中国中央电视台
China Central Television Ltd.

中国中央电视台新台址
China Central Television New Headquarter

建筑结构超限设计可行性报告
Feasibility Study for Building Structure Exceeding Code Limited

CCTV Tower

DRAFT 1
草稿 1

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1. 引言

荷兰大都会建筑事务所/奥雅纳工程顾问/华东建筑设计研究院有限公司 (ECADI) 受甲方中国中央电视台委任为中央电视台新台址提供顾问服务。奥雅纳工程顾问将负责两座主塔楼的结构设计至扩初阶段。ECADI 为国内设计单位，在前阶段提供建议，在扩初阶段后负责施工图。

1.1 项目概况

新台址建设工程位于北京市朝阳区东三环中路 32 号（原为北京市汽车摩托车联合制造公司光华路厂址），地处东三环路以东、光华路以北、朝阳路以南，在北京市中央商务区（CBD）规划范围内。

中央电视台新台址用地面积总计 187,000 平方米，总建筑面积约 55 万平方米，最高建筑约 230 米，工程建筑安装总投资约 50 亿元人民币（约计 6 亿美元）。

中央电视台新台址工程包括：

(1) 一幢 230 米高的中央大楼

(2) 一幢 150 米高的电视文化中心

(3) 一幢警卫楼

(4) 一幢多媒体外景地

各楼于地下室车库连为一体通。

1. INTRODUCTION

Office for Metropolitan Architecture (OMA)/ Arup Group/ECADI were appointed by China Central Television as the consultant team to design the new CCTV headquarter. Arup will be responsible to the design of two main towers to Extended Preliminary Stage. ECADI will act as the China Local Design Institution, before the EPD stage to provide advice on design, and responsible for the construction drawings after the EPD.

1.1 Outline of the Project

The new CCTV development is located in the Central Business District (CBD) Beijing, No. 23 of East Third Ring Road, East Chao Yang Qu (Originally the Beijing Jeep Factory). The site is on the East side of the East Third Ring Road, North of Guang Hua Road and south of Chao Yang Road.

The new CCTV development comprises of 187,000m² site area, which provides a total of 550,000 m² GFA. The tallest tower would be around 230m high and the estimated construction cost would be around 500M RMB (or 60M US).

The development includes:

(1) a 230m tall CCTV main Tower

(2) a 150m tall TVCC tower

(3) a Security Building

(4) an open area for external feature.

All areas will be linked by a underground car parking basement.

2. 设计规范

本发展项目将按中国国家已颁布之各规范及北京地方规范进行设计，并同时参考规范修改送审稿，具体如下：

(1)	建筑结构设计术语和符号标准	GB/T50083-97
(2)	建筑抗震设防分类标准	GB50223-95
(3)	工程结构设计基本术语和通用符号	GBJ132-90
(4)	建筑结构可靠度设计统一标准	GB 50068-2001
(5)	建筑结构荷载规范	GB50009-2001
(6)	建筑抗震设计规范	GB50011-2001
(7)	混凝土结构设计规范	GBJ50010-2002
(8)	钢结构设计规范	GBJ17-88
(9)	高层民用建筑钢结构技术规程	JGJ99-98
(10)	高层建筑箱形与筏形基础技术规范	JGJ6-99
(11)	建筑桩基技术规范	JGJ94-94
(12)	北京地区建筑地基基础勘察设计规范	DBJ01-501-92
(13)	高层建筑混凝土结构技术规程	JGJ3-2002
(14)	人民防空地下室设计规范	GB 50038-94
(15)	地下工程防水技术规范	GB 50108-2001
(16)	建筑地基基础设计规范	GB50007-2002
(17)	型钢混凝土组合结构技术规程	JGJ138-2001

如结构设计部分不包括在中国国家规范之内，则参考

(1)	美国 – 国际建筑规范	IBC-2000
(2)	美国 – 统一建筑规范	UBC-1997
(3)	欧洲规范 8	Eurcode 8
(4)	美国 – 房屋抗震加固设计标准及注释	FEMA356
(5)	NEHRP 新建建筑及结构抗震设防条例建议	NEHRP 1997

2. DESIGN STANDARDS

The design of the proposed development will comply the following current code of practices in China:

(1) General definitions & symbols Architectural Design	GB/T50083-97
(2) Standard for classification of seismic protection of Buildings	GB50223-95
(3) General definitions & symbols Structural Design	GBJ132-90
(4) Unified Standard for Reliability Design of Building Structures	GB 50068-2001
(5) Load Code for the Design of Building Structures	GB50009-2001
(6) Code for Seismic Design of Buildings	GB50011-2001
(7) Code for Design of Concrete Structures	GBJ50010-2002
(8) Code for Design of Steel Structures	GBJ17-88
(9) Technical Specification for Steel Structure of Tall Buildings	JGJ99-98
(10) Technical Specification for Box and Raft Foundation for Tall Buildings	JGJ6-99
(11) Specification for Pile Foundation of Buildings	JGJ94-94
(12) Beijing Standard – Specifications for Foundation Design	DBJ01-501-92
(13) Technical Specification for Tall Reinforced Concrete Buildings	JGJ3-2002
(14) Specification for Civil Defence Basements	GB 50038-94
(15) Technical Code for Waterproofing of Underground Works	GB 50108-2001
(16) Code for Design of Building Foundation	GB50007-2002
(17) Technical Specification for Steel Reinforced Concrete Composite Structures	JGJ138-2001

In case where elements are not covered by Chinese Standard, reference to the following standard would be made.

(1) US – International Building Code	IBC-2000
(2) US – Uniform Building Code	UBC-1997
(3) Eurcode 8	Eurcode 8
(4) Prestand and Commentary for the Seismic Rehabilitation of Buildings	FEMA356
(5) NEHRP Recommended Provisions for Seismic Regulation for New Buildings and other Structures	NEHRP 1997

3. 材料

3.1 混凝土

结构构件所选用之混凝土将不少于 C30，按 GB50010-2002 材料参数如下：

强度种类	标准值	设计值 (N/mm^2)		弹性模量
	f_{ck} (N/mm^2)	f_c	f_t	E_c (N/mm^2)
C30	20.1	14.3	1.43	3.00 x 10^4
C35	23.4	16.7	1.57	3.15 x 10^4
C40	26.8	19.1	1.71	3.25 x 10^4
C45	29.6	21.1	1.80	3.35 x 10^4
C50	32.4	23.1	1.89	3.45 x 10^4
C60	38.5	27.5	2.04	3.6 x 10^4

3.2 钢筋

钢筋(国产)材料应符合中国规定 GB50010-2002

钢筋种类	符号	直径 (mm)	标准值，f_{yk} (N/mm^2)	设计值，f_y，f_y' (N/mm^2)	弹性模量，E_s(N/mm^2)
HPB 235	ϕ	8~20	235	210/ 210	2.1x10^5
HRB335	ϕ	6~50	335	300/ 300	2.0x10^5
HRB400	ϕ	6~50	400	360/ 360	2.0x10^5

3.3 钢材

结构用钢材将采用中国之标准钢材，其设计值如下：

3.3.1 钢材的物理性能指针

弹性模量 E (N/mm^2)	剪变模量 G (N/mm^2)	线膨胀系数α（以每°C 计）	质量密度ρ（kg/m^3）
206×10^3	79×10^3	12×10^{-6}	7850

3. MATERIAL DATA

3.1 Concrete

Grade of concrete for structural components must not less than C30, according to GB50010-2002 the design parameters are as follows:

Concrete Grade	Standard Value	Design Value		Young's Modulus
	f_{ck} (N/mm²)	f_c (N/mm²)	f_t (N/mm^2)	E_c (N/mm²)
C30	20.1	14.3	1.43	3.00 x 10^4
C35	23.4	16.7	1.57	3.15 x 10^4
C40	26.8	19.1	1.71	3.25 x 10^4
C45	29.6	21.1	1.80	3.35 x 10^4
C50	32.4	23.1	1.89	3.45 x 10^4
C60	38.5	27.5	2.04	3.6 x 10^4

3.2 Reinforcement

All reinforcement complies with Chinese Standard GB50010-2002

Type	Symbol	dia (mm)	Standard Value, f_{yk}(N/mm^2)	Design Value, f_y, f_y' (N/mm^2)	Young's Modulus, E_s(N/mm^2)
HPB 235	ϕ	8~20	235	210/ 210	2.1x10^5
HRB335	ϕ	6~36	335	300/ 300	2.0x10^5
HRB400	ϕ	6~50	400	360/ 360	2.0x10^5

3.3 Structural Steel

Typically structural steel would be Chinese Standard with the mechanical properties as follow:

3.3.1 Mechanical Properties for Structural Steel

Elastic Modulus E (N/mm^2)	Shear Modulus G (N/mm^2)	Coefficient for thermal expansion α （°C）	Density ρ （kg/m^3）
206×10^3	79×10^3	12×10^{-6}	7850

3.3.2 钢材强度设计值(N/mm^2)

钢 材		抗拉、抗压和抗弯 f	抗 剪 f_v	端面承压（刨平顶紧）f_{ce}
牌 号	厚度或直径(mm)			
Q235 钢	≤16	215	125	320
	>16～40	205	120	320
	>40～60	200	115	320
	>60～100	190	110	320
Q345 钢	≤16	310	180	400
	>16～35	295	170	400
	>35～50	265	155	400
	>50～100	250	145	400
Q390 钢	≤16	350	205	415
	>16～35	335	190	415
	>35～50	315	180	415
	>50～100	295	170	415
Q420 钢	≤16	380	220	440
	>16～35	360	210	440
	>35～50	340	195	440
	>50～100	325	185	440

3.3.3 型钢钢号

所有热轧型钢按 GB/T 11263-1998

3.3.2 Structural Steel Design Strength (N/mm^2)

Steel		Tensile, Compressive and Bending f	Shear f_v	Bearing（Flat Comfort Surface）f_{ce}
Grade	Thickness or Diameter (mm)			
Q235	≤16	215	125	320
	>16～40	205	120	320
	>40～60	200	115	320
	>60～100	190	110	320
Q345	≤16	310	180	400
	>16～35	295	170	400
	>35～50	265	155	400
	>50～100	250	145	400
Q390	≤16	350	205	415
	>16～35	335	190	415
	>35～50	315	180	415
	>50～100	295	170	415
Q420	≤16	380	220	440
	>16～35	360	210	440
	>35～50	340	195	440
	>50～100	325	185	440

3.3.3 Steel Sections

For rolled steel sections, follow GB/T 11263-1998.

4. 荷载

4.1 建筑设计分类

	中央电视台主楼
建筑结构安全等级	一级
结构重要性系数	1.1
结构设计使用年限	50 年
建筑类别	乙类

4.2 楼面荷载

按照中国国家规范 GB50009-2001 选用下列之荷载标准值。

中央主楼面荷载标准值 (kN/m^2)

位置/荷载	活荷载	机电设备及吊顶	找平层	间隔墙
屋面	1.5	5.0	4.8	–
停机坪	待定	–	待定	–
A 区办公室	3.0	0.5	1.2	1.0
B 区信息中心	1.5	7.5	1.2	1.0
C 区设备室	5.0	2.0	1.2	2.5
C 区公用房	3.0	0.5	1.2	1.5
C 区消防疏散楼梯	3.5	0.5	1.2	–
机电设备房	1.5	不少于 7.5 或实际重量	1.2	–
避难层	5.0	0.5	1.2	–
幕墙				1.5

有关演播用房荷载请参考附录中结构设计依据文件。

地下室楼面荷载标准值 (kN/m^2)

位置/荷载	活荷载	机电设备	找平层	间隔墙
地面绿化区	4.0	–	覆土按 20 kN/m^3	–
消防车道/大型车道	35	–	5.0	–
停车库				
上、落货	20	0.5	2.0	–
小车	4.0	0.5	1.8	–

4.3 风荷载

北京市基本风压, 按 100 年一遇 ω_0 为 0.50 kN/m^2。风压高度变化系数根据 C 类地面粗糙度采用。

4. LOADS

4.1 Building Classification Details

	CCTV Building
Safety Category	1st Class
Importance Factor	1.1
Design Working Life	50 years
Building Class	Grade B

4.2 Floor Loads

The following live and superimposed loads based on the Chinese Code GB500009-2001 are used.

