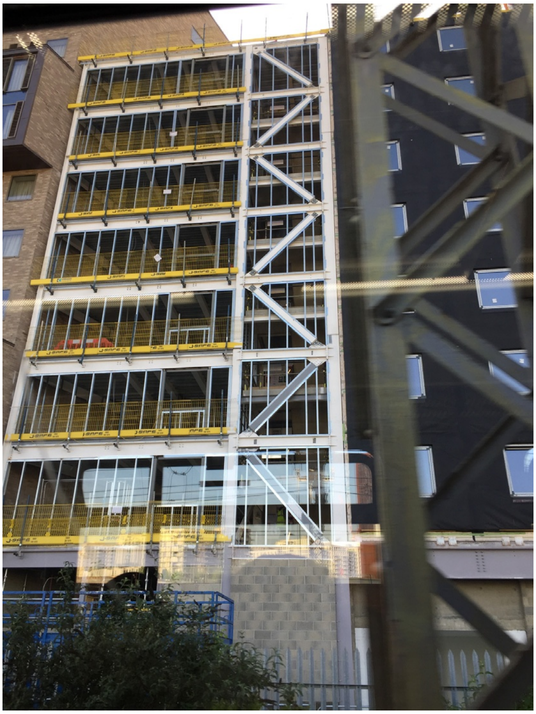

Fig. 5.1 An example of a multistory building using bracing as the lateral stability system. *(Photo taken by the author, at Stratford, London, U.K.)*

lateral forces. This is because the diagonal members work primarily in axial stress, resulting in minimum member sizes in the structural system.

There are two major categories of bracings: concentric bracing and eccentric bracing. The differences between them are: the typical concentric bracing can only take axial loading in the braces. Eccentric bracing comprises both axial loading members and bending loading members (the horizontal members). They are heavily used in earthquake zones due to the high ductility they provide. The features of these two types of bracing system are explained in the following sections.

5.2.1 Concentric Bracing

Concentric bracing is oriented in such a way that all members (beams, columns, and bracing) meet at a common point. They provide the lateral resistance mainly through the axial force in the braces. The two major categories of concentric bracing are diagonal bracing and K-bracing. In addition, there is another type of bracing which is called cross bracing (X-bracing). As shown in Fig. 5.2, this is a construction site, where

Fig. 5.2 Examples of typical concentric bracings. *(Photo taken by the author in London, U.K.)*

two concentric bracings can be identified, as cross bracing and diagonal bracing. It can be seen that all these bracings meet at a common point. The vertical cross bracing provides the lateral resistance to lateral load from both X and Y directions, mainly through the axial force in the structural members. Therefore, the diagonal member of this type of bracing is easy to design. It is also easy to assemble in the construction site. The detailing of the connections (Fig. 5.3) is simple when compared with eccentric bracing members.

One of the disadvantages of concentric bracing is that the behavior of such bracing under cyclical loading is unreliable. In addition, efficient energy dissipation is difficult to achieve in concentrically braced frames. Therefore, they are rarely used in the seismic zones.

5.2.2 Eccentric Bracing

As shown in Fig. 5.4, different from concentric bracing, in eccentric bracing, the braces are offset from the columns or they do not intersect at the floor beams. Therefore, it results in an eccentrically connected bracing. Examples of several typical eccentrically connected bracings are shown in Fig. 5.4.

It can be seen that different from concentric bracing, it comprises both axial loading members and bending loading members (the horizontal

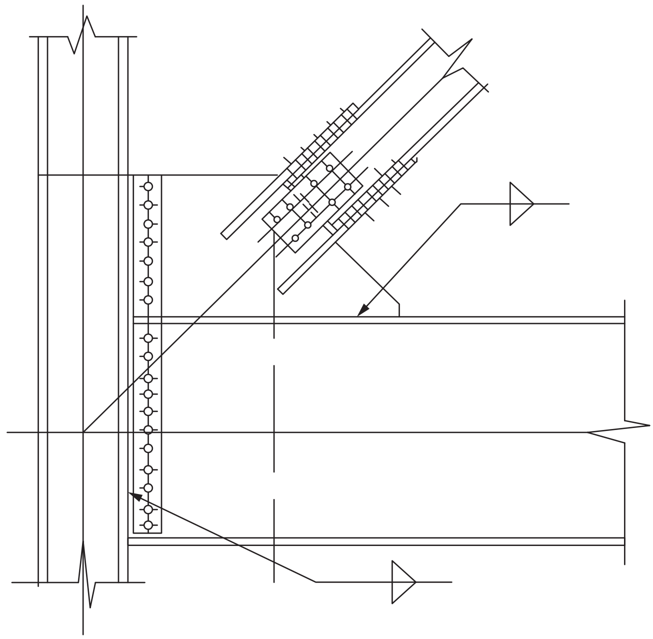

Fig. 5.3 Typical connection details of concentric bracings.

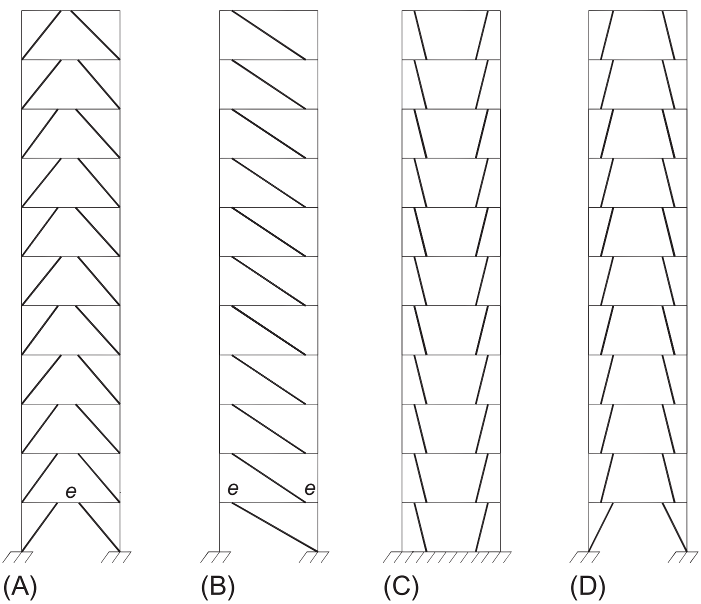

Fig. 5.4 Typical eccentric bracing.

members). Eccentric bracing can offer the same advantages as concentric bracing, while also providing significant ductility capacity and greater flexibility with architectural openings. Eccentric bracing is designed in a way that they do not buckle under extreme loading conditions. The axial forces induced in the braces are transmitted either to a column or to another brace largely through shear and bending in a segment of the beam called a link. The length of the link is notified by the letter e in Fig. 5.4. In designing this type of bracing, the designer needs to ensure that under severe loading conditions the major inelastic activity takes place in the link. Therefore, the links can work as fuses to prevent buckling of the braces [1].

As discussed in Chapter 2, in earthquake design, energy dissipation is one of the key design measures to resist earthquake loads. Eccentric bracing exhibits more ductile characteristic and greater energy dissipation capabilities than a concentrically braced frame of the same material. Therefore, this type of bracing is heavily used in earthquake zones due to the high ductility they provided through the link elements. However, concentrically braced frames can be used in moderate seismic regions.

The critical element for eccentric bracing is the link and therefore it has been subject to intensive research. Several cyclic tests have been performed in University of California, Berkley, to study the hysteretic behavior of this type of bracing systems. Fig. 5.4A and B shows the most common eccentric bracing used in the seismic design. In designing an eccentric bracing, the length of the link e as shown in Fig. 5.4A and B is critical. Fig. 5.4 shows the typical connection details of eccentric bracings. In Ref. [2], the author studied the behavior of a building with different e lengths. In Ref. [3] cyclic tests on eccentric brace were conducted with four link lengths ($e=400$, 500, 600, and 700 mm). Their test results show that the stiffener spacing at the connection is important for their performance. Response of short links was governed by web shear. Bolt failure by shank rupture would have caused a more brittle response of long links. It is also stated in Ref. [1] that

When

$$e/L > 0.5$$

where e is the length of the link and L is the length of the beam.

It is found that under this situation little benefit is gained from the bracing. However, as the length of the link decreases, the elastic stiffness increases.

Fig. 5.5 is the typical connection details of eccentric bracings, which is much more complicated than the connection details of concentric bracing.

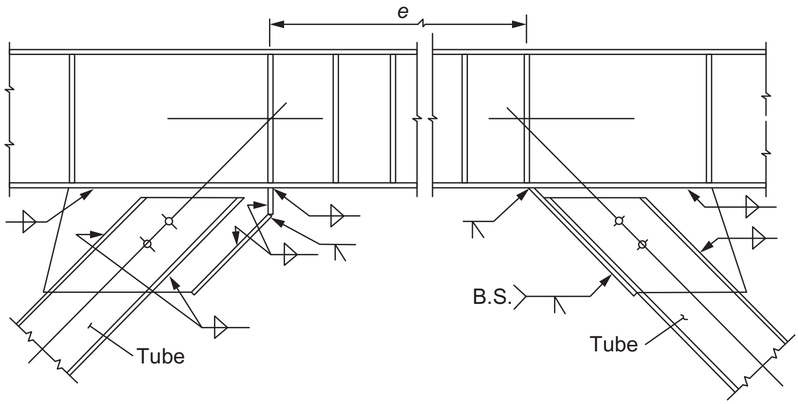

Fig. 5.5 Typical connection details of eccentric bracings.

5.2.3 Project Examples

Fig. 5.6 shows Neo Bankside housing project design, a luxury apartment in London. The structural engineering design is provided by my previous employer Waterman Group Ltd. It is located on the south bank of the Thames in London. The architect for this project is Rogers Stirk Harbor + Partners. It uses the exterior bracings in conjunction with a nondiagrid primary structural system. This bracing system uses elliptical tube sections. When designing this exterior bracing, the issues of corrosion protection, differential thermal expansion, and connection back to the primary structure was considered by the design team. It can be seen that these large exterior bracings extend between three stories. This exterior bracing is easily confused with the so-called diagrid system introduced in the latter section of this chapter. The major difference is, this type of bracing system is only designed to take the lateral load rather than any gravity load.

Fig. 5.7 is another example of a tall building using concentric cross bracings as the major lateral stability system. Similar projects can be found as in the Hancock Tower in Chicago.

5.3 3D SPACE TRUSS SYSTEM

It is widely known that the 3D space truss system is widely used for long-span roof; this system has been further evolved by Leslie Robertson, who further developed this system into tall buildings. One of the famous examples is the Bank of China Tower Hong Kong.

Fig. 5.6 Typical concentric bracings of NEO Bankside Tower in central London, U.K. *(Photo taken by the author.)*

Fig. 5.7 Typical concentric bracings of a tower near Liverpool Street, London, U.K. *(Photo taken by the author.)*

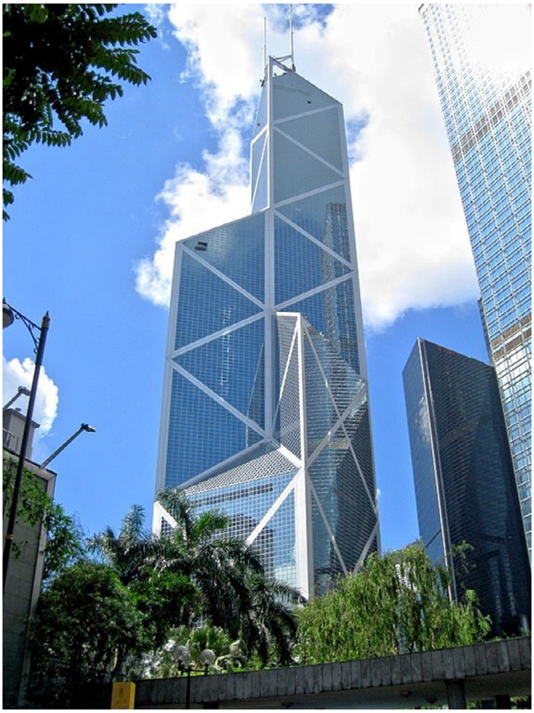

Fig. 5.8 HK Bank of China Tower (https://commons.wikimedia.org/wiki/File:HK_Bank_of_China_Tower_View.jpg, Free domain).

Fig. 5.8 shows the Bank of China Tower built in Hong Kong by the famous Chinese-American architect I.M. Pei. The structural engineer is Leslie Robertson. The height of the building is 315.0 m with two masts reaching 367.4 m. This was the tallest building in Hong Kong and Asia during 1990–92, and the first composite space frame high-rise building. While its distinctive look makes it one of Hong Kong's most controversial landmarks today, it earned the nickname "One Knife" due to its peculiar sharp shape.

It uses a 3D space truss system. The building is in composite steel and reinforced concrete. It is the most unique work of the structural system. Fig. 5.9 shows the typical floors along different heights of the building. It is noticed that the building's superstructure comprises four interlocking triangular shafts, terminating at various heights. This is similar to the bundle tube system used in Wills Tower discussed in Chapter 4. The only difference is that one uses tube, whereas the other one uses triangular shafts. It is built of composite metal tray and reinforced concrete floor system.

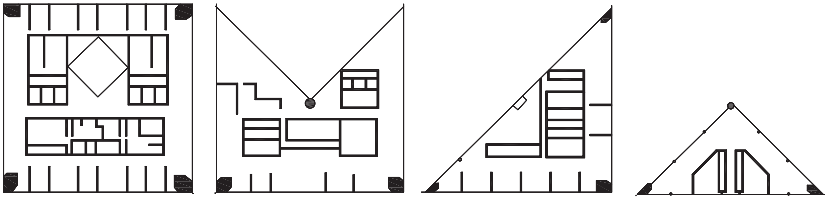

Fig. 5.9 Typical floors along different heights of the building.

The structural concept is also similar to the 'mega fame' structure system, discussed in the latter part of this chapter. To a certain extent, it is also similar to a diagrid structure, which is also dealt with in the latter part of this chapter. However, as it is made up of a space truss, it acts to carry both vertical loads and horizontal forces. The whole structure is supported by the four steel columns at the corners of the building, with the triangular frameworks transferring the weight of the structure onto these four columns. It is covered with glass curtain walls.

In Hong Kong, all buildings need to be designed against typhoon and hence the structure is relatively light in resisting typhoon wind. As introduced in Ref. [4] for this particular project, 127 permanent VSL rock anchors were also installed to strengthen the building against extreme loads including typhoon winds and earthquakes. Diaphragm walls were used in the foundation. Caissons of 9-m diameter were used to support the four main columns in the superstructure.

5.4 DIAGRID STRUCTURES

Among the different lateral stability of the tall buildings, diagrid structure is a unique structural system, which is increasingly popular in the design of tall buildings from the past decades. The term "diagrid" is a combination of words "diagonal" and "grid." To a certain extent, a diagrid system is also part of the bracing system, which originated from the conventional bracing system. It consists of huge diagonal bracings sitting on the exterior of the building, which is normally exposed to the public; therefore, it also becomes one of the aesthetical components for architects to use.

It is originally explored by the Russian Engineer Vladimir Shukhov. Norman Foster referred to his idea and applied in the Swiss Re Tower (Fig. 5.10). This photo was taken by me when I first visited London in 2002. The reason I took this photo was that it was a new type of structure,

Fig. 5.10 Swiss Re Tower, under Construction, London, U.K. *(Photo taken by the author.)*

which I have ever seen at that time. It shows the building was under construction. It can be seen that there are large diagonal members sitting on the façade.

5.4.1 Difference Between Exterior Braced Frame Structure and Diagrid Structure

It is easy to get confused between the conventional exterior braced frame structure and diagrid structure. Fig. 5.6 shows a typical exterior braced frame structure. Fig. 5.10 shows the diagrid structure Swiss Re Tower under construction. The major difference between them is that in a diagrid structure, vertical columns can be eliminated. This is because in diagrid structures, diagrid bracings can also take the gravity load in addition to the lateral load due to their triangulated configuration. However, the diagonals in the conventional bracing system could not take any gravity load.

5.4.2 Structural System of a Diagrid Structure

As most diagrid structures will still have core as partial lateral stability, the diagrid structure is an extension of the tube-in-tube structure; however, the outer tube is made by the diagrids. Normally, the diagonal members in the diagrids extend through several storys, they are normally not sufficient to achieve the stability alone without the help of the core in buildings taller than 200 m, due to combined lateral and gravity loads. As buckling may occur in the diagonal member, the horizontal ring beams are also used at the perimeter of the floor edge. They can tie the diagonal members and restrain them.

5.4.3 Diagrid Structure in the World

The first diagrid structure may be accredited to the IBM Building in Pittsburgh, which is built in the 1960s. The other project examples are Swiss Re London, the Hearst Tower in New York and the latest built Guangzhou International Finance Centre, Poly International Plaza.

5.4.3.1 Gherkin, Swiss Re, 30 St Mary Ax

The most famous project using diagrid structure is Swiss Re Tower, known as 30 St Mary Ax or the Gherkin (Fig. 5.10). It was completed in 2003, which is located in the heart of Central London with a height of 180 m. It is designed by the architects at Foster + Partners. The structural designer is Arup and the contractor is Skanska.

The building is built with a circular floor plan, which widens in profile as it rises and then tapers toward the top, giving it the distinctive 'gherkin' shape. However, despite the building's curved shape, the only piece of curved glass is the cap at the very top. Except for the aesthetic reason, this shape also minimized the effect of the wind to the building, reducing the need to stiffen the structure, and resist wind loads. Due to the diagonal braces at the perimeter, the building is free from columns. There is an internal steel core in the center which is connected at the perimeter diagrid system, to make a similar tube-in-tube-type lateral resistance system.

The Gherkin is well recognized as a milestone for the diagrid structure, as it perfectly satisfies both architectural and structural design requirements for a tall building; therefore, it is considered an iconic building for diagrid structures.

5.4.3.2 Hearst Tower

Hearst tower (Fig. 5.11) was completed in 2006, which is also designed by Foster + Partners. It is situated in Manhattan New York. The structural design was of my previous employer WSP Group. The contractor is Turner Construction. It is 46 stories tall, standing 182 m with 80,000 m^2 of office space. The structural steel of the diagrid system was reportedly about 20% less than a conventional steel frame. The whole diagrid system is sitting

Fig. 5.11 Hearst Tower. (https://upload.wikimedia.org/wikipedia/commons/thumb/b/b1/Hearstowernyc.JPG/352px-Hearstowernyc.JPG, public domain.)

on 14 huge mega columns. There are also internal concrete cores inside the bundling which are connected to the diagrid systems through the concrete floor.

5.4.3.3 Guangzhou International Finance Centre

Guangzhou International Finance Centre (IFC) is a 103-story, 438.6-m-tall building completed in 2010 in Guangzhou, Guangdong Province China (Fig. 5.12). It is currently the tallest diagrid tower in the world. The designers for this project is a consortium of local Chinese design firms together with international design companies, such as Architect Wilkinson Eyre and structural engineer Arup. In the center, it has an internal reinforced concrete (RC) core (from the ground floor to the 69th floor) with an interior diagrid structure all over it. This diagrid structure consists on diagrid members of steel tubes filled with concrete. In addition, the reinforced

Fig. 5.12 Guangzhou International Finance Centre. *(This file is licensed under the Creative Commons Attribution-Share Alike 3.0 Unported license, free to use, https://en.wikipedia.org/wiki/Guangzhou_International_Finance_Center.)*

concrete (RC) core is connected to the diagrid through beams. In addition, aerodynamic optimization of the shape of the building is made based on the wind tunnel test, therefore, minimizing the effect from the wind.

5.4.3.4 CCTV Building

The CCTV headquarters is a 234-m tall 44-story skyscraper located in the Beijing Central Business District (CBD). The tower serves as the headquarters for China Central Television (CCTV). The headquarters was completed in May 2012. The architect of this project is the Office for Metropolitan Architecture and the structural engineer is from Ove Arup & Partners. As shown in Fig. 5.13, the structural system of this building is also using the diagrid framing system. It is composed of two L-shaped towers.

Fig. 5.13 Central China Television headquarters under construction. *(This file is licensed under the Creative Commons Attribution 2.0 Generic license, https://commons.wikimedia.org/wiki/File:CCTV_Beijing_April_2008.jpg.)*

The two towers lean at 60-degree angle and kink at the right angles at the top, which made a cantilever overhang start after 36 floors and is 13 floors high. The length of the overhang is 75 m outwards. Due to this special shape, the diagrid system becomes definitely one of the only possible options, as it needs to tackle the large overhang, as well as the lateral stability of the whole building. Since Beijing is located in the high seismic activity zones, most of the buildings are required to resist eight magnitude earthquakes.

It is worth noting that to accommodate the sophisticate diagrid connections, the CCTV tower is used as an innovative butterfly plate connections to assist smooth load transfer [5]. This connection will be discussed further in Section 5.4.4.

5.4.3.5 Poly International Plaza

The Poly International Plaza was built in Beijing (Fig. 5.14). It is designed by the joint design consortium of SOM and by my previous employer, Beijing Institute of Architectural Design (Group) Co., Ltd. (BIAD). I was a

Fig. 5.14 Poly International Plaza. *(Photo taken by the author in Beijing, China.)*

structural engineer at BIAD, when I was in China. Its structural system is also a combination of internal concrete shear core with a perimeter diagrid. As introduced in Ref. [6], the diagrid perimeter has four modules with an 18-m span between the diagrid nodes, it is designed to resist gravity and lateral loads axially, with only minor bending effects due to rigid welded nodal connections. Every alternative floor is connected to the joints. The diagrid members are using the concrete-filled steel tube (CFT) to resist high compression loads. Due to the importance of the node, when designing the diagrid node, a special finite element modeling has been performed, as well as the full-scale test of the typical node under cyclic loading. The nodes are loaded monotonically until failure occurred (Fig. 5.14).

5.4.4 Structural Design Consideration of a Diagrid Structure

In this section, the structural feature and the design consideration of the diagrid structure will be discussed.

5.4.4.1 Structural Features

Diagrids use diagonal bracing members to resist both compression and tension. Therefore, an isotropic material such as steel was used by most diagrids.

As shown in Fig. 5.15, conventionally, the external diagonal bracings are working together with interior core which provides extra stiffness; therefore, the whole structure can achieve greater heights. Such systems retain greater structural effectiveness in comparison with the traditional diagonal bracing located within the cavity of central cores or in the façade. The triangular geometry of diagrid structures can effectively prevent structural

Fig. 5.15 A diagrid structure under construction in Beijing, China. *(Photo taken by the author.)*

failure as a result of both lateral and gravity loads. Due to this unique feature, when compared with conventional bracing, it considerably minimizes the use of columns especially less requirement of corner columns.

It can be seen that the lateral stability of diagrid structures is similar to the conventional framed tube structure introduced in Chapter 4; however, it is more efficient, as it can resist shear distortion through the axial action of diagonal members. In addition, less rigid core can be used in diagrid structures as the shear forces resulting from the wind or earthquake can be partially carried by the large diagonal members located on the perimeter. It helps to reduce the number of columns, especially the corner columns, thereby, allowing flexibility in the floor plan.

Diagrid systems can replace the conventional outrigger system in the design of tall buildings. This is because the most efficient way to resist lateral loads is to provide the resisting elements with maximum stiffness at the building exterior. Thereby eliminating the need for other systems such as outriggers. The stiff lateral and gravity load resisting diagonal members help to increase the lateral and torsional stiffness of the tower. The study from Rupa Garai [6] shows that the exterior frame takes significant proportion of the overturning moment on the structure, thereby, reducing the amount of the core wall.

The main advantages of using diagrid structures can be summarized as follows:

- The combination of gravity and lateral stability system provide efficiency in the design, therefore, reducing the steel usage.
- Triangulated configuration enhances the stability, through maximum resistance against torsion and overturning moment. Therefore, reduce the size of the core.
- Reducing the weight of the structure, therefore saving the cost of the foundation.
- Structures are more robust due to the redundancy of the diagonal members, as it can easily transfer the load from the failed portion to other parts of the structure.
- The reduction on the requirement of the columns results a more flexible design of space for the client.

The disadvantages of the diagrid structure are the following:

- As all the diagonal members intersect into nodes, the design of the nodes becomes especially essential for this type of structures.
- Due to its triangular shape, the design of the façade becomes challenging.
- The member of the diagrid should be designed to not buckle in the rare wind and seismic events, therefore, they are designed to perform elastically rather than plastically; as a result, it has less energy dissipation capacities.
- For buildings over 70 stories this system becomes uneconomical, mainly due to its complicated joints and complications that occur during construction increases the overall cost

5.4.4.2 Diagonal Member Design

As it has been explained, the member of the diagrid should be designed to not buckle in the rare wind events, in most of the projects, such as the Poly International Plaza Tower and The Guangzhou International Financial Center the concrete filled diagonal members are used due to their better resistance to buckling. In addition to this, they are also excellent in fire resistance.

5.4.4.3 Node Design

In diagrid structures, the nodes (also called joints) connecting the structural members of the diagrid is the critical components in a diagrid system. As each node at least needs to accommodate four incoming large diagonal members, as well as the horizontal beams, a clear distribution of the load needs to be provided by the nodes to enable them to accommodate these huge horizontal and diagonal forces, sometimes also vertical forces. Therefore, some special modification or adaption may be necessary. If the concrete-filled tube is used, the nodes also need to be designed as hollow and allow for a full fill when the concrete is pumped [7]. There are primarily two types of nodes: welding nodes and bolted nodes. Therefore, the sufficient weld design and bolt design are also required.

On the basis of the above considerations, in almost all the diagrid structure projects such as Gherkin and Poly International Plaza, the nodes are all designed based on the full-scale testing as well as the finite element modeling, to make sure that the nodes can perform effectively under different loading conditions.

As an example, in the CCTV building, the designer Arup use staggered splices in some joints to reduce concentration of weld stress [5]. They fabricated the so-called butterfly joint. It is a light and flexible connection, which allows the member to move. This minimized disruption to the stress in the column, the braces and edge beam was connected with the flanges of the steel column; with large butterfly plates no connection is made to the web of the column to simplify the details and concrete construction around the steel section (Fig. 5.16).

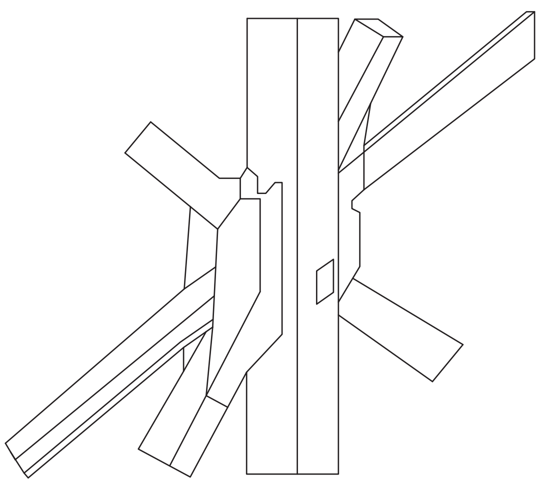

Fig. 5.16 Nodes using butterfly plates in the CCTV building [5].

5.4.4.4 Optimal Angle of Diagonal Members for Maximum Shear Rigidity

The angle of the diagonal member is an importance design consideration, as it will determine the stiffness of the structure and the usage of the material. The optimal angle is 69 degrees as recommended by Moon [8]. However, it depends on the requirement from the client and the architect. As shown in Ref. [9], when designing the Poly International Plaza, the architect wanted to design the exterior facade of the building resembling a Chinese paper lantern (as shown in Fig. 5.14), and therefore, the angle is very shallow.

Federal Emergency Management Agency [10] provided the way for optimization of the angle of the diagonal members in the diagrid structure to maximize shear rigidity (Fig. 5.17).

Considering only shear rigidity, the optimal angle for diagonal members can be estimated using.

$$V = 2F_d \cos\theta$$

where V is the cross-sectional shear force shown in Fig. 5.17; F_d is the diagonal member forces by assuming that the members carry only axial forces.

$$F_d = A_d\sigma_d = A_d E_d \varepsilon_d$$

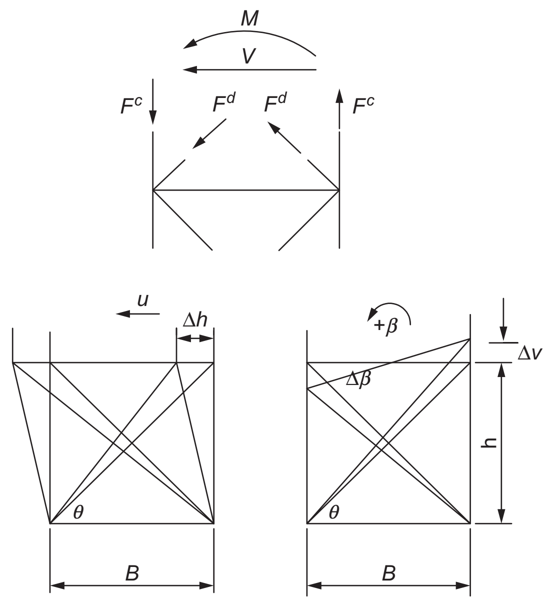

Fig. 5.17 Diagrid angle optimization [9].

where ε_d is the axial strain.

$$\varepsilon_d = \frac{e_d}{L_d} = \frac{\Delta_h \cos\theta}{h/\sin\theta} = \frac{\Delta_h \cos\theta \sin\theta}{h}$$

If the shear strain is approximately as follows:

$$\gamma \approx \frac{\Delta_h}{h}$$

$$\varepsilon_d = \gamma \cos\theta \sin\theta \approx \frac{\gamma \sin 2\theta}{2}$$

So the shear force.

$$V = \gamma A_d E_d \sin 2\theta \cos\theta$$

So by plotting the $\sin 2\theta \cos\theta$, one can calculate the optimal angle for maximum shear rigidity of the system.

5.4.4.5 Design of the Internal Core in Diagrid Buildings

The use of the diagrid system can diminish the dependency on the core to provide the overall lateral stiffness of the buildings. However, it is rare to purely rely on the perimeter tube to resist all the lateral loads. It is common to share the loads with a reinforced core or a steel core. In particular, if a building is located in a seismic zone, a core is essential for stability in the seismic design. Most of seismic codes have the design requirement which relies on the core to resist the seismic load. The perimeter diagrid structure that is assuming all of the lateral loadings is functioning as a bearing wall-type system and is therefore not addressed in the current seismic codes and practices.

5.4.5 Case study of Guangzhou International Finance Centre

As discussed in the previous section, the building has 103 floors with 432 m of height. In the center, it has a reinforced concrete core (from the ground floor to the 69th floor, as shown in Fig. 5.21), a transition area (from floors 69 to 72), and then steel columns (from floor 72 to the roof). These columns create a hole in the center (as shown in Fig. 5.22). The whole project was designed based on the Chinese code.

As shown in Fig. 5.18, the diagrid structure consists of diagonal members of steel tubes filled with concrete. In addition, as in Fig. 5.21, the reinforced concrete core is connected to the diagrid by tie beams. At the perimeter of the slab, there are also ring beams. These beams not only connect the RC core with the diagrid, but also support the slabs.

Fig. 5.18 Guangzhou International Finance Centre under construction. *(This file is licensed under the Creative Commons Attribution-Share Alike 4.0 International, 3.0 Unported, 2.5 Generic, 2.0 Generic and 1.0 Generic license. https://upload.wikimedia.org/wikipedia/commons/thumb/1/18/IFC_Guangzhou.jpg/682px-IFC_Guangzhou.jpg.)*

The unbalanced force between the core and the diagrid can be resisted by the ring beam and the tie beams. The roof of the center of the building is made of steel.

In addition, the overall exterior shape of the building has been aerodynamically optimized to minimize the wind effect.

5.4.5.1 3D ETABS Model

The building is a model in the ETABS as shown in Fig. 5.19. The model is set up based on the literatures and drawings found on the web. Due to the limited access to real design documents, the dimensions were estimated based on the available documents and pictures found online. It cannot represent 100% the original design; however, it is accurate enough for a case study purpose.

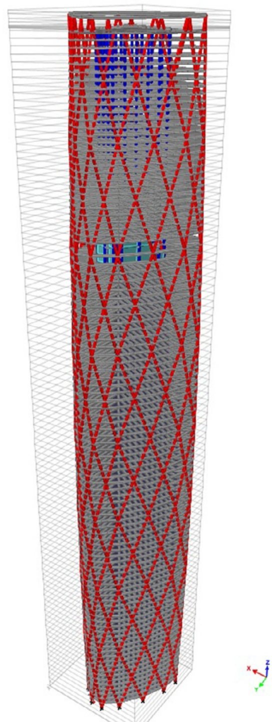

Fig. 5.19 Model of the Guangdong Tower in ETABS. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

Fig. 5.20 shows the lateral stability system of the building, which is core + diagrid. Figs. 5.21 and 5.22 show the typical plan layout of the building.

5.4.5.2 Modeling Result

Fig. 5.23 shows the axial force distribution of the diagrid under wind loading case. It can be seen that the structural members in the diagrid are in compression in the leeward side, but in tension in the windward side, which effectively resist the wind-induced overturning moment and shear force.

Fig. 5.24 shows the axial force distribution of the diagrid under dead load. It can be seen that the structural members in the diagrid are primarily in compression due to gravity load, this is different from the normal bracing system introduced in Section 5.2, which are primarily taking the lateral load rather than gravity load. As explained in the previous sections, this is one of

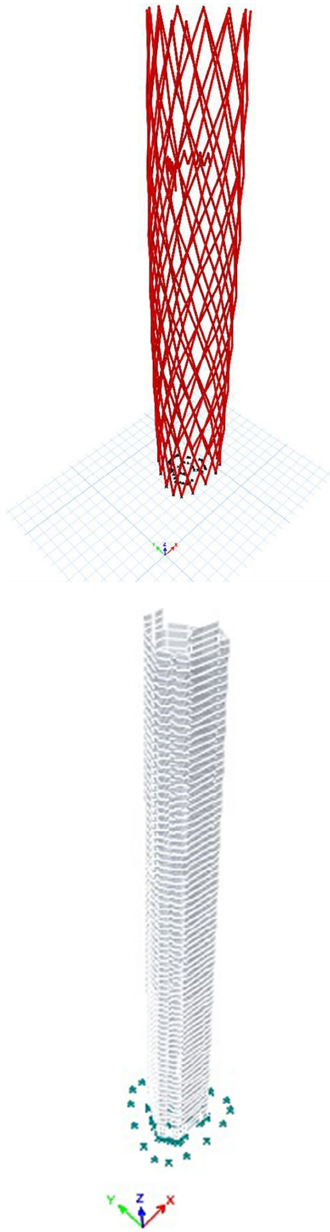

Fig. 5.20 Diagrid and core wall for the Guangdong Tower in ETABS. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

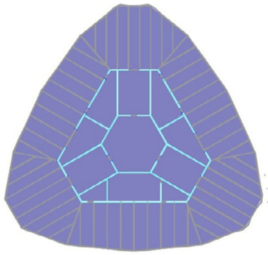

Fig. 5.21 Structural plan of grand floor of the Guad Dong Tower in ETABS. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

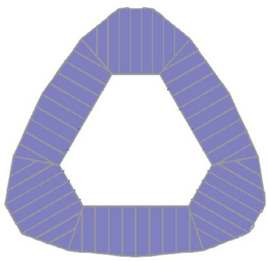

Fig. 5.22 Structural plan of level 100 of the Guangdong Tower in ETABS. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

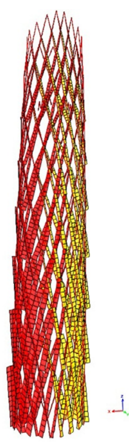

Fig. 5.23 Axial force distribution under wind loading in a diagrid. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

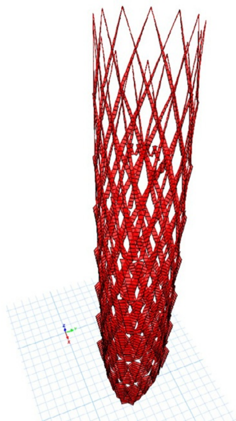

Fig. 5.24 Axial force distribution under dead load in a diagrid. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

the great advantages of the diagrid system over the conventional bracing systems.

Figs. 5.25 and 5.26 show shear and axial force distributions of the core under wind load condition. It can be seen that the core wall is in compression in the leeward side but in tension in the windward side.

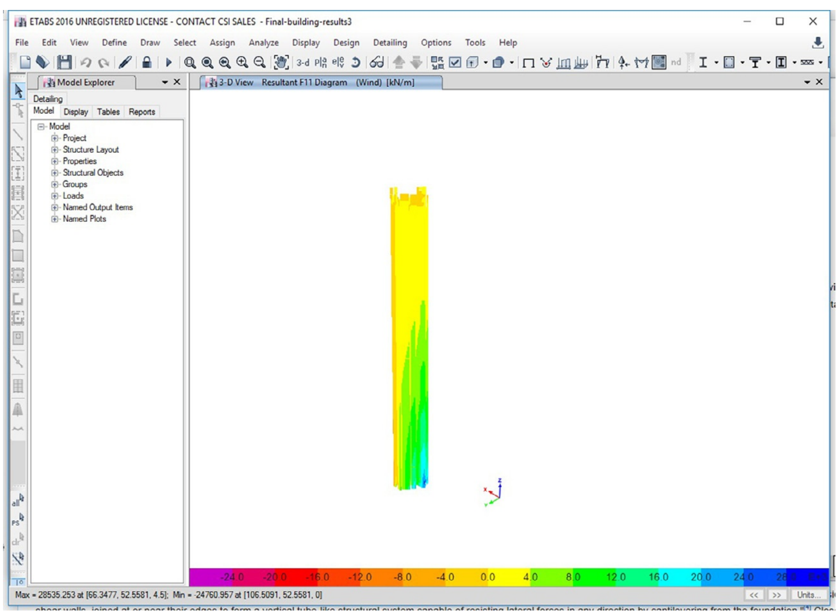

Fig. 5.25 Shear force distribution under wind loading. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

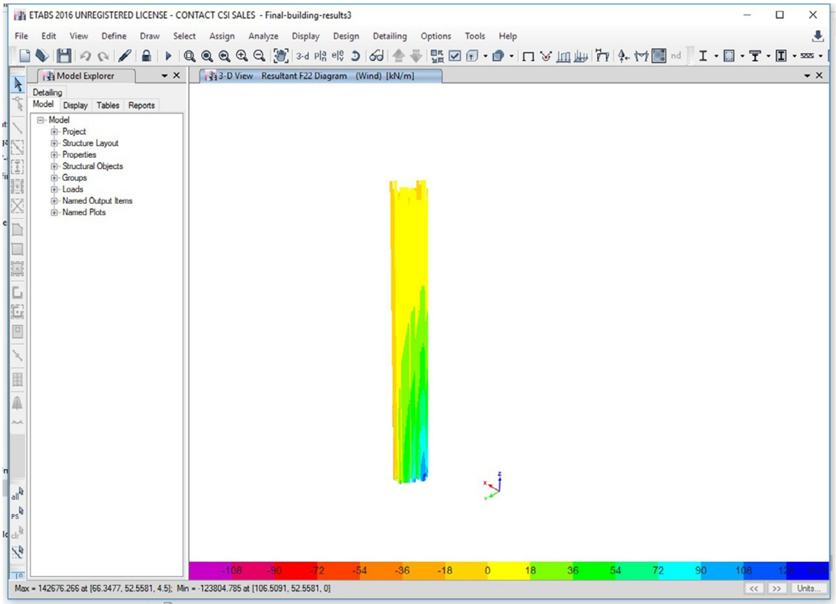

Fig. 5.26 Axial force distribution under wind loading. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

5.5 MOMENT RESISTING FRAMES

The moment resisting frame (MRF) system using the moment-connected frame as the major lateral stability system provides lateral resistance. It is also called a rigid frame. MRF can be of steel or concrete frames. Both the columns and beams must be designed to give strong resistance to bending. The beam-to-column connections are also needed to be detailed specially to accommodate the bending moment caused by the lateral load.

The disadvantage of the MRF system is that the rigid frame requires expensive detailing when assembling the connections. In addition, lateral drift due to the P-Delta effect is hard to control, which will increase the sway of the building, it will also produce additional bending of the beams and columns. Therefore, it is not used as a sole lateral resistance structure in tall buildings. In practice, this system has been frequently used together with core wall or bracing as a combined lateral resisting system in most tall buildings as shown in Figs. 5.27 and 5.28. This coupled system is called the shear

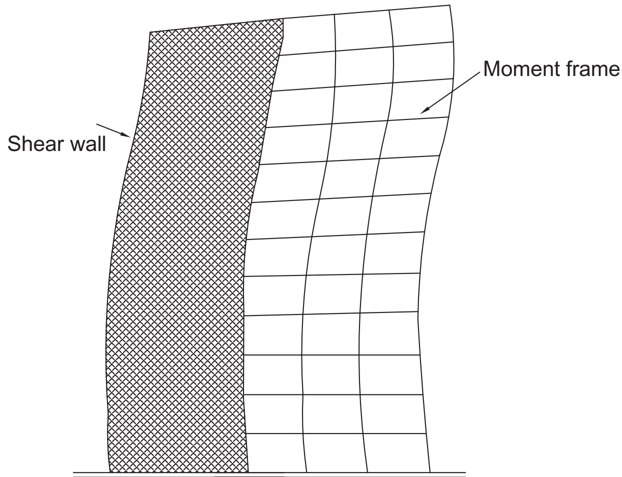

Fig. 5.27 Coupled shear wall and moment frame.

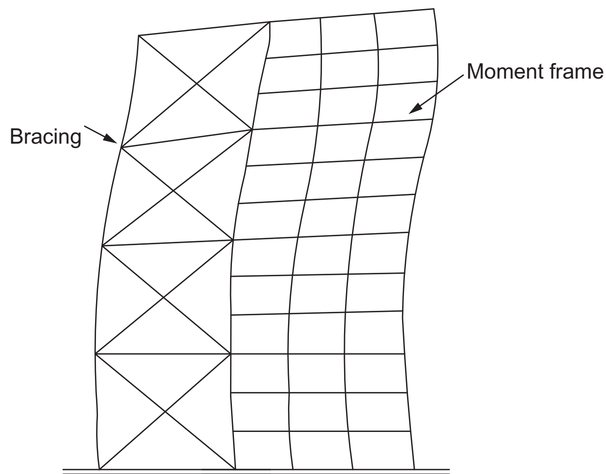

Fig. 5.28 Coupled bracing and moment frame.

frame interaction system [11]. Each system controls the lateral stability of the building at different locations which provide exceptional lateral rigidity for tall structures. A famous project example is One World Trade Center in New York. It comprises a high-strength concrete core surrounded by a perimeter steel moment frame which is introduced in Chapter 4.

There are two primary types of moment frames, ordinary and special. The special MRFs are detailed to ensure ductile behavior of the beam-to-column joints and are normally used in zones of higher seismicity. It should be detailed to make sure that the ductility of the connections allows for sufficient resistance of excessive movement and swaying of the structure.

It is worth noting that the steel moment-frame buildings damaged in the 1994 Northridge earthquake are a special type, known as welded steel moment frames (WSMF). This is because the beams and columns in these structures are connected with welded joints. Generally, WSMF buildings constructed in the period 1964–94 should be considered vulnerable to this damage [10].

5.6 MEGA FRAME STRUCTURES (SUPERFRAME STRUCTURES)

5.6.1 Introduction

Mega frame structure (also known as superframe structure) is an ideal structural system for supertall buildings, as it can provide efficient rigidity against the lateral loads with minimum amount of structural materials [11]. It consists of mega columns and mega girders. They are rigid connected together at approximately every 10–25 stories to build large moment-resisting frames, which are primary lateral resisting structural systems for this type of structure. This large frame also supports the secondary structures (or substructures). Therefore, this kind of structure consists of large number of members. One of the famous examples is the Shanghai Tower shown in Fig. 5.34. The other famous example is the HSBC headquarters in Hong Kong.

5.6.2 Case Study of the HSBC Headquarters in Hong Kong

As shown in Fig. 5.29 HK HSBC main building, the major structural system of this building composed of eight structural steel masts carry all the structural loads, which allows the column-free floor area on each floor. Floor slabs are suspended by pairs of trusses located at five different stories. The trusses are supported by the structural steel masts, which transfer all vertical forces to the foundation. It can be seen that one of the advantage of the mega frame structure lies in that it maximizes the flexibility to adapt to different spatial arrangements [12].

Fig. 5.29 HK HSBC Main Building. *(This file is licensed under CC-BY 3.0, produced be Wikipedia user -Wpcpey, https://commons.wikimedia.org/wiki/File:HK_HSBC_Main_Building_2008.jpg).*

5.6.3 Case study of China Zun Tower (A Mega Frame Structure)

5.6.3.1 Introduction

As shown in Fig. 5.30, China Zun is a supertall skyscraper under construction in the Central Business District of Beijing, Capital of China. It has 108 stories with a height of 528 m, a gross floor area of 350,000m^2. The structural engineer is a design consortium consists of my previous employer Beijing Institute of Architectural Design (Group) Co., Ltd. and Arup. The shape of the building resembles the ancient Chinese wine vessel. It is currently the tallest building in the location, where the seismic fortification intensity is 8 magnitudes.

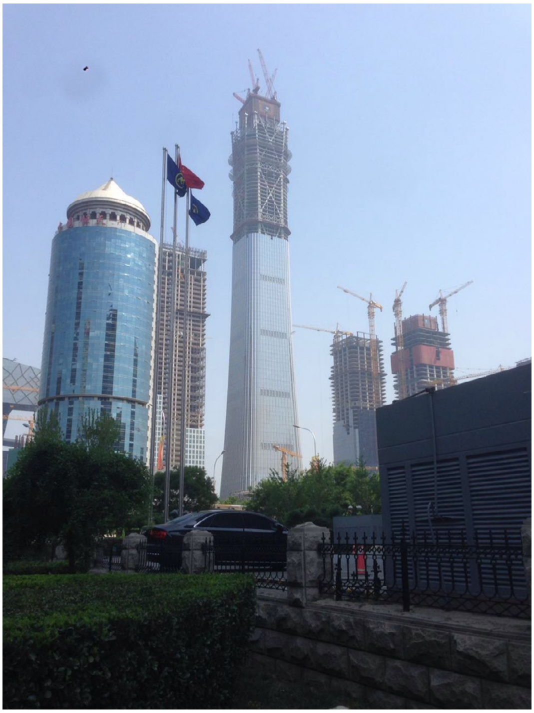

Fig. 5.30 China Zun Tower under construction. *(Photo taken by the author's father in Beijing, China.)*

As introduced by [13], the geometry of the mega frame system was built in Rhinoceros shape, which is dealt with in Chapter 6. Parametric modeling design was performed using Grasshopper.

5.6.3.2 The Structural System

Beijing has the highest seismic fortification requirement of China's major cities (PGA = 0.20 g for a 475-year return period). The structural system of a tall building with such a height must find a proper balance between stiffness and ductility. Beijing is in a location where magnitude 8 seismic fortification intensity is required by the Chinese code [14]; therefore, as introduced by [15] for this particular tall building, the design consortium introduced a highly efficient dual system for lateral force resistance. As shown in Fig. 5.32, it is composed of a fully braced mega frame and a concrete core. Composite steel-concrete floor system is used for the floor system.

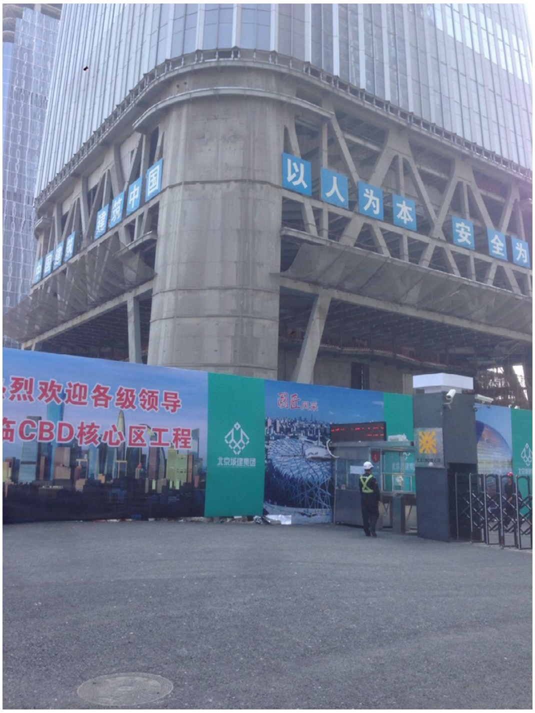

Fig. 5.31 Mega column, belt truss and core of China Zun Tower. *(Photo taken by the author's father in Beijing, China.)*

As shown in Fig. 5.33, the mega columns are concrete-filled steel boxes. There are two mega columns at each corner, which merged into four huge concrete columns, as shown in Figs. 5.31 and 5.32. It can also be seen that the mega columns are connected through horizontal transfer truss as well as huge braces, which extends to 10 stories (Figs. 5.30 and 5.31).

The Chinese code [14] requires that there be no shear failure during a maximum credible earthquake event. Especially, as the shape of the building resembles a Zun cup, the enlarged top zone encompasses extra mass at the top of the tower. This will have adverse effects when the building is subject to earthquake loading, which may generate extra shear to the neck zone, so special measure need to be made. To tackle this problem, as introduced by [13], steel plates with a thickness of 30–60 mm are embedded in the core at the low or top floors for further strengthening of the core, which greatly enhances the shear resistance of the core. The wall thickness is also reduced, because of the addition of steel plates.

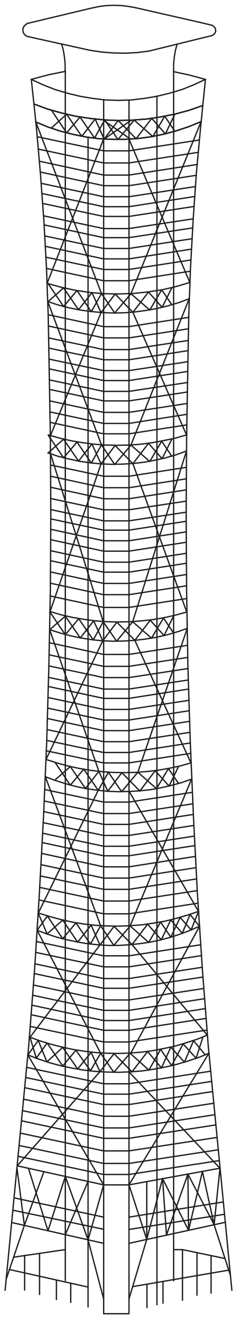

Fig. 5.32 Structural system of China Zun Tower (the drawing is made based on the FE model in Ref. [13]).

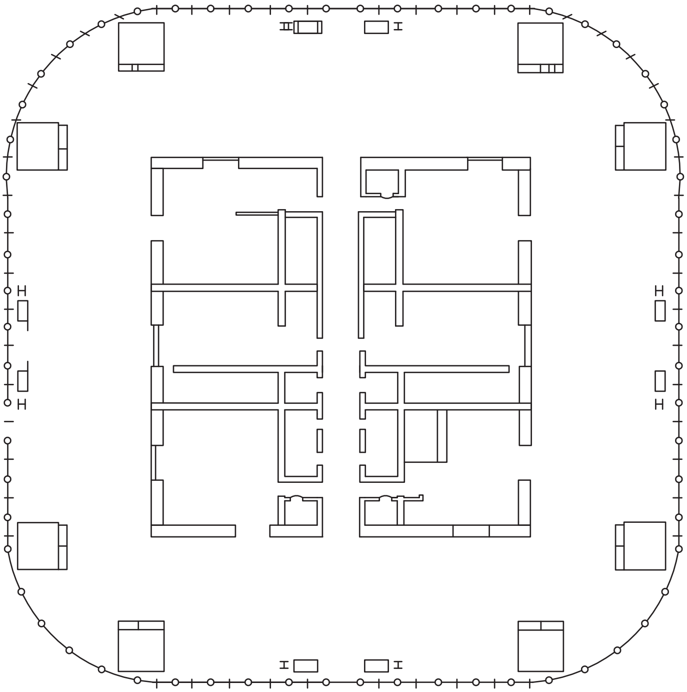

Fig. 5.33 Structural plan layout of China Zun Tower (the drawing is made based on the architectural drawing of KPF/BIAD) [13].

5.6.4 Case Study of Shanghai Tower—Mega Frame Structure

The 632-m tall Shanghai Tower is the tallest building in China and the world's third tallest structure. The building has 128 stories with five underground floors. The total floor area is 380,000 m^2. The tower is design by American architectural firm Gensler. The structural engineer for this tower is Thornton Tomasetti. The Shanghai Tower also use tuned mass damper to limit swaying at the top of the structure, it was the world's largest at the time of its installation (Fig. 5.34).

5.6.4.1 The Structural System of the Shanghai Tower

As shown in Fig. 5.35, the major structural system of this building is the internal concrete core wall and outer mega frame system. Outside the main structural system, there is another secondary cladding system to support the curtain wall, which is directly connected to the mega frame. Shanghai Tower has the tallest mega frame structure in the world and the largest

Fig. 5.34 Shanghai Tower. *(With the permission of "Baycrest"-Wikipedia user-CC-BY-SA-2.5.)*

and highest flexible curtain wall in the world. Strictly speaking, Shanghai Tower and China Zun are not pure mega frame structures, as they have massive core wall works as part of the lateral stability system. This is different from the HSBC headquarters in Hong Kong, which solely relies on the mega frame to resist both lateral load and gravity load.

As shown in Fig. 5.36, to enable the case study, a 3D model of Shanghai Tower was built in ETABS; it is based on images and drawings from the internet including paper [15]. Due to the limited access of the designed files, the model does not 100% represent the real design; however, it is accurate enough for a case study purpose.

As shown in Fig. 5.36, the building is divided into nine separate zones along its height, which are separated by eight zones strengthening the floors. Each zone is about 80-m high. The core of the structure is approximately

Fig. 5.35 Shanghai Tower under construction. *(This file is licensed under the Creative Commons Attribution-Share Alike 3.0 Unported license. https://commons.wikimedia.org/wiki/File:Baustelle_des_Shanghai_Towers_am_01.09.2012.JPG.)*

30 m^2. The mega frame is made up of supercolumns and diagonal columns along with double belt truss at each zone.

As shown in Fig. 5.37 there are eight supercolumns along with four corner columns with two-story high-belt truss to connect these columns. The supercolumns are composite structures made up of steel sections encased in concrete.

As shown in Fig. 5.37, the internal core and outer mega fame are also connected by outriggers at six levels (at zones 2, 4, 5, 6, 7, and 8).

As shown in Fig. 5.38, the floors are designed as composite metal deck slabs. They are supported on the steel beams supported in between the internal core and ring beams, which are supported on the mega columns. The inner layer of the glass curtain wall is attached along the periphery of the floor slabs; the outer layer of the glass curtain wall is attached to the radial trusses.

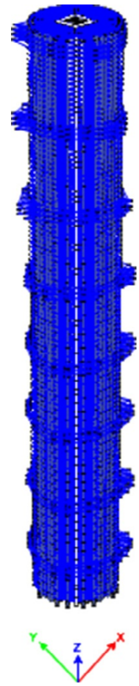

Fig. 5.36 3D model of Shanghai Tower in ETABS. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

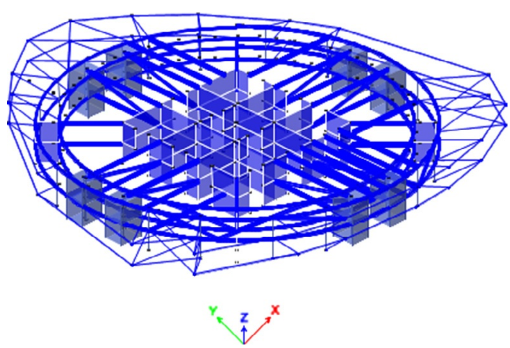

Fig. 5.37 Outrigger, central core, and mega frame at zone 2 level. *(ETABS screenshot reprinted with permission of Computer and Structures.)*

5.6.4.2 Model Setup Methods

In this section, the way to set up this complicated model will be briefly dealt with. The model is first built using the Revit structure provided by Autodesk. The model is set up based on the structural drawings obtained

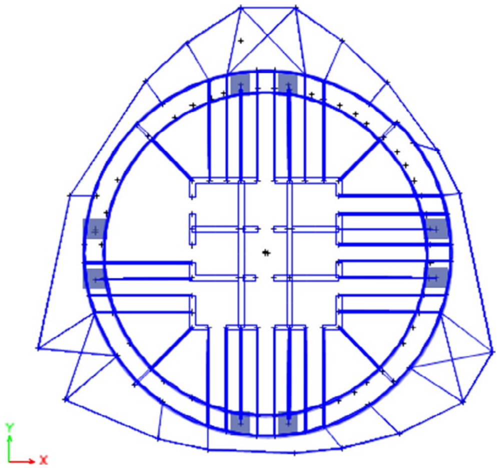

Fig. 5.38 Typical floor plan of the Shanghai Tower, zones 1–4.

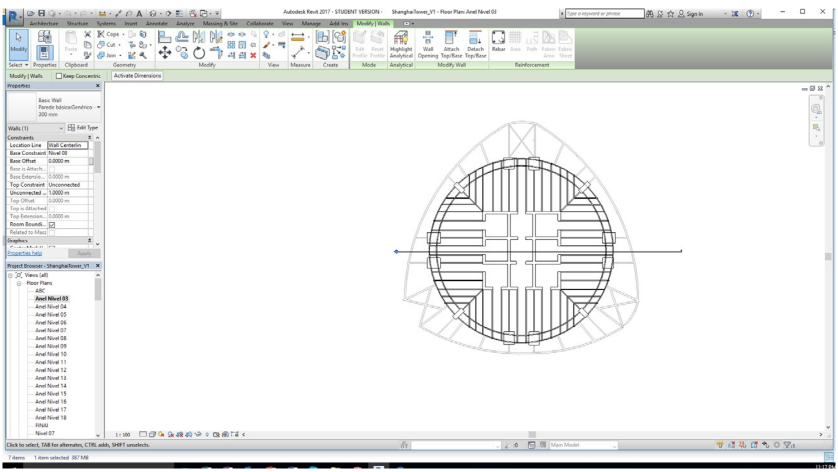

Fig. 5.39 Ground floor layout of the 3D Revit Model.

(Figs. 5.39 and 5.40). After the 3D model is set up (Fig. 5.41), the model can be imported into ETABS, for further analysis. Using Revit, one can directly define the member sizes. This is an advantage over AutoCAD or Rhino, as these two draughting programs can only define the polylines, the user has to define the member size in the analysis software.

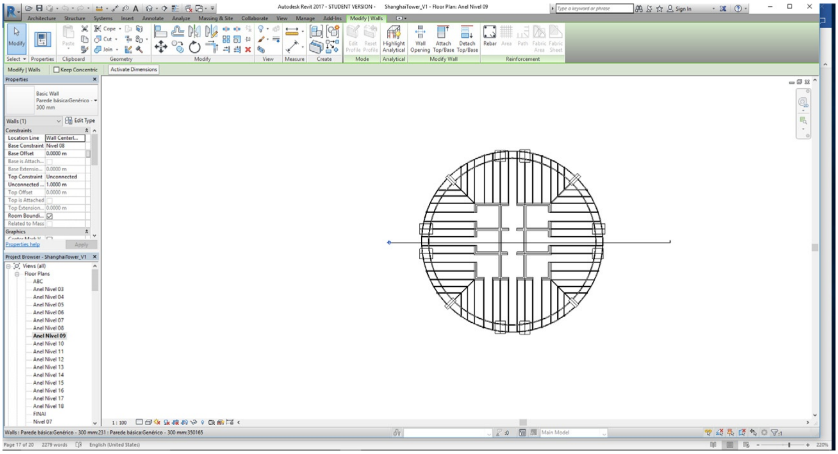

Fig. 5.40 Level 25 plan layout of the 3D Revit Model.

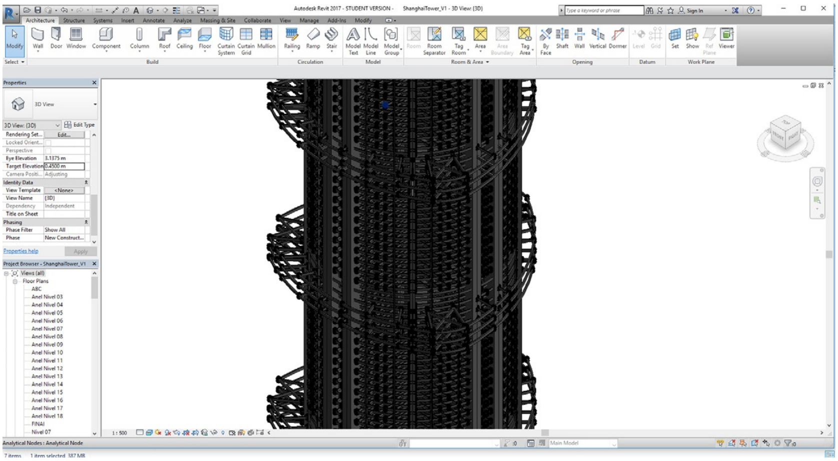

Fig. 5.41 3D Revit Model of the Shanghai Tower.

5.7 CONCLUSION

In this chapter, several different structural systems for tall buildings are introduced, which include the bracing system, moment frame system, diagrid system, 3D space frame, and mega frame systems. It can be seen that with the increase in structural height, the diagrid structure and the mega frame structure are the most effective structural systems. Although a pure diagrid or mega frame structure does not necessarily require the assistance of the core to resist lateral loads (the project example can be the HSBC headquarters in Hong Kong, which is a pure mega frame structure), for supertall buildings, both of them need to work together with the internal concrete core to guarantee sufficient lateral resistance. It can also be seen that the core and the diagrid or mega frame are normally connected through beams rather than purely relying on the floor diaphragm, making it a better coupling between the internal and external lateral systems. Similar design principles can be found for the outrigger system discussed in Chapter 3. The dual system composed of the core and mega frame or the diagrid frame is also beneficial for gravity load resistance, as the internal and external systems share the gravity loads that come from the floor, which gives the support to the floor system.

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CHAPTER SIX

Design and Analysis of Complex Structures

Abstract

In this chapter, the definition of complex structure is discussed. Examples of current complex structures in the world are demonstrated, which are: Sydney Opera House, Kyoto and Shin-Osaka Train Station, Marina Bay Hotel, Beijing Aquatic Center and Emirates Airline Cable Car Supports. It also introduces major design considerations for complex structures, such as the design of space structure, arches, support, and connections. An introduction to the complex structure analysis methods is also made. In addition, the building information modeling and parametric modeling techniques are given in detail followed by the topics of the development of the application program interface and graphical user interface. In the final part of this chapter, some case studies are discussed, which is to help the readers to understand fully the effective way to analyze the complex structures.

Keywords: Complex structure, BIM, Parametric modeling, API, GUI, Multiphysics fire analysis

6.1 INTRODUCTION

With the increasing demands from both clients and architects, structural projects are becoming more and more complex. Complex and irregular geometry is common for most of the large structural projects (e.g., Beijing Aquatic Center and Shanghai Tower). When we are talking about the complex structure, the first thing that comes to people's mind is the structure with complex geometry, such as the Bird Nest Stadium and Aquatic Center in Beijing.

A broader definition of a complex structure can be either a structure with complex geometries or a structure that needs to be analyzed using complex structural analysis methods and theories (such as multiphysics modeling, coupled FEM (finite element method) and SPH (smoothed particle hydrodynamics) modeling analysis, and nonlinear geometrical analysis). In addition, large-scale complex structures, such as the large station complex in Kyoto Train Station, large-scale stadiums and airport terminals can also be treated as complex structures due to the complexity of their structural type.

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Some special structure types such as tensile membrane structures and tensegrity structures are complex in terms of their geometry as well as analysis method. This is because the nonlinear geometrical analysis method is needed to perform form finding and stress analysis for these types of structures. As the analysis theories for tensile structure, tensegrity structure, and offshore structure are different for conventional structures these are being discussed in detail in Chapters 7 and 8.

The other special structural types such as offshore oil platforms and offshore wind turbines are subject to more severe environmental loadings such as huge waves, strong winds, etc.; they need special design considerations, and therefore can also be treated as complex structures.

All these complex structures mentioned require efficient structural analysis software packages, which can represent the original structure with sufficient accuracy. These software packages should also have the capacity to perform advanced numerical analysis.

When analyzing complex structures, the first priority in the analysis is to set up the correct geometry that can replicate or maximally represent the original structure in the analysis packages. However, due to their complexity in terms of the geometry, it is consequently hard to set up the model directly in most of the current analysis software packages such as ANSYS, Abaqus, Autodesk Robot, and SAP2000. As the developers for these packages focus more on the capacity of their software to perform complicated structural analysis using different analysis theories, most of these analysis packages lack the power in their preprocessing modules. Therefore, in the current design practice, special draughting software such as Rhino3D, AutoCAD, and Microstation are widely used by the structural engineers as they provide powerful functions such as parametric modeling which enable an engineer to set up the structural geometry swiftly. The 3D geometric model can be consequently imported into the analysis packages such as SAP2000, Autodesk Robot, Abaqus, or ANSYS for structural analysis. The structural engineers can also easily re-iterate the structural design option with architect and clients after analysis.