Standard Loads for CCTV Main Building (kN/m^2)

Location/Loads	Live Load	Mechanical & Ceiling	Finishes	Partitions
Roof	1.5	5.0	4.8	–
Helipad	TBC	–	TBC	–
Zone A Office	3.0	0.5	1.2	1.0
Zone B Data Center	1.5	7.5	1.2	1.0
Zone C Equipment Room	5.0	2.0	1.2	2.5
Zone C General Purpose	3.0	0.5	1.3	1.5
Zone C Fire Escape Stair	3.5	0.5	1.2	–
E&M Room	1.5	min 7.5 (or actual)	1.2	–
Refuge Area	5.0	–	1.2	–
Facade				1.5

Refer to Appendix: Structural Design Brief for specific studio loads

Standard Loads for Basement (kN/m^2)

Location/Loads	Live Load	Mechanical & Ceiling	Finishes	Partitions
Landscape Area	4.0	–	Fill = 20 KN/m³	–
EVA/HGV Access	35.0	–	5.0	–
Car Park	4.0	0.5	2.0	–
Loading Bay	20.0	0.5	2.0	–
Private Car	4.0	0.5	1.8	–

4.3 Wind Loads

The basic wind pressure, based on 100 years return period， ω_0 for Beijing area is 0.50KN/m^2. For variation of wind pressure based on elevation, degree of ground roughness used is Type C.

4.4 雪荷载

按 100 年一遇，雪荷载采用 0.45kN/m^2。

4.5 地震荷载

地震荷载以国标 GB50011-2001 为标准，参考邻近场地地震安全性评估报告的结果：

设计等级	地震烈度	50 年设计基准期超越概率	重现周期 (年)	地面最高加速度 PGA (g)
1	多遇地震	63.2%	50	0.07
2	设防烈度	10%	475	0.2
3	罕遇地震	2%	2475	0.4

<u>地震设计参数</u>

- 抗震设防烈度　8 度
  设计基本地震加速度 = 0.2g
- 建筑场地类别　III 类场地，第一组，Tg = 0.45s
- 地震反应谱 (按 GB50011-2001，5.1.1 条)－图 4.1
- 2 条天然时程波
  - 待定
  - 待定
- 人工波
  - 待安评结果
- 时程分析法中步长不宜大于 0.02 秒
- 阻尼
  - 弹性分析，阻尼比，ξ_L = 0.02 (全钢), = 0.05 (混凝土)
  - 弹塑性分析，阻尼比，ξ_{EP} = 0.05　（JGJ99-98，第五章 37 条）
- 在计算高层钢结构多遇地震力时, 周期时应乘以 0.9 折减系数以考虑非结构构件对刚度的影响

4.4 Snow Loads

Based on a return period of 100 years, snow Loads equal to 0.45kN/m^2.

4.5 Earthquake Loads

The data below is based on Chinese Code GB50011-2001 and the recommendations from the Seismic Hazard Report from adjacent site.

Reference Level	Earthquake	Probability of Occurrence in 50 yr	Return Period (yr)	Peak Ground Acceleration (g)
1	Frequent	63.2%	50	0.07
2	Design Intensity	10%	475	0.2
3	Severe	2%	2475	0.4

<u>Specific Seismic Design Parameters</u>

- Basic seismic intensity (code)　Zone 8
  Design ground acceleration = 0.2g
- Site type:　Type III, Group I, Tg = 0.45s
- Response spectrum (According to GB50011-2001, cl 5.1.5)-Figure 4.1
- Real time history curves used for analysis
  - TBC
  - TBC
- Artificial Time History Curve
  - Awaiting seismic Hazard Study
- For all Time History, time step should not be more than 0.02s.
- Damping
  - For linear analysis, ξ_L = 0.02 (Steel), = 0.05 (Concrete)
  - For nonlinear elasto-plastic analysis, ξ_{EP}= 0.05　(JGJ99-98, cl 5:37)
- In calculating the Level 1 seismic force of high-rise steel structures, the Chinese Code requires that the periods should be multiplied by a reduction factor (0.9) to account for the stiffness due to non-structural elements.

4.5.1 楼层水平地震剪力

结构抗震验算时，任一楼层的水平地震剪力应符合下式要求：

$$V_{Eki} \geq \lambda \sum_{j=i}^{n} G_j$$

式中：

V_{Eki}—第 i 层对应于水平地震作用标准值的楼层剪力；

λ—剪力系数，对于扭转效应明显或基本周期小于 3.5s 的结构取为 0.032，对 于基本周期大于 5.0s 的结构取为 0.024，对于基本周期介于 3.5s 和 5.0s 之间的结构，可插入取值；对竖向不规则结构的薄弱层，尚应乘以 1.15 的增大系数；

G_j—第 j 层的重力荷载代表值。

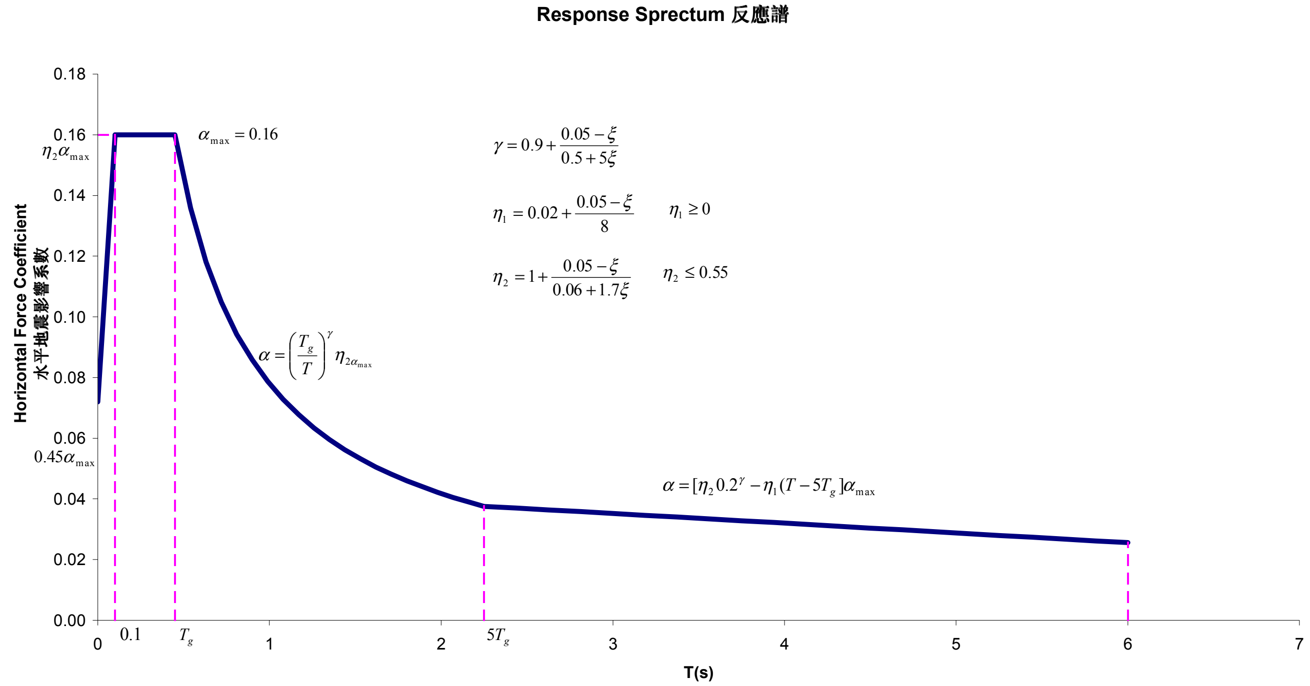


Figure 4.1 Response Spectrum
图 4.1 地震影响系数曲线

4.5.1 Horizontal Seismic Story Shear

Under seismic load condition, the shear force in each story should satisfy the following:

$$V_{Eki} \geq \lambda \sum_{j=i}^{n} G_j$$

where

V_{Eki}— The standard horizontal seismic shear force in i[th] story

λ — Shear Force Coefficient, for structures with torsional behaviour or fundamental period less than 3.5s equals to 0.032. For structures with fundamental period larger than 5s use 0.024, for structures with period between 3.5s and 5s, use linear interpolation; for soft story structure with vertical irregularity, a 1.15 factor should also be applied.

G_j — The standard vertical load in j[th] story.

4.5.2 竖向地震作用

长悬臂结构及大跨度结构的竖向地震作用标准值，取结构、构件重力荷载代表值之10%。

参考 JGJ3-2002 第 10.5.2 条对连体结构之要求，CCTV 主楼亦应按下列方程考虑竖向地震作用：

$$F_{vi} = \frac{G_i H_i}{\sum G_j H_j} \alpha^v_{\max} G^v_{eq}$$

式中	F_{vi}	=	质点 i 的竖向地震作用标准值
	G_i, G_i	=	分别为质点 i、j 的等效重力荷载代表
	H_i, H_j	=	分别为质点 i、j 的计算高度
	$\alpha^v_{\max}$	=	竖向地震影响系数的最大值或 0.65 $\alpha_{\max}$
	G^v_{eq}	=	结构等效重力荷载或 0.75 G_e

楼层的竖向地震作用效应可按各构件承受的重力荷载代表值比例分配，并宜乘以增大系数 1.5。

4.5.3 地震的性能目标

因中央电视台主楼的外形较为特别，故将以基于性能的设计方法来证明结构系统在罕遇地震下的性能。基于性能的设计方法是通过复杂的非线性动力分析软件对结构进行分析，以证明结构可以达到预定的性能目标。对各结构构件会进行充份的研究以对结构的整体性能得出定性的结论。

以下针对中央电视台主楼在不同水平的地震作用下的设计提出了性能指标。这些指标作为设计的目标，代表了结构在相关地震设计等级下的预期抗震性能。

4.5.2 Vertical Seismic Effect

The standard load for long cantilever structure and long span structures under vertical seismic effect should be taken as 10%of the standard value of gravity load.

With reference to JGJ 3-2002 clause 10.5.2, vertical seismic effects should be considered using the following equation:

$$F_{vi} = \frac{G_i H_i}{\sum G_j H_j} \alpha^v_{\max} G^v_{eq}$$

where	F_{vi}	=	Vertical seismic load standard value
	G_i, G_i	=	Vertical load representative value for mass i and j respectively
	H_i, H_j	=	Height of mass i and j respectively
	$\alpha^v_{\max}$	=	Maximum vertical seismic load coefficient or 0.65 $\alpha_{\max}$
	G^v_{eq}	=	Structure vertical load representative value or 0.75 G_e

The vertical seismic load at each storey should be calculated in accordance with the structural vertical load representative value storey, a factor of 1.5 should be applied.

4.5.3 Seismic Performance Objectives

Give the atypical nature of the CCTV tower configuration it is proposed that a performance based design (PBD) approach is adopted to justify the structural systems adopted especially under extreme seismic events. The PBD will be based on a first principles justification of the design using complex non-linear dynamic analysis software to demonstrate certain target performance objectives are achieved. Various elements of the design will be studied in substantial detail so that overall statements in terms of buildings qualitative performance can be made.