In addition, the capacities of the structural analysis packages are also essential in the analysis of the complex structures. This is because that the stress states of structural members and structural connections are generally complicated for complex structures. Some engineering problems such as nonlinear geometrical analysis and fluid structure interaction require efficient analysis algorithms. This also brings difficulties in design and analysis. Therefore, the analysis program should be capable of solving the aforementioned problems.

In general, the design and analysis of complex structures require the engineers to be capable of using the latest draughting programs and advanced analysis programs. In the meantime, they need to fully understand the analysis theories that underpin these analysis programs. Therefore, in this chapter, the effective methods of analyzing the complex structure will be addressed, which include the building information modelling (BIM) framework, the latest developed parametric modeling techniques, and the development of application program interface (API) and graphical user interface (GUI) for commercial analysis software. The modeling examples of some prestigious projects will also be demonstrated.

6.2 EXAMPLES OF COMPLEX STRUCTURES

In this section, some real project examples of the complex structures will be demonstrated. This is to help the reader to have a general background of the complex structures.

6.2.1 Sydney Opera House, Australia

The Sydney Opera House is one of the 20th century's most famous and distinctive buildings. As shown in Fig. 6.1, it features a complicated roof structure. The building situated in Sydney Harbor is designed by a Danish architect Jørn Utzon and the structural engineering company is Arup. The building was formally opened on October 20, 1973. This is an ironic example of complex structures; particularly it was designed in 1973, when design tools such as finite element software and AutoCAD software are not available, let alone the parametric modeling technique which is going to be discussed in this book.

Fig. 6.1 The Sydney Opera House, Sydney, Australia. *(Photo taken by the author.)*

The ironic roof structure is constructed through a series of large precast concrete shells, each composed of sections of a sphere of 75.2-m radius. The building covers 183-m length and 120-m width, which was a huge building complex at that time. The precast concrete shells are literally made by a number of smaller precast concrete panels supported by precast concrete ribs. The precast concrete panels are assembled on site.

As in Ref. [1], initially, no definite geometry for the shells had been established due to its complexity. As work progressed, the shells were developed according to a spherical geometry. In order to design this complex structure, the designer has to make several physical full-scale mock-up models, which together with the drawings enabled them to solve specific problems. As engineers practicing in current time, we are very fortunate to have so many powerful software packages to help us, which I am going to demonstrate in this chapter.

6.2.2 King's Cross Western Concourse, UK

As in Fig. 6.2, the architect of this project is John McAslan + Partners and the structural engineer is Arup. The structure has a height of 20 m and spans of 150 m in the western range. The concourse occupies 75,000 m^2 [2]. It is currently the largest single-span station structure in Europe. This building

Fig. 6.2 Single-layer grid in King's Cross Station Western Concourse, London, UK. *(Photo taken by the author.)*

features in its unique umbrella shape roof, as shown in Fig. 6.2. The vast canopy is made up of a single-layer lattice shell, which is supported by 10 steel columns at the circumference of the roof and a central vertical truss column as another support. In the latter part of this chapter, this project will be selected for a case study, which will demonstrate how to set up a model to replicate the roof effectively and analyze using the general purpose software ANSYS.

6.2.3 Shin-Osaka Station, Japan

As shown in Fig. 6.3, Shin-Osaka Station also called Osaka Station City is the main railway station in Yodogawa-ku, Osaka, Japan. It is the western terminal of the high-speed Tokaido Shinkansen line from Tokyo, and the eastern terminal of the Sanyo Shinkansen. The whole station was designed as the so-called station city in Japan. This is a planning strategy for major cities in Japan. A railway station conceived as a "station-city" has many facilities. They play a role like a city, ranging from above the ground to underground, with urban functions accommodated into this large station complex [3].

"Osaka Station City" is one of the largest stations in Japan. The station's exterior shape is largely determined by the advanced structural roof. It consists of a 28-story North Gate Building and a 16-story tower at the south side [4]. As shown in Figs. 6.2 and 6.3, these two buildings are connected by a large, 180-m long and 100-m wide roof. The roof is made of steel space

Fig. 6.3 Shin-Osaka Station, Japan. *(Photo taken by the author.)*

Fig. 6.4 Space truss roof Shin-Osaka Station, Japan. *(Photo taken by the author.)*

truss and is covered by glass, which shields the platforms and a large hall from rains. The total floor space of the new station (without the towers) is 42,300 m^2 (Fig. 6.4).

6.2.4 Kyoto Station, Japan

Kyoto Station is a major railway station and transportation hub in Kyoto, Japan. It is also one of the so-called station cities in Japan. It consists of a shopping mall, hotel, movie theater, and departmental store. It is considered as one of the largest railway stations in the world. The new Kyoto Station building was designed by the architect Hiroshi Hara Construction in 1995. The 70-m-high station building stretches 470 m from east to west and covers a huge floor area of 238,000 m^2 [5].

As shown in Fig. 6.5, it features a striking giant glass cubic façade. The huge facade was constructed with glass plates installed on the steel double-layer grid system. It spans from the internal tower blocks and external huge truss columns. The pace structures are composed of both a vertical and horizontal double-layer grid, which is one of the largest glass façades in Japan.

6.2.5 Marina Bay Sands Hotel and Art Science Museum, Singapore

Fig. 6.6 shows two ironic landmarks of Singapore, Marina Bay Sands Hotel and the ArtScience Museum; they are both part of the Marina Bay Sands complex.

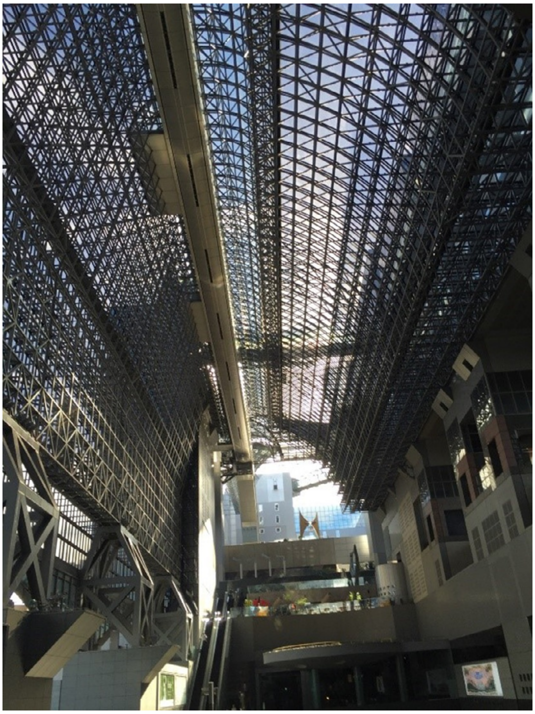


Fig. 6.5 Double-layer grid in Kyoto Station, Japan *(Photo taken by the author.)*

Fig. 6.6 ArtScience Museum and Marina Bay Sands Hotel, Singapore. *(Photo taken by the author.)*

6.2.5.1 Marina Bay Sands Complex

The Marina Bay Sands complex was designed by Moshe Safdie of the Las Vegas Sands Corporation. It is a 929,000-m^2 urban district anchoring the Singapore waterfront. The complex integrates the waterfront promenade, a multilevel retail arcade and the iconic Museum of ArtScience and the Marina Bay Sands Hotel. It also consists of two theatres, a casino, an enormous convention and exhibition center, and a hydraulically adjustable public event piazza [6].

6.2.5.2 ArtScience Museum

As shown in Fig. 6.7, the ArtScience Museum is a museum located within the integrated resort of Marina Bay Sands in Singapore. The shape of the building resembles a form reminiscent of a lotus flower which is made of 10 "fingers"; the tallest one is 60 m above ground level. As mentioned in Ref. [7], the fingers' primary surfaces are made of special glass fiber reinforced polymer (GFRP), which has never been used in a project in Singapore. This is probably the largest structure in the world using GFRP panels. The traditional cladding material would be too heavy for such large cantilever structures.

It is also a so-called green structure. The roof of the ArtScience Museum allows rainwater to be harvested and channelled down through the center of the structure to the reflecting pond at the foot of the building. The rainwater is also recycled for use in the museum's bathrooms.

As shown in Fig. 6.6, opened on June 23, 2010, the 55-story Marina Bay Sands Hotel towers is composed of three towers with a 2.5-acre Sky Park at the top. The sky park accommodates a public observatory, gardens, a 151-m-long swimming pool, restaurants, and jogging paths and offers sweeping panoramic views of Singapore. The Sky Park is designed as a sailboat; so most local people call it the sailboat hotel.

6.2.6 Emirates Air Line Cable Car Supports

The Emirates Air Line is a cable car link across the River Thames in London, England, built by Doppelmayr with sponsorship from the airline Emirates. The architect of this project is WilkinsonEyre.

As shown in Fig. 6.8, this structure features a unique shape of the supporting towers, which are formed from a series of steel ribbons which

Fig. 6.7 ArtScience Museum, Singapore. *(Photo taken by the author.)*

Fig. 6.8 The supporting spiral tower for a cable car across the Thames River in London, UK. *(Photo taken by the author.)*

are arranged as a tapering spiral. The ribbons are lined and restrained together by a helical tie that screws up the inside of the structure. The effect is of lightness and movement, with the loose curves of the central helix providing the necessary stiffness of the tower but with the least amount of steel used. This tower is similar to the conventional truss column, which can withstand the huge compression force that comes from the cables.

6.2.7 Beijing National Aquatics Center (Water Cube) Beijing, China

As shown in Fig. 6.9, the Beijing National Aquatics Center (also called as the Water Cube) is an aquatics center that was built in Beijing Olympic park. It was designed for the swimming competitions of the 2008 Summer Olympics. The Center was handed over for use on January 28, 2008. The architect of Water Cube is the design consortium composed of PTW Architects &

Fig. 6.9 Water Cube Stadium, Beijing, China. *(Photo taken by the author.)*

China State Construction International and Shenzhen Design Consulting Co. The structural engineer is the consortium composed of Arup and China State Construction International.

This structure features in a polyhedron steel space frame structure using the translucent ETFE (ethyl tetra fluoro ethylene) inflammable cushion membrane panels as cladding. As explained in Ref. [8], the polyhedron steel space frame is designed very robustly and is ideally suited to the seismic conditions found in Beijing.

As discussed in Ref. [9], the polyhedron space frame structure uses rectangular hollow sections as chords, circular hollow sections as web members, and welded hollow balls as joints. Except for its unique shape, this kind of space structure is different from the conventional space structures discussed in the earlier sections, such as the space structure used in Kyoto Station, Japan. For traditional space truss, the members are designed to primarily take the axial force. However, the polyhedron space frame using rigid connection is a 3D vierendeel space frame with the dimensions of 176 m × 176 m × 29 m. The structural elements take combined bending moment, axial force, shear force, torsional moment, and the moment stress is larger than axial stress.

Such a complicate structure form would have been a mission impossible, if a computer program had not been used. Especially, the whole structure composed of 24,000 beam members and 12,000 nodes and during the design process the structural options was constantly changing.

The structure is designed using a series of computer programs such as ETABS, SATWE, SAP2000, STRRAND7, Midas, and Microstation [9].

It is particularly worth noting that Microstation VBA script was used to set up and evaluate the 3D wireframe model. The 3D model created by the scripts consisted of members and nodes spheres of the same size at the initial stage [10]. Each wireframe element was denoted by its element number and section type. The information of the members is retained during the export to the DWG format. The final output from the script was an Excel spreadsheet detailing each of the spherical (or hemispherical) nodes. The correspondent data was tabulated before being written to a file using Excel's VBA.

After the 3D wireframe model was set, the analysis model was imported into Microstation via the VBA script. Using Microstation's Structural package with libraries of section sizes, the size of each member can be assigned. Nonstandard sections such as fabricated sections can also been added into text files and subsequently used within the structural model. As explained in Ref. [11], except for software Microstation, the other design-orientated software such as SAP2000 and ETABS all have the capacities to define nonstandard sections.

As in Ref. [9], in the design stage, the member sections are further optimized follow the "strong wall and weak roof" principle, in which the control stress levels in wall members are lower than those of the roof members.

The design of the Water Tube was made around 2004; at that time the so-called parametric modeling probably was still in its infancy. It can be seen that with help of the computer software such complicated structures were built efficiently.

6.3 DESIGN CONSIDERATIONS IN COMPLEX STRUCTURES

From Section 6.2, it is clear that the complex structures are always involved with sophisticate design issues, due to features such as irregular geometry, large span, or complex relations between the main structure and substructures. This mostly includes special connection design and special member design considerations. It is clear that the traditional code-based design methods and traditional analysis software would not be able to handle such a degree of complexity. It requires special design and analysis software. Therefore, in this section, the major design issues such as how to design connections, supports, and special structural members such as arches and space structures will be discussed.

6.3.1 Setting Up the Geometry

As stated in Ref. [11], to be able to effectively model complex structures, it is easier to set up the model in these draughting programs such as Rhino, Revit, AutoCAD, and Microstation, then import them into the analysis program, rather than set up the model directly from the analysis programs. Irregular geometry is one of the features for complex structures. It is obvious that with the traditional method it would also be difficult to set up their geometry. Therefore, there is no doubt that new analysis techniques such as parametric modeling or BIM has to be used. They are going to be discussed systematically in the latter part of this chapter.

6.3.2 Space Structure

Fig. 6.10 depicts the double-layer lattice shell used for the terminals in ChengDu International Airport, in China. This is an example of using space structure in a public building complex. From the review of the existing

Fig. 6.10 Double-layer lattice shell in Chengdu International Airport terminal building, Chengdu, China. *(Photo taken by the author.)*

complex structures in the world and the project explained in Section 6.2 most of these complex structures are composed of space structures, such as a double-layer grid or tensile structure. This is because space structures such as space truss consist of a large number of structural members, which can easily achieve much more complicate geometry through changing the number of members, the orientation of the members, and the location of the members. Space structure can easily be adapted to any shape compared to other normal structural members such as single beam, slab, or shell.

The space structures are widely used in a variety of building types, such as sports arenas, exhibition pavilions, assembly halls, transportation terminals, and stations [11]. Some façades in tall buildings are also supported by the so-called vertical space structures.

Compared with other type of structures, such as tall buildings and bridges, stability is one of the important design issues needed to be considered in space structure design. For certain types of space structure such as the lattice shell, stability is of pressing importance.

These are following major types of buckling needed to be checked during the design:

1. member buckling
2. local bucking of certain members in a certain area
3. global buckling

Whenever the structural geometry is set up, these bucklings can be checked using the design-orientated program such as SAP2000, Robot, etc., as all of them incorporate the above bucking analysis in their functions.

6.3.3 Arch Structure

Arches have been used heavily for ancient structures. You can find them in churches or bridges. However, it can be noticed that they are also one of the key components for modern complex structures. In this section, examples of complex structure using arches will be demonstrated.

6.3.3.1 St Pancras International Train Station

Fig. 6.11 depicts the roof of the St Pancras International Train Station; where the roof is primarily made of long-span arches. These arches sit in the transverse direction across the station wall covering all the platforms.

6.3.3.2 Wembley Stadium

Wembley Stadium is a football stadium in London, England, which opened in 2007. It replaced the original stadium, which was demolished.

Fig. 6.11 Arch roof in St Pancras International Train Station, London, UK. *(Photo taken by the author.)*

Fig. 6.12 Wembley Stadium in London, UK. *(Photo taken by the author.)*

As shown in Fig. 6.12, it features a huge arch structure, the arch is 133-m high and it is made of 504 steel tubes and 41 steel rings. The components of the arch are prefabricated members. It took engineers 10 months to complete the assembly of the arch at Wembley. The tapered ends of the arch are attached to 70-ton hinges that are embedded into concrete bases [12].

The arch is designed to be a structural support of the roof of the stadium. The entire weight of the north roof and the retractable roof's weight are supported by the arch. Therefore, there is no need for pillars to be constructed and obstruct the view for the audience.

Although the arch works as a support to the roof, on the contrary, the weight of the roof also helps to balance the arch truss weight. Therefore, when designing this stadium, the tilting angle of the arch has also been optimized, as it is needed to provide effective support to the roof, as well as maintain adequate tension in the cable systems.

The roof structure of the Wembley Stadium is a complicated space structure system; particularly the huge truss arch is the key component of this system, making it difficult to analyze. Therefore, the draughting program and the advanced finite element analysis programs are needed. In the latter part of this chapter, the roof of the Wembley Stadium will be selected as a prototype for our case study, which will demonstrate the way to analyze a complex space structure effectively.

6.3.3.3 Tied arches, Heathrow Airport T5

As it is widely known that a large thrust force will be produced by the arch when the span of the arch increases. In order to counteract the thrust force, tied arches are quite common in some of the large span space structures. Fig. 6.13A shows the main skeleton of the Heathrow Airport Terminal 5. The main terminal building is the largest free-standing structure in the UK. It is 396-m long, 176-m wide, and 40-m tall [13]. The whole terminal building is covered by a series of space-curved tied arches supported by columns at the perimeter. Owing to the large span of the arch, a tie is used for each arch in Fig. 6.14A.

6.3.3.4 Summary

In this section, different types of arch structures are discussed. It can be seen that arches play an important role in complex structures, particularly for long-span space structures. Due to its complexities, such as long span, it would be challenging to design it without the help of advanced modeling techniques.

6.3.4 Design of Supports and Connections

Supports and connections are the most important design considerations for complex structures. Special support conditions are required for complex structures, as the load path of a complex structure is often different from the conventional structures. In addition, the connections or joints of the complex structure often have complicated stress states. For an example,

(A) Steel columns to roof arches connections

(B) Common support of roof and cladding

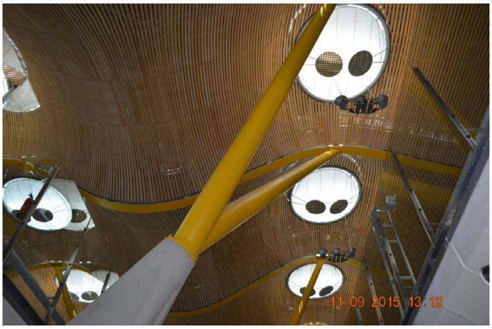
(C) Base connection details of steel columns

Fig. 6.13 Roof support at Heathrow T5. *(Photo taken by the author.)*

for a structure such as the Water Cube, many structural members merged into a single connection; it is not possible to analyze this using a conventional method, making the general-purpose finite element (FE) analysis, such as using ANSYS and ABQUS as the only viable option to verify their structural adequacy.

(A) Internal view

(B) Outside view



(C) Connection details of steel columns

Fig. 6.14 Roof support at Barcelona International Airport, Spain. *(Photo taken by the author.)*

6.3.4.1 Heathrow Airport Terminal 5

As discussed in the earlier part of this chapter, Heathrow Airport Terminal 5 is constructed using a tied arch supported high above the concourse on inclined structural columns to keep the interior space free of columns, maximizing layout flexibility and cost effectiveness. Fig. 6.13B shows the cladding and the roof of Heathrow Airport Terminal 5 is supported by a common support: the column. Fig. 6.13A shows the column to arch connections. Fig. 6.13C shows the base connection of the columns. It can be seen that the whole support system is like a 3D space truss; therefore it is different from conventional single column support. It can be seen that the 3D space truss system integrates the roof and cladding into a unified system; therefore effectively enhance the stiffness and reduced the material.

As it is demonstrated in Fig. 6.13B; it can be seen that six axial members come into each of the roof connection; therefore a special connection was designed as shown in Fig. 6.13, which can accommodate the axial force transmitted through six structural members.

6.3.4.2 Barcelona International Airport

Fig. 6.14 depicts the roof support of Barcelona International Airport. It can be seen that the wave shaped long-span roof is supported by the exterior steel columns and interior steel columns. The steel columns are anchored to the concrete blocks through the base plates. Actually, they can be considered to be a kind of composite columns. In the upper level of Fig. 6.14C pin connections are used to connect the steel columns to the roof beams; in the lower level, bolted steel base plate connections are used for the connection between the steel columns and the concrete blocks.

6.4 COMPLEX STRUCTURAL ANALYSIS METHODS

As mentioned in the introduction section of this chapter, in a broader definition, complex structures include structures which needs to be analyzed using complex structural analysis methods. In this section, the different analysis theories and methods which a complex structure may need will be briefly discussed here.

The engineering problem of the fluid-structure interaction needs to be considered when designing offshore wind turbines or offshore oil platforms under wave loading or tall buildings under wind load. The more accurate way to model their behavior would be computational fluid dynamics (CFD) using commercial software such as OpenFOAM, Star CCM+, or ANSYS.

For the engineering problem of structure under fire loading, both ABAQUS and ANSYS software can be used. Particularly, ANSYS is featured in the so-called multiphysics fire analysis (coupled thermal-stress analysis), which can accurately monitor the behavior of each individual structural member.

For the engineering problem of structure under blast loading, coupled finite element and SPH method is considered to be the most accurate method, the major analysis program are LS_DYNA, ANSYS, AutoDYN, or Abaqus.

The engineering problem of form finding for geometrical nonlinear structures, methods of force density, or dynamic relaxation methods can be used. The readers may need to develop their own coding. Some software such as SAP2000 can be used. The detailed information of some of the analysis method can be further referred to other books of mine [11,14].

6.5 BUILDING INFORMATION MODELING

British Standards Institute gives an accurate definition of BIM, the definition is: "the process of generating and managing information about a building during its entire life. BIM is a suite of technologies and processes that integrate to form the 'system' at the heart of which is a component-based 3D representation of each building element; this supersedes traditional design tools currently in use."

In other words, BIM is a 3D digital modeling method for modeling, controlling a building project. Each design team member creates and maintains its own BIM model as part of a "central model." The BIM models should also have the capacity of clash detection in a central model by different contributors.

Government construction strategy [15] has started to promote the adoption of BIM since 2011. Therefore, BIM will dominant the construction industry development in the next several decades, changing the way of the interaction between different disciplines of the construction industry. In this section, the BIM will be discussed in detail.

6.5.1 Introduction

BIM allows users to build a model using software such as Revit. The model contains all the project information, including drawings and specifications. All different stakeholders have access to the central model made in Revit, enabling project participants from all disciplines such as architects, facility managers, M & E Engineers, and structural engineers to coordinate their work. BIM integrates designs from initial design to construction and until the project finishes. Using a program such as Revit, updates of drawing can be done automatically to reflect each discipline's input, enabling integrated management of information of building components.

The use of the BIM increases the productivity of the design activities, consequently resulting in efficient building designs which, in turn, saves the material cost. It can also result in shorter construction times and a safer construction process. As systems are increasingly digitized, BIM is seen as fundamental to the development of future smarter cities.

6.5.2 Standard Methods and Procedures' Protocols

When using BIM, a standard protocol is important for the whole BIM process. The protocol should consist of document naming, data file naming, and CAD layer naming, origin, scale, orientation of structure model, etc. Standard procedures should also be defined between different disciplines. All of these are required by effective data sharing through a common data environment.

It should make sure to

- unify layer naming and file naming
- collect, manage, and disseminate data effectively in the required formats
- ensures compliance to agreed standards
- able to aid design managers in the timely delivery of the construction schedule
- for members of the supply chain not using BIM (such as small contractors) to find a way to integrate them into the process
- set up the approval process and the design and sign-off processes to improve the project management and documentation control

6.5.3 Design Liability and Legal Issue of BIM

When BIM becomes widely used, some legal issues emerged such as

- obligations to create/contribute to BIM models in agreed forms and deadlines
- liability for each team member
- how to insure the work on BIM models by an insurance company
- ownership of BIM models and data and licensing for agreed purposes
- legal status of BIM approach to collaborative working

The construction industry council issued the first edition of "Building Information Modeling (BIM) protocol" [16] in 2013. The protocol covers below the main issues: contract, intellectual property, electronic data exchange, change management, liability for the use of models.

The primary objective of the protocol is to enable the production of BIMs at defined stages of a project. It requires the employer to appoint a party to undertake an information management role such as an "information manager." Another objective is to support the adoption of effective collaborative working practices in project teams, making an explicit contractual requirement under the protocol.

It is worth noting that it is required that all project team members are required to have a BIM protocol appended to their contracts. This will ensure that all parties producing and delivering models adopt any common standards or ways of working described in the protocol and that all parties using the models have a clear right to do so.

6.6 PARAMETRIC DESIGN PROCESS

Parametric design is a design process, which utilize the latest parametric modeling techniques to set up the structural geometry of the project, to determine the structural form and structural members, to propose different structural form options, to iterate these structural design options with architect and clients, and to finalize the design options.

Parametric modeling is a newly developed modeling technique. It was originally used by architects to set up complex 3D models such as a canopy and a stadium roof with irregular curve forms. In order to quickly, easily, and precisely control the form of the structure, computer programming was used as a tool, allowing the designer to quickly and creatively generate the geometry. This technique has been gradually adopted by the structural engineers since the past few years.

6.6.1 What is Parametric Modeling

Parametric modeling is a modeling process with the ability to change the shape of model geometry as soon as the dimension value is modified. Parametric modeling is implemented through the design computer programming code such as a script to define the dimension and the shape of the model. The model can be visualized in 3D draughting programs to resemble the attributes of the real behavior of the original project. It is quite common that a parametric model uses feature-based, modeling tools to manipulate the attributes of the model.

Parametric modeling was first invented by Rhino, which is a 3D draughting software that evolved from AutoCAD. The key advantage of parametric modeling is, when setting up a 3D geometric model, the shape of model geometry can be changed as soon as the parameters such as the dimensions or curvatures are modified; therefore there is no need to redraw the model whenever it needs a change. This greatly saves time for engineers, especially in the scheme design stage. Before the advent of parametric modeling, the scheme design was not an easy task for designers, as the model is prone to be changed frequently. Therefore, changing the shape of a construction model was very difficult. Particularly, parametric modeling allows the designer to modify the entire shapes of the model, not just individual members. For example, to modify a roof structure, conventionally, the designer had to change the length, the breadth, and the height. However, with parametric modeling, the designers need to only alter one parameter; the other two parameters get adjusted automatically.

6.6.2 Available Parametric Modeling Programs

Nowadays, more and more software start to incorporate the parametric modeling functions, which enables the computer to design structure projects with a model component closely representing the real projects. Some of the leading industry software will be discussed here.

6.6.2.1 Rhino

Rhino is a 3D draughting software designed by Robert McNeel & Associates. Rhino can create, edit, analyze, document, render, animate, and translate NURBS* curves, surfaces, and solids, point clouds, and polygon meshes [17]. One of the important features of Rhino is the Grasshopper plug-in for computational design. A parametric modeling is made available through the platform of Grasshopper. Therefore, it is extremely useful at the stage of scheme design, where the design is always changing according to the requirement of the architects and clients.

6.6.2.2 SolidWorks

Introduced in 1995, it is a low-cost competitor to the other parametric modeling software products. SolidWorks was purchased in 1997 by Dassault Systems. It is primarily used in mechanical design applications and has a strong following in the plastics industry.

6.6.2.3 CATIA

Dassault Systems created CATIA in France in the late 1970s. This sophisticated software is widely used in the aeronautic, automotive, and shipbuilding industries.

6.6.2.4 AutoCAD

AutoCAD is a 3D computer-aided design (CAD) and draughting software developed by Autodesk, Inc. [18]. AutoCAD is used by various architects, project managers and structural engineers, and mechanical engineers. In the latest AutoCAD version, the parametric modeling method is developed, which can find the parametric module easily from the main menu of AutoCAD.

6.6.2.5 Bentley MicroStation V8

MicroStation is one of the major CAD software products for design and draughting, developed by Bentley Systems [19]. MicroStation is the platform for architectural and engineering. It generates 2D/3D vector graphic objects and elements. The latest version of MicroStation enables the parametric modeling functions. As discussed in Section 6.2 Water Cube was designed primarily using MicroStation when setting up the 3D wireframe model.

6.6.3 Lean Production Work Flow Using Parametric Modeling Process

Lean production is a systematic method for waste minimization within a manufacturing system without sacrificing productivity. Lean manufacturing makes obvious what adds value by reducing everything else. This management philosophy is derived from the Toyota Production System (TPS) for its focus on reduction of the original Toyota seven wastes to improve overall customer value.

Different from mass commodity production lines such as Toyota, building design is one that often needs to fit the requirements of the client and every construction site is different. It seems that the lean production is difficult to be implemented, although it has a great advantage. However, as explained in Ref. [20] "by dissecting and interrogating the whole process of creating building structures, we can still draw efficiency and marginal gains at each step of the way to the lean production of construction product by implementing the parametric design."

Currently, parametric design is applied to more projects by many structural design consultancies. In parametric design, the computer begins to take part in the design roles, following a script developed by the designer. The computer makes decisions based on the rules and parameters created by the designer to perform a task. As we have discussed, these tasks are highly repetitive and time consuming to perform manually. Therefore, the introduction of the parametric modeling into the construction industry enables lean production in this old-fashion industry.

6.6.3.1 Case Study of Scheme Design of Great Canopy, West Kowloon Cultural District in Hong Kong

A example of using parametric modeling can be demonstrated from Foster + Partner's scheme design of project of Great Canopy, west Kowloon Cultural District in Hong Kong [21]. This project is to design a great canopy, which is 1.4-km long. The geometry varies from one end to the other. Therefore, it is very critical to control the form of the curved canopy surface, to enable the structural elements such as the trusses to be correctly fitted underneath the canopy.

The canopy was first generated as a sphere; based on that a series of scripts were developed to create multiple structure and cladding options. A system of modular parametric program was developed by Foster + Partner's and was used to control the complex geometry of the design surface. Therefore, it can easily generate the minimum number of control points required to create the desired curvature characteristics.

The second stage is to generate the structural system. From the common engineering knowledge, we know that a space truss solution was the most suitable solution, as it can suit the varying geometry of the canopy. However, due to the geometrical complexity, it is very hard to fit in these trusses using the traditional computer modeling method. Through using this parametric modeling scripts developed by the staff in Foster + Partner, it is much easier to accommodate the varying depths and spans in the head and tail areas of the canopy.

The advantage of the parametric modeling is showcased well in this project. When the detailed structural investigations were taking place, the geometry of the design surface was still changing; this means a large amount of work for the structural engineers. Without using the parametric modeling technique, it would become a very difficult task. The parametric models enable the design to be developed in many areas simultaneously. The geometry can be modified freely simply through changing the variables through the script; the structural members can be freely and swiftly inserted or removed without having a large impact on other parts of the process.

6.6.3.2 Case Study of Stadium Design by AECOM

When designing the stadium and long-span roofs, the structural geometry is deeply correlated with the architectural shape; so parametric optimization techniques are vital to allow various design options to be tested and prototyped from concept design through to construction documentation stages.

As introduced by Jon Leach et al. [20], in stadium design, the design company AECOM developed their own scripts based on Rhinoceros 3D and its plugin Grasshopper4, which act as data hub with a multifunctional, multiobjective approach of the interoperability processes. It can create the roof trusses from an initial architectural curve. The truss can be further adjusted by changing the variables such as the structural depth at defined points and curved profiles of truss chords. Once the structural geometry is defined, the other components of the script are used to direct the information into further processing, most importantly structural analysis and documentation. They claimed that through using the parametric modeling, it significantly optimizes their workflow.

Structural geometry formation is the first step of the process. The structural analysis parameters such as analytical centerlines, local axes, eccentricities, releases, restraints, and loading are defined in the Grasshopper canvas. They are written as XML database files and exported into SCIA for design. Grasshopper can control the changes made in SCIA.

In the model, a common reference system for all nodes and members was used, which enable the smooth transmit of the geometry information into other draughting packages such as Autodesk or Revit.

During the design, architects use scripting to generate the baseline geometry the stadium, and use parametric tools to develop the design above the surface of the precast concrete terrace units or seating units. Structural engineers use parametric tools to develop the terrace unit and raker frame below the surface and to generate the structural geometry.