The following general performance objects are proposed for the design of the CCTV tower to the various levels of seismic events. These objectives serve as targets for the design, representing predicted seismic performance with respect to associated earthquake design levels.

地震烈度 (设计等级)	1 = 多遇地震 (T = 50 年)	2 = 设防烈度 (T = 475 年)	3 = 罕遇地震 (T = 2475 年)
性能等级	没有破坏	有破坏但可修补	不可倒塌
允许层间位移	h/300	h/100	h/50
层间延性	< 1 (弹性)	有待确定	有待确定
梁性能	弹性	有待确定	有待确定
斜支撑性能	弹性	有待确定	有待确定
柱性能	弹性	有待确定	有待确定
转换结构的性能	弹性	弹性	弹性

于设防烈度及罕遇地震的设计复核中将不考虑风荷载，竖向重力 G_e将视为如下：

G_e = 恒载 + λ_L 活载 + λ_s 雪荷载

λ_L=0.5(一般情况), 0.8(藏书室、文件档案部份) λ_S=0.5

(G_e 计算中不考虑顶层的活荷载及活荷载折减)

4.5.4 双向水平地震作用的扭转效应，可按下列公式确定：

$$S_{EK} = \max\left(\sqrt{S_x^2 + (0.85S_y)^2}, \sqrt{(0.85S_x)^2 + S_y^2}\right)$$

4.6 设计荷载组合

4.6.1 多遇地震 (弹性) 的构件设计

所有结构构件的承载力应按下表要求，按以下的最不利的荷载组合进行设计。

结构设计依据文件第 4.4.1 节可作为参考。

Seismic Intensity (Reference Level)	1 = Frequent (T = 50 years)	2 = Design Intensity (T = 475 years)	3 = Severe (T = 2475 years)
Qualitative Performance Levels	No Damage	Repairable Damage	No Collapse
Allowable Story Drift Ratio	h/300	h/100	h/50
Story Ductility	< 1 (Elastic)	To be determined	To be determined
Beam Performance	Elastic	To be determined	To be determined
Brace Performance	Elastic	To be determined	To be determined
Column Performance	Elastic	To be determined	To be determined
Transfer Structure Performance	Elastic	Elastic	Elastic

In Level 2 and Level 3 design checks, wind load can be ignored and the vertical load, G_e taken to be.

G_e =Dead Load + λ_L Live Load + λ_s Snow Load

λ_L=0.5(for general area), 0.8(book store, file storage area) λ_S=0.5

(Roof's Live Load and Live Load reduction cannot apply to G_e.)

4.5.4 For torsional effect due to bi-directional horizontal seismic, the following equation can be used:

$$S_{EK} = \max\left(\sqrt{S_x^2 + (0.85S_y)^2}, \sqrt{(0.85S_x)^2 + S_y^2}\right)$$

4.6 Design Load Combination

4.6.1 Level 1 (Elastic) Elemental Design

The capacity of all structural elements should be designed according to the table below for the worse case load combination given below.

Reference should be made to section 4.4.1 of the Structural Design Brief.

设计荷载分项系数

组合	恒载 γ_G	活载 γ_{Q1}、γ_{Q2}	水平地震 γ_E	竖向地震 γ_{Ev}	风 γ_W
1. 恒载 + 活载 (恒载控制)	1.35 (1.0)	0.98 (0)			
1A. 恒载 + 活载 (活载控制)	1.20 (1.0)	1.40 (0)			
2. 恒载 + 活载 + 风 (恒载控制)	1.35 (1.0)	0.98 (0)			±1.40
2A. 恒载 + 活载 + 风 (活载控制)	1.20 (1.0)	1.26 (0)			±1.40
3. 恒载 + 风	1.35 (1.0)				±1.40
4. 恒载 + 活载 + 竖向地震	1.20 (1.0)	0.6 (0) *		±1.30	
5. 恒载 + 活载 + 风 + 水平地震	1.20 (1.0)	0.6 (0) *	±1.30		±0.28
6. 恒载 + 活载 + 风 + 水平地震 + 竖向地震	1.20 (1.0)	0.6 (0) *	±1.30	±0.50	±0.28

指号内的数值将用于荷载有利设计的组合上。

* 活荷载不折减。

Design Load Factor

Combination	Dead Load γ_G	Live Load γ_{Q1}、γ_{Q2}	Horizontal Earthquake γ_E	Vertical Earthquake γ_{Ev}	Wind Load γ_W
1. Dead and Live (DL governs)	1.35 (1.0)	0.98 (0)			
1A. Dead and Live (LL governs)	1.20 (1.0)	1.40 (0)			
2.Dead, Live and Wind (DL governs)	1.35 (1.0)	0.98 (0)			±1.40
2A. Dead, Live and Wind (LL governs)	1.20 (1.0)	1.26 (0)			±1.40
3. Dead and Wind	1.35 (1.0)				±1.40
4.Dead, Live and Vertical. Earthquake	1.20 (1.0)	0.6 (0) *		±1.30	
5.Dead, Live, Wind, and Horizontal Earthquake	1.20 (1.0)	0.6 (0) *	±1.30		±0.28
6.Dead, Live, Wind, Horizontal Earthquake and Vertical. Earthquake	1.20 (1.0)	0.6 (0) *	±1.30	±0.50	±0.28

Values in parentheses are to be taken if load is beneficial.

* Live load is assumed to be unreduced in cases marked thus.

5. 场地工程地质条件(参考初步工地勘察报告)

5.1 地形地貌

本场地地貌上为永定河冲洪积扇中下部，第四系地层厚度大于 130m。场地地形较平坦，自然地面标高在 39.00m 左右。

5.2 地层构成

拟建场地地基土层主要为永定河多次冲洪积形成的冲洪积层，在勘探深度 130m 范围内地基土层主要为粉土、粉质粘土、细中砂及卵石、砾石组成，在垂直方向上形成多次沉积韵律。

5.3 水文地质条件

根据初步钻探资料，自然地面下 30m 深度范围内主要有 3 层地下水；第一层为上层滞水，初步勘察未发现该层水；第二层为潜水，静止水位标高在+24.40 – +23.99m 之间，埋深 14.38 ~ 14.74m；第三层为承压水，静止水位标高约在 22.58m，埋深约为 16.20m。

上述各层地下水水质对混凝土结构及钢筋混凝土中的钢筋均无腐蚀性，但在干湿交替情况下对钢筋混凝土结构中的钢筋均具有弱腐蚀性。

5.4 场地抗震性能

根据建筑抗震设计规范（GB50011-2001），拟建场地的抗震设防烈度为 VIII 度，场地土类型为中软土，建筑场地类别为 III 类。根据现场标准贯入试验与剪切波速试验结果分析判定：拟建场地属非液化地基。

有关场地地震安全性评估，参见地震局发出的相关报告。

5.5 场区地层分布情况

岩土工程初步勘察所揭露的 130 米深度范围内地层，按沉积年代、成因类型可分为人工堆积层和第四系沉积层两大类，而按地层岩性及工程特性又可进一步划分为 18 个主要土层，自上而下分布情况如下表。

5. GROUND CONDITION (WITH REFERENCE TO THE PRELIMINARY SI REPORT)

5.1 Topography

The site is located at the middle and lower area of the alluvial/diluvial fan of Yongding River. The thickness of the Quaternary deposits is estimated to be more than 130m. The site is flat and the current ground level is at about +39.0m.

5.2 Geology

The soils are mainly the alluvial/diluvial sediments of the Yongding River. Within the 130m exploration depth, the subsoil is composed of silt, silty clay, fine to medium sand, cobble and gravel, among which multi sedimentary rhythms were formed vertically.

5.3 Hydrology

Based on the preliminary site investigation, there are 3 ground water layers within 30m in depth. The first one is perched water, which was not found in the preliminary site investigation; the second layer is phreatic water, which has a hydrostatic water table of +24.40 to +23.99m(14.38 to 14.74m in depth); and the third one is confined water, whose hydrostatic water table is at about +22.58m (16.20m in depth).

The above-mentioned ground water layers have no corrosion to reinforced concrete and the reinforcements inside concrete members.

5.4 Earthquake

According to the “Code for seismic design of buildings” (GB50011-2001), the seismic fortification intensity is VIII for the site. The site soil is classified as medium-soft soil and the site is classified as Construction Site Category III. Based on the results of Standard Penetration Tests (SPT) and shear wave velocity tests, the proposed site is considered to be non-liquefiable.

Regarding the site-specific seismic hazard study, refer to the report by the Seismic Bureau.

5.5 Stratigraphy

Within 130m in depth, soil layers can be classified into two main types: man-made fill and Quaternary deposits. Under the consideration of their lithologic characters and engineering properties, the soil layers can be further divided into 18 sub-layers as shown in the following table.

层序	土层名称	层底标高 (m)
1	人工填土	+37.2 – +35.2
2	粉土夹粉质粘土	+30.7 – +27.9
3	中砂夹粉细砂	+26.3 – +24.4
4	圆砾	+20.8 – +19.4
5	粉土	+14.8 – +13.3
6	卵石夹中粗砂	+6.5 – +1.3
7	粉质粘土	+0.8 – -1.6
8	卵石	-4.5 – -6.5
9	粉质粘土	-12.5 – -14.5
10	中砂	-15.4– -17.9
11	粉质粘土	-28.5 – -31.0
12	圆砾夹中砂	-42.7 – -44.4
13	粉质粘土	-49.0
14	中砂	-57.1
15	粉质粘土	-71.7
16	卵石	-80.2
17	粉质粘土	-90.9
18	圆砾	<-91.3

Layer No.	Soil description	Bottom level (m)
1	Man-made fill	+37.2 – +35.2
2	Silt with silty clay interlayer	+30.7 – +27.9
3	Medium sand with silty to fine sand interlayer	+26.3 – +24.4
4	Rounded gravel	+20.8 – +19.4
5	Silt	+14.8 – +13.3
6	Cobble with medium to coarse sand interlayer	+6.5 – +1.3
7	Silty clay	+0.8 – -1.6
8	Cobble	-4.5 – -6.5
9	Silty clay	-12.5 – -14.5
10	Medium sand	-15.4– -17.9
11	Silty clay	-28.5 – -31.0
12	Rounded gravel with medium sand interlayer	-42.7 – -44.4
13	Silty clay	-49.0
14	Medium sand	-57.1
15	Silty clay	-71.7
16	Cobble	-80.2
17	Silty clay	-90.9
18	Rounded gravel	<-91.3

6. 基础

根据初步勘察结果，塔楼基底天然地基承载力不足以承受上部荷载。因此，建议塔楼采用桩基基础。典型的桩基直径为 1200 毫米，桩端深度约 75 米深 (从地面开始)。

建筑物总沉降量可控制在 100mm 以内。桩基承载力与沉降计算中并未考虑采用后压浆。若采用后压浆，可增加桩的侧端阻力，同时降低建筑物的沉降和沉降差。

塔楼以外区域将采用一般筏式基础。塔楼与其它相连结构间之差异沉降将尽可能控制在 50 毫米内。

6.1 地基承载力特征值与土的压缩模量

根据初步勘探资料，地基承载力特征值 f_{ak} 与土的压缩模量 E_s 见下表。

土层名称	f_{ak}(kPa)	Es(p_0~$p_0+\Delta p$) (MPa)
粉土②	200～250	6～14
粉质粘土②$_1$	160～220	6～14
中砂（细中砂）③	240～280	45
粉细砂③$_1$	240～280	35
粉质粘土③$_2$	160～220	6～14
圆砾④（含砾砂④$_1$）	300～350	58
粉土⑤（粉质粘土⑤$_1$、粘土⑤$_2$）	200～280	13～33
中粗砂⑥$_1$	300～350	45
卵石⑥（圆砾⑥$_2$）	450～500	88
粉质粘土⑦（粉土⑦$_1$）	230～300	13～18
卵石⑧（中砂⑧$_1$）	500～550	101～113
粉质粘土⑨（粉土⑨$_1$）	250～340	11～33
粉砂⑨$_2$	300～350	63
中砂⑩	350～400	75
粉质粘土⑩$_2$	220～260	22～36
粉质粘土⑪（粉土⑪$_1$）	220～260	22～36
圆砾⑫	300～350	180～193
细中砂⑫$_1$	270～350	75
粉质粘土⑫$_2$	220～260	25～32
卵石⑫$_3$	400～600	180～193
粉质粘土⑬	220～260	25～32
中砂⑭（含细砂⑭$_1$）	350～400	75
粉质粘土⑮（粘土⑮$_1$、粉土⑮$_2$）	220～320	27～50
粉砂⑮$_3$	250～350	63
卵石⑯（含细砂⑯$_1$砾砂⑯$_2$）	400～600	200
粉质粘土⑰（粉土⑰$_1$、粘土⑰$_3$）	220～320	27～50
粉砂⑰$_2$	250～350	63
圆砾⑱	300～350	200

6. FOUNDATION

According to the preliminary SI report, the bearing capacity of the soil under the various high-rise towers is not sufficient to support the superstructure load requirements. Piled foundations are proposed for the all the tower structures. Typically the bored reinforced concrete piles are 1200mm in diameter and up to approximately 75m in length (measured from Ground Level).

The total average settlement of the building is estimated to be less than 100mm. Pile grouting methods are under consideration to increase the shaft friction and tip resistance of piles and to reduce the building settlement and differential settlement.

For areas outside the high-rise towers such as podium structures, piled raft foundations are proposed. The differential settlement between the tower structures and any adjacent podium/linked structures would be controlled to within 50mm.

6.1 Characteristic values of subsoil bearing capacity and compression modulus of soils

From the preliminary site investigation results, the characteristic values of subsoil bearing capacity (f_{ak}) and compression modulus of soils (E_s) are listed in the table below.

Soil description	f_{ak}(kPa)	E_s (p_0~$p_0+\Delta p$) (MPa)
Silt②	200～250	6～14
Silty clay②$_1$	160～220	6～14
Medium sand(fine to medium sand)③	240～280	45
Silty to fine sand③$_1$	240～280	35
Silty clay③$_2$	160～220	6～14
Rounded gravel④（including gravelly sand④$_1$）	300～350	58
Silt⑤（Silty clay⑤$_1$、clay⑤$_2$）	200～280	13～33
Medium to coarse sand⑥$_1$	300～350	45
Cobble⑥（Rounded gravel⑥$_2$）	450～500	88
Silty clay⑦（Silt⑦$_1$）	230～300	13～18
Cobble⑧（medium sand⑧$_1$）	500～550	101～113
Silty clay⑨（silt⑨$_1$）	250～340	11～33
Silty sand⑨$_2$	300～350	63
Medium sand⑩	350～400	75
Silty clay⑩$_2$	220～260	22～36
Silty clay⑪（silt⑪$_1$）	220～260	22～36
Rounded gravel⑫	300～350	180～193
Fine to medium sand⑫$_1$	270～350	75
Silty clay⑫$_2$	220～260	25～32
Cobble⑫$_3$	400～600	180～193
Silty clay⑬	220～260	25～32
Medium sand⑭（including fine sand⑭$_1$）	350～400	75
Silty clay⑮（clay⑮$_1$、silt⑮$_2$）	220～320	27～50
Silty sand⑮$_3$	250～350	63
Cobble⑯（including fine sand⑯$_1$ gravelly sand⑯$_2$）	400～600	200
Silty clay⑰（silt⑰$_1$、clay⑰$_3$）	220～320	27～50
silty ⑰$_2$	250～350	63
Rounded gravel⑱	300～350	200

6.2 灌注桩极限阻力标准值

根据初步勘探资料，灌注桩极限阻力标准值见下表。

土层名称	侧阻力 q_{sik}(kPa)	端阻力 q_{pK}(kPa)
圆砾④	120～135	
粉土⑤	4～78	
卵石⑥	120～135	2000～3000
中粗砂⑥$_1$	70～80	650～800
粉质粘土⑦	64～78	700～800
卵石⑧	120～135	2000～3000
粉质粘土⑨	70～78	700～800
粉砂⑨$_2$	72～80	1600～1700
中砂⑩	80～90	1600～1700
粉质粘土⑪	72～78	800～900
粉土⑪$_1$	72～80	800～900
圆砾⑫	120～135	2500～3000
细中砂⑫$_1$	72～85	1700～1900
卵石⑫$_3$	120～135	2500～3000

6.2 Standard values of shaft friction and tip resistance of subsoil for bored piles

Standard values of shaft friction and tip resistance of subsoil for bored piles are listed in the table below.

Soil description	Shaft friction q_{sik}(kPa)	Tip resistance q_{pK}(kPa)
Rounded gravel④	120～135	
Silt⑤	4～78	
Cobble⑥	120～135	2000～3000
Medium to coarse sand⑥$_1$	70～80	650～800
Silty clay⑦	64～78	700～800
Cobble⑧	120～135	2000～3000
Silty clay⑨	70～78	700～800
Silty sand⑨$_2$	72～80	1600～1700
Medium sand⑩	80～90	1600～1700
Silty clay⑪	72～78	800～900
Silt⑪$_1$	72～80	800～900
Rounded gravel⑫	120～135	2500～3000
Fine to medium sand⑫$_1$	72～85	1700～1900
Cobble⑫$_3$	120～135	2500～3000

7. 结构系统

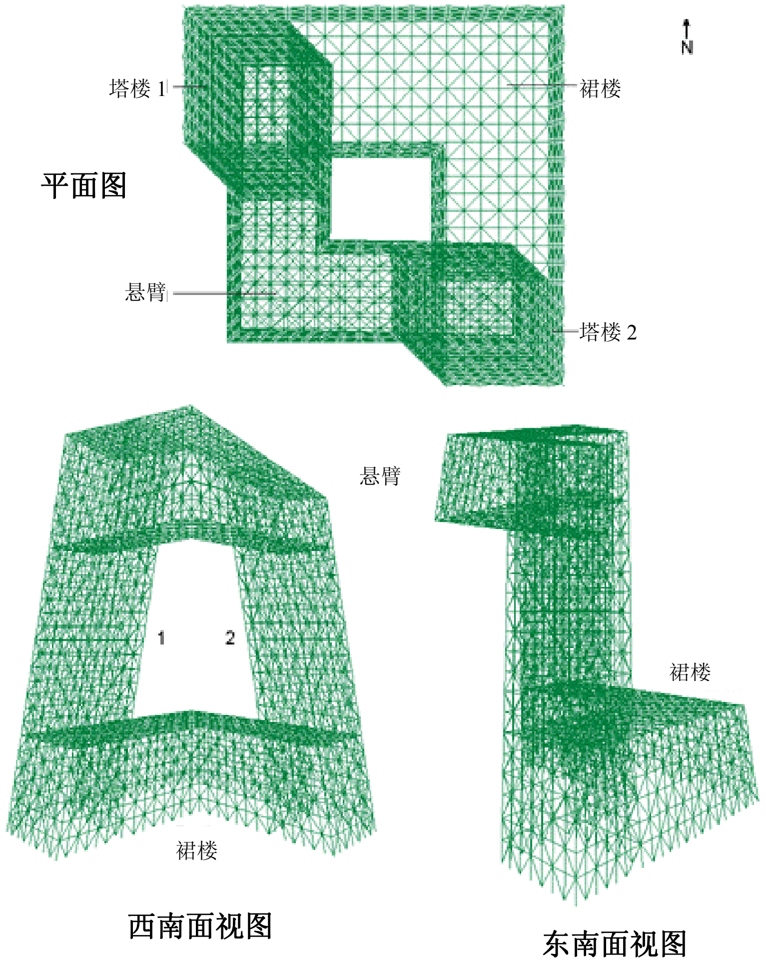


图 7.1 模型三维视图

7.1 描述

中央电视台（CCTV）主楼的最高点为 234 米，建筑面积约 40 万平方米，当中包括地下室及裙楼。其外形按建筑师图纸设计。主楼包括两座斜塔，连接两座斜塔顶部的 14 层高的悬臂结构，以及 9 层裙楼与三层地下室，占据了整个大楼的底部面积。按照国内规范，本塔楼属于乙级建筑。

主楼楼层围绕垂直筒体均以办公室用途为主，同时主楼内设有多个演播室，分别布置于主楼不同位置，整个主楼从上到下均设有用来放置建筑设施的专用楼层。

从结构角度看，最后竣工时，这两座斜塔、悬臂结构、9 层的裙楼底座及地下室均可整体看作一个完整结构系统。

7. STRUCTURAL SYSTEM

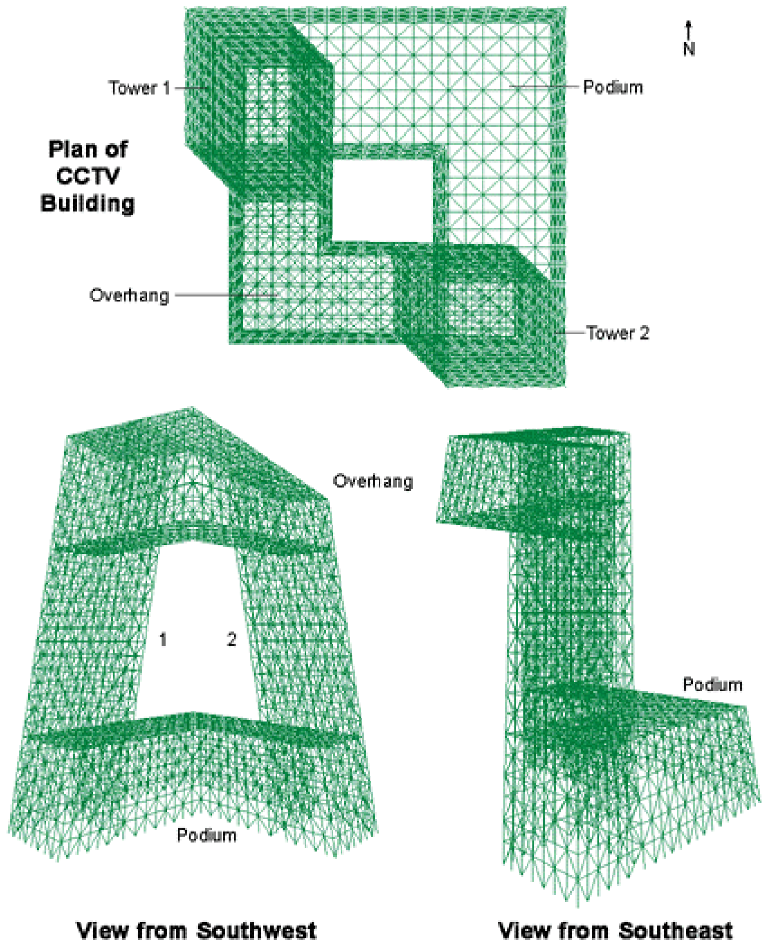


Figure 7.1 Isometric Views of the model

7.1 Description

The CCTV Tower is 234m tall at its highest point and has approximately 400,000 m² of floor area including basement and podium structures. Its geometrical definition is as per the architect's drawings. It consists of two leaning towers, which are linked together at the top via a 14-storey cantilever link element. The building has a 9-storey podium and 3 levels of basement across the entire footprint of the building site. The CCTV tower is defined as a Class B type of building to the Chinese code.