6.6.3.3 Summary of the Workflow Using Parametric Design

From the above case studies parametric design greatly changes the workflow of the design process. Fig. 6.15 is the summary of the workflow of the scheme design process using parametric modeling; it demonstrates the whole design process by utilizing the parametric modeling techniques. Based on the discussions in previous sections, with the help of parametric modeling, the whole design process has been greatly simplified, which enhances the productivity as well as the quality of the design. As discussed in the earlier sections, before the parametric modeling has been emerged, complex

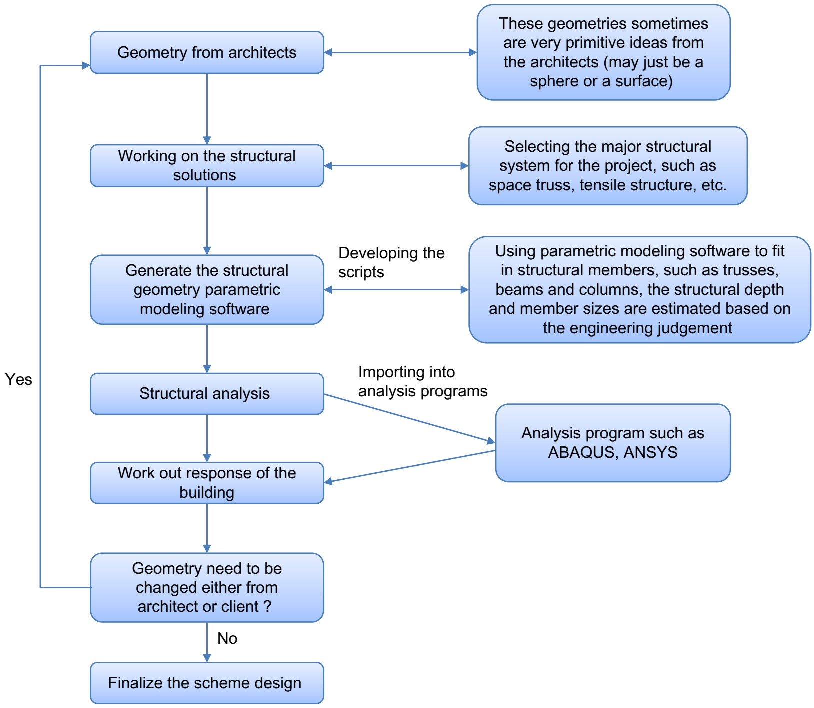


Fig. 6.15 Workflow of scheme design process using parametric modeling.

structural projects such as the Sydney Opera House were considered as a very challenging design project; engineers and architects had to make several physical full-scale mock-ups models to help them to design the structure. It is very expensive and less efficient. With the help of the parametric modeling, the scheme design becomes much easier. Apart from that, parametric modeling guarantees sufficient feasible options for both architects and engineers, as changing the model becomes much quicker, therefore enhancing the quality of the project.

Apart from the scheme design stage, it is very common that the architects continue to change their design throughout the whole project duration, parametric modeling also enables structural engineers to swiftly adapt to the changes from the architect.

6.7 API AND GUI DEVELOPMENT FOR ANALYSIS PROGRAM

To effective analyze the complex structures, sometimes a further development of API and GUI is needed. It will be introduced here.

6.7.1 API

API is a set of routines, protocols, and tools for user to design software applications. An API specifies how software components should interact. They are widely used when designing programming GUI components. Most of the numerical analysis package allows API development, such as Abaqus [22], which allows the user to develop API using Python language. This enables the users to maximize their capacity to use the relevant software packages and to meet their special requirement.

Abaqus can be tailored to meet the specific needs of the user through this API development. With the Abaqus Scripting Interface, an extended version of the Python language, the user can automate repetitive tasks or generate parametric models.

6.7.2 GUI

A GUI is a type of user interface that allows users to interact with graphical icons and visual indicators.

The Abaqus GUI Toolkit allows the modification of the GUIs which is Abaqus/CAE and Abaqus/Viewer in the Abaqus program. From the development of workflow to the automatic post-processing of analysis, the extensions are unlimited and provide the user with suitable software for his particular applications.

For another general-purpose program ANSYS [23], the GUI can be used to execute all interactive ANSYS work. The GUI provides an interface between the user and the ANSYS program, which commands drive internally. The GUI enables user to perform an analysis with little or no knowledge of the ANSYS commands. Each GUI function—a series of picks resulting in an action—ultimately produces one or more ANSYS commands that the program executes and records on the input history file. The ANSYS GUI uses OSF/Motif, a graphics programming standard that allows programmers to create interfaces for software applications. Distinguishing characteristics of Motif include the three-dimensional appearance of its menu components and its use of the Window System.

6.8 EXAMPLE OF SET UP A 3D MODEL FOR LONG-SPAN STRUCTURES—ROOF OF WEMBLEY STADIUM

As in Fig. 6.12, the roof of the Wembley Stadium was constructed by two space frames (north and south) and a retractable roof sitting in the middle. Both north and south roofs are sitting on the large ring truss; one is further supported by the huge truss arch. There are seven pairs of cables hanging from the space frame from the arch. Due to its complexity, it is difficult to generate this special shape using conventional draughting software. Therefore, Rhino 5.0 was chosen to set up the model. This is because Rhino features in some unique tools such as surface, arch and solid. It also has strong mesh tools for further meshing the model.

As shown in Fig. 6.16, the model was first set up as a number of polysurfaces. They help to determine the base control points of the space frames and the ring truss. The width of these polysurfaces is chosen to have the same value of the height of the space frame and the ring truss.

Based on these polysurfaces, the space frames and the ring truss can be further developed, as shown in Fig. 6.17. In the meantime, an arch solid is set up in Fig. 6.17 as well. This solid is the basis for the further development of the truss arch.

Based on the model in Fig. 6.17, the arch solid is dissected into curves and further meshed, therefore, enabling us to set up the structural members of the arch truss (as shown in Fig. 6.18).

When the truss arch is determined, the hanging cables can be drawn accordingly, as shown in Fig. 6.19, and further finalization of the model can be made.

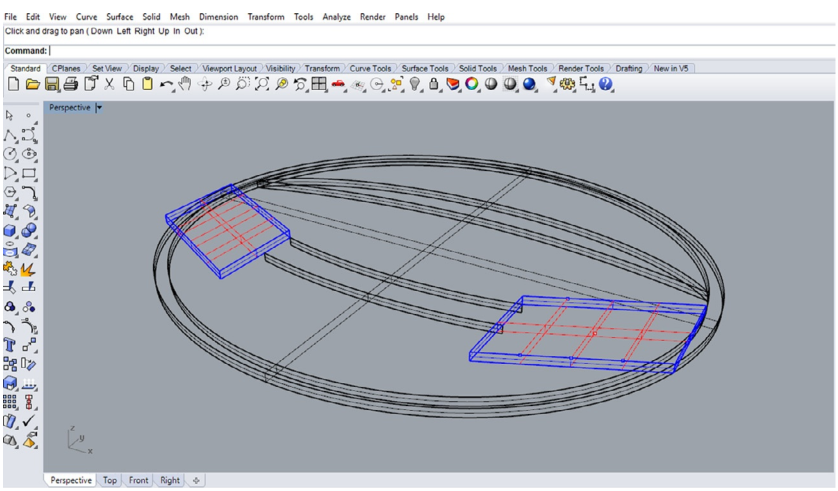


Fig. 6.16 Setting up poly-surface in Rhino. *(Screenshot reprinted with permission of Robert McNeel & Associates.)*

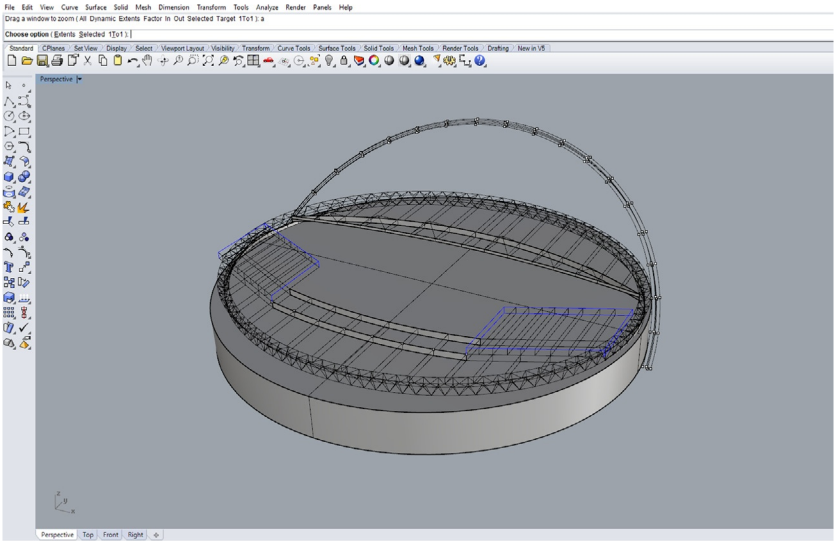


Fig. 6.17 Setting up the space frame and ring truss and a solid arch. *(Screenshot reprinted with permission of Robert McNeel & Associates.)*

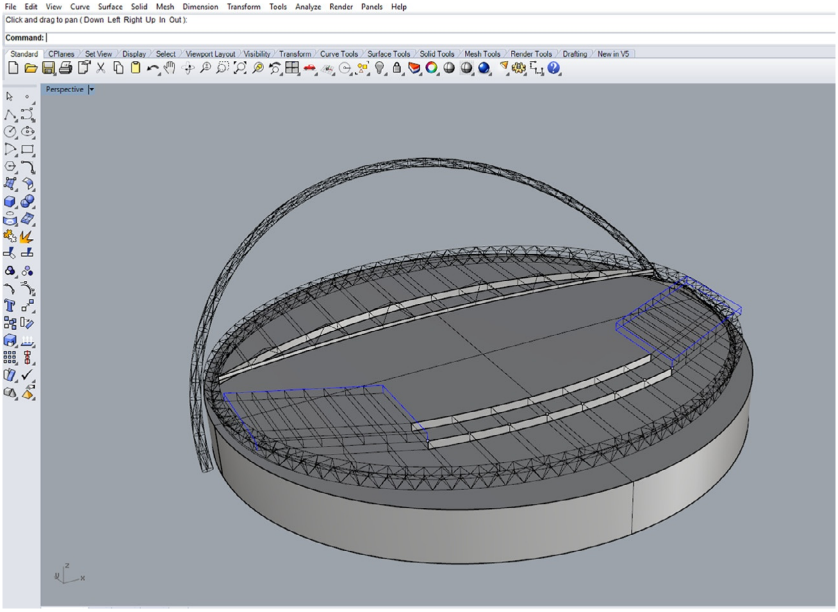


Fig. 6.18 Setting up the truss arch. *(Screenshot reprinted with permission of Robert McNeel & Associates.)*

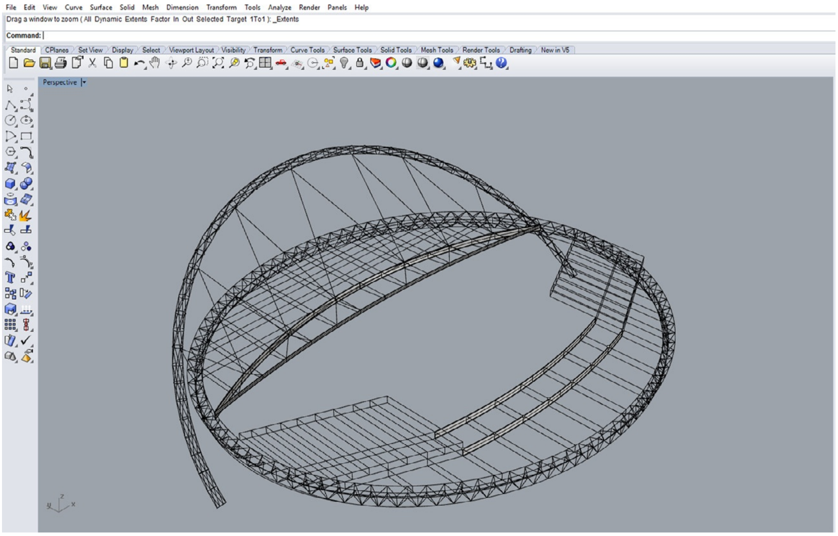


Fig. 6.19 Setting up the 3D wireframe using Rhino. *(Screenshot reprinted with permission of Robert McNeel & Associates.)*

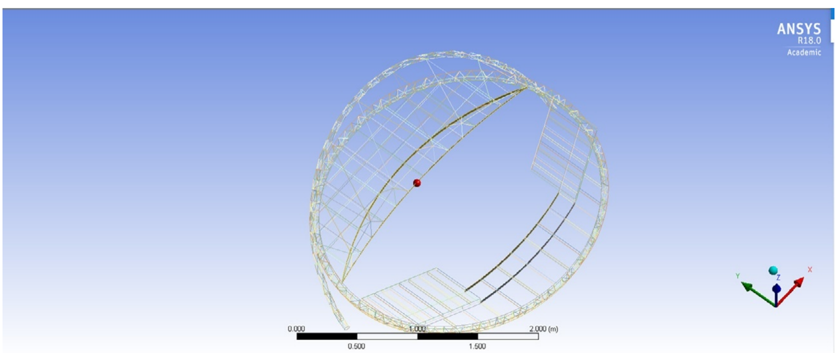


Fig. 6.20 Importing the geometry into ANSYS. *(Images used courtesy of ANSYS, Inc.)*

When creating the final 3D wireframe model shown in Fig. 6.19, it is important to make sure to store the different type of structural members into different layers, so it is easy to manage them, if any changes are needed.

As shown in Fig. 6.20, after the structural geometry is generated in Rhino 3D, the geometry can be further imported into ANSYS, Abaqus, or SAP2000 for further structural analysis.

6.9 CASE STUDY OF LONG-SPAN ROOF IN KING'S CROSS ST PANCRAS STATION USING ANSYS

As discussed earlier in this chapter, the roof of the King's Cross western concourse features a complex single-layer lattice shell; although it is a complex structure due to its symmetric shape, the structure geometry in terms of wireframe is generated using AutoCAD, as shown in Fig. 6.21.

After the 3D wireframe was set up in AutoCAD, the model was imported into finite element analysis software ANSYS, where the section of the structural member can be defined (Fig. 6.22). After that further structural analysis can be made.

6.10 CASE STUDY OF MULTIPHYSICS FIRE ANALYSIS OF BUILDING IN CARDINGTON FIRE TEST USING ANSYS

As discussed in Chapter 2, ANSYS features in the so-called multiphysics fire analysis (coupled thermal–stress analysis). In this method, the

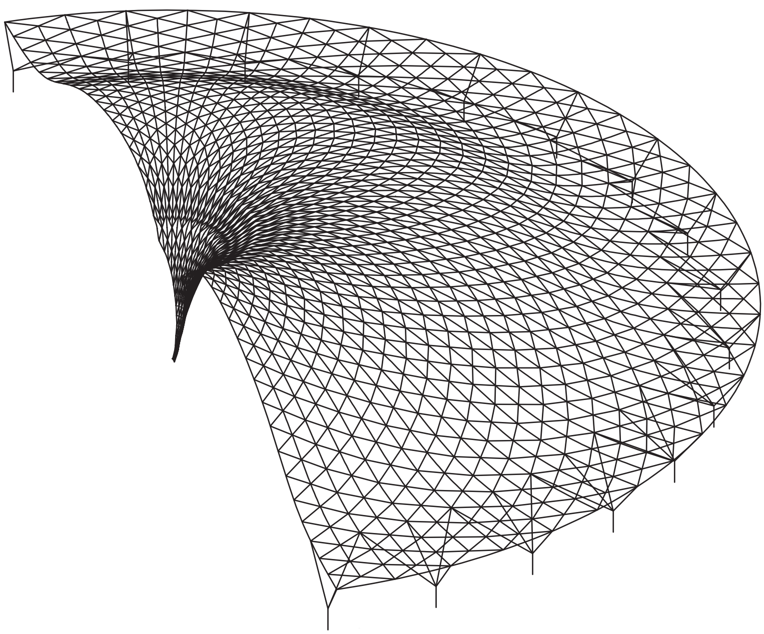


Fig. 6.21 3D wire frame model of the roof of King's Cross western concourse in AutoCAD.

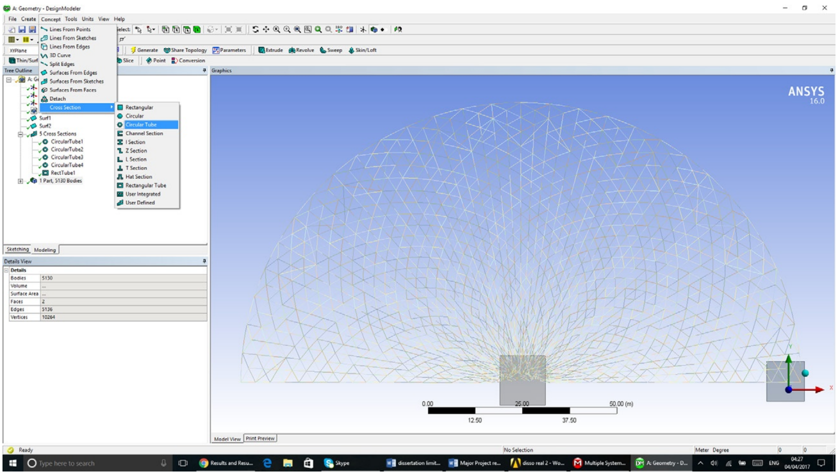


Fig. 6.22 Geometry imported into ANSYS defining the member section. *(Images used courtesy of ANSYS, Inc.)*

thermal analysis (heat transferring analysis) such as convection, radiation, and conduction is performed; temperature increase for each structural member will be worked out. In the meantime, the stress analysis can be performed.

To demonstrate this analysis process, in this section, a model of a multistorey building was set up in ANSYS, which replicates the multistorey building used in the Cardington fire test. A multiphysics fire analysis is performed in ANSYS based on the same fire scenarios of Cardington fire test. The modeling technique will be demonstrated here.

First, a wireframe of the multistorey building was made in AutoCAD (Fig. 6.23); the reason to choose AutoCAD rather than Rhino is due to the simplicity of the building. As shown in Fig. 6.23, columns, beams, and bracings are drawn in AutoCAD, then transferred to ANSYS for analysis.

After the wireframe has been transferred to ANSYS, the member size of the columns, beams, and bracings can be defined in ANSYS. Similarly, the slabs are defined using a shell element with the same thickness of the real slabs (Fig. 6.24). It can also be seen that from the left-hand column of the ANSYS (see Fig. 6.24), the thermal analysis module (including convection and radiation), and the static stress analysis module are set up as well.

When all the model parameters have been defined, the multiphysics fire analysis (coupled thermal–stress analysis) can be performed in ANSYS. Part of the analysis result is shown in Fig. 6.25.

Fig. 6.23 3D wireframe mode of the Cardington test building.

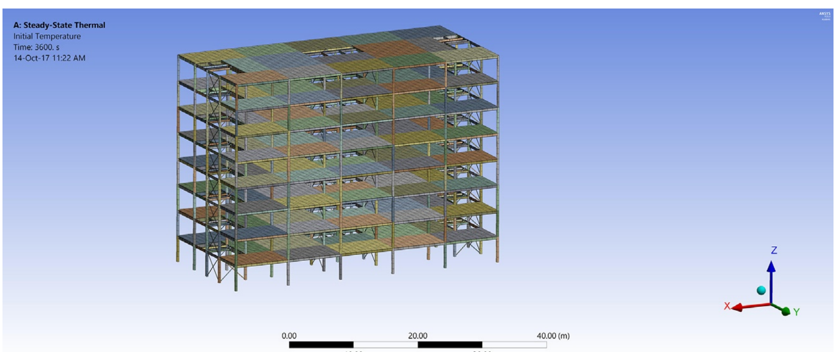


Fig. 6.24 Setting up Cardington test building in ANSYS. *(Screenshot reprinted with permission of ANSYS, Inc.)*

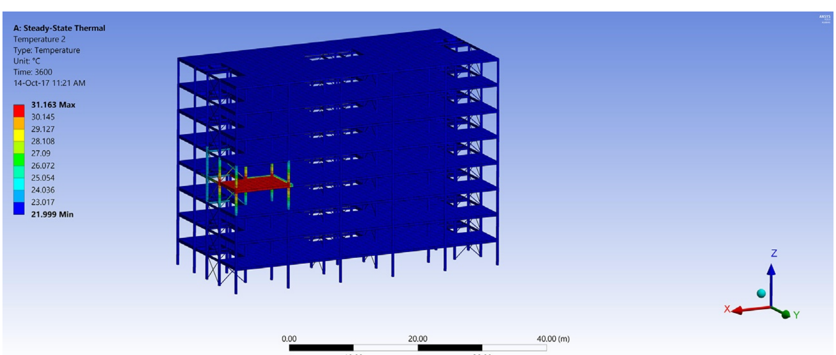


Fig. 6.25 Temperature distribution of the building. *(Images used courtesy of ANSYS, Inc.)*

6.11 CONCLUSION

In this chapter, different types of the complex structure are discussed, the design considerations and modeling example of complex structures are demonstrated. It can be seen that the design and analysis of complex structures are greatly involved with the advanced modeling techniques such as using parametric modeling and advanced finite element software. Detailed modeling technique can refer to another book of the author [11].

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CHAPTER SEVEN

Design and Analysis of Tensile Structures and Tensegrity Structures

Abstract

In this chapter, the design and analysis of two special types of structures: tensile and tensegrity structures are explained. It starts with the introduction of tensile structure, including different membrane materials, different types of support. This is followed by discussion of major loadings need to be considered for tensile structural. Then, the general design considerations are illustrated which covers some of the important design issues such as wrinkling of the membrane. Detailed analysis theories for form finding and static analysis are illustrated. They are nonlinear finite element method, force density method, and dynamic relaxation method. The way to model membrane structure in ANSYS and Abaqus are also described. In the final two sections of this chapter it covers the topic of tensegrity structures. Design and analysis methods are examined. The analysis methods for tensegrity structure are demonstrated using Georgia Dome as a prototype, which is modeled using design orientated program SAP2000.

Keywords: Tensile structures, Tensegrity structures, Form finding, Geometrical nonlinearity, Nonlinear finite element method, Force density method, Dynamic relaxation method

7.1 INTRODUCTION TO TENSILE STRUCTURES

Tensile structure is one of the unique complex structures. Its structural members can only carry tension but there can be no compression or bending. It normally consists of exterior fabric material with its framework such as cables. The fabric and the cables are maintained in tension in all directions to provide stability. Membranes are one of the common fabric material used in the tensile structure.

A membrane is tensioned between rigid structural elements such as steel frames or flexible structural elements cables. The membrane itself can resist external loading only through increasing tension in the membrane, which results compression and bending in supporting elements. To achieve adequate stiffness, its surface curvatures must be relatively high. Therefore,

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doubly curved forms are conventional in design as it provides greatest stability. As this type of material can only carry tension but no compression or bending, its surface shape must be generated through form finding to find the equilibrium position of a structure for a given stress state.

7.1.1 Type of Fabric Material

There are several different kinds of membrane materials such as polytetrafluorethylene (PTFE)-coated fiberglass, polyvinylchloride (PVC)-coated polyester fabric, silicon-coated glass, etc. Ethylene-tetra-fluorethylene (ETFE) film is another type of membrane material; it can be used as either single layer or cushion form (which is inflammable). The two famous stadiums for 2008 Beijing Olympic Birds Nest and Water Cube have used ETFE membrane. In Water Cube, the inflated cushions provide good insulation and beautiful aesthetic effect. Through the volume control of inflated air, the transparency of the membrane can be changed, therefore the natural light can be effectively used, and in addition, the reflection of sound can also be controlled.

The material of membrane structure has unique feature in terms of insulation, light transmission, and fire protection. Therefore, the selection of the membrane material is important in the design of this type of structure. Among them, PTFE, PVC, and ETFE are widely used with a wide range of tests performed; therefore, they are the preferable material for an engineer to choose. For ETFE material, the stains can be simply washed off by the rain. Although the ETFE is susceptible to punctures, these can be easily fixed with ETFE tape.

7.1.2 Type of Support

The maximum span of the fabric itself can achieve 30 m. To cover longer span, the fabric should be supported by cables (Figs. 7.1 and 7.2) or steel frames (Fig. 7.5). These cables are further supported to the main supports (Fig. 7.1A) or directly anchored to the foundation (Fig. 7.1B). For the main support, there are different types, such as mast support, compression rings, beams, arch support, or hanging by cables. They are going to be explained here.

7.1.2.1 Supported by Hanging Cables

As shown in Fig. 7.1, in O_2 arena, the membrane is directly supported by cables, these cables are further supported by hanging cables which are supported back to masts (Fig. 7.1A), which will result huge compression force to be resisted by these masks. Therefore, the masks are normally supported by heavy concrete block (Fig. 7.2B), which shows another

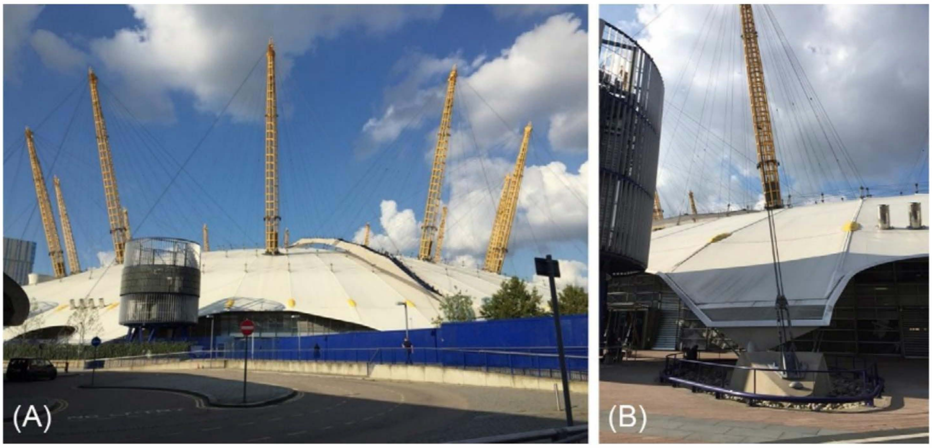

Fig. 7.1 (A) Tensile membrane structure, the O_2 dome, London, UK and (B) the anchorage of the cable. *(Photo taken by the author.)*

Fig. 7.2 A tensile membrane canopy of a toll station at Highway of Beijing International Airport, Beijing, China: (A) overview of the entire structure, (B) details of mast connection to concrete foundation support, and (C) details of cable support. *(Photo taken by the author.)*

membrane structure with a similar support system. In this shape of structures, the membranes are anchored back to the foundations through cables to maintain its shape (Figs. 7.1B and 7.2C).

This kind of support is quite common. Fig. 7.2 is another example of this type of support. It shows a tensile membrane canopy of a toll station on a highway to Beijing International Airport.

7.1.2.2 Supported Directly by Mast

Most tensile structures are supported by some forms of compression or bending elements, such as masts (Fig. 7.3), compression rings, or beams. Fig. 7.3 is another example of membrane structure. It is at the Entertainment Center in Astana, Kazakhstan designed by Architect Foster + partners. The whole membrane structure is supported on a large compression masts which consists of three truss columns to resist the huge compression force come from the supported membrane structures (Fig. 7.4).

7.1.2.3 Supported by Steel Frames

As shown in Figs. 7.5 and 7.6 steel frames are used to support the large inflammable cushion membrane panels.

7.1.2.4 Supported by Arches

Arch support is one of the typical supports for membrane structures (Fig. 7.7). This is because by using the arch support, saddle curvature can

Fig. 7.3 Membrane structure Khan Shatyr Entertainment Center, Astana, Kazakhstan. *(Photo taken by the author.)*

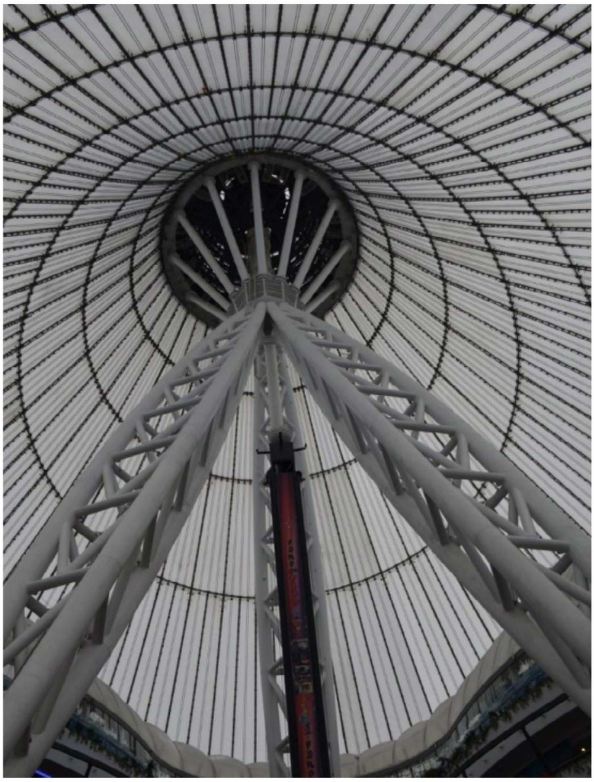

Fig. 7.4 Interior of Entertainment Center in Astana, Kazakhstan. *(Photo taken by the author.)*

Fig. 7.5 Membrane roof in Manchester Victoria Train Station, Manchester, UK. *(Photo taken by the author.)*

Fig. 7.6 Membrane structure in Eton Project, Cornwall, UK. *(Photo taken by the author.)*

Fig. 7.7 A bridge with arch supported membrane canopy at Greenwich, London, UK. *(Photo taken by the author.)*

be formed in between the support of arch, which allows adequate resistance to buckling of the membrane.

7.1.3 Introduction of Different Projects Using Membrane Material

In order to give the readers a clear explanation of membrane structure, in this section, different projects across the world are using the membrane material that will be discussed. Different membrane material and supporting structures are chosen for clear demonstration of membrane structures.

7.1.3.1 Khan Shatyr Entertainment Center in Astana Kazakhstan

As shown in Figs. 7.3 and 7.4, Khan Shatyr is a giant transparent membrane structure in Astana, Kazakhstan. This project was completed on December 9, 2006. The Khan Shatyr Entertainment Center was designed by an UK architect Foster + Partners and UK structural engineer Buro Happold.

The height of the structure is 150 m, it has a 200-m elliptical base covering 140,000 m^2. As shown in Fig. 7.4, the area is larger than 10 football stadiums, is an urban-scale internal park, shopping, and entertainment venue with squares and cobbled streets [1].

The fabric roof is constructed from ETFE-cushions. It can also be seen in Fig. 7.4 that the fabric roof is suspended on a network of cables, they are further supported by a central mask. The transparent membrane allows sunlight through, working with air heating and cooling systems, it can maintain an internal temperature between 15 and 30°C in the main space and 19–24°C in the retail units.

7.1.3.2 Eden Project (Cushion Membrane)

As shown in Fig. 7.6, Eden Project is located in Cornwall, England. It is designed by architects Nicholas Grimshaw & Partners. The project was built by McAlpine as a joint venture. It is featured in its two biome—the Rainforest Biome and the Mediterranean Biome, each of them consist of several domes which are joined in the middle by the link building. The cladding panels are made from the thermoplastic ETFE large cushion panels, up to 9 m across, were made by several layers of thin ETFE film, which are sealed around their perimeter and inflated. These panels are supported on steel space structures using tubular steel, forming hexagonal shape frame. These cushions provide good thermal insulation to the structure. It can be seen in Fig. 7.6 that the structure is completely self-supporting, with no internal supports. Owing to its lightweight, wind uplift has been considered, therefore, the domes are tied into the foundations with ground anchors.

7.1.3.3 O_2 Arena Dome

As shown in Fig. 7.1, the O_2 arena hosted the 2012 Summer Olympics Games. It is an indoor arena covered by the tensile membrane dome using PTFE-coated glass fiber supported by the hanging cables. Its structure was designed by BuroHappold. It is supported by high-strength cables spanning from outer edge to the center, which are supported by 12 100-m-high yellow steel support towers. As shown in Fig. 7.1, the cables are anchored back to concrete block, to support the fabric.

According to Ref. [2], the arena provides 100,000 m^2 of enclosed space. The structure is 365 m in diameter, with a circumference of 1 km and a maximum height of 50 m.

It was the largest structure in the world when it was built. It can be seen that for such a large span structure, membrane material would be the first option due to its super lightweight.

7.1.3.4 Jeju World Cup Stadium

As shown in Fig. 7.8, the Jeju World Cup Stadium located on Jeju Island, with a 35,657-person capacity. It was built in the city of Seogwipo on Jeju Island in South Korea. It was designed resembling the shape of the mouth of a volcano and its roof in the shape of nets of traditional fishing boats in Jeju [3]. The stadium was built 14-m below ground level to endure strong winds. It used the FGT-800 membrane material. FGT series are the glass fiber B yarn clothes impregnated with fluoroplastic PTFE and then sintered. The B yarn has the specific strength even excellent than steel. It has the resistance of high temperature up to 700–800°C and can resist ultraviolet rays. This kind of membrane also has the anti-adhesion and water-repellent features. Therefore, it is a good option for stadium roof.