The building is predominately for office tasks arranged on regular floors planned around a vertical core. The building also consists of a large number of studios, which are located at various positions around the building. A number of dedicated building services floors are located at various levels throughout the building height.

Structurally the 2 towers, the cantilever overhang, the 9-storey podium, and basement predominantly act as one structural system in the final as built case.

7.2 侧向支撑稳定系统

由于两座塔楼本身倾斜，而悬臂位置较高，同时当承受到风和地震时将会产生侧向荷载力，所以本塔楼必须保证足够的结构稳定性，以抵抗由于重力及其他侧向荷载力作用下产生的倾覆力。

斜塔、悬臂与裙楼四周均分布了多个利用三角形状结合形成的钢结构外筒体，此结构系统能提供塔楼的整体稳定性，其中部份钢结构外筒体表面延续至筒体内部，以加强大楼转角部份的结构及保持钢结构外斜筒体的连续性。

这些三角表面由水平梁、柱及斜支撑组成。并有规律性地分布于建筑表面。斜支撑则按不同的剪力分布需求而分布于建筑表面。

该建筑的所有核心筒体都是垂直的，有助于保持各层位于主要支撑节点间的楼板稳定。在发生大地震时，这种核心筒体结构可以有助整体的建筑物稳定性。

现正考虑在塔楼的核心筒体和外钢筒的表面使用吸收能量的阻尼器。

楼板将作为横隔板，把风力以及地震的惯性力转移到主要的稳定系统上。塔楼 2 中某些楼层设有双层高的演播室，所以该处每两层提供一层楼板。

本塔楼地基基础稳定性依靠由深桩筏形地基。地库四边的钢筋混凝土墙将进一步加强塔楼基础，同时亦增加整体杆臂长度，有助抵抗塔楼颠覆。地基系统之所以这样建造，是为了保证桩不承受永久的拉力。只有在大震下部份地桩才会承受较小的拉力。

7.3 楼盖系统

这两座斜塔主要为钢结构。外筒体结构为一完全支撑框架，其中包括柱、梁及交叉支撑形成一系列的刚性面板，这外筒体结构将伸至塔楼悬臂及裙楼部份，从而承担悬臂和裙楼的重力，形成动能直接转达下层结构，从总体上保证大楼的稳定性。

大楼整个上层建筑内的楼板均用组合钢梁与及混凝土板制造。楼板的重量由外部钢筒体与及内部柱子承撑。尽管外钢筒体的结构构件在两个平面间有 6 度的倾角，然而大楼的内柱，包括中心带的内柱，却均保持垂直。

由于塔楼倾斜，同时亦考虑到楼板的深度限制在合理的范围内，故部份内柱需于塔楼内终止及换转作配合。这些内柱所承撑的重量将被转移到内筒心和外部钢筒体上。转换结构大多采用二层高的桁架结构，位置多设于适当位置如设备用房内。转换结构数目将尽量控制于最小。

悬臂内的柱子由设于悬臂底层的桁架支承。而这些悬臂底层的桁架则将重力荷载传递给周边外筒体钢结构，后者再把这些重量转移到地基上。设于悬臂底层的转换桁架深度至少为 2 层。

7.2 Lateral Stability System

The tower structures are required to provide stability for both the permanent overturning under gravity loads, due to the sloping nature of the towers and the high level cantilever overhang, and the lateral loads imposed by wind and seismic loading.

The overall building stability system is provided by a triangulated external tube surface on all four sides of the towers, overhang and podium. Some of the external surfaces that form the tube continue in plane into the building volume in order to reinforce corners and provide continuity of the tube actions.

The triangulated surface is made up of horizontal edge floor beams, columns, and diagonals. These are arranged in a regular pattern on the external surface of the building and the diagonal distribution pattern reflects the varying shear forces distributed in the tube and the full envelope of the design load cases.

All of the cores in the building including the towers are vertical and will contribute to stabilise floors, between the primary braced node floors. The core structure may also be used to contribute to the stability system in an extreme seismic event.

The use of energy absorbing elements (dampers) is being considered for the tower cores and also the surface of the external tube.

The floor plates will act as diaphragms transferring wind and seismic inertial forces into the primary stability system. Some diaphragm will only be effective every two levels in the shorter tower 2 where double height studios are planned.

Foundation stability is provided via a deep piled raft. Reinforced concrete walls within the basement structures are used to further stiffen the base of the towers effectively increasing the lever arm resisting overturning. The foundation system is arranged such that no permanent tension is allowed in the piles. Limited tensions in some piles are only envisaged in the most extreme seismic event.

7.3 Floor Systems

The building is constructed primarily in structural steelwork. The perimeter structural tube, which consists of a fully braced framework of columns, beams, and bracing diagonals, acts as a set of diaphragms. The perimeter, braced tube extends around the cantilever overhang and the podium components of the building providing both a gravity load path, and overall stability system for the building as a whole.

Throughout the tower superstructure the floor plate construction consists of composite steel beam and concrete slab. Vertical gravity loads from the floor plates are taken to ground via a combination of the column elements in the external tube and internal columns within the core and internal floor plates. The towers internal columns including those in the core are vertical whereas the ones in the external tube slope in 2 planes at 6 degrees.

Due to the slope of the towers some of internal columns, required to limit floor plate structure to reasonable depths, will need to terminate within the height of the towers. The loads from these terminated columns will be transferred into the core and external tube. Transfer structure is typically constructed from 2-storey deep trusses located in suitable program areas such as plant rooms. The number of transfer structures within the towers will be kept to a minimum.

The internal columns required within the cantilever overhang will be supported via transfer trusses at the lower level of the overhang. The gravity load from the overhang transfer trusses will in turn be supported by the perimeter tube, which will transfer it down to foundation level. The overhang transfer trusses will be at least 2-storeys deep.

裙楼主要也采用钢结构建造而成。大演播室上面的桁架将该具有足够大的深度保证能够跨过大演播室。按照设计需要，有些情况下这些桁架还将承受从上部传递下来的楼面荷载。

7.4 地基系统

地基设计将综合考虑因恒载、活载荷、浮力、土壤隆起、风力以及地震力等共同产生作用组合。同时亦考虑到因施工过程中对邻近建筑物、道路以及地下设施的影响。初步场地勘查已经完成，详细场地勘查正在进行中。根据初次勘查的结果以及从邻近工地所得到的勘查及地质资料，现正研究适合本项目的地基系统。

根据现阶段的设计所得，估计需要在整个场地内使用 1.2 米直径的直轴钻孔灌注桩。同时，位于斜塔之下估计需用桩长度大于 75 米 (从地面开始)。

两座斜塔将采用深桩筏基。典型桩中心距为 5 米，将布置于整片筏基以下。筏基面积比斜塔底部为大，其厚度大约为 8 米以提供足够的刚度和强度，把斜塔所承受的荷载转达到筏基以下的桩格上。但为为能节省材料及进一步优化地基方案，正在考虑采用华夫饼式地基，此方案将更具效率，亦能于 8 米深筏基上去除不必要的空间节省材料。

地下室和裙楼则采用传统独立桩帽，而独立桩帽将由 500mm 厚的面板互相连接固定。

基于地下室和地基坑的深度较深，将有可能产生因高水位导致的向上浮力。故必要时在将有规律地布置小口径的抗拔桩，以抑制因水压产生的浮力及减小地面板的跨度。

因斜塔承受的重量压力将会与邻近裙楼产生互相压力影响作用，而且两者之间亦可能发生不均匀沉降现象。这种因互相压力影响而发生的不均匀沉降作用现正进行详细研究，施工期间于塔楼与裙楼将设有后浇带，以减小不均匀沉降所造成的不利影响。

地下室三层结构系统将利用钢筋混凝土梁、柱与及平板。地下室的周边墙体和地库底板亦将会用抗水混凝土浇筑。

The podium superstructure will be constructed primarily in structural steelwork. Deep trusses will be used to span across the large studios where they occur. In some cases these trusses will be designed to support the required floor loads transferred from above.

7.4 Foundation System

The design of foundation will consider the combination dead and live load, floatation, heave, wind load and seismic load. The implication to adjacent buildings, road and under ground utilities during construction will also be considered. An initial site investigation had been performed and a detailed SI is underway. The foundation systems now being investigated are based on the initial SI and information from adjacent sites.

In the current stage of the design, straight-shafted bored piles of approximately 1.2m diameter are expected to be used extensively across the site. Under the two towers it is expected that piles in excess of 75m in (measured from Ground Level) length will be considered.

The two towers will be founded on deep piled rafts. Piles will be placed typically at 5m centres under the full extent of the raft. Typically the raft will extend well beyond the immediate footprint of the towers. The overall depth of the raft required to provide the stiffness and strength necessary to spread the tower loads into the pile grillage will be approximately 8m. However to save material and optimise the design it is envisaged that a waffle type raft design will be adopted effectively creating large voids within the overall 8m depth.

Under the basement and podium structures more conventional, discrete pile caps will be used. The individual pile caps will be tied together via a 500mm thick ground slab.

Due to the deep nature of the basement/foundation excavation the potential for high water uplift forces exists. Where necessary smaller diameter tension piles on a fairly regular grid will be used to restrain the uplift forces and limit the span of the ground slab.

The interaction between the heavily loaded inclined tower and the adjacent podium and the possible differential settlement between the two will be studied in detail. A late cast strip between the tower and the podium will be maintained during construction to minimize the undesirable effect of differential settlement.

The structural system for the 3-storey deep basement will be of reinforced concrete beam and column frames with flat slab. Water-resistant concrete, drained cavity, or tanking will be used for the basement walls and the base slab to provide water tightness as appropriate.

8. 与规范要求的比较

本节将探讨有关结构系统中不符合规范的要求。目的是将中央电视台主楼的结构就不符合规范的要求作出比较。同时，亦对不满足规范要求的部份进行研究及了解其对结构的影响。

以下就中央电视台主楼的结构系统平面规则性及竖向规则性进行比较。

正如本报告内其它章节所提及，中央电视台大楼采取基于结构性能的设计方法，明确确定其结构不规则的影响以考虑相关措施。

正如不能完全达到符合规范要求的其他建筑结构，中央电视台主楼在考虑所有荷载情况下的设计与施工将会进行全面的专家讨论及审批。

8.1 平面不规则

8.1.1 扭转不规则

当考虑扭转不规则时，层板将采用刚性横隔板。

当中最大层间移位包括考虑偶然偏心后 (即 5%平面偏移后) 大于结构两端层间位移的平均值的 1.2 倍时，将考虑存有扭转不规则情况。

如果上述比值超出 1.4 时，将视为严重扭转不规则。

8. COMPARSION WITH STANDARD CODE REQUIREMENTS

This section of the report investigates which aspects of the structural system are outside of standard code requirements. An attempt is made here to quantify the amount by which the CCTV structure is outside of these requirements and a study is made of the implications of any non-compliance.

The CCTV structural system is compared to the standard plan and vertical regularity criteria as set out below.

It should be noted, as discussed elsewhere in this report, that a fully compressive performance based design will be undertaken of the CCTV building and that the effects of any irregularity on the structural performance will be determined explicitly. As per other structures that do not fully comply with the simplified code requirements the CCTV building is to be fully reviewed by Expert Panel Review EPR with regard to it's construction and performance under all loading criteria.

8.1 Plan Irregularity

8.1.1 Torsional Irregularity

Torsional Irregularity shall be considered when diaphragms are not flexible.

Torsional Irregularity shall be considered to exist when the maximum storey drift, computed including accidental torsion (5% plan dimension offset), at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts at the two ends of the structure.

Extreme torsional irregularity shall be considered if the above ratio exceeds 1.4.

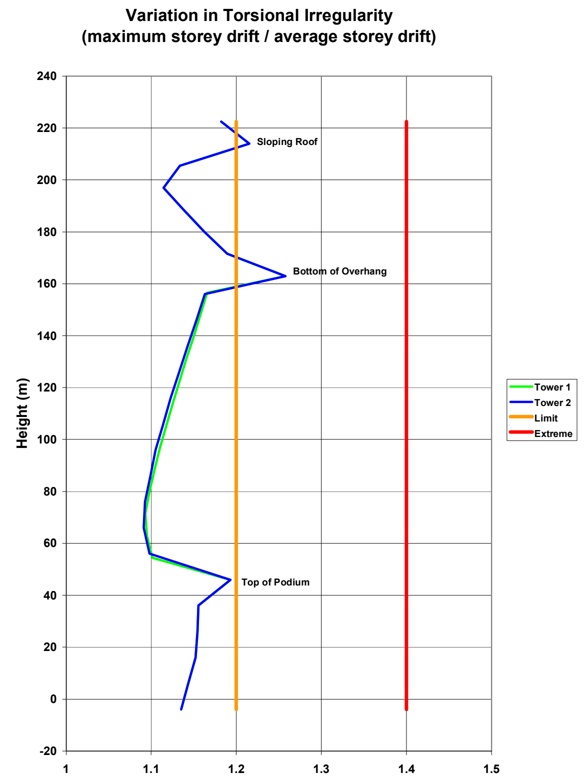


Figure 8.1 Torsional Irregularity Index
图 8.1 扭转不规则性

依据图 8.1 所示，中央电视台主楼结构只有两处超出规范 1.2 的扭转不规则标准。一般而言，本塔楼楼板的布置与刚度是相当吻合及一致。

本大楼的全高从上到下无任何一处出现严重扭转不规则的情况。

8.1.2 凹凸不规则

凹凸不规则的定义是指平面布置中凹凸结构的一侧尺寸于指定方向上大于相应投影方向总尺寸的 30%。

位于两座塔楼内的矩形楼面板，并不存有凹凸不规则的情况。

悬臂结构部份的楼板明显并不是平面矩形，严格按照规范要求此部份可视为凹凸不规则。但在考虑这一定义时应当注意，侧向外筒体系统对层板形成了完整的维护结构，因此有助稳定凹凸不规则的影响。

于塔楼结构分析模型中均对悬臂楼板已作出详细分析及模拟相关凹凸特性。

8.1.3 楼板不连接

楼板不连接是指横隔板发生尺寸及刚度的突变，包括有开口或开洞尺寸超出该层楼板典型宽度的 50% 或开洞面积大于该层楼面面积约 30%，或有较大的楼层错层。

一般情况下，本大楼楼面积范围内的横隔板，其平面内刚度是合理分布的。电梯内的开口及其他的开洞面积均少于 30%的总楼面面积。

当最终设计与建筑布置确定后，将采用二维壳体单元作出对横隔板进行明确模拟分析，同时，评估各个开洞的影响。横隔板内的薄弱部位将通过钢结构斜支撑构件，采取平面增强/加固方式作解决。

8.1.4 平面外偏移

平面外偏移是指侧向力路线形成间断，如竖向构件的平面外偏移。

一般来说，本大楼横向支撑系统是由平面内的全斜支撑横隔板组成，此将与外结构筒体相连，故本系统并不存在平面外偏移。

8.1.5 非平行系统

非平行系统是指抗侧力竖向构件于抗侧力系统的主要正交轴线上形成不平衡或不对称的情况。

如前所述，本大楼的横向支撑系统是由平面内的全斜支撑横隔板组成。

As demonstrated in Figure 8.1 the CCTV structure only exceeds the code torsional irregularity criteria of 1.2 in two locations. Generally the floor diaphragms are arranged fairly concentrically with regard to the centre stiffness.

There is no location up the height of the CCTV building that the extreme irregularity criterion is reached.

8.1.2 Re-Entrant Corners

A re-entrant corner is defined as plan configurations where both projections of the structure beyond a re-entrant corner are greater than 30 percent of the plan dimension of the structure in the given direction.

The CCTV floor plates within the towers are rectangular in plan and do not contain re-entrant corners.

The overhang floor plates clearly are not rectangular on plan and could be defined in strict abidance with the code as possessing a re-entrant corner. It should however be noted whilst considering this definition that the lateral “tube” system fully envelopes the floor plates at these levels and thus helps to stabilise the effects of the re-entrant corners.

The re-entrant nature of the overhang floor plates has been modelled explicitly in all the structural models used for the analysis of CCTV.

8.1.3 Diaphragm Discontinuity

A diaphragm discontinuity is defined as occurring when a diaphragm has an abrupt discontinuity or variation in stiffness, including those having cut out or open areas dimension greater than 30%/50% respectively of the gross enclosed diaphragm area/dimension, or changes in effective diaphragm stiffness of from one storey to the next.

Generally within the tower areas diaphragms are well arranged in terms of their in plane stiffness and cut outs for cores and other openings are generally less than 30% of the gross diaphragm area.

As the design and architectural layouts are finalised it is the intention of the analysis to model the diaphragms explicitly with 2D shell elements to assess the effects of all openings. Any shortcomings within the diaphragm will be dealt with via in plane strengthening/stiffening via steelwork bracing elements.

8.1.4 Out-of-Plane Offsets

Out of plane offsets are defined as discontinuities in lateral force path such as out of plane offsets of vertical elements.

Generally the CCTV lateral system consists of in-plane fully braced diaphragms, which combine to make an integrated external structural tube. This system by its nature is free of out-of-plane offsets.

8.1.5 Non-Parallel Systems

Non-parallel systems are defined as those where the vertical lateral force resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force resisting system.

As discussed above the CCTV lateral system consists of in plane fully braced diaphragms.

8.2 竖向不规则

8.2.1 刚度不规则

刚度不规则用于确定软弱层的存在。软弱层是指当本楼层刚度小于上一楼层刚度的70%或其上三层平均值的 80%。严重软弱层是指上述数值分别降低到 60%和 70%。

经过验算现有模型的侧向刚度分析结果及与其上部大楼各构件的刚度与高度对照。

如预料一样，竖向不规则发生在悬臂结构与及两座塔楼连接介面上。故考虑到此部份的重要性，于塔楼设计时该部份的非线性行为将严格控制。因此在性能目标设计分析时需特别注意此部份设计。

如以上表示，裙房的刚度明显表示随高度有所增加。此方面并不理想，故随着设计进入到扩大初步设计阶段，需对裙楼的侧向支撑系统进行调整，使得最终竖向刚度在此部份内得到更好的分配。

8.2 Vertical Irregularity

8.2.1 Stiffness Irregularity

Stiffness irregularity is used to determine the presence of soft storeys. A soft storey is one in which the lateral stiffness is less than 70 % of that in the storey above or less than the 80% of the average of the 3 storeys above. An extreme soft storey is defined by lowering the values above to 60% and 70% respectively.

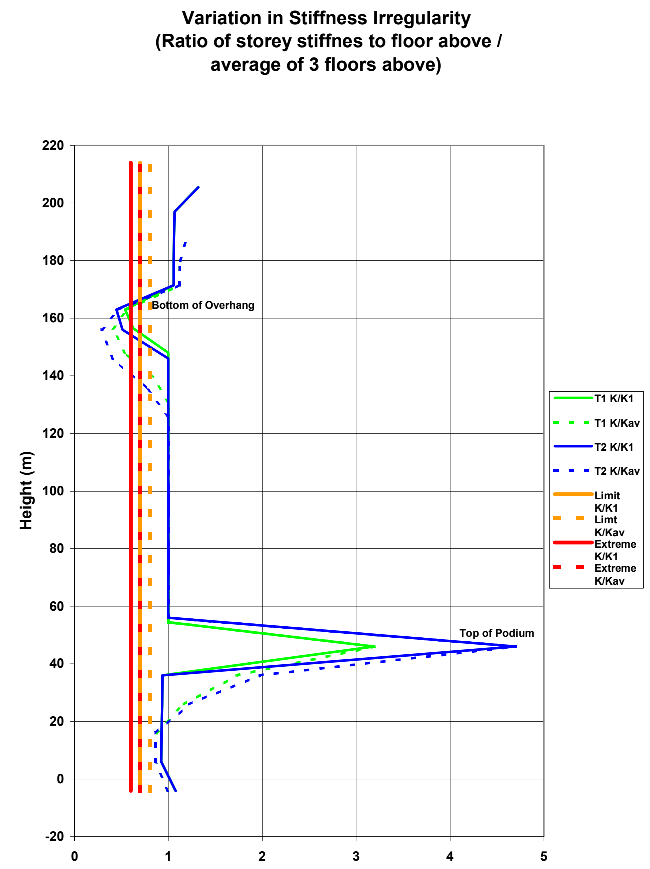


Figure 8.2 Stiffness Irregularity Index
图 8.2 刚度不规则性

The lateral stiffness of the existing analysis model has been examined and values of stiffness ratio plotted against height for the various elements of the superstructure.

As would be expected the only level where a defined stiffness irregularities occur is at the intersection of the towers with the overhang element of the structural system. It is the general intention that this zone of the building will only be allowed to undergo very limited non-linear behaviour. Particular attention is given to this area in the performance based design analysis.

As illustrated above the podium stiffness exhibits some increase in stiffness up its height. It is appreciated that this is not ideal and the podium lateral system will be tuned, as the design develops into the EPD phase, to better apportion the vertical stiffness in this region.

8.2.2 重量（质量）不规则

当任何楼层的有效质量超出相邻楼层的有效质量的 150%时，并视为存在质量不规则。

如预料之内，中央电视台大楼唯一可能存在质量不规则的情况位置是位于双塔与悬臂结构连接部位和与裙楼连接部位。

正如其它章节所述，此部份结构将需要于大震设计中深入讨论及研究，并利用结构性能目标为基础进行分析及设计。

8.2.3 抗侧力竖向构件中的平面突变

当构件的偏移超出其长度时，或者当下一层抗侧力构件发生重大刚度下降时，应考虑抗侧力构件存在突变。

由于中央电视台主楼的抗侧力系统是一个平面的斜撑式筒体，因此抗侧力竖向构件中不会存有重大突变。

8.2.2 Weight (Mass) Irregularity

Mass irregularity shall be considered to exist where the effective mass of any storey is more than 150% of the effective mass of an adjacent storey.

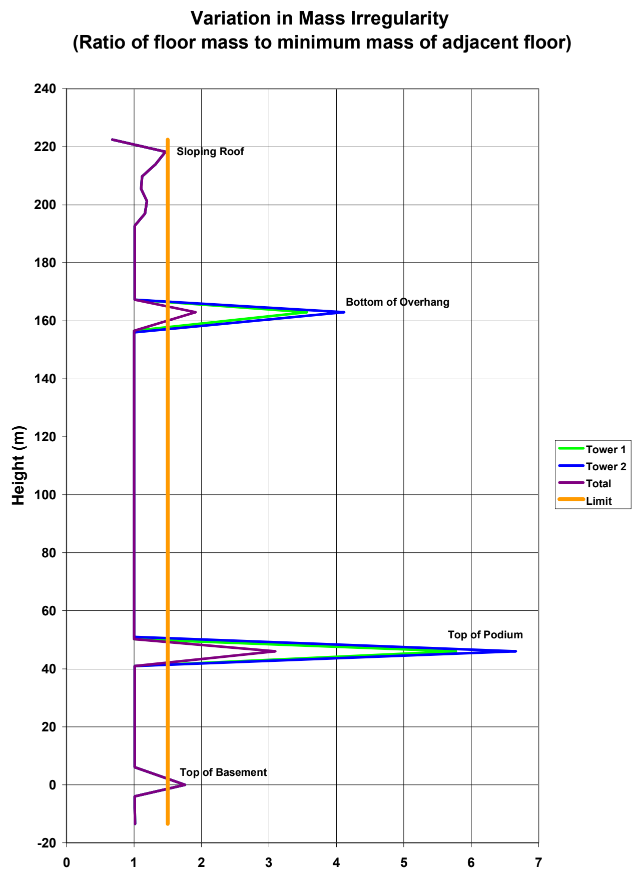


Figure 8.3 Mass Irregularity Index
图 8.3 质量不规则性

As expected the only areas in the CCTV building that are defined as having mass irregularity are at the intersection of the towers with the overhang and podium elements of the structural system.