As we can see that the membrane structures are also supported on the arch which is supported by the cable hanging from six large masks, the arch itself is also anchored back to the mass concrete blocks at the two ends of the structure.

Fig. 7.8 Jeju World Cup Stadium, Seogwipo, Jeju, South Korea. *(Photo taken by the author.)*

7.1.3.5 Birds Nest Stadium, Beijing

As shown in Fig. 7.9, the Beijing National Stadium was completed in early 2008. It has a gross floor area of 254,600 m^2 with seating capacity for 91,000 including 11,000 temporary seats. CITIC Internationals Contracting Inc. was the major construction contractor. The structural engineering, mechanical, and electrical engineering, fire safety engineering, and acoustic designs were made by Ove Arup & Partners.

The National Stadium's main structure is an enormous saddle-shaped elliptic steel structure weighing 42,000 t. The stadium extends 333 m from north to south and 294 m from east to west, with a height of 69.2 m [4]. It uses the ETFE cushion membrane as the roof structure. As shown in Fig. 7.10, it provides a half transparent but water-resistant roof, allow sunshine to penetrate through, but reduce the intensity of the ultraviolet from the sun.

7.1.4 Pros and Cons of Using Tensile Structures

From the explanation of Section 7.1.3, it can be seen that there are plenty of benefits in using tensile structures. Most membrane structures have high sun reflectivity and low absorption of sunlight. They also have benefits such as flexible design aesthetics, good translucency, excellent durability and low maintenance, and cost effective. In addition, daylight is maximized in building interiors, thus reducing the costs for electric lighting.

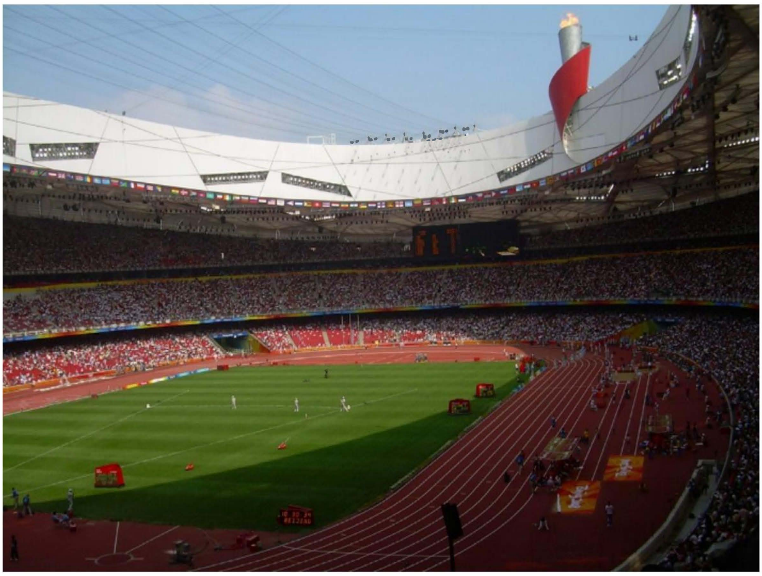

Fig. 7.9 Interior of Birds Nest Stadium in Beijing, China. *(Photo taken by the author.)*

Fig. 7.10 Membrane roof of Birds Nest Stadium in Beijing, China. *(Photo taken by the author.)*

In term of structural design, the lightweight membrane is a cost-effective solution that requires less structural steel to support the roof compared to conventional building materials, therefore, it is ideal to use in long-spans structures. Owing to its flexible feature, it can also be used as a deployable structure.

However, there are also some disadvantages of using membrane structures. Owing to the lightweight of the structure, wind becomes critical for loading, and several accidents of membrane structure can be damaged due to wind that have been reported.

7.1.5 Design Code for Membrane Structures

Currently, there are not many design guidance for membrane structures cross the world. In China, CECS 158:2004 [5] "Technical specification for membrane structures" is one of the detailed design code for membrane structure. It covers the basic design specifications, structural analysis methods, fabrication, construction, procedures for approval, and structure maintenance. It is one of the most detailed design codes for membrane structures.

The European Standard, EN 13782:2005 [6] "Temporary structures Tents-Safety," covers the safety requirements for tents during design, calculation, manufacture, installation, maintenance, operation, and examination for temporary installed tents of more than 50-m^2 ground areas. The contents cover fundamental terms and definitions, the general requirements for design, principles of numerical analysis, the design actions, the verification of stability, and equilibrium, the ground anchorages, the other structural components, the special design and manufacture criteria, the manufacture and supply, the examination, the competence, procedures for approval, examination and tests, and the aerodynamic coefficients for round-shape tents.

There is another European design guidance, EN 15619:2008 + A1:2010 "Rubber or plastic coated fabrics—Safety of temporary structures (tents)—Specification for coated fabrics intended for tents and related structures" [7]. It covers the level of performance for different fabrics.

Membrane structure has been widely used in Japan for long time, especially inflammable membrane structures. MSAJ/M-03:2003 "Test methods for membrane materials (coated fabrics)—qualities and performances [8]" describes the testing procedures for membrane materials.

In the United States, the ASCE Standard 55-10 "Tensile Membrane Structures" [9] covers the design approach and the prescriptions for an appropriate fabrication and erection of the structure. It includes the properties of membrane materials, the connections details, load combinations and strength reduction factors. It also suggests recommendations about the use of wind tunnel analysis in order to investigate the wind pressures (where the shape of the membrane does not fall within the limits of the prescriptive load requirements), the flutter at free edges and the resonance of the entire structures.

ASCE Standard 17-96 "Air Supported Structures" [10] and ASCE Standard 19-96 "Structural applications of Steel Cables for Buildings" [11] are the other two design guidance related to the design of membrane structures.

7.2 GENERAL CONSIDERATIONS IN STRUCTURAL DESIGN OF TENSILE MEMBRANE STRUCTURE

Owing to its unique feature, the design and analysis of membrane structure is different from conventional structures. The support, geometry and prestressing of the membrane are three key design considerations. In addition, patterning is another important design process. Structural fabrics are manufactured as flat panels, for this reason, tensile fabric structures cannot be formed without incurring stresses in the surface. The conventional structural fabrics are manufactured with a typical width of 2–3 m, and a maximum width of 5 m [12]. If it is a large structure, multiple panels are needed. The shape of these panels affects the final form and stress distribution of the membrane. Therefore, a specialist design process, patterning, is needed for this type of structure.

The flow chart of the whole design process is shown in Fig. 7.11, which will be introduced in the following sections.

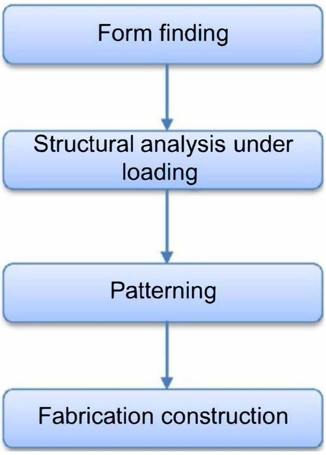

Fig. 7.11 Flow chart of design analysis process of tensile membrane structure.

7.2.1 Design Loading

In this design, the loadings below need to be considered, dead load, snow load, wind load, earthquake load, and thermal loading. The dead loads are mainly from the self-weight of the membrane, as membrane is a super-light material, so it is quite small, around 1 kg/m^2. The membrane roof is not accessible in reality; therefore, the live load can be ignored. Owing to its lightweight, the inertial force is quite small during the earthquake; therefore, the earthquake loading is not a major consideration.

Wind loading is major load which needs to be considered in the design; this is due to its smaller stiffness compared to other kind of structure. Wind is one of the major causes for the failure of membrane structures. The wind load can be resistant through the wrinkling of the membrane. There are quite a few incidents of damage to membrane structure under wind loading that has been reported. Therefore, wind is a dominant load in the design consideration discussed in detail further.

7.2.2 Wind Loading

As mentioned, because membrane structures are lightweight and flexible, wind resistance is critical to their structural design. Wind is a dominant load in the design.

In terms of structural analysis of membrane structure under wind loadings, there are two approaches available to use: quasi-static and dynamic approach.

Quasi-static approach is to assume that the dynamic wind effects can be ignored in designing a membrane structure, so the structure only undergoes deformation, therefore wind load can be considered as static load case. In this case, the wind load can be derived directly using the normal wind code such as EN1991-1-4 [13]. The mean wind load at a certain point on the surface of a structure is computed as the product of a mean dynamic pressure at a specific point, a dimensionless pressure coefficient, and further coefficients.

Dynamic approach membrane structures have a considerably low natural frequency (typically 0.5–1.5 Hz). This damping results from the crimp interchange of the yarns. The high flexibility of membranes leads to considerably large deformations for external loadings, which makes membrane structures susceptible to wind-induced aeroelastic effects. Therefore, to accurately analyze the structure under wind load, the dynamic effect of the wind on the membrane structure needs to be considered. Wind tunnel experiments are necessary to determine the wind load on membrane structures, due to their individual geometry. In addition, numerical approach computational fluid dynamics for the analysis of membrane structures is also necessary. Examples are the works of Saberi-Haghighi [14], Hübner [15], and Glück and Halfmann [16,17]. They analyze membrane-wind interaction through fully coupled fluid-structure interaction simulation, however, with simplified boundary conditions for the fluid analysis. In Ref. [18], Alexander Markus Kupzok also simulated membrane-wind interaction through surface-coupled fluid-structure interaction method.

In real design practice, for large-scale complex membrane structures, the geometry coefficient and wind pressure coefficient under different directions of wind loading is determined using wind tunnel test. However, for simple membrane structure, different design codes such as "Temporary structure Tents Safety" [6], gives the aerodynamic coefficients for round-shape tents for wind design.

7.2.3 Wrinkling

Owing to its low compressive strength, wrinkles can be formed in membrane material in certain loading conditions. The wrinkle often gives a negative image; therefore, to understand what would result in wrinkle occurrence is important for membrane structural design.

Wrinkling theory was the iterative materials properties (IMP) model firstly developed by Miller and Hedgepeth [19,20]. They presume that if a simulation of a membrane element is deemed to be wrinkled, the geometric strain in the direction perpendicular to the direction of the wrinkles, due to out-of-plane deformation of the material, can be modeled by introducing a variable efficacious Poisson's ratio for the element. Hence, the standard "taut" modulus matrix, predicated on Hooke's law for plane stress and given by.

$$M_t = \frac{E}{1-\nu^2}\begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \frac{1}{2}(1-\nu) \end{bmatrix} \tag{7.1}$$

was replaced by "wrinkled" modulus matrix.

$$M_w = \frac{E}{4}\begin{bmatrix} 2(1+A) & 0 & B \\ 0 & 2(1-A) & B \\ B & B & 1 \end{bmatrix} \tag{7.2}$$

Based on above theory, Guo [21] gives two methods to make wrinkle analysis:

1. Tensile field theory method

when the principle strain. $\varepsilon_1 > 0, \varepsilon_2 \leq 0$ wrinkle occurs

2. Finite element static buckling analysis method of membrane

Using FE analysis package such as LS-DYNA, a buckling analysis of membrane can be performed.

The main objective of wrinkle analysis is to find the regions affected by wrinkles and the direction of wrinkles.

7.2.4 The Initial Geometrical Equilibrium—Form Finding

Before analyzing under the loading condition, the membrane and its reinforcing members (cables) should be prestressed to provide adequate stiffness. Therefore, a certain shape of the structure can be formed. Without prestress, the structure does not have adequate stiffness to resist the external loading. For this reason, initial geometrical equilibrium of the structure position need to be found before the load can be applied to the structure, this process is called form finding. Through the form finding, the adequate stiffness can be achieved. Therefore, no wrinkling will be formed under the normal loading.

According to Ref. [22], there are certain assumption made when perform form finding:

1. small strain behavior in a model undergoing large deformations, and
2. the form-finding system significantly more flexible than the form-found shape while still maintaining the desirable relative element stiffness.

There are several form-finding theories discussed in Section 7.3.

7.2.5 Patterning of Tensile Fabric Structures

Patterning is a unique design stage for tensile fabric structures. In the patterning design stage, the three-dimensional surface of the fabric members (membrane), found by means of form finding is flattened obtaining a two-dimensional cutting pattern for the fabric. This process is generally based on mathematical studies, which is included to determine the arrangement of planar fabric panels when the panels are assembled, the desired 3D form to be achieved, ensuring that the stress distribution is as close as possible to that intended during form finding. Fabric usage should also be minimized, so that the membrane can be fabricated.

Based on Ref. [12], the patterning process can be divided into the following steps:

Step 1: Seam definition concerns the division of the form-found membrane surface into panels. The divisions are defined by seams which generally take the form of sewn and/or welded overlapping panels of fabric.

Step 2: Flattening, developing a portion of the 3D form-found membrane surface into the 2D plane.

Step 3: Stress reduction, using iterative methods to the flattened panel geometry to reduce stresses.

Step 4: Compensation, concerns the shrinking of the pattern to account for tensioning of the membrane during construction.

Step 5: Pattern assembly, can be included in patterning schemes to calculate the final geometry and stresses in the constructed membrane.

There are two numerical analysis methods used for patterning: dynamic relaxation method (DRM) and force density method (FDM), which is explained in Section 7.3.

7.3 STRUCTURAL ANALYSIS OF TENSILE MEMBRANE STRUCTURE

Analysis of cable-membrane structure is a geometrical nonlinear problem due to its large deformation when resisting the external loading. Therefore, this type of structure is very sensitive to the geometry shape.

In the analysis, the structure can be idealized into the elements and nodes. The membrane can be discretized into a system of nodes and 2D finite elements, traditionally, triangle elements are chosen for the simulation of membrane, as shown in Fig. 7.12.

As shown in Fig. 7.13, cables can be simulated using 1D finite elements such as cable elements or tension-only elements, both consist of two nodes with six degrees of freedom.

The equilibrium position of each node under loading condition can be searched iteratively using different numerical tools such as DRM and FDM discussed here.

7.3.1 Nonlinear Finite Element Method

For each node of the cable-membrane structure, the equilibrium equation of a node under external loading can be seen in Ref. [23] and Eq. (7.3).

$$\left([K_E]^t + [K_G]^t\right)\{\Delta U\}^t = \{P_0\}^t + \{\Delta P\}^t + \{R\}^t \tag{7.3}$$

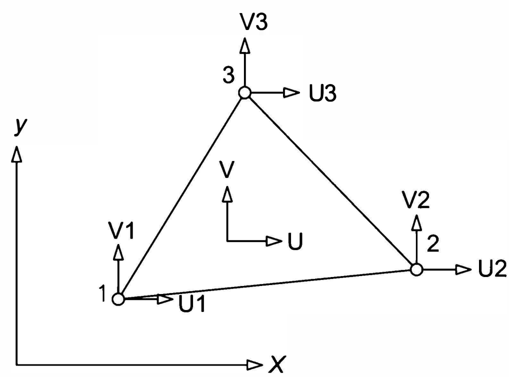

Fig. 7.12 2-D triangular elements.

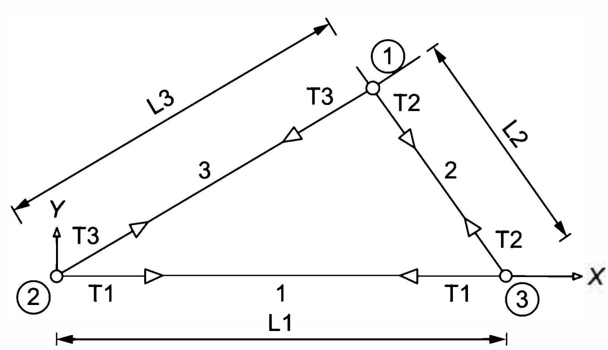

Fig. 7.13 Triangular membrane element and cable elements.

where

$[K_E]$ is the elastic stiffness matrix,

$[K_G]$ is the geometrical stiffness matrix,

$\{\Delta U\}^t$ is the increment of displacement of the nodes,

$\{P_0\}^t$ is the initial nodal force,

$\{P\}^{\mathrm{t}}$ is the external loading force on the node, and

$\{R\}^0$ is the residual force.

At the form-finding stage, there is no external loading, therefore,

$$\left([K_E]^t + [K_G]^t\right)\{\Delta U\}^t = \{R\}^t \tag{7.4}$$

Eq. (7.4) is the basic governing equations for form finding of tensile cable membrane structure using the nonlinear finite element method.

7.3.2 Force Density Method in Form Finding

The force density method (FDM) was first introduced by Schek [24]. In this method, the force/length ratios or force densities are defined for each branch of the cable net and membrane structure. The associated node coordinates of the structure are obtained by solving the topological branch-node matrix [24]. This method makes a simple linear "analytical form finding" possible. This method is used in tensile membrane structures to find the equilibrium shape of a structure consisting of a network of cables and membrane.

7.3.2.1 Cable Net

For cable net, the form-finding equations can be analytically determined by using the force density ratio for each cable element [25]. The basic principle is introduced here.

For any joint i, its equilibrium in the x direction can be represented as.

$$\sum_{k=1}^{n_i} \frac{X^i - X^k}{L_{ik}} t_{ik} = F^i \tag{7.5}$$

where

n_i is the number of cables connected to joint i,

t_{ik} is tensile force in the cable,

L_{ik} is the length of cable, and

$X^i - X^k$ is the coordinate difference of joint i and joint k for cable ik in the X direction.

If $q = t/L$, where t and L are the cable force and length of a cable element, respectively, therefore, for joint i.

$$\sum_{k=1}^{n_i} \left(X_j^i - X_j^k\right) q_{ik} = F_j^i \tag{7.6}$$

If there are n nodes and m cables, the equilibrium equation in the X direction for each node can be deduced:

$$[C]^T [U][L]^{-1}\{t\} = \{F_x\} \tag{7.7}$$

where

$\{F_j\}$ is the load vector,

$[U]$ is the coordinate difference matrix,

$[C]$ is the topology matrix of this network, and $[U]$ and $[C]$ will be explained in the following sections.

If we call $Cs = [C\ C_f]$ branch-node matrix [24], which gives the positions of nodes (vertices) which connect the cables (edges) of a cable network. Cs is constructed following the below rules:

$C(i,j) = -1$ (when j is the smaller node number of element i).

$C(i,j) = 1$(when j is the lager node number of element i).

$C(i,j) = 0$ (for other situation).

where i and j are the joint numbers, from 1 to n_t and

$$n_t = n + n_f$$

where

n is the number of free nodes, these nodes are not constrained, and

n_f is the number of fixed nodes, these nodes are supported.

Therefore, $[C_s]$ can be further divided into $[C]$ and $[C_f]$.

So the coordinate's difference of all the connected joints in the cable net can be obtained using the branch-node matrix C and C_f.

$$\{u\} = [C]\{x\} + \left[C_f\right]\left\{x_f\right\}$$
$$\{v\} = [C]\{y\} + \left[C_f\right]\left\{y_f\right\}$$
$$\{w\} = [C]\{z\} + \left[C_f\right]\left\{z_f\right\}$$

where

x, y, z are the vector for the coordinate of free joints, and

x_f, y_f, z_f are the vector for the coordinate of fixed joints.

In the X direction.

$$\{u\} = [C]\{x\} + \left[C_f\right]\left\{x_f\right\}$$

If the length of the element is $[l]$ and the internal force of the element is $\{t\}$, external loading is $\{F_x\}$, and convert vector $\{u\}$ into $m \times m$ matrix $[U]$, $[U]$ is the coordinate difference matrix:

$$[C]^T[U][L]^{-1}\{t\} = \{F_x\} \tag{7.8}$$

The force density.

$$\{q\} = [L]^{-1}\{t\}$$

$[L]$ can be worked out through $\{x\}$.

Therefore,

$$[C]^T[U]\{q\} = \{F_x\} \tag{7.9}$$

Because.

$$[U]\{q\} = [Q]\{u\}$$

where $[Q]$ is the diagonal matrix for $\{q\}$.

Therefore,

$$[C]^T[Q]\{u\} = \{F_x\} \tag{7.10}$$

So

$$[C]^T[Q][C]\{x\} + [C]^T[Q]\left[C_f\right]\left\{x_f\right\} = \{F_x\} \tag{7.11}$$

If we let $[D] = [C]^T[Q][C]$ and $[D] = [C]^T[Q][C_f]$, $[D]$ is the generalized Gaussian transformation of $[C]$.

So

$$[D]\{x\} = \{F_x\} - \left[D_f\right]\left\{x_f\right\} \tag{7.12}$$

Eq. (7.12) can be used to determine the node coordinate for all the free nodes by using following equations:

$$\{x\} = [D]^{-1}\left\{\{F_x\} - \left[D_f\right]\left\{x_f\right\}\right\} \tag{7.13}$$

Similarly, we can obtain.

$$[D]\{y\} = \{F_z\} - \left[D_f\right]\left\{y_f\right\} \tag{7.14}$$

$$[D]\{z\} = \{F_z\} - \left[D_f\right]\left\{z_f\right\} \tag{7.15}$$

Therefore.

The coordinate for the nodes in the y direction can be:

$$\{y\} = [D]^{-1}\{\{F_y\} - [D_f]\{y_f\}\} \tag{7.16}$$

The coordinate for the nodes in the z direction can be:

$$\{z\} = [D]^{-1}\{\{F_y\} - [D_f]\{z_f\}\} \tag{7.17}$$

Therefore, based on above procedures, the equilibrium shape of a cable net can be determined.

7.3.2.2 Membrane Elements

The basic formulation of cable nets can be extended to the analysis of the membrane structures. As it is described, membrane can be discretized into a system of number of 2D triangular membrane elements. For each triangle element the stress inside can be expressed as [25].

$$\{\sigma\} = \{\sigma_0, \sigma_0, 0\}^t \tag{7.18}$$

As shown in Fig. 7.14, for any joint i, of the triangular element, the prestressed force within the membrane element m produce a tensile force to that joint which can be expressed as.

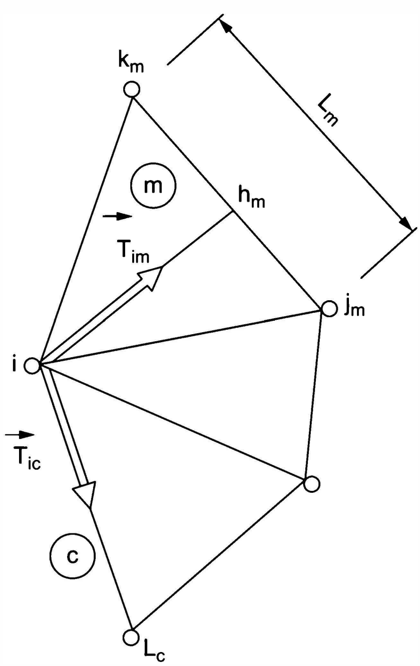

Fig. 7.14 Equilibrium of joints.

$$T_m = \frac{1}{2} L_m t \sigma_0 \frac{h_m}{H_m} \tag{7.19}$$

where

H_m is the length of the perpendicular bisector, and

t is the thickness of the membrane.

The area of the membrane element.

$$S_m = \frac{1}{2} L_m H_m$$

Therefore,

$$T_m = q_m L_m^2 i h_m$$

where q_m is the force density of the membrane elements.

7.3.2.3 Equilibrium of Joints

As shown in Fig. 7.14, for any joint i, the prestressed force in cable element c produce a tensile force to that joint which can be expressed as.

$$T_{ic} = q_c L_c$$

Therefore, the equilibrium of joint i can be expressed as.

$$\sum_{m=1}^{k} q_m L_m^2 i h_m + \sum_{n=1}^{j} q_c L_c = 0. \tag{7.20}$$

where K and j are numbers of membrane elements and cable elements connected to joint i.

Based on Eq. (7.20), the equilibrium of all the joints of.

7.3.3 Dynamic Relaxation Analysis Method

The load analysis of cable-membrane structures is a geometrical nonlinear problem due to their large deformation. The dynamic relaxation method (DRM) is an iterative process that is used to find static equilibrium. The theory of this method was first described by Day [26]. In this method, a dynamic solution is used for a fictitious damped structure to achieve a static solution. The stability of the method depends on the fictitious variables (such as mass and damping) and time step.

Dynamic relaxation is based on the Newton's second law of motion and stress-strain relations of the structural components under consideration.

For motion in the x direction:

$$F_i = M_i \ddot{X}_i \tag{7.21}$$

If a tensile structure is not in equilibrium, there will be some residual force R acting on the node. The static configuration of an elastic system can be treated as a consequence of a previous dynamic state, by presuming an artificial dynamic system which could be expressed as.

$$R_i^t = [M]\ddot{X}_i^t + [C]\dot{X}_i^t \tag{7.22}$$

where

$[M]$ is the artificial mass matrix, oscillating around their equilibrium position,

$[C]$ is the artificial damping matrix, it will dissipate the energy until a steady equilibrium state is reached, X is the displacement,

$\dot{X}_i$ is the velocity,

$\ddot{X}_i$ is the acceleration, and

R_i is the residual of internal and external forces.

$$\ddot{X}_i^t = [M]^{-1}\{R_i^t - [C]X_i^t\} \tag{7.23}$$

For a small time interval Δt, the equation can be rewritten in central difference form.

For velocity:

$$\dot{X}_i^{t+\Delta t/2} = \dot{X}_i^{t-\Delta t/2} + \Delta t \ddot{X}_i \tag{7.24}$$

Substitute Eqs. (7.6) into (7.4), therefore,

$$R_i^t = M_i \frac{\left(\dot{X}_i^{t+\Delta t/2} - \dot{X}_i^{t-\Delta t/2}\right)}{\Delta t} + C_i \frac{\left(\dot{X}_i^{t+\Delta t/2} + \dot{X}_i^{t-\Delta t/2}\right)}{2} \tag{7.25}$$

Further, rearrange Eq. (7.7), we get the nodal velocity that.

$$\dot{X}_i^{t+\Delta t/2} = \dot{X}_i^{t-\Delta t/2}\left\{\frac{M_i/\Delta t - C_i/2}{M_i/\Delta t + C_i/2}\right\} + R_i^t\left\{\frac{1}{M_i/\Delta t + C_i/2}\right\} \tag{7.26}$$

Therefore, the displacement of nodal coordinates at time $t+\Delta t$ can be worked out as

$$X_i^{t+\Delta t} = X_i^t + \Delta t \dot{X}_i^{t+\Delta t/2} \tag{7.27}$$

For the first iteration.

Therefore, the current nodal residuals $R_i^{t+\Delta t}$ can be worked out, and a new iteration starts.

Based on the above equations, the DRM can be divided into the following steps:

1. Setting the displacement of the node X and residual R at their initial value at $t=0$
2. Using displacement X and Eq. (7.23) to work out $\ddot{X}_i^t$ at time t
3. Calculate the new velocities $\dot{X}_i$ at time $t+\Delta t/2$ using $\dot{X}_i^{t+\Delta t/2} = \dot{X}_i^{t-\Delta t/2} + \Delta t \ddot{X}_i$ Eq. (7.24)
4. Calculate the residuals force R at time $t+\Delta t/2$ using $R_i^t = M_i \frac{\left(\dot{X}_i^{t+\Delta t/2} - \dot{X}_i^{t-\Delta t/2}\right)}{\Delta t} + C_i \frac{\left(\dot{X}_i^{t+\Delta t/2} + \dot{X}_i^{t-\Delta t/2}\right)}{2}$ Eq. (7.25)
5. Calculate the new velocities $\dot{X}_i$ at time $t+\Delta t/2$
6. Using the velocity $\dot{X}_i$ to Calculate the new displacement X at time $t+\Delta t/2$
7. Go back to step 2
8. Repeating the above steps unless the residual R close enough to zero

The iterations proceed until the required convergence, is determined by the allowable residual forces are smaller than the required value, has been achieved [27].

7.4 MODELING EXAMPLES OF TENSILE MEMBRANE STRUCTURE

Owing to its flexibility, membrane structure is difficult to model without helping the advanced analysis software. In this section, the modeling example of form finding for membrane structure will be shown using two popular general purpose software ANSYS and Abaqus.

7.4.1 Modeling Example of Form Finding for Membrane Using ANSYS

We have introduced several form-finding theories, here, the form finding will be performed using general purpose software ANSYS. There are several steps to set up the membrane model in ANSYS [28]

1. Set up the geometrical model
2. Using *link10* element to model Cable
3. Using *shell41* element to model membrane, in the *Option*, choose *Tension Only*

4. Meshing the membrane elements using triangular mesh
5. Applying the constrain by
   Load > Apply > Displacement > on Keypoints,
6. Apply the prestress force to the membrane through decreasing the temperature by
   Load > Apply > temperature > on area > then choose the membrane,
   Choose temperature value −0.375°C
7. Analysis using static large displacement analysis procedure
8. Checking the stress distribution of the membrane

7.4.2 Modeling Membrane Elements Using Abaqus

In Abaqus, the membrane can be modeled using several different elements available in Abaqus:

M3D4 membrane element which is a four-node quadrilateral element,
M3D8R, an eight-node quadrilateral membrane, and
M3D3 membrane element which is a three-node triangular element.

These elements are surface elements that transmit in-plane forces only and have no bending stiffness so they could not resist moments. Therefore, they are ideal to model membrane.

It is necessary to prestress these elements before any analysis is carried out. The mesh for the square membrane consists of triangular or square elements.

Uniform biaxial prestress force is applied to the membrane elements directly using the.

**INITIAL CONDITIONS, TYPE = STRESS* option.

This option should be used before applying any of the **STEP* options, to account for the geometric stiffness induced by the prestress.

The nonlinear calculation procedures should be used. Before the eigenvalue extraction calculations, a nonlinear static analysis step is carried out by using the **STEP, NLGEOM* option after the **INITIAL CONDITIONS* option.

In the above static analysis step, the applied initial prestress is maintained by restraining the outer edges of the membrane. Then, in the next step (*frequency extraction step*), the boundary conditions are changed to the actual ones, by using *BOUNDARY, OP = NEW option.

If vibration of the membrane is to be checked, frequency analysis step needs to be defined, using linear analysis step, number of eigenvalues can be extracted with the choice of *eigensolver*. There are two solution methods for solving eigenvalues can be used in Abaqus: the subspace iteration method and the Lanczos method. The Lanczos eigensolver is faster and more effective for models with many degrees of freedom.

7.5 INTRODUCTION TO TENSEGRITY STRUCTURES

Tensigrity is a structural system which was developed in the past 50 years. The concept of 'tensegrity' was first invented by architect B. Fuller [29]. The word tensegrity comes from two words "Tension + Integrity = Tensegrity". He came up with an idea of "nature relies on continuous tension to embrace islanded compression elements." In Fuller's idea, for this type of structure, stresses are evenly distributed throughout the entire structure rather than accumulating at certain points, therefore, maintain the balance of tension members.

7.5.1 Difference Between Tensegrity Structure and Tensile Structure

Most of the tensegrity structures are covered with membrane material, therefore, it sometimes is confused with tensile structures. The major difference between tensegrity structure and tensile structure is that it is a self-equilibrium structure, a system composed of continuous prestressed cables (in tension) and individual compression struts, as shown in Fig. 7.15, with the cables attached to the compression struts. Therefore, different to tensile structures, it consists tensional members as well as compression members, however, the compression member is part of the self-equilibrium system. On the contrast, for tensile structures, compression member such as masts are primarily work as the support structures to the entire tensile cable nets.

7.5.2 Application of Tensegrity Structure in Construction Projects

Tensegrity structure is widely used in long-span space structures in civil engineering, this is due to its super lightweight features, making a clear span of more than 200 m easily achievable which outperform other long-span structures such as double-layer grid and single-layer lattice domes. As shown in Fig. 7.16, the first long-span space structure using project using Fuller's tensegrity structure concept is a "cable dome" in the circular roof structures of Gymnastic and Fencing Arenas for the Seoul Olympic Games in 1986, designed by Geiger [30]. However, many arguments have been made on this

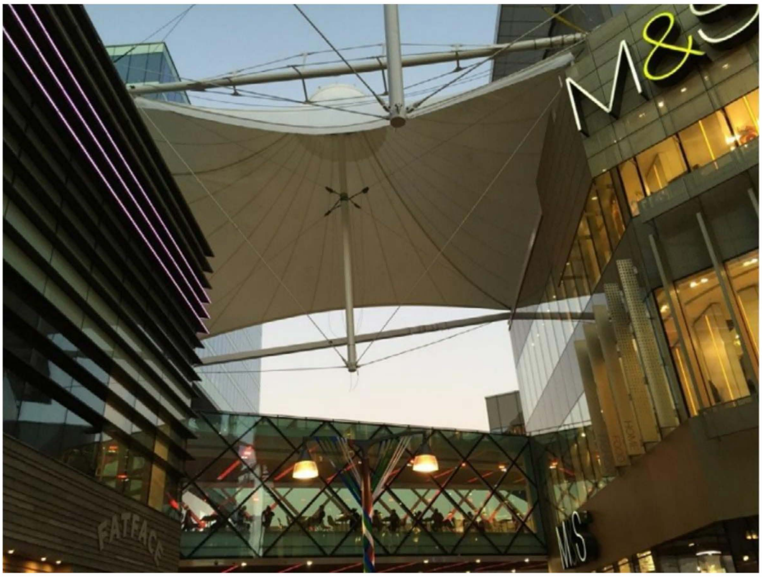

Fig. 7.15 Unit tensegrity structure in Westfield Shopping Center, Stratford, London, U.K. *(Photo taken by the author.)*

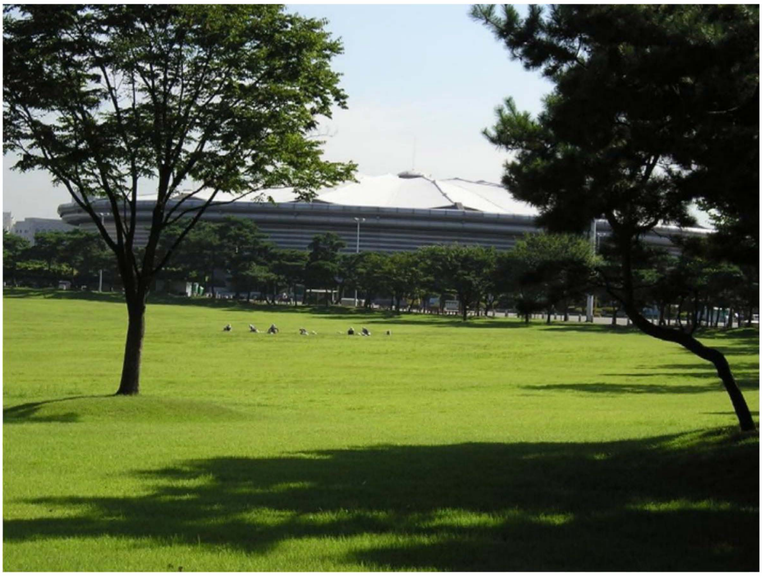

Fig. 7.16 Fencing Arena for Seoul Olympic Games, Seoul, South Korea. *(Photo taken by the author.)*

structure, as most engineers did not recognize it as an actual tensegrity system, because the compression ring is not inside the set of cables. In 1992, Levy [31,32] further improved the layout of the cable dome and built the Georgia Dome in quasi-elliptical shape for Atlanta Olympic Games, which is considered to be the first tensegrity dome built in the world.

Except in long-span structures, tensegrity concept have been used applications in fields such as sculpture, architecture, aerospace engineering, marine engineering, and biology [33]. Owing to its flexible and easily controllable feature, it is also used in active and deployable structures.

7.6 STRUCTURAL ANALYSIS OF TENSEGRITY STRUCTURES

Similar to tensile structure, the tensegrity structure is a geometrical nonlinear system, large deformation need to be considered. Tensegrity structure is a type of statically indeterminate structure, it also requires an initial form-finding procedure to create a state of self-equilibrium. There are several methods in form finding, similar to tensile structures, force density and DRM are also widely used in tensegrity structures.

In spite of that researchers have also discovered certain tools in form finding. Paper [34] presents a novel numerical form-finding procedure which only needs the type of each member, with both equilibrium geometry and force densities are iteratively calculated. A condition of a maximal rank of the force density matrix and minimal member length were included in the form-finding procedure to guide the search of a state of self-stress with minimal elastic potential energy.

In Ref. [35], a review for form-finding methods for tensegrity structure has been made. They are three so-called kinematical methods which include DRM, static method including FDM, energy method, and reduced coordinate method. The most popular methods used in design and analysis will be explained in this section.

7.6.1 Force Density Method in Form Finding

The FDM can be extended in analyzing the tensegrity structures, where similar equation as it introduced in cable-membrane structures can be used:

$$[C]^T[Q][C] = \{F_x\}$$

where $[C] = [C_{ij}]_{n\times m}$ is the topology matrix of this network, following the same rule as it is described in Section 7.3.

As it is a self-equilibrium system, in the form-finding stage, no external loading is applied. Therefore, the equation:

$$[D]\{x\} = \{F_x\} - [D_f]\{x_f\}$$

can be changed into.

$$[D]\{x\} = -[D_f]\{x_f\}$$

When determining the [Q], we need to be clear that the cables are always in tension and the struts are always in compression; therefore, the elements in the matrix is not always positive, singularity maybe found.

when $[D] \neq 0$, we will have nontrivial solution. $[D] = 0$, we will have trivial solution.

7.6.2 Dynamic Relaxation Analysis Method

As discussed in Section 7.3.3, dynamic relaxation is based on Newton's second law of motion and the iteration of it over many time steps, it has been used for form finding of pure tensile structures such as cable nets. It is later been used to tensegrity structures.

René Motro [36] proposed to apply the DRM to tensegrities, this marked an important step in form finding of tensegrity structures. When applying it to tensegrity structures, it is one of the so-called kinematical method by Tibert and Pellegrino [35]. In this method, the lengths of the cables are kept constant, while the strut lengths are increased until a maximum is reached. Alternatively, the strut lengths may be kept constant while the cable lengths are decreased until they reach a minimum, in summary, an initial layout member lengths are gradually altered until an equilibrium condition is reached.

In a form-finding analysis, one can prescribe for each element of the structure a constitutive relationship of the type:

$$t = t_0 + ke \tag{7.28}$$

where

t is the axial force,

e is the extension, t_0 is the desired prestress, and

k is a fictitious small axial stiffness.

Nodal equations of equilibrium are used to compute out-of-balance forces from which the current acceleration can be obtained through.

$$M\ddot{x} + D\dot{x} + Kx = f$$

where

K is a stiffness matrix,

M is a mass matrix,

D is a damping matrix,

f is the vector of external forces, and

$\ddot{x}$, $\dot{x}$, x are the vectors of acceleration, velocity, and displacement from the initial configuration, for simplicity, and the velocities and displacements are initially set to zero in the analysis.

Motro [36] applied the DRM to the form finding of the triangular tensegrity prism. The lengths of the cables were held constant while the struts were gradually elongated, until a state of prestress was set up in the structure. This analysis converged when the ratio between length of the strut to the length of the cable are 1.468.

As in Ref. [37] form finding through DRM depends on member stiffness, damping, and length ratio. Among them, the stiffness property assignment is crucial to the relaxation procedure. As tensegrity structures consist of both rigid and nonrigid members (struts and cables). At the beginning of the analysis two member stiffness parameters need to be defined: EA (tensile) and EA (compressive). At latter loading application stage during the analysis, this will ultimately decide the element's deformation under external load and hence the displacement of the nodes. Experiments show that the least successful procedures are those in which the relative difference in stiffness between the members is the greatest. Gustav Fagerström [37] suggests that not only relative stiffness differences between members but also absolute stiffness in relation to overall structure dimension and topology govern the properties of a tensegrity assembly.

Different to the form-finding processes in pure tensile structures, it is important to set appropriate interrelation between tension and compression members in the assembly. It is necessary to specify that elements are to be in pure compression and pure tension, respectively. To achieve this, in Ref. [37] for each relaxation cycle, if a compression member switches into tension or a tension member switches into compression, the forces acting on that member are re-set to zero. This can solve the problem of unwanted forces that are present.

7.6.3 Nonlinear Dynamic Finite Element Method

This method is to set up the nonlinear finite element equation for the tensegrity structures [38]. This procedure is to set up the basic nonlinear finite element equation for the entire structure, using stiffness of matrix method, and then using different numerical method such as Newton-Raphson approach to solve the equation.

7.6.3.1 Nonlinear Finite Element Analysis Equations

In the procedure of the determination of the initial equilibrium, the coordinate can be firstly presumed with an ideal distribution of the prestressed force which means it can be in any form of shape. Nevertheless, the node in the structural grid will not be balanced under this condition, imbalance force will be resulted, hence, it results in displacement of the node. Thus, the coordinate and the prestressed force need to be adjusted step by step until the whole structure is balanced.

The formula to determine the initial equilibrium is.

$$\left([K_L]^0 + [K_{NL}]^0\right)\{\Delta U\}^0 = -\{P\}^0 + \{R\}^0 \tag{7.29}$$

where

$[K_L]^0$ is the initial linear stiffness matrix,

$[K_L]^0$ is the initial nonlinear stiffness matrix,

$\{\Delta U\}^0$ is the variation of the coordinate,

$\{P\}^0$ is the prestressed force, and

$\{R\}^0$ is the residual force or imbalance force at each node.

After the determination of the initial equilibrium of the structure, the structure can be further analyzed under the load conditions. The fundamental equation for static analysis is.

$$([K_L] + [K_{NL}])\{\Delta U\} = -\{P\} + \{R\} \tag{7.30}$$

where

$[K]$ is the total stiffness matrix,

$\{U\}$ is the displacement vector of the node,

$\{P\}$ is the external load vector, and

$\{R\}$ is the residual force.

7.6.3.2 Newton-Raphson Approach to Solve the Equation

The Newton-Raphson approach can be used for solving the above two equations. The total load is divided into small increments and the calculation procedure is divided into correspondent steps, and for each increment a new $[K]_i$ is used. The nonlinearity is therefore treated as piece-wise linearity and a constant $[K]_i$ is used in all increments. After each iteration, the "unbalanced" portion of the external force is estimated and applied in the next increment.

For each step:

$$[K]_i\{\Delta U\}_i = \{\Delta P\}_i \tag{7.31}$$

$$\{U\}_i = \{U\}_{i-1} + \{\Delta U\}_i \tag{7.32}$$

where

$[K]_i$ is the stiffness matrix when $n = i$,

$\{\Delta U_i\}$ is increment of the displacement when $n=i$,
$\{\Delta P\}_i$ is imbalance load when $n=i$,
$\{U\}_i$ is displacement when $n=i$, and
$\{U\}_{i-1}$ is displacement when $n=i-1$.

As $\{\Delta U\}_i$ is obtained, $\{\Delta F\}_i$, the increment of the internal force when $n=i$ can be therefore obtained. For each step, the internal force can be obtained as

$$\{F\}_i=\{F\}_{i-1}+\{\Delta F\}_i \tag{7.33}$$

where $\{F\}_i$ is the internal force of member $n=i$, $\{F\}_{i-1}$ is the internal force of member $n=i-1$, and $\{\Delta F\}_i$ is the increment of member when $n=i$.

When all the steps are completed, the increment of the displacement and the increment of the internal force in a different step will be added together, so the final result can be obtained.

$$\{U\}=\sum_{i=1}^{n}\{\Delta U\}_i=\{\Delta U\}_1+\{\Delta U\}_2+\cdots+\{\Delta U\}_n \tag{7.34}$$

$$\{F\}=\sum_{i=1}^{n}\{\Delta F\}_i=\{\Delta F\}_1+\{\Delta F\}_2+\cdots+\{\Delta F\}_n \tag{7.35}$$

7.6.4 Case Study of Georgia Dome

As shown in Fig. 7.17, Georgia Dome is a decommissioned domed stadium located in Atlanta, Georgia, United States. The dome is supposed to be demolished in 2017. It was completed in 1992. The stadium seated

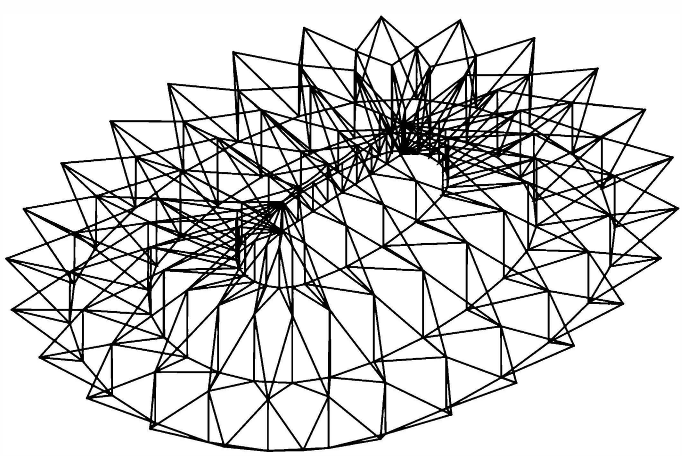

Fig. 7.17 3D AutoCAD Drawing of Georgia Dome.

74,228 for football, and could hold approximately 80,000 for concerts, and 71,000 for basketball [39].

7.6.4.1 Structural System of Georgia Dome

The dome was the largest cable-supported dome in the world and as mentioned in the earlier section, it is considered to be the first tensegrity dome in the world. The dome was designed by Levy [31,32]. On top of the tensegrity skeleton, it was covered by Teflon-coated fiberglass fabric and with an area of 34,800.000 m^2. It was the largest membrane structure in the world before the completion of Millennium Dome, London in 1999.

From Fig. 7.18, it can be seen that the longitudinal length of the dome is 239.35 m, the transvers length of the dome 186.97 m. As shown in Fig. 7.17, the dome can be divided into four layers, each layer composes of vertical compression struts connected by diagonal ridge cables. These cables are joined into a center cable truss, as shown in Fig. 7.20. It can be seen that the center truss is constructed using nine vertical compression struts connected by diagonal cables and cables as the top and bottom cord. The reason is evident, as the center truss needs to resist the huge tension forces come from the ridge cables. At the perimeter, these ridge cables are anchored back into a large concrete compression ring beam (7.9-m wide and 1.5-m deep). Therefore, the ring beam and the dome compose a self-equilibrium system. This fulfilled the design philosophy of Fuller for nature relies on continuous tension to embrace islanded compression elements.

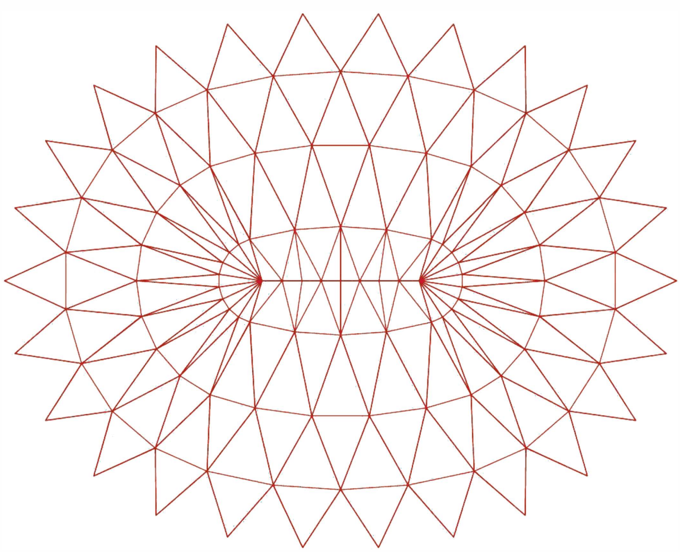

Fig. 7.18 Plan view of Georgia Dome.

To accommodate the complex connection between the compression struts and tension cables, a special joints are designed by Levy [31,32]. This type of connection can resist the huge tension force come from the cables, as well as the compression forces from the compression struts.

7.6.4.2 3D Modeling of Georgia Dome Using SAP2000

In this section, the nonlinear dynamic finite element method discussed in Section 7.6.3 was used to analyze the tensegrity structure Georgia Dome. The analysis was implemented in SAP2000. As SAP2000 can perform geometric nonlinear analysis, which also have the cable element, which can also define the prestress force of the cable and the perform form-finding analysis.

The model was first built using AutoCAD. As shown in Fig. 7.17, a 3D AutoCAD model was built, then imported into SAP2000, as shown in Figs. 7.19 and 7.20.

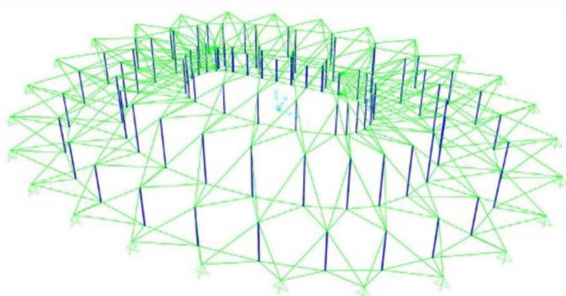

Fig. 7.19 3D SAP model of Georgia Dome. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

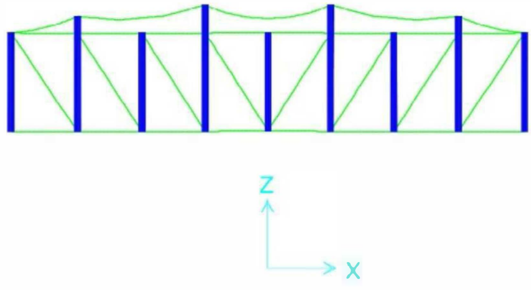

Fig. 7.20 Center Truss of Georgia Dome. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

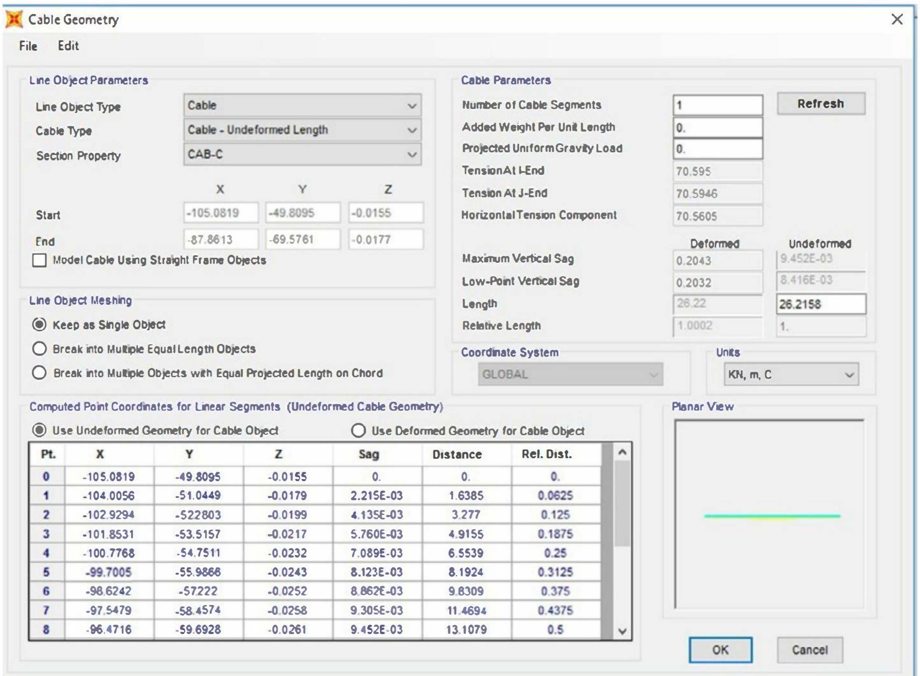

Fig. 7.21 Defining cable element. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

7.6.4.2.1 Defining Cables and Struts

After the dome has been imported into the SAP2000, the cable element can be defined (Fig. 7.21) as well as the compression struts. The size of the cables and struts are based on Levy (1989, 1991).

7.6.4.2.2 Defining Prestress Force for the Cable

Then select the cables, choose Assign → Cable Loads → Target Force, then define the initial prestress forces for the cables. The initial prestress force of a cable is calculated using below empirical formula:

$$F = \sigma \times A \times 60\%$$

where

F is the initial prestress force,
σ is the yield stress of the cable, and
A is the cross-sectional area of the cable.

7.6.4.2.3 Form Finding

When all the structural members are set up, there are two analysis steps, one is form-finding analysis, another is the analysis under the actual loading in

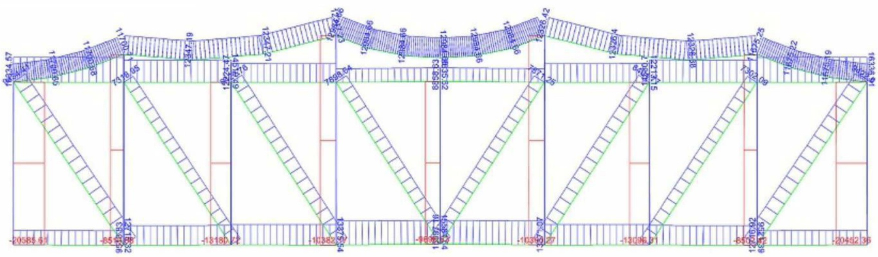

Fig. 7.22 Result of form finding for center cable truss. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

SAP2000, one can define the *target load case* step for nonlinear form-finding analysis, however, remember to include the dead load when running the target load case, as the self-weight of the structure need to be inclusive in the form-finding analysis. Fig. 7.22 shows the analysis result, which shows the tensile and compression force distribution in the cable truss, the unit is in kN.

7.6.4.2.4 Analysis Under Loading Case

After the form-finding analysis completes, one can perform an analysis under the loading case, however, when defining the load cases such as wind or live load, make sure to choose the option "*Continue from State at the End of Nonlinear Case.*" Fig. 7.23 shows the analysis result of the dome under the live load. In this case study, 0.6 kN/m^2 live load was applied to the dome, which shows the tensile and compression force distribution in the cable truss, the unit is in kN. It can be seen that when the live load is applied, the tensile force inside the cables are reduced. The tensigrity structure primarily relies on the prestress force to resist the external loading. Therefore, a proper form finding is important, as it will optimize the right level of the prestress force.

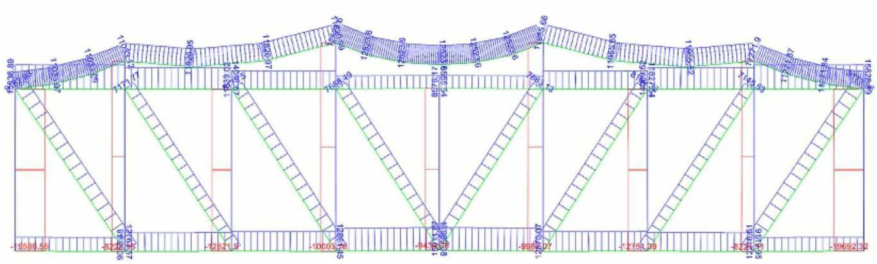

Fig. 7.23 Result of analysis under live load for center cable truss. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

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CHAPTER EIGHT

Design of Offshore Structures

Abstract

This chapter focuses on the introduction of the design and analysis of the offshore structures, including offshore oil platforms and offshore wind turbines. The incidents of offshore structures are demonstrated and a detailed explanation of major loadings and design guidelines are made. The wave theory models on fluid and structure interactions are also introduced. A modeling example of an oil platform using SAP2000 is discussed in the final part of this chapter.

Keywords: Offshore structure, Wind turbine, Oil platform, Wave theory, Accidental load, Wave load, Fatigue, Morrison's equation, Fluid-structure interaction

8.1 INTRODUCTION

Offshore structures consist of offshore oil platforms and offshore wind turbines (OWTs). They are special type of complex structures. The offshore oil platforms are large-scale structures situated in the sea. They can accommodate the facilities and equipment to drill, extract, and process oil and natural gases. It also provides living spaces for the workforce. With the need for clean energy, offshore wind farms were constructed across the world which consists of large number of OWTs. Therefore, both oil industry and clean power supply industry has great demand for structural engineers with the ability to design offshore oil platform or OWTs.

In this chapter, the details of designing and analysis of offshore oil platforms and wind turbines are discussed. The major loadings in the design consideration such as wave loading, seismic loading, and blast loading will be discussed, and the way to analyze the offshore oil platform using the conventional software such as SAP2000 will be introduced.

8.2 OIL PLATFORM

8.2.1 History of Offshore Platforms

In the past, oil was the primary resource in America, due to increase of gasoline by combustion engines. In 1887, H.L. Williams decided drilling

Design and Analysis of Tall and Complex Structures
https://doi.org/10.1016/B978-0-08-101018-1.00008-3

in the sea by use of building a wharf; the first wharf extended 90 m into the ocean. In1947, Kerr McGee Corporation drilled the first well from a fixed offshore platform. In 1949, 11 fields were found in the Gulf of Mexico with 44 exploratory wells. They are the pioneers in offshore oil drilling. After that the different offshore drilling methods have been developed, from then on different oil platform types have been designed.

8.2.2 Type Oil Platforms

While choosing the correct type of platform, several factors need to be considered: such as its function and the water depth. Based on these two factors the oil platforms are designed into two major categories: one is movable oil platform, the other kind is fixed platform. They will be discussed in detail here in this chapter.

8.2.3 Movable Oil Platform

8.2.3.1 Drilling Barges

Drilling barges are shown in Fig. 8.1. They are mostly used for shallow, inland waters; this would typically be lakes, rivers, and canals. When the drilling barge tends to be large floating platforms must be moved using a tugboat

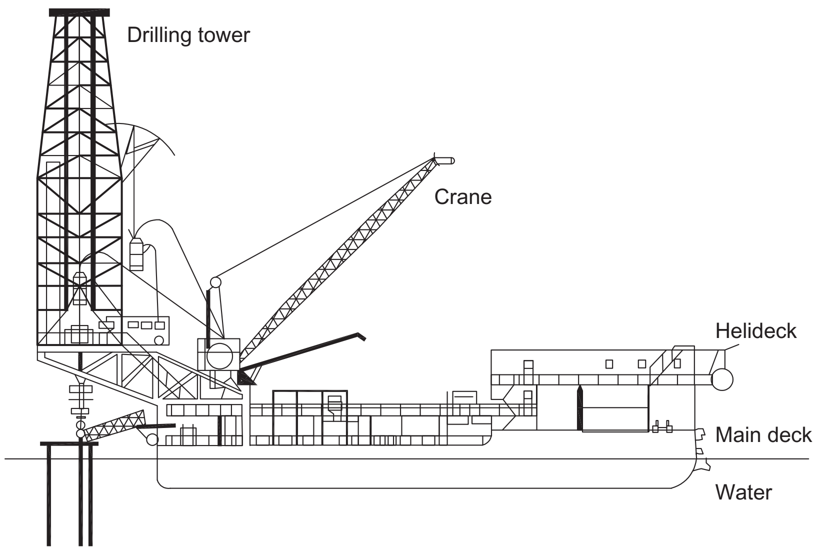

Fig. 8.1 Drilling barge.

to the location of drilling. They are not suitable for large open waters as they are not able to withstand water movement.

8.2.3.2 Drillship

As shown in Fig. 8.2, drillships are ships with the drilling platform installed in the middle of the ship deck to allow the drilling string to reach the sea bed through ship's. Drillships can drill even in deep waters. Drillships maintains its position using dynamic positioning systems and other sensors. The ships also use satellite-positioning technology with motors integrated below the hull to position the ship directly above the drill site.

8.2.3.3 Jack-Up Platforms

Jack-up platforms (Fig. 8.3) are mobile drilling platforms made of floatable deck with three or four legs lowered onto the seabed upon being towed to the drill site. The platform is then towed to the location with raised legs. It can raise its hull over the surface of the sea. Jack-up rigs are suitable for shallow waters with a maximum depth of water of up to 130 m in gentle water and up to 70 m in harsh waters. Jack-up platforms tend to be a safe alternative than drilling barge since during operation they operate similarly to a fixed platform.

8.2.3.4 Submersible Platforms

As shown in Fig. 8.4, submersible platforms are mobile structures which are designed that can be floated to location and lowered onto the sea floor for offshore drilling activities. However, due to its own feature, it is only limited to shallow waters. It normally consists of two platforms one is on top

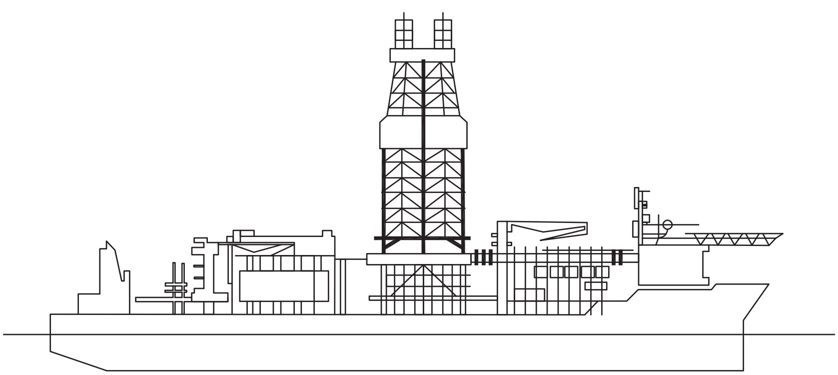

Fig. 8.2 Drillship.

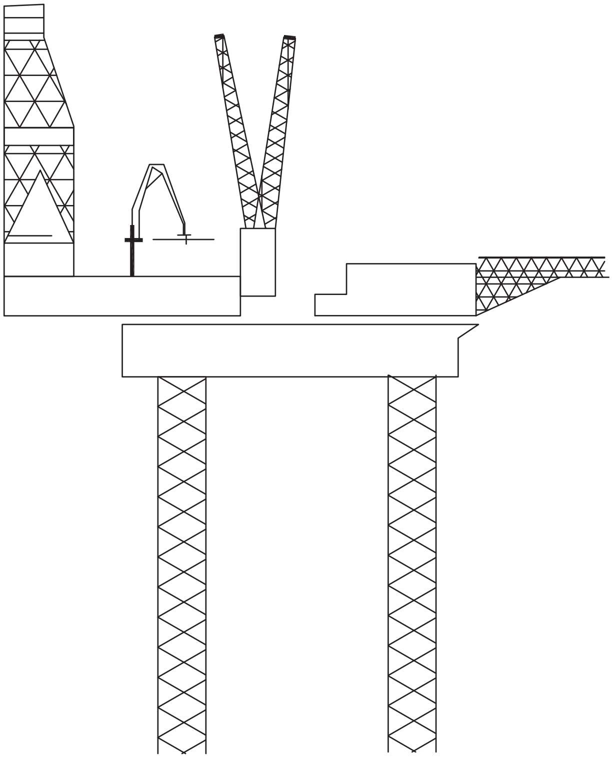

Fig. 8.3 Jack-up rig.

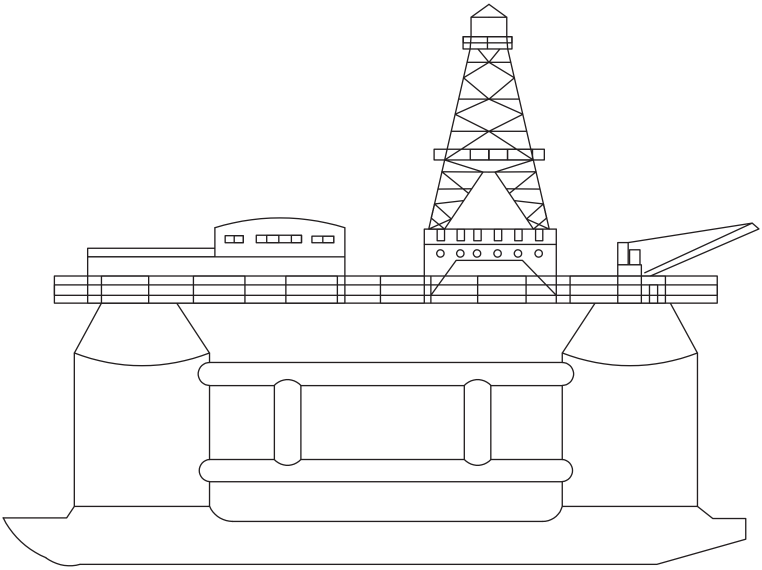

Fig. 8.4 Submersible platform.

of another. The upper platform is the actual drilling platform, the lower platform provides buoyancy during towing of the platform between drilling sites. Once the platform has been positioned, air is let out of the hull and then it submerges to the sea floor. Due to the design of the platform, it is limited to shallow waters.

8.2.3.5 Semi-Submersible Platforms

As shown in Fig. 8.5, semi-submersible platforms are similar in design to submersible platforms with same principles. As the lower hull can be inflated and deflated this type of platform is only partially submerged and uses water to fill the bottom hull for buoyancy. A semi-submersible platform can be used to drill in deep water depths of 1800 m. The depth of the lower hull that submerges into the water is predetermined, and the platform is held in position by anchors weighing upwards of 10 tones. The dynamic positioning can also be used for the same purpose, and this can ensure safe and stable conditions of the platforms.