As discussed elsewhere the behaviour of these areas in a major seismic event will be examined in suitable detail as part of the performance based approach to the analysis and design.

8.2.3 In-Plane Discontinuity in Vertical Lateral Force Resisting Elements

Lateral force resisting elements are considered to have a discontinuity when the offset of the elements is greater than the length of those elements or when a significant reduction in stiffness exists in the resisting element in the storey below.

Generally as the CCTV lateral force resisting system is an in plane braced tube there are no significant discontinuities in vertical lateral force resisting elements.

8.2.4 承载能力的突变－薄弱层

薄弱层是指楼层受剪承载力低于上一楼层的 80%。楼层受剪承载力指该地震方向上所有抗侧力构件的抗剪承载力总和。

总体而言塔楼结构内的斜撑式筒体系统都可以调整到符合以上要求的楼层承载力分布。但很明显，在双塔与悬臂连接部份，由于斜撑面体长度的增加，将存在楼层承载力变化。设计时需严格限制此部份连接的非线性行为，同时限制整体强度或刚度的变化下降程度。故此将进行全方位的非线性分析作研究及验证，作为基于性能设计方法的一部份。

8.2.4 Discontinuity in Capacity – Weak Storey

A weak storey is defined as one in which the storey lateral strength is less than 80% of that in the storey above. The storey strength is the total strength of the seismic resisting elements sharing the storey shear for the direction under consideration.

Generally within the tower structures the braced tube system is tuned to provide a distribution of lateral strength in line with the requirement above. Clearly at the overhang intersections with the towers a discontinuity in lateral strength will exist due to the increase in braced wall length. It is the intention of the design to limit the amount of non-linear behaviour that exists at this intersection in order to limit any global sudden decrease in strength or stiffness. This will be demonstrated via a full non-linear pushover analysis of the CCTV building performed as part of the performance based design.

9. 弹性分析

中央电视台主楼电脑模型采用整体的三维有限元模型 (见图 9.1 及 9.2)。结构分析已按照本文件的 4.0 节列明的标准荷载规范要求进行深入分析。采用反应谱分析方法进行多遇地震下的静力分析。分析中亦考虑到因施工过程所产生的附加荷载的影响。

目前，中央电视台主楼的弹性分析采用了两个标准有限单元软件作分析之用。软件包括奥雅纳公司自行开发的有限元分析软件 GSA，以及商用结构分析软件包括 SAP2000。SAP2000 由加利福尼亚的 CSI 公司开发，适用于地震区的复杂建筑结构分析。

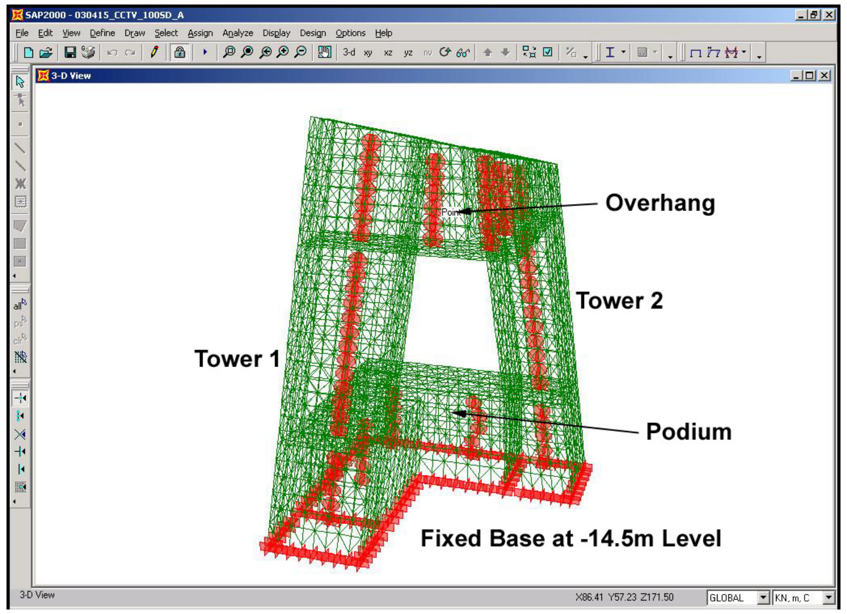


Figure 9.1 Computer Model (1)
图 9.1 电脑模型 (1)

除了主要的上述上部结构模型以外，还特别制作了若干单独模型用作研究特定问题如地基系统。同时，为了确定整体的趋势和建筑的几何特征，还特别利用简化的"杆"式模型进行了若干参数性的研究。

9.1 分析模型

上部结构模型使用梁单元模拟所有柱、梁和斜撑构件。同时，采用两种包括柔性和刚性楼盖隔板的假定对模型进行了研究。总体而言，两项假定的结果相似。

为了确定三维有限元模型中构件的有效刚度，采用了适当的软件对钢混凝土组合柱进行了研究。柱的刚度和承载能力容量表与及现行布置大样见附件表示。典型的组合柱详图及位置参见附件内初步设计图纸。

上部结构模型假定固定于桩承台顶，标高为－14.5 米。基础变形对上部结构变形的影响及产生的附加荷载已用一个简化的 "杆" 模型进行了研究。一般而言，基础变形对上部结构的影响很小，而就上部塔楼的附加地震力而言，现有的模型假定固定于基础顶是足够安全的。

9. ELASTIC ANALYSIS

A full 3 dimensional finite element model has been built of the CCTV building (Figures 9.1 ~ 9.2). The structural model has been analysed in detail under all the standard code loading requirements as detailed in section 4.0 of this document. Level 1 seismic loads have been accounted for via the use of response spectrum analysis procedures, all other loads are represented by static load cases. The construction effects of pre-loading the towers in their partially constructed state have been applied to the analysis.

To date two standard finite element software packages have been used in the elastic analysis of the CCTV building. GSA, an Arup in-house finite element package, has been used in parallel with SAP2000, a commercially available building engineering FE package developed by CSI in California for use on complex buildings in seismic regions.

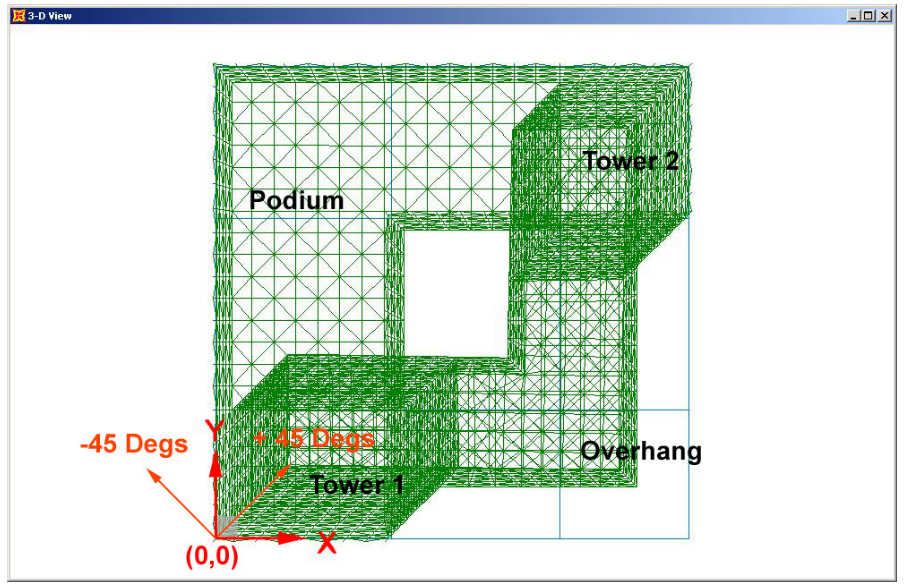


Figure 9.2 Computer Model (2)
图 9.2 电脑模型 (2)

In addition to the primary super-structure models a number of discrete models have been developed to study specific issues such as the foundation systems. A number of parametric studies have also been undertaken on more simplified "stick" models to determine global trends and characteristics of the building geometry.

9.1 Analysis Model

The superstructure has been modelled as a series of beam elements representing columns, beams, and braces. Models have been studied with both flexible and rigid floor diaphragm assumptions. Globally the results of these two assumptions are similar.

Composite steel and concrete columns have been studied using a suitable software package to determine effective stiffness properties to be used within the full 3D analysis model. Refer to appendix for column stiffness and capacity tables and for a current arrangement diagram. Refer to the SD drawings in the appendix for typical composite column details and locations.

The superstructure model is assumed to have a rigid base at top of pile cap level, –14.5m. The effects of foundation flexibility on tower rotations and secondary loads imposed on the towers have been studied on simplified stick models. Generally the foundation flexibility effects are relatively small and in terms of the seismic forces generated within the towers it is a perceived as conservative to assume base fixity.

建筑物的地震质量是一连串连系于楼板上的集中质量。为了确定扭转特征，集中质量具有平移质量和质量惯性矩。地震质量的数值采用 100%的恒载与附加恒载质量加上 50%的活载质量。

振型分析利用三维模型分析进行，采用特征值分析方法，计算了标准的 50 个振型。振型数目以保证获得超过 95%的结构地震质量为标准。

当采用标准反应谱分析法时，将按本报告内第 4.0 节的规定，进行符合中国规范的多遇地震反应谱作弹性模型分析。选择了不同输入地震谱方向，作配合主要反应振型及各塔楼交角轴线。基本上而言地震方向为以 45° 为增量的所有方向。

按中国规范规定，于多遇地震的反应谱分析时，需要考虑因非结构构件造成对基本周期的影响及相应的折减，折减系数为 0.9。此折减系数通过对反应谱适当调整而实现。结果表明此调整对整体建筑结构反应影响很微。

整个多遇地震分析考虑了 2%的极限模型阻尼。

9.2 分析结果

9.2.1 整体结果一览表

为了确定前 50 个平移和转动为主的振型，采用了标准特征向量法进行模型分析。以下列出了中央电视台主楼的前 9 个振型，并列出与每个振型相关的模型参与质量比。

包括整个裙楼结构在内的中央电视台主楼的总地震质量为 42 万吨。大楼的总建筑面积约为 40 万平方米，因此每平方米建筑面积为 1.05 吨。

The seismic mass of the building is represented by a series of lumped masses coupled to the diaphragms. The lumped masses have both translational mass and mass moment of inertia properties in order to pick up any torsional behaviour. In general the seismic mass is taken as 100% dead and superimposed dead mass plus 50% of the imposed floor mass.

A modal analysis has been undertaken on the full 3D analysis model. The Eigen solution technique has been used and typically 50 modes computed. The number of modes computed has been set to capture in excess of 95% of the participating seismic mass of the structure.

The input Chinese code compliant level 1 seismic response spectrum as defined in section 4.0 of this report has been applied to the elastic model using standard response spectrum analysis techniques. The various directional orientations of the input signals have been chosen to coincide with the orientation of the primary modes of response as well as the orthogonal axes of the individual towers. Effectively this means that the earthquake is applied at 45-degree increments around the clock.