8.2.4 Fixed Type Oil Platform

The fixed platforms are normally used in shallow waters. They have the legs which are made of concrete or steel tubular structures anchored to the sea bed to ensure stability of the platforms. However, the cost of these types of permanent platform would be too much to use in deep water conditions, therefore, if most of them are used in the shallow sea water, except for compliant tower (CT), which can be used for >600 m. There are several types of fixed platforms such as, Jacket platforms, tension leg platform (TLP), gravity platform, or even an artificial island.

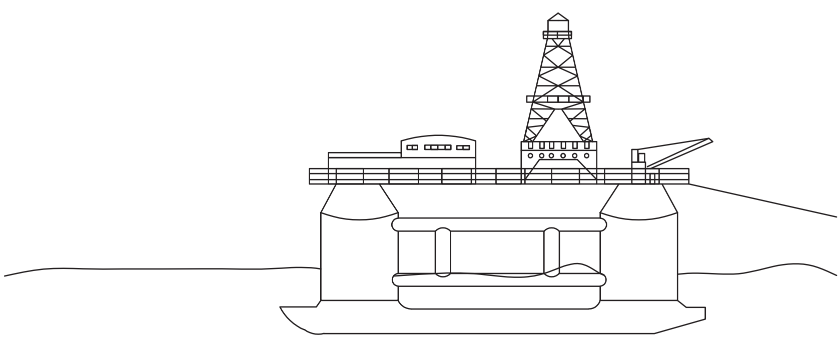

Fig. 8.5 Semi-submersible platform.

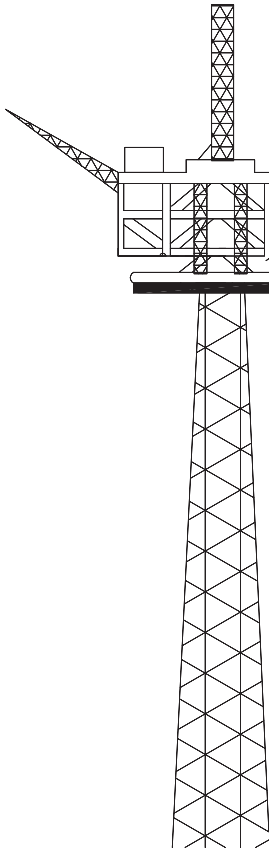

Fig. 8.6 Jacket offshore platforms.

8.2.4.1 Jacket Offshore Platforms

As shown in Fig. 8.6, jacket platforms are one of the most commonly used fixed type of platform, about 95% of the offshore platforms in the world are using jacket platform. The platform is supported by steel space frame which consists of plate girder or deck truss structure supported by welded tubular that is piled to the seafloor. The steel frame is called jacket. It is used in water when the depth does not exceed 500 m.

8.2.4.2 Tension Leg Platforms

Fig. 8.7 shows the TLPs. As explained in Ref. [1], TLPs is evolved from semi-submersible platforms, which consist of a buoyant floating platform kept in position by pretensioned anchors fixed to the seabed using vertical mooring lines called tension leg. Although vertical movement is restricted,

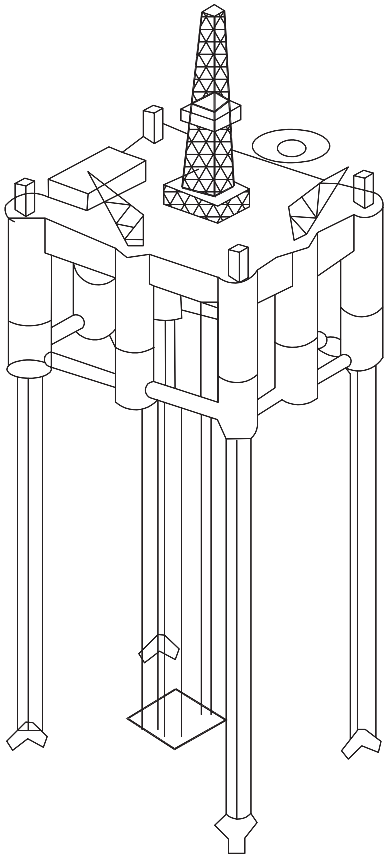

Fig. 8.7 Tension leg platform.

they do allow for significant sideway movements. The pretensioning of the tethers is huge in size to avoid compression due to waves. Typically, the tethers consist of a 12- to 15-m-long section connected by welds or threaded joints, with a pile or gravity skirt foundation into the seabed.

To further increase the stability of the platform the lower hull is filled with water during drilling to resist movement of ocean and wind forces. The TLPs can operate in waters up to 2.13-km-deep waters. Sea star platforms are a miniature version of a TLP made for waters of depths up to 1 km.

8.2.4.3 Gravity Platforms

Gravity platforms consist of a steel deck and concrete framework as well as a steel skirt foundation where petroleum is normally stored in the skirts. Gravity platforms have been made for water depths of up to 300 m in harsh

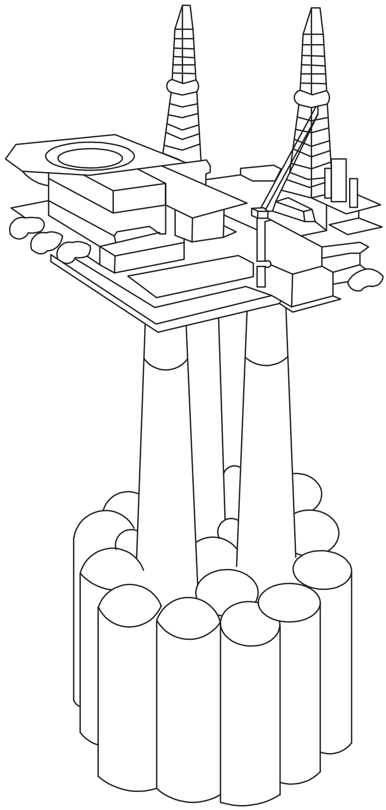

Fig. 8.8 A gravity platform (Geo Prober, 2016).

waters. They are built in an upright position and towed out to the offshore drilling site and installed by ballasting, where a heavy substance is used to provide floating stability. Gravity platforms are cost-efficient option due to reusability of a platform as the structure can be towed to another location. A gravity platform is shown in Fig. 8.8.

8.2.4.4 Spar Platform

This is one of the largest fixed type offshore platforms. The spar is a low motion floater that can support full drilling assistance of flexible or steel catenary risers. A large platform consists of a large cylinder that does not extend all the way to the seafloor but is tethered to the seabed by cables and lines. A cylinder is used to stabilize the platform in the water and allows the movement to absorb the force of the hurricanes. There are different types of spars:

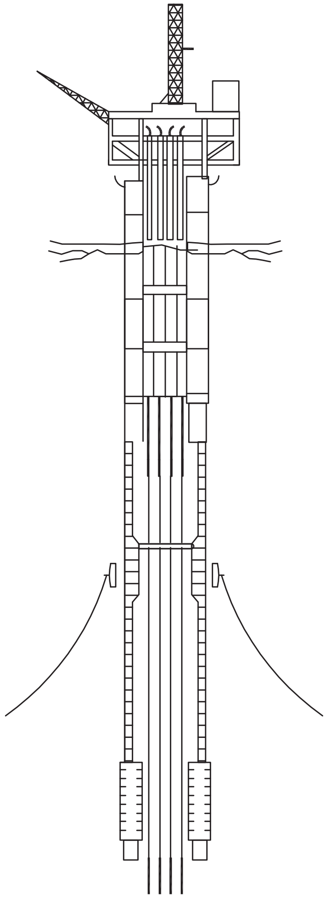

Fig. 8.9 Conventional spar.

original cylindrical classic spar, truss spar, the cell spar, the Arctic Spar, and spar with storage. Three main types of configurations of Spar platform cylinders are mentioned below:

1. Conventional spar: One-piece cylindrical hull, as shown in Fig. 8.9.
2. Truss spar: Midsection is made of truss elements as shown in Fig. 8.10.
3. Cell spar: Midsection made of multiple vertical cylinders, as shown in Fig. 8.11.

8.2.4.5 Compliant Towers

A CT [2] is a fixed rig which is similar to the fixed platform, but consists of a narrow tower attached to piled foundations on the seafloor and extend up to

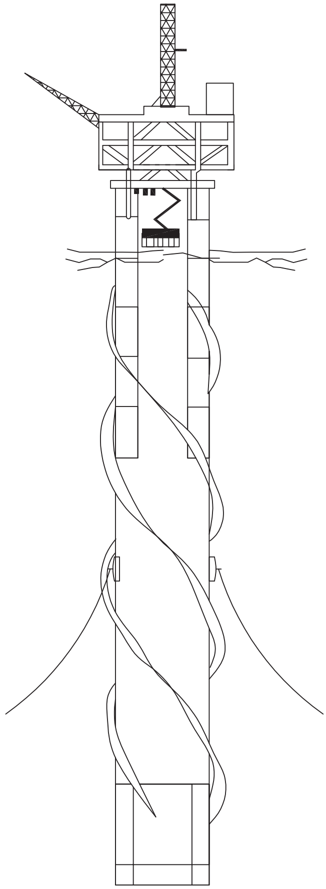

Fig. 8.10 Truss spar.

the platform shown in Fig. 8.12. In deep water, in order to prevent excessive amplification of wind, waves, and current, the natural period of the bottom founded structure should be substantially different to the dominant period of hurricane. Therefore, one of the methods to achieve this is to be able to control the mass and stiffness of the rigs.

The advantage of CTs use the flex elements such as flex legs and axial tube, which can control its natural periods therefore resonance is reduced and wave forces are de-amplified. Therefore, they are designed to sustain significant lateral deflections and forces, and are typically used in water depths ranging from 450 to 900 m.

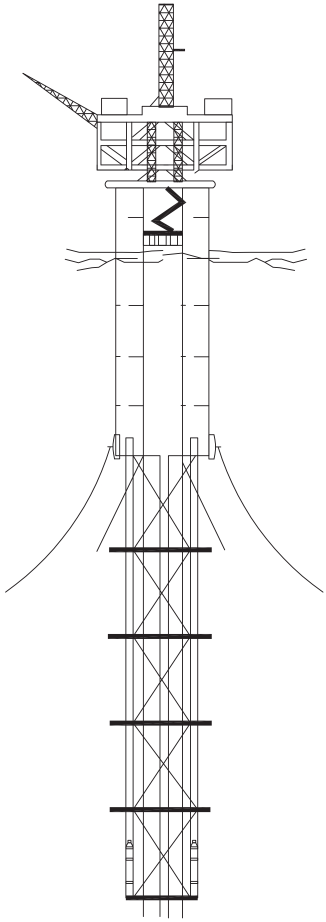

Fig. 8.11 Cell spar (Skaug, 1998).

At present, the deepest is the Chevron Petronius Tower in waters 623-m deep. However, because of cost, it becomes uneconomical to build CTs in depths >1000 m. In such a case, a floating production system is more appropriate.

8.2.5 Summaries

From the introduction in this section, there are various types of offshore oil platforms that are discussed. The choice between them is based on the depth of the water and the function as well as budget of the clients. From these factors, the depth of water is one of the important issues, as different oil

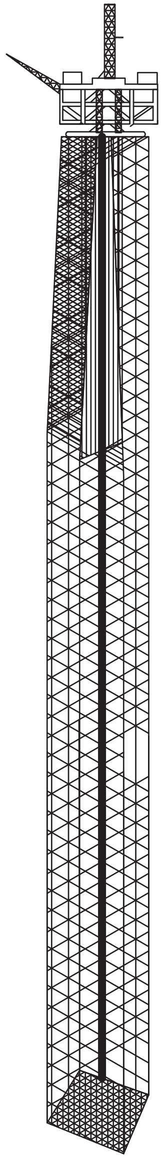

Fig. 8.12 Compliant tower.

platform can only be used in different water depth. Based on what we have discussed in the earlier sections, for different types of oil platforms and the maximum depth of water they can be used and is summarized in Fig. 8.13, while designing the oil platform, the readers can refer to it.

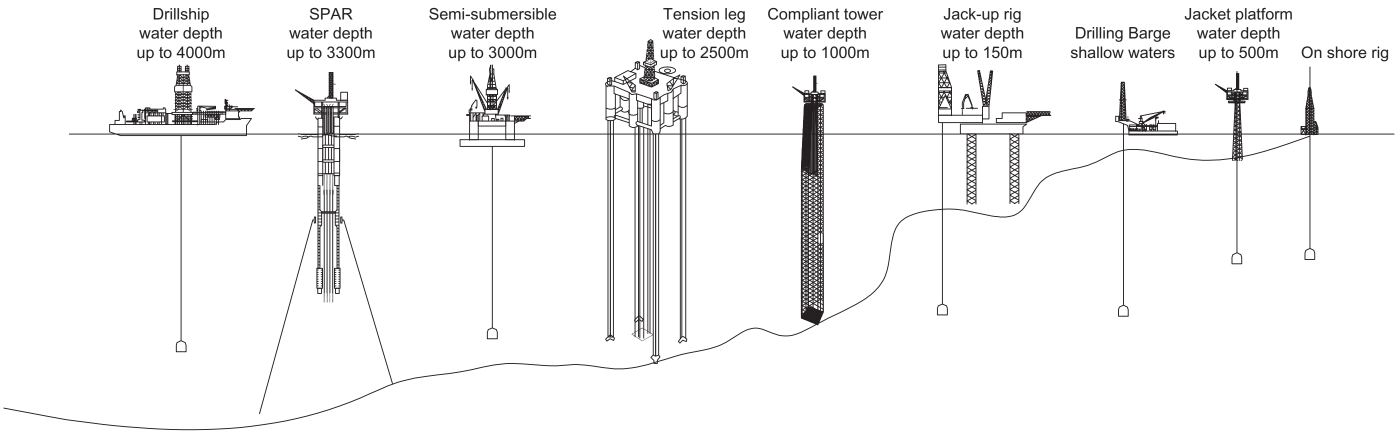

Fig. 8.13 Different types of offshore oil platforms used in different water depths.

8.3 INCIDENTS OF OFFSHORE STRUCTURES

There are certain reasons that cause incidents on the oil platform, heavy wave and oil leakage induced blast are the two major reasons. In this chapter, serval collapse incidents of oil platform is discussed.

8.3.1 Heavy Wave-Induced Incidents

Offshore structures are subject to very harsh marine environment. Therefore, heavy wave is a major cause for incidents on the oil platform. Fatigue-induced incidents have also been reported on several occasions.

8.3.1.1 Ocean Ranger Platform Collapse, Canada

Ocean ranger was a semi-submersible mobile offshore drilling unit that sank in on the Grand Banks of Newfoundland on February 15, 1982. In all, 84 crew members were on board when it sank and all of them died. The United States Coast Guard Marine Board of Investigation Report explained the causes for the sink was due to rogue waves (abnormal waves), which appeared to have a broken port light allowing the ingress of seawater into the ballast control room, the ballast control panel malfunctioned or appeared to be malfunctioning to the crew. The pumping of the forward tanks using the usual ballast control method was not possible, as the magnitude of the forward list created a vertical distance between the forward tanks and the ballast pumps located astern that exceeded the suction available on the ballast system's pumps; flooding of the chain lockers and subsequent flooding of the upper deck resulted in a loss of buoyancy cause the platform to capsize.

8.3.1.2 Sleipner A—North Sea Oil Platform Collapse

The Sleipner A (SLA-1) platform was used for drilling in the Sleipner gas field in the North Sea. It is a gravity base structure, built in the fashion of typical Condeep platforms which are reinforced concrete structures floating in water up to 300-m deep. These platforms consist of a number of buoyancy cells that serve as the floating mechanism. It was found that a combination of poor geometry and an inadequate design were the causes of the platform's failure. The failure mechanism was then concluded to be a shear failure that split open several walls in one of the platform shafts, which led to leakage of water which induced the final collapse.

8.3.2 Explosion-Induced Incidents

The explosion due to oil leak is another reason for incidents in oil platforms.

8.3.2.1 The Piper Alpha Disaster

The piper alpha disaster is one of the serious oil rig accident which killed 167 workers on July 6, 1988, the rigs were located off the coast of Aberdeen. Piper alpha was once Britain's biggest single oil and gas producing platform. A report on the disaster was submitted by Lord Cullen a judge mentions that the operator of Occidental Petroleum was inadequate in maintenance and safety procedures. The main reason for the incident was due to a shift change, the staff used a key piece of pipework which had been sealed with a temporary cover and no safety valve. Gas leaked and ignited while firewalls that could have resisted fire on an oil platform failed to cope with the ensuing gas explosion.

8.3.3 Fatigue-Induced Incident

Due to the adverse environment on the offshore structures, fatigue is one of the major reasons that cause their collapse. One of the famous examples is Alexander Kielland offshore rig collapse, it is a submersible oil rig. In 1980, the fatigue-caused cracks developed in six bracings caused the rig to heeled 30°, which broke six anchor cables, thereby making the rig to collapse further.

8.4 OFFSHORE WIND TURBINE STRUCTURES

In the next 50 years, with the growth of world population, the domestic environment and living conditions across the world will change dramatically. Compared with the conventional electricity generating methods for both residential and commercial communities, clean energy generated by wind turbine will provide a sustainable renewable energy option which will gradually replace, to substantial extent the traditional energy generating methods. Offshore wind farms have great advantage as they do not occupy existing land resources. This renewable energy solution will become a dominant power supply resource in the next 20–30 years. Therefore, in this section, the design and analysis method for offshore wind structures is discussed.

Wind turbine is a machine used to convert kinetic energy in the wind into electrical power. Wind energy has been used in the history such as

Fig. 8.14 Offshore wind farm near of coast of Kent, UK. *(Photo taken by the author.)*

"windmill," which can pump water and grind grain. Beginning of the 20th century numerous research projects explored the possibility of generating electricity through wind. The large-scale generation of electricity from the wind started in the early 1980s. Fig. 8.14 shows a typical wind farm that is used to generate electricity from the wind located near Kent, UK.

8.4.1 Features of Offshore Wind Turbines

OWTs are big and placed in deep waters in harsher sea areas and in extreme weather conditions. These factors make projects potentially hazardous and more expensive. From the energy producing point of view, larger wind turbines can capture more energy from the wind. Therefore, they must be able to withstand hard loading conditions due to the higher wind speeds and extreme wave conditions in areas afar from the shore. Any defects in the foundation will cause failure of the tower; therefore, a good foundation design is a very important.

8.4.2 Design Guidelines

When designing wind turbine structures, detailed guidelines of foundation design, tower, and blade design can be found in some design guidelines such as "Design of Offshore Wind Turbine Structures" [3] and "design of offshore steel structures general (LRFD method)" [4]. In terms of seismic design, there are three guidelines with details on design guidance for a design engineers to refer to: Guidelines for Design of Wind Turbines [5], Guideline for the Certification of Wind Turbines [6], and IEC 61400-1 Ed.3: Wind

Turbines—Part 1: Design Requirements [7]. In this chapter, the relevant structural design knowledge is discussed .

8.4.3 Different Support Structures

As the OWTs are located on the sea, therefore, support structure and its foundation are important for design considerations. According to Ref. [3], there are several conventional bottom-mounted support structures, which can be categorized into five basic types:

- monopile structures (using pile foundation),
- tripod structures (using pile foundation),
- lattice structures (using pile foundation),
- jacket foundations,
- gravity structures, and
- floating structures.

There are also hybrid support structures which use the combined features of the above categorized structures.

For the first three type of supporting structures (monopole, tripod, lattice structures) pile as the foundation is usually used. Pile foundations are one of the most common forms of offshore structures; they are widely used for both offshore oil platform and OWT. The standard method of piling method is to lift or float the structure into position and then drive the piles into the seabed using either steam or hydraulic powered hammers.

8.4.3.1 Monopile Structures

As shown Fig. 8.15, monopole has the simple fabrication and installation. The tower of the turbine directly sits on one pile.

Monopile foundations are one of the most frequently used support structure to date. Most of the offshore wind farms in shallow waters are monopole structures, which have the advantage of simple design for manufacturing. However, failure of the grouted connections between the monopile and the transition piece is one of its disadvantages. This transition piece is responsible for connecting the monopile to the turbine tower. In addition, there is no proven solution using monopiles for larger turbines with 5 MW or more powerful turbines.

8.4.3.2 Tripod and Lattice

As shown in Fig. 8.16, the turbines directly sit on a tripod or a lattice, which are supported on the pile foundations. The tower can be further stabilized by the tripod.

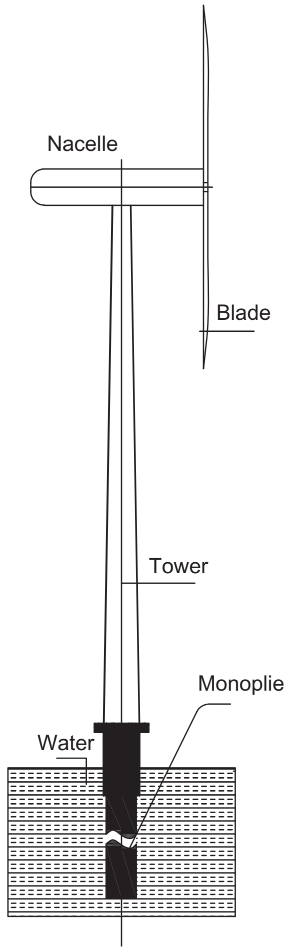

Fig. 8.15 Wind turbine on the monopile foundation.

8.4.3.3 Gravity Foundations

As shown in Fig. 8.17, this type of foundation achieves its stability solely by providing sufficient dead loads by means of its own gravity. Ballast can be pumped-in sand, concrete, rock, or iron ore to add extra weight.

Gravity structures are suitable for modest environmental loads such as wave load that are relatively small and dead load is significant or when additional ballast can easily be provided at a modest cost.

The gravity base structure is especially suited where the installation of the support structure cannot be performed by a heavy lift vessel or other special

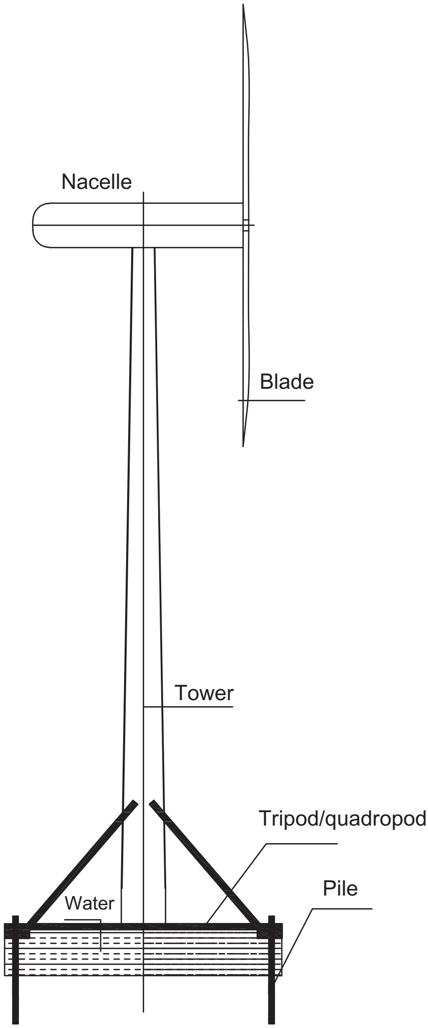

Fig. 8.16 Wind turbine on tripod.

offshore installation vessels, either because of nonavailability or prohibitive costs of mobilizing the vessels to the site.

Gravity-base foundations are the second most popular sort of support structure to date. They have been used mostly to support smaller turbines in shallow waters near shore locations with a rocky seabed where the operation of piling is extremely complicated and expensive. However, for water depth beyond 35 m, the new generation of wind farms are needed.

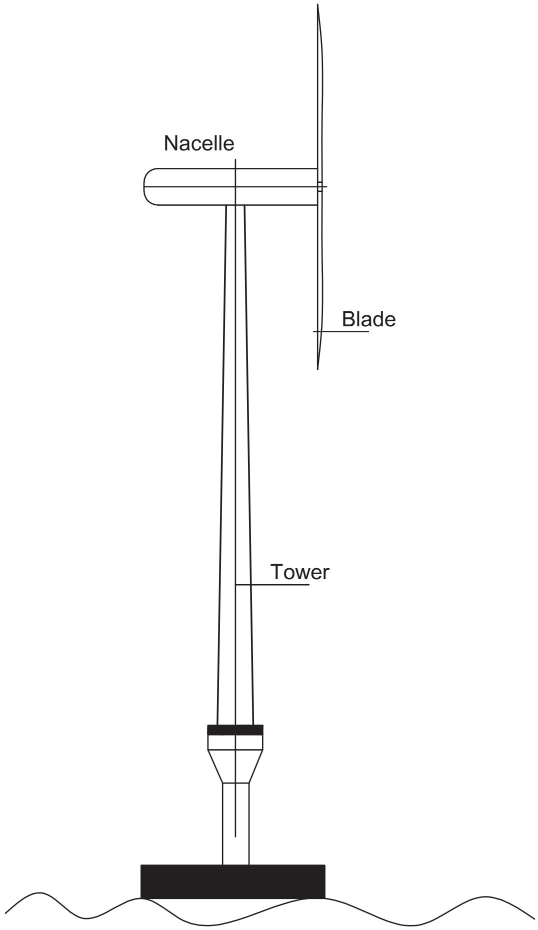

Fig. 8.17 Wind turbine on the gravity-base foundation.

8.4.3.4 Floating Structures

Floating structures are especially competitive at large water depths where the depth makes the conventional bottom-supported structures non-competitive. Detailed design guideline can refer to DNV-OS-J103," Design of Floating Wind Turbine Structures" [8].

Fig. 8.18 shows a floating form using space frame style floater. Fig. 8.19 gives the other two examples.

8.4.3.5 Jacket Foundations for Offshore Wind Structure

As shown in Fig. 8.20, jacket foundation uses four-legged jackets to support the OWTs, which can support larger OWTs such as 6 MW turbines. Jacket

Fig. 8.18 Floating wind turbines. *(This file is licensed under the Creative Commons Attribution-Share Alike 3.0 Unported license, https://commons.wikimedia.org/wiki/File:Diagram_of_Principle_Power%27s_WindFloat.jpg.)*

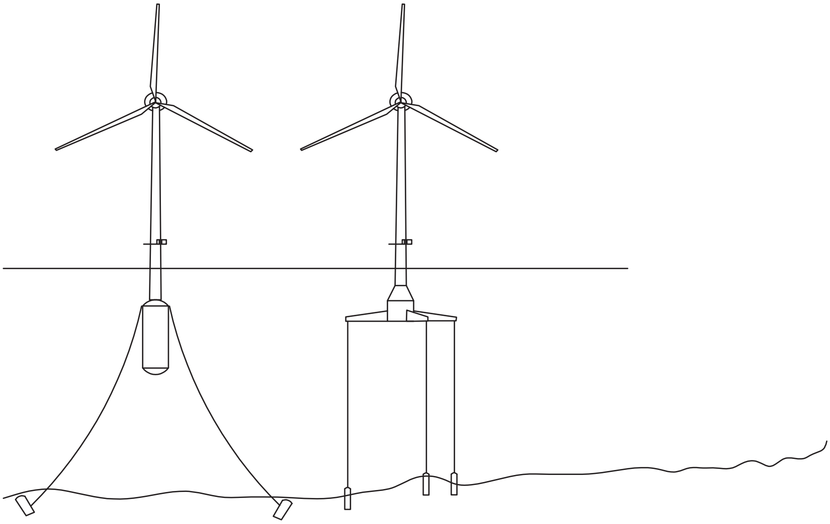

Fig. 8.19 Floating wind turbines examples.

Fig. 8.20 Steel jackets on a barge. *(This file is made available under the Creative Commons CC0 1.0 Universal Public Domain Dedication, https://commons.wikimedia.org/wiki/File:Steel_jackets_on_a_barge_(1).jpg.)*

foundations provide a solution for foundations in offshore wind farms in water depths of 35 m and beyond which is less risky, less expensive, and more reliable than monopiles and gravity-base foundations [9].

8.5 MAJOR DESIGN CONSIDERATION AND DESIGN GUIDELINES FOR OFFSHORE STRUCTURES

Different to conventional structures, offshore structures are subject to very harsh marine environments. Their design lifetimes are normally 25 years or more. Therefore, some important design considerations include

- peak loads created by hurricane wind and waves and
- fatigue loads generated by waves over the platform lifetime and the motion of the platform.

The platforms are sometimes subjected to strong currents which create loads and vortex shedding. Therefore, in special design procedure needs to be followed, to ensure the fully understanding of the loads on the platforms or OWTs and to reduce the potential collapse of this type of structure. Placing heavy structures on the seabed also requires a thorough investigation of the soil characteristics, as well as whether it should be piled.

8.5.1 Design Guidance for Offshore Structures

There are several design guidelines for offshore structures. A few recommended standards are:

1. BSI BS-6235: 1982 Code of Practice for Fixed Offshore Structures [10].
2. DNV-OS-C101: Design of offshore Steel Structures, General (LRFD Method) [4].
3. DoE Offshore installation: Guidance on Design and Construction, 1990 [11].
4. NORSOK N-004: Design of steel structures, 2013 [12].

"Guidance notes on Accidental load analysis and design for offshore structures" by ABS [13] is the major guidance on design offshore structure against accidental loads, which will be explained in the latter section. The Steel Construction Institute, Health and Safety Executive, issued a document [14] "topsides structures as part of explosion risk reduction methods" for the Health and Safety Executive. The report introduced the method for reassessment and strengthening methods for blast walls, decks, and floors of offshore oil platform.

Reader can also refer to the below mentioned relevant design guidance:

- API RP 14 J Recommended Practice for Design and Hazards Analysis for Offshore Production Facilities, 2007.
- ISO 19901-3 Petroleum and natural gas industries—Specific requirements for offshore structures—Part 3: Topsides structure, 2010.
- API RP 2FB Recommended Practice for the Design of Offshore Facilities against Fire and Blast Loading, 2006.
- API RP 2A—WSD Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms—Working Stress Design: Section 18, Fire, Blast, and Accidental Loading, 2008.
- ABS (1) Guidance Notes on Risk Assessment Applications for the Marine and Offshore Oil and Gas Industries, 2000.
- ABS (2) Guidance Notes on Review and Approval of Novel Concepts, 2003
- ABS (3) Guide for Risk Evaluations for the Classification of Marine-Related Facilities, 2003.

Beside these codes, there are several design guidelines for buildings that can be referred to, as the basic design principles are similar. Among them, UFC 3-340-02 [15], is the one of the most detailed design guidance in US to introduce the design of building against blast loading. FEMA 427 [16] provides the design measurements to reduce physical damage to the structural and nonstructural components of building and related infrastructures during conventional bomb attacks, as well as attacks using chemical, biological, and CBR agents.

The UK SCI publication 244 [17] provides guidance on the design of commercial and public buildings where there is a requirement to provide protection against the effects of explosions caused by the detonation of explosives. A philosophy for the design of buildings to reduce the effects of attack is introduced and a design procedure is proposed. The robustness of buildings and the prevention of disproportionate collapse are also discussed.

8.5.2 Summary of Design Loadings

In design an offshore structure, below loadings need to be considered:

1. Environmental loads: such as wave load, wind loads, ice loads, water level load, and earthquake loads. DNV-RP-C205, ENVIRONMENTAL CONDITIONS AND ENVIRONMENTAL LOADS [18] gives detailed instruction on different environmental loads.
2. Accidental load: impact such as ship and airplane, blast, fire, and dropped objects. American Bureau of Shipping gives detailed guidance on accidental load in Ref. [13] "Guidance notes on Accidental load analysis and design for offshore structures."
3. Dead load.
4. Live load.
5. Fatigue spectra.
6. Temporary conditions: Transportation and lifting loads, installation, removal.

8.5.3 Dead Load

For offshore structure, the dead load needs to be considered are the weight of the platform, tower, blade, and equipment, helicopter platform, etc.

8.5.4 Live Load

The live load consists of personnel and equipment, oil and gas storage, helicopter, etc.

8.5.5 Wave Load

Wave loading is the most important type of environmental loadings on offshore structures. They are designed with maximum load occurrence frequencies taking into account both 50 and 100 years wave and weather scenarios, so that a maximum level of safety is reached.

When analyzing the wave loading, as required by Offshore Standard DNV-OS-J101 [3], "For calculation of wave loads, a recognized wave theory with due consideration of the water depth and of the range of validity of the theory should be used. Methods for wave load prediction shall be applied that properly account for the size, shape and type of structure."

Therefore, in this section, the relevant theories will be discussed.

8.5.5.1 Wave Theory

As mentioned in Ref. [3], in the analysis of wave structure interactions, "a recognized wave theory with due consideration of the water depth and of the range of validity of the theory should be used." In this section, different wave theories will be explained .

8.5.5.1.1 Regular Linear Waves

In this theory, the regular waves can be defined as.

$$\eta(x, t) = \frac{H}{2}\cos(kx - \omega t) \tag{8.1}$$

where

H is the wave height

T is the wave period, and

L is the wavelength.

8.5.5.1.2 Airy Wave Theory

The theory describes which often referred as linear wave theory, it gives a linearized description of the propagation of the flow of gravity waves on the water surface using the potential flow approach [19].

Airy wave theory gives the profile dynamic pressure and particle velocities and acceleration in linear regular harmonic waves which is used in locations where the depth of the water is much greater than the wavelength and the wave height of the waves. Small amplitude waves are considered in two dimensions x and z directions, where the z-axis starts at the mean water level. Then the mean water level is assumed to be uniform with fluid flow is inviscid, incompressible, and irrational. The accuracy of the theory is sufficient for the application of offshore structures in shallow waters loaded by random waves and can even be used for second-order nonlinear properties of waves.

8.5.5.2 Morison's Equation

When analyzing the wave structure interactions [3] requires that:

> *For slender structures, such as jacket structure components and monopile structures, Morison's equation can be applied to calculate the wave loads. For large volume structures, for which the wave kinematics is disturbed by the presence of the structure, wave diffraction analysis shall be performed to determine local (pressure force) and global wave loads. For floating structures wave radiation forces must be included. Both viscous effects and potential flow effects may be important in determining the wave-induced loads on a wind turbine support structure. Wave diffraction and radiation are included in the potential flow effects.*

Therefore, the Morison's equation is used to calculate the wave loading for slender structures. Morison's equation is commonly applied when $D/L < 0.2$, where D is the diameter of the member and L is the wavelength. The force in the direction of wave propagation on a segment of a cylindrical object submerged in water can be worked out using the Morison's equation, where the horizontal force per unit length on the vertical member is the sum of the drag and inertia force components. Based on Ref. [3], the horizontal force on a vertical element dz of the structure at level z is expressed as.

$$dF = dF_M + dF_D = C_M \rho \pi \frac{D^2}{4} \ddot{x} dz + C_D \rho \frac{D}{2} |\dot{x}| \dot{x} dz \tag{8.2}$$

where

dF is the horizontal force,
dF_M is the inertial force,
dF_D is the drag force,
C_M is the inertial coefficient,
C_D is the drag coefficient,
D is the diameter of the cylinder,
ρ is the density of the water, and
z is the level measured from the still water level and z axis point upward.

It should be noted that Morison's equation does not account for flow history or that the instantaneous velocity vector arises as a superposition of several flow processes, it does account for some flow nonlinearity by way of drag term [20]. The drag coefficient for cylindrical components range from 0.5 to 1.2.

Inertia forces acting upon the structure are due to movement of water body onto the structure. Inertial forces can be further split into two parts; Froude-Krylov force and the added mass force. The drag and inertia coefficients are in general functions of the Reynolds number, the Keulegan–Carpenter number and the relative roughness. The coefficient also depends on the cross-sectional shape of the structure and of the orientation of the body.

8.5.6 Wind Load

Most of the offshore structures are in the area where the server whether condition will cause extreme wind loading conditions. Therefore, wind loading is another important load in the design consideration. As explained in Ref. [3] "Design of Offshore Wind Turbine Structures." A distinction should be made between normal and extreme wind conditions. Normal wind conditions are used as basis for determination of primarily fatigue loads, but also extreme loads from extrapolation of normal operation loads. Extreme wind conditions are conditions that can lead to extreme loads in the components of the wind turbines or oil platforms and in the support structure and the foundation.

"Design of Offshore Wind Turbine Structures" [3] also gives the formula to calculate the wind speed,which are briefly discussed here.

1. The mean wind speed U with averaging period T at height z above sea level as

$$U(T,z) = U_{10}\left(1 + 0.137\ \ln\frac{z}{h} - 0.047\ \ln\frac{T}{T_{10}}\right) \tag{8.3}$$

where $h = 10$ m, $T_{10} = 10$ min, and U_{10} is the 10-min mean wind speed at height h.

This expression converts mean wind speeds between different averaging periods.

2. For extreme mean wind speeds corresponding to specified return periods in excess of approximately 50 years:

$$U(T,z) = U_0\left(1 + C\ \ln\frac{z}{h}\right) \times \left(1 - 0.41 I_U(z)\ \ln\frac{T}{T_0}\right) \tag{8.4}$$

where

$h = 10$ m,

$T_0 = 1$ h, and

$T < T_0$ and where

$$C = 5.73 \times 10^{-2}\sqrt{1 + 0.15U_0}$$

and

$$I_U = 0.06 \times (1 + 0.043U_0)\left(\frac{z}{h}\right)^{-0.22}$$

where U will have the same return period as U_0.

In designing the offshore structures, the spectral density of the wind speed process expresses how the energy of the wind turbulence is distributed between different frequencies. The spectral density of the wind speed process may be represented by the Kaimal spectrum:

$$S_U(f) = {\sigma_u}^2 \frac{4\dfrac{L_k}{U_{10}}}{\left(1 + 6\dfrac{fL_k}{U_{10}}\right)^{5/3}} \tag{8.5}$$

in which f denotes frequency, and the integral scale parameter L_k is to be taken as.

$$L_k \begin{Bmatrix} 5.67z & \text{for} & z < 60\,\text{m} \\ 340.2m & \text{for} & z \geq 60\,\text{m} \end{Bmatrix}$$

where z denotes the height above the seawater level.

8.5.7 Ice Load

Some of the offshore structures are in the cold area such as North Sea or Newfound Bay, Canada. Therefore, ice load is another kind of loading consideration for consideration for the offshore structures. The load is primarily caused by laterally moving ice. Ice load are difficult to assess, it depends on the nature and quality of the ice, including the size of the ice, the age of the ice, the salinity of the ice and the temperature of the ice. Therefore, this type loading is assessed primarily on model experiments which can be reliably scaled. Detailed information about ice loads can be found in ISO 19906 “Petroleum and natural gas industries—Arctic offshore structures” [21].

8.5.8 Earthquake Loads

8.5.8.1 Wind Turbine Structures

When a wind turbine structure is to be designed for installation on a site which may be subject to an earthquake, such as in China, Japan, or California USA, the structure shall be designed to withstand the earthquake loads. Similar to the analysis method in buildings, response spectra in terms of the so-called pseudo-response spectra may be used for this purpose which can be calculated from the ground acceleration history by Duhamel's integral. When a wind turbine structure is to be installed in areas which may be subject to tsunamis set up by earthquakes, the load effect of the tsunamis on the structure shall be considered.

In analyzing the wind turbine structure under earthquake, it usually suffices to reduce the analysis in two horizontal directions to an analysis in one horizontal direction, due to the symmetry of the dynamic system. If vertical earthquake need to be considered for certain countries such as Japan, the buckling in the tower caused by the vertical acceleration needs to be checked in the design. Because there is likely dynamics involved with the vertical motion, the tower may be analyzed with respect to buckling for the load induced by the maximum vertical acceleration caused by the earthquake. The horizontal motion may also cause local buckling [22] due to bending, and the buckling analysis for the vertical motion may then not be relevant.

8.5.8.2 Oil Platform

For earthquake-resistant design of oil platform, the horizontal ground motion in both directions should be considered. In addition, vertical earthquake need to be considered if the site is inside the regions with high vertical seismic activities.

For offshore oil platforms, earthquake resistant design with a very low probability of collapse must be secured. So, the ductility earthquake level (level where progressive collapse shall be avoided) should be selected [23].

8.5.9 Accidental Loads

When designing an offshore structure, accidental loads that need to be considered are:

- dropped objects,
- collision impact,
- explosions,
- fire,
- change of intended pressure difference,
- load from rare, large breaking wave, and
- accidental impact from vessel, helicopter or other objects.

"Design of Offshore Wind Turbine Structures" Ref. [3] recommended that relevant accidental loads should be determined on the basis of an assessment and relevant experiences.

"Guidance notes on Accidental Load Analysis and Design for Offshore Structures" Ref. [13] gives detailed guidance on how to design the offshore structure against accidental loadings.

It states that: "The purpose of assessing accidental loadings is to understand the extent of initial damage and verify that the accident does not escalate in terms of personnel health and safety, environmental concerns, or facility damage."

American Bureau of Shipping [13] gives the procedure of making hazard risk assessment for:

- ship collision hazards,
- dropped object hazards,
- fire hazards, and
- blast hazards.

Fig. 8.21 shows the three distinct activities in the accidental hazard evaluation for the offshore structures.

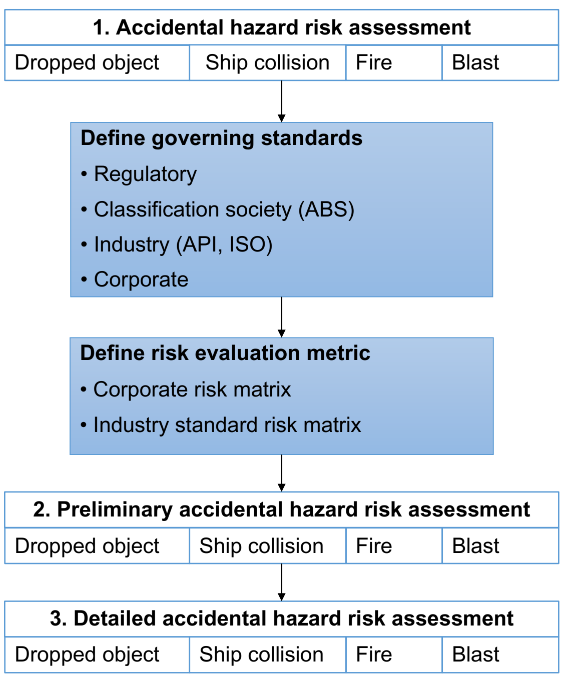

Fig. 8.21 Hazard evaluation process. *(Reproduced based on reference Guidance Notes on Accidental Load Analysis and Design for Offshore Structures, American Bureau of Shipping, Incorporated by Act of Legislature of the State of New York 1862, 2013.)*

8.5.9.1 Accidental Loads From Ship Collision

Ship collision may happen to offshore structures such as oil platform or wind turbines. As required by American Bureau of Shipping [13], the characteristic impact load shall be taken as the impact load caused by unintended collision by the maximum authorized service vessel in daily operation. For this purpose, the service vessel shall be assumed to be drifting laterally and the speed of the drifting vessel shall be assessed. The speed shall not be assumed <2.0 m/s. Effects of added mass shall be included. Effects of the maximum authorized service vessel shall be considered.

There are two methods to assess the collision of the ship, one is using the formula recommended by American Bureau of Shipping [13], this method is suitable for a simple estimation of the ship collision. Another is to perform a nonlinear dynamic finite element analysis, which gives more accurate assessment result.

8.5.9.1.1 Ship Collision Assessment

American Bureau of Shipping [13] gives the formula for the calculation of the ship collision energy balance:

$$E_{KE,\,init} = E_{ext} + E_{\mathrm{int}} \tag{8.6}$$

where

$E_{KE,init}$ is the initial colliding vessel kinetic energy,
E_{ext} is the external mechanical energy, and
E_{int} is the internal mechanical energy.
$E_{KE,init} = E_{ext} + E_{\mathrm{int}}$ Eq. (8.1) can be further derived as.

$$E_{KE,\,init} - \left(E_{KE,f} + E_{KE,\nu}\right) = E_{SE,f} + E_{SE,\nu} \tag{8.7}$$

where

$E_{KE,f}$ is facility kinetic energy after contact,
$E_{KE,f}$ is colliding vessel kinetic energy after contact,
$E_{SE,f}$ is facility strain energy, and
$E_{SE,f}$ is colliding vessel strain energy.

8.5.9.1.2 Nonlinear Dynamic Finite Element Analysis

Nonlinear finite element analysis can be used to assess the internal mechanics of the collision event as it can capture failure mechanisms a ship collision event by consideration of the subsequent stress state during the event. As required by American Bureau of Shipping [13], a comprehensive model including such as material yielding, large deformations of plate and stiffener components, connection failures, and material rupture should be built. The model should be able to capture not only the structural deformations but also the changing contact area between the bodies and the desired failure modes. The dynamic simulation will be able to trace the whole collision process from its initial contact through to the termination of the striking vessel's velocity.

8.5.9.2 Accidental Load From Fire

Fire is one of the typical incidents for offshore oil platforms. Two types of fires are normally considered for offshore platforms: pool fire and jet fire:

- The pool fire develops when liquid inventory (e.g., liquid hydrocarbon) released on a deck forms a pool.
- Jet fire is a high-pressure release of gas or liquid hydrocarbons forms a jet that is subsequently ignited.

As explained in Chapter 2 "fire safety design for tall buildings", thermal loads will increase the temperature of engulfed structural members as well as non-engulfed members near through heat transferring such as radiation, convection, and/or conduction. Owing to thermal load, the structural members will experience degradation in both strength and stiffness as a function of exposure time. The degradation of individual structures can then result in the entire structural system failing to meet its service requirements, ultimately incur either a local or global failure.

8.5.9.2.1 Acceptance Criteria

American Bureau of Shipping [13] requires that the performance of the following shall be considered in the acceptance criteria for designing offshore structures under fire:

(i) individual structural members,
(ii) safety critical elements, and
(iii) global structural system.

8.5.9.2.2 Fire Assessment Inputs

Similar to fire design of tall buildings, which has been explained in Chapter 2, it consists of four primary inputs:

(i) fire scenario definition,
(ii) structural configuration,
(iii) material Properties, and
(iv) applied loading.

8.5.9.2.3 Structural Fire Analysis Methods

There are three structural fire analysis methods recommended in American Bureau of Shipping [13], which are zone method; linear-elastic method (strength-level analysis); elastic-plastic method (ductility-level analysis). All these three methods must be implemented using correspondent computer software.

Primary inputs for the linear-elastic analysis:

- structural member temperature profiles for the given fire scenario and
- structural utilization levels prior to the fire.

Primary inputs for nonlinear analysis:

- structural member temperature time histories,
- temperature-dependent stress-strain curves, and
- other temperature-dependent material properties.

It can be seen that similar analysis methods for buildings (Chapter 2) can be used for offshore structures.

8.5.9.2.4 Mitigation Methods

There are two major mitigation methods for fire load:

Improves the structural response:

Using fire protection method.

Mitigation of the fire load:

- reducing the potential inventory for the fire,
- modify the placement of equipment to reduce or eliminate potential secondary release sources or
- isolate the release from critical elements (e.g., firewalls or deck plating), and
- using active fire control systems.

8.5.9.3 Explosion or Blast Loading

Explosion or blast loading is another type of the typical incident for offshore oil platforms.

8.5.9.3.1 Acceptance Criteria

The performance of the following shall be considered in the acceptance criteria development:

- local structural members,
- global structural system, and
- critical systems that must remain functional directly following an event.

When blast is detonated, it normally will introduce a thermal load component; however, the short duration of the event is not sufficient to raise the member's temperature significantly unless a secondary fire is introduced by the explosion.

Failure modes for individual member are overstressing, buckling, formation of plastic hinges, connection failures, among others, etc. All failure modes for the structural component must meet the required response limit.

8.5.9.3.2 Blast Assessment Methods

Blast event assessments including the assessment of both the individual structural member and a global structural system. There are three different assessment methods are given in Ref. [13]:

(i) Screening analysis

This method applies an equivalent static load and evaluates the response utilizing accidental limit state design checks.

(ii) Strength-level analysis

This is a linear-elastic analysis of an equivalent static load corresponding to the blast overpressure that incorporates plastic code checks.

(iii) Ductility-level analysis

It assesses the dynamic structural response accounting for geometric and material nonlinearities. Explicit computer program checks should be performed to verify that the structural performance is within acceptable limits. This analysis assumes the structure will undergo plastic deformation and acceptance criteria would be set using deformation limits. The potential for buckling and connection capacities should also be checked. This analysis is to ensure that the structure will not collapse and that no local collapse escalates the personnel health and safety or environmental risk exposure.

8.5.9.3.3 Mitigation Methods

There are three mitigation methods:

The structural design methods placement of equipment/structures (including blast walls), ventilation, utilization of blast walls and relief panels, and utilization of proper design detailing to maximize ability of the structure to develop its full plastic capacity.

The process design method is to minimize the likelihood of explosion.

The active and passive safety measures are through the detection devices, quickly shutdown the facility to minimize the likelihood of escalation.

8.5.9.4 Dropped Objects

Objects dropped or debris falling overboard during a storm is a common incident in offshore oil platform. A dropped object may significantly impair the performance of structures and equipment. In the design, the effect of a dropped object on topsides structures and equipment should be assessed. Proper study is needed to address the dropped object trajectory and subsequent likelihood of striking additional structure and equipment as well as predicting the consequences of such subsequent impacts.

8.5.10 Fatigue and Material Degradation

In designing the offshore structures, as they are exposed to severer environmental load than the conventional structures, fatigue and material degradation is the other important considerations.

8.5.10.1 Fatigue

The reader can refer to DNV-RP-C203, "Fatigue Design of Offshore Steel Structures" for guidance on the fatigue design [24],. here, just briefly introduces the fatigue damage accumulation formula from Ref. [24]

There are different ways of fatigue analysis, they are primarily based on the S-N curve (Fig. 8.22). The S-N curves are obtained from fatigue tests. The design S-N curves which follows are based on the mean-minus-two-standard-deviation curves for relevant experimental

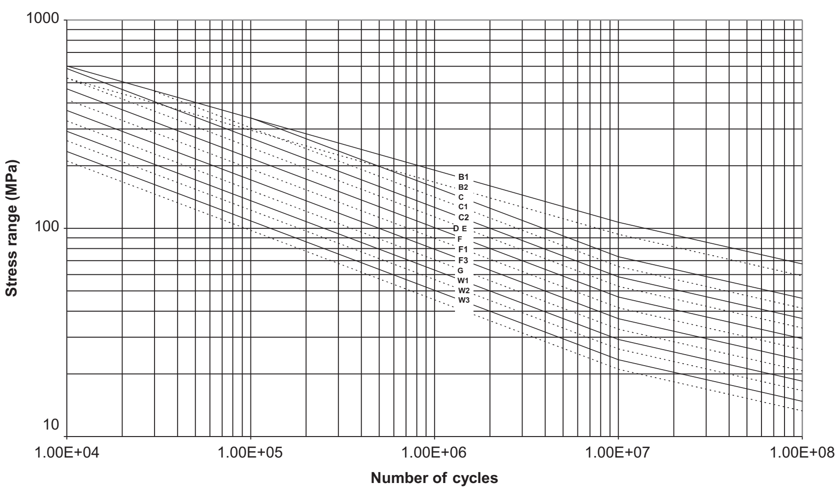

Fig. 8.22 S—N curves in air *Reproduced based on Figure 2.7 of DNV-RP-C203 "Fatigue Design of Offshore Steel Structures" [24].*

data. The S-N curves are thus associated with a 97.7% probability of survival.

The fatigue life can be calculated based on the S-N fatigue approach under the assumption of the linear cumulative damage, using the below formula:

$$D = \sum_{i=1}^{k} \frac{n_i}{N_i} = \frac{1}{\bar{a}} \sum_{i=1}^{k} n_i \cdot (\Delta\sigma_i)^m \leq \eta$$

where

D is the accumulated fatigue damage,
$\bar{a}$ intercept of the design S-N curve with the log N axis,
m is the negative inverse slope of the S-N curve,
k is the number of stress blocks,
n_i is the number of stress cycles in stress block i, and
N_i is the number of cycles to failure at constant stress range $\Delta\sigma_i$.

8.5.10.2 Material Degradation

The degradation of a component can take place externally and internally. The external damage is mainly caused by environment., DNV-RP-G101 "RISK-BASED INSPECTION OF OFFSHORE TOPSIDES STATIC MECHANICAL EQUIPMENT APPENDIX A" [25] gives details of how to make degradation assessment, which include the method such as how to calculate the probability of failure for different material based on the external degradation models.

DNV-RP-G101 [25] also gives the direction of the internal damage assessment. The internal damage mechanisms are based on combinations of material of construction, operating conditions, and fluids flowing in the pipe. It also refer to the product service codes used on offshore topside systems to give an indication of the type of fluid that can be expected to flow in the pipe.

8.5.11 Stability and Buckling Analysis of Offshore Structures

Hydrostatic and dynamic stability are needed to be considered for offshore structures under severe weather conditions. Strict procedures should be followed for weight control during construction and throughout the life of structure to avoid bucking of the support members. It should prevent the loss of the platform arising from flooding caused by limited damage to structural members.

Majority of the offshore structures such as oil platform consists of truss or space truss, which have large numbers of the tension and compression members. As these structural members are mainly subjected to axial load, therefore, the bucking analysis is essential in the design of offshore oil platform.

Similar to space structure in Chapter 6, several types of buckling which may occur in the oil platform are:

1. member buckling,
2. local bucking of certain members in certain area, and
3. global buckling.

As it is widely known, buckling analysis involves the solution of the generalized eigenvalue problem, which can be performed using design-orientated analysis programs such as SAP2000 and ETABS or StaadPro.

For detailed buckling design, refer to "design of offshore steel structures general (LRFD method)" [4] and NORSOK standards [26].

8.6 STRUCTURAL ANALYSIS OF OFFSHORE STRUCTURES (MODELING EXAMPLE OF AN OIL PLATFORM USING SAP2000)

8.6.1 Analysis Software

The structural analysis of offshore structures should be carried out using a commercial software, as it is will be difficult to analyze it manually due to the complicate loading scenarios and their complex structural system. There are several analysis packages available for offshore such as Staad_Pro, SAP2000 and Bentley. Staad_Pro can perform nonlinear structural analysis. Dynamic response analysis due to environmental loads, impact effects analysis and severe accidental loadings analysis.

Program SAP2000 is another option. In this section, we are going to describe how to set up a Jacket Platform model and analyze using SAP2000, the global buckling analysis will also be covered.

8.6.2 Modeling Example of Jacket Platform

In this section, a modeling example of a jacket platform will be presented here. The geometry of the oil platform is created as a 3D model in AutoCAD. As shown in Fig. 8.23, the model is imported into SAP2000. To simplify the analysis, in the model, only the jacket and the platform are modeled, the other components such as crane, helicopter pad, and staff roof are not modeled, but converted into correspondent loads applied on the structure.

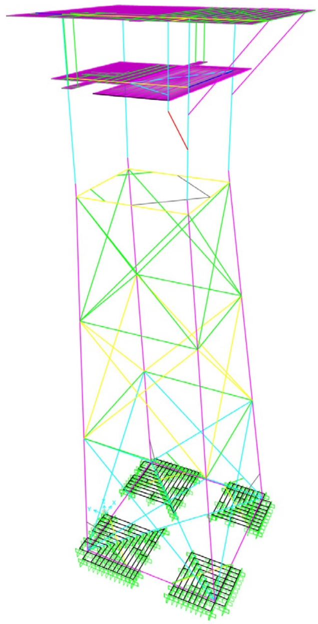

Fig. 8.23 SAP2000 model for jacket platform. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

8.6.3 Setting Up Section Properties and Support Conditions

The proper section properties need to be defined as shown in Fig. 8.24, then the support conditions.

8.6.4 Setting Up Wave Loading

In SAP2000, choose *Define->load Patterns*, in the pop up window (Fig. 8.25), further defining the wave load, Choose code API WSD2000.

When clicking on *Modify Lateral Load Pattern*, in the pop up window (Fig. 8.26), the readers can further define the wave load parameters.

In Fig. 8.26, under *Wave Characteristics* choose *modify/show,* a new window pops up (Fig. 8.27), one can define the wave characteristic and chose the suitable wave theories.

If you choose *Show Wave Plot* from the window shown in Fig. 8.26, you can also check the wave profile (Fig. 8.28).

You can also define the Current, Marine Growth, Drag and Inertia Coefficients or Wave Wind, as shown in Fig. 8.29, it defines the wave wind.

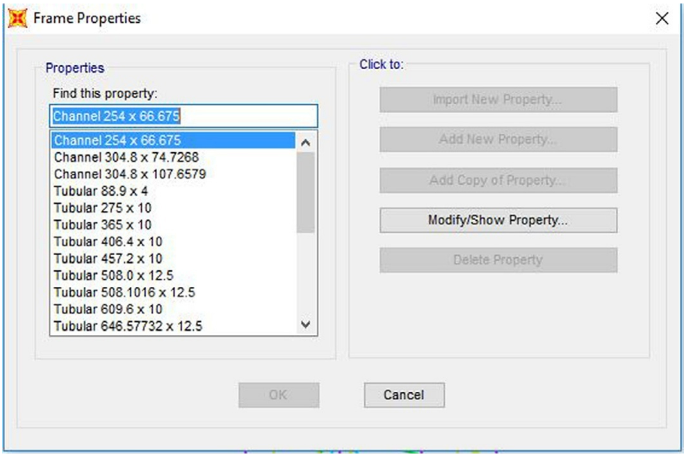

Fig. 8.24 Defining frame sections. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

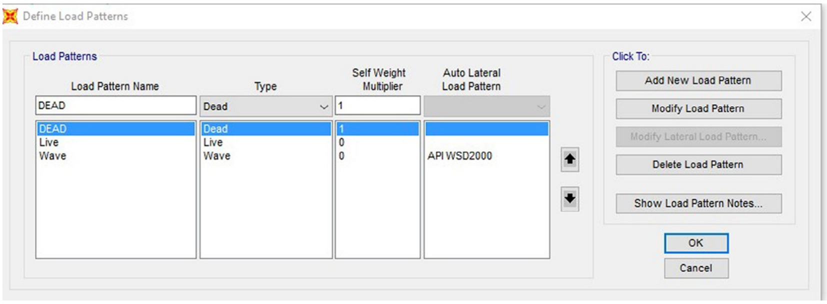

Fig. 8.25 Defining the load pattern. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

8.6.5 Setting Up Other Loads

You can also set up other loads such as earthquake and dead or live load accordingly. For fire and blast load, one has to choose a general purpose software or a special software to perform the analysis.

8.6.6 Running Analysis and Getting Result

When everything is set up, the reader can start to run the analysis and get the required results.

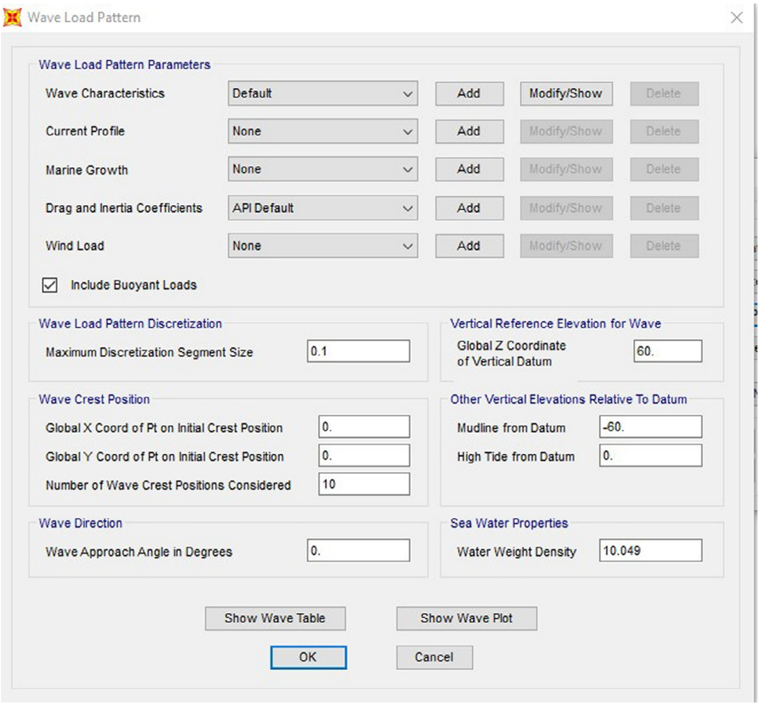

Fig. 8.26 Defining wave load pattern. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

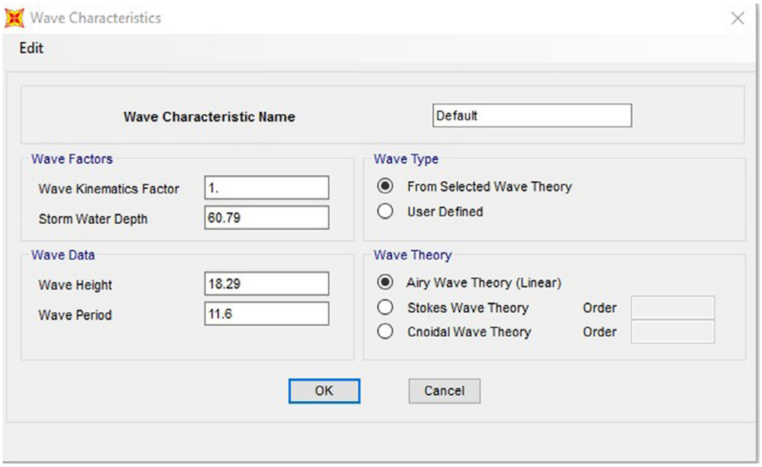

Fig. 8.27 Defining Wave Characteristic. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

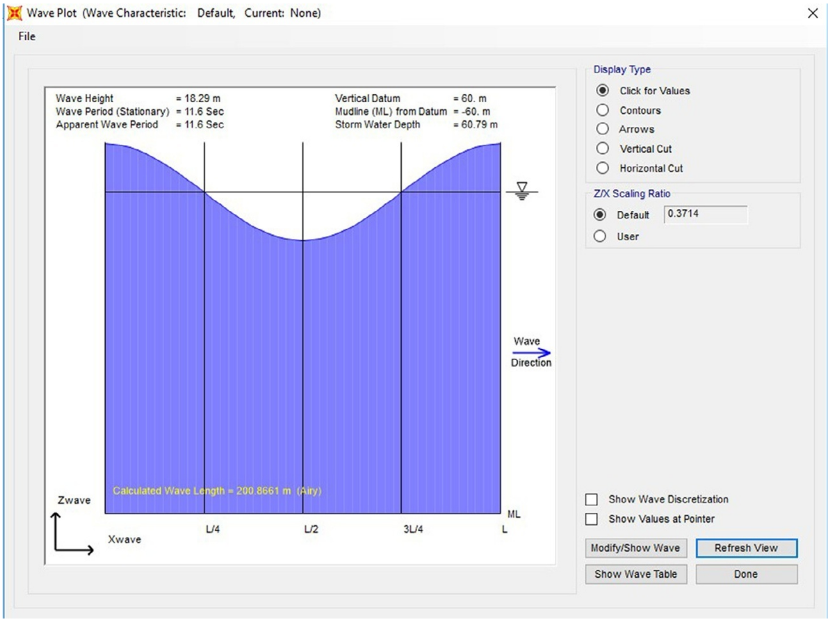

Fig. 8.28 Wave profile. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

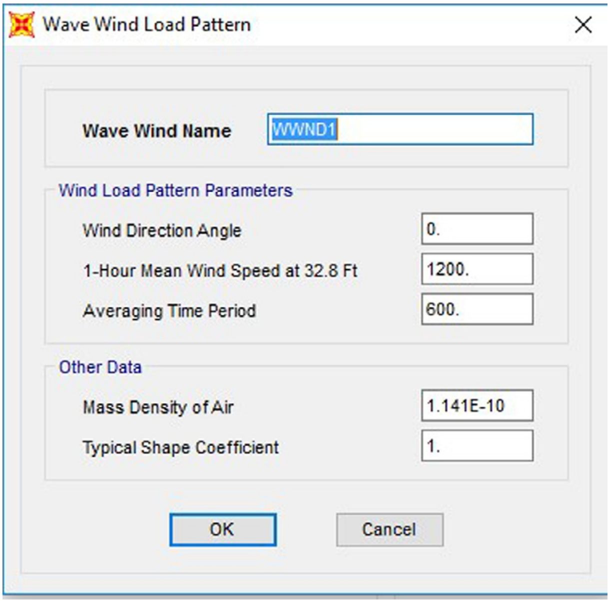

Fig. 8.29 Defining wave wind parameters. *(SAP2000 screenshot reprinted with permission of Computer and Structures.)*

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INDEX

Note: Page numbers followed by *f* indicate figures, and *t* indicate tables.

C