The Chinese code states that the level 1 response spectrum analysis must take account of any inaccuracies in calculating the building stiffness via the application of a reduction factor of 0.9 taken on the fundamental periods. The effect of this has been approximated via running a response spectrum analysis with an appropriately shifted corner frequency. Generally the effects on the global response of the building are found to be small.

Modal damping of 2% of critical has been considered for all Level 1 analysis.

9.2 Analyses Results

9.2.1 Global Results Summary

A modal analysis has been undertaken using standard Eigen vector techniques to establish the first 50 translational and rotational primary modes of vibration. The following table lists the first 9 modes of vibration of the CCTV building and shows the modal participating mass ratios associated with each mode.

The total seismic mass of the CCTV building including all podium structure is approximately 420,000 tonnes. Based on a gross floor area of approximately 400,000m² this equates to 1.05 tonnes/m² of floor area.

振型数	周期 (秒)	模型参与质量比		
		X-轴反应	Y-轴反应	转动反应
1	4.48	0.26	0.30	0.00
2	3.86	0.31	0.29	0.11
3	2.93	0.02	0.00	0.40
4	1.35	0.06	0.15	0.01
5	1.24	0.16	0.01	0.00
6	1.20	0.01	0.08	0.01
7	1.07	0.02	0.00	0.18
8	0.85	0.02	0.07	0.01
9	0.81	0.06	0.03	0.08

第一最低平移振型与主要转动振型之比为 0.76，低于规范内的 0.85 的限值。因此扭转振型将不会与主侧向振型形成偶联。

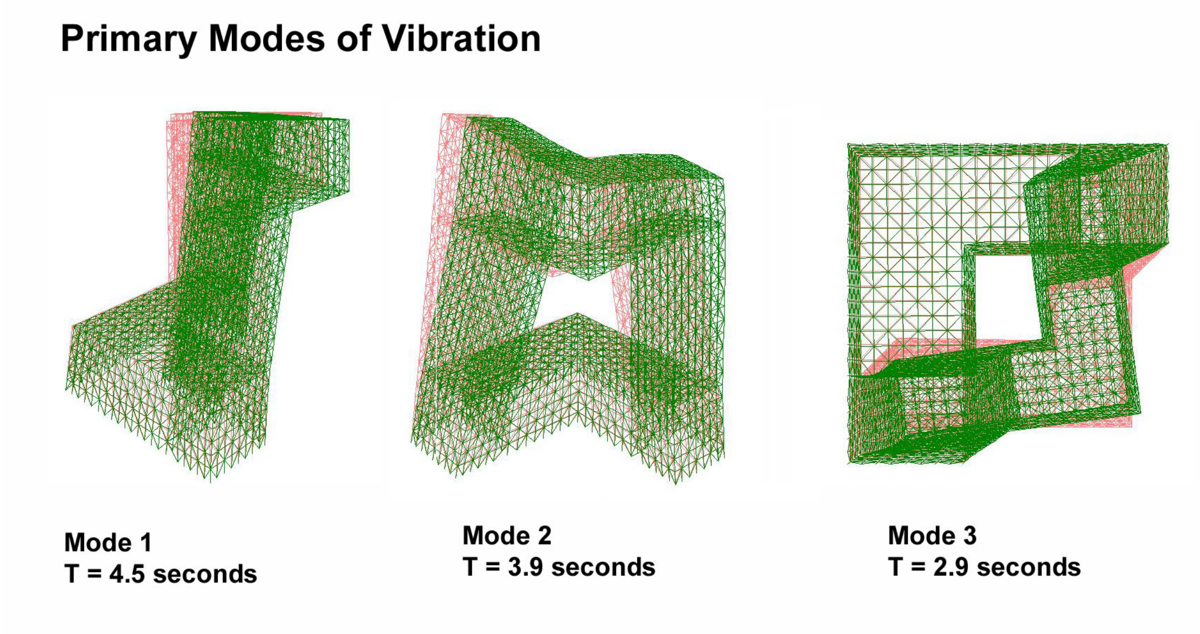


Figure 9.3 Horizontal Primary Modes
图 9.3 主水平振型

Mode Number	Period (seconds)	Modal Participating Mass Ratios		
		X-Direction Response	Y-Direction Response	Rotational Response
1	4.48	0.26	0.30	0.00
2	3.86	0.31	0.29	0.11
3	2.93	0.02	0.00	0.40
4	1.35	0.06	0.15	0.01
5	1.24	0.16	0.01	0.00
6	1.20	0.01	0.08	0.01
7	1.07	0.02	0.00	0.18
8	0.85	0.02	0.07	0.01
9	0.81	0.06	0.03	0.08

The ratio of the lowest first translation mode to the primary rotational mode is 0.76, which is less than the Chinese code limit of 0.85. Hence the torsional mode is deemed to be uncoupled from the primary lateral modes of vibration.

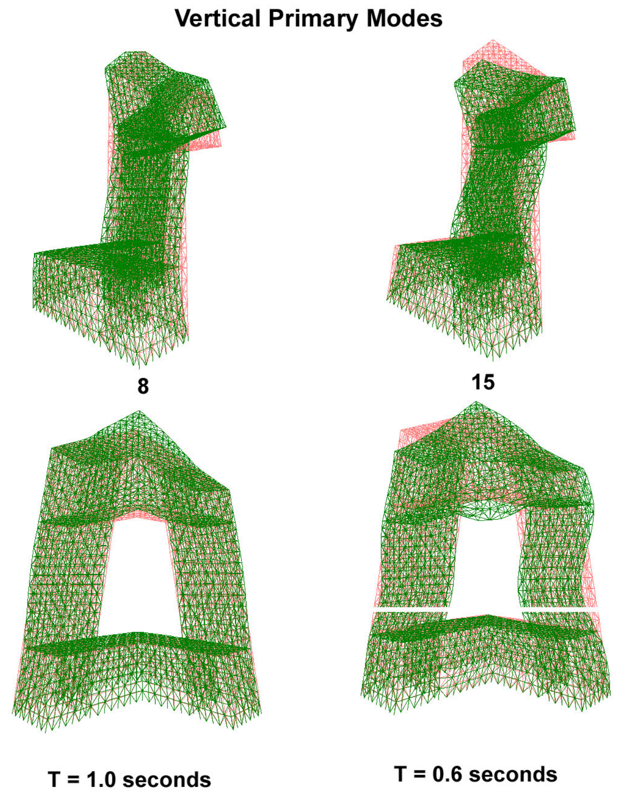


Figure 9.4 Vertical Primary Modes
图 9.4 主竖向振型

为模拟简便原因，单独制作了一个分析模型，用作研究多遇地震下的竖向地震的反应。主竖向振型和其后的振型形态列于以下供参考。

竖向地震模型的分析结果将结合多遇地震下应力验算结果进行荷载组合作日后设计之考虑。

For modelling simplicity reasons a separate analysis model was constructed to study the Level 1 vertical seismic response effects. The primary vertical modes and the subsequent mode shapes are shown below.

The analysis results from the vertical earthquake model are combined with the other load combinations in the Level 1 stress checks.

9.2.2 层间剪力

现将各地震荷载的层间侧向剪力见图 9.5a~g。

最大多遇地震下的底部剪力为大概 3.5%G。

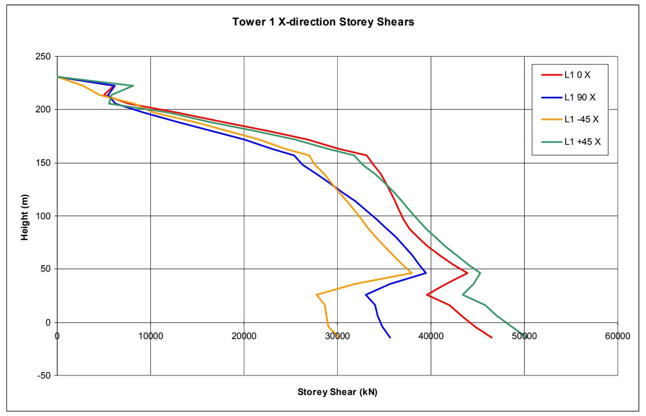


Figure 9.5a X dir. Storey Shears of Tower 1
图 9.5a 塔楼 1 X 向地震剪力

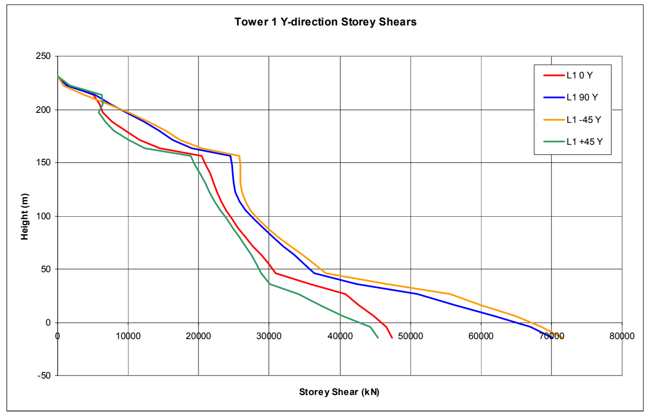


Figure 9.5b Y dir. Storey Shears of Tower 1
图 9.5b 塔楼 1 Y 向地震剪力

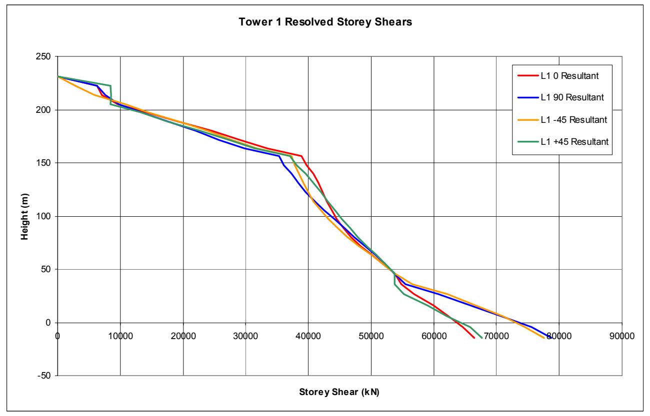


Figure 9.5c Resolved Storey Shears of Tower 1
图 9.5c 塔楼 1 组合层间剪力

9.2.2 Inter Storey Shear

The inter storey lateral shear plots for the various seismic load cases are shown in Figure 9.5a~g.

The maximum Level 1 base shear is seen to be approximately 3.5%G.

Figure 9.5d X dir. Storey Shears of Tower 2
图 9.5d 塔楼 2 X 向地震剪力

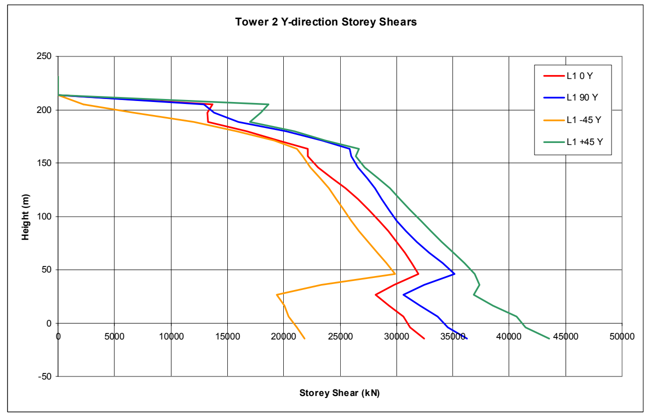


Figure 9.5e Y dir. Storey Shears of Tower 2
图 9.5e 塔楼 2 Y 向地震剪力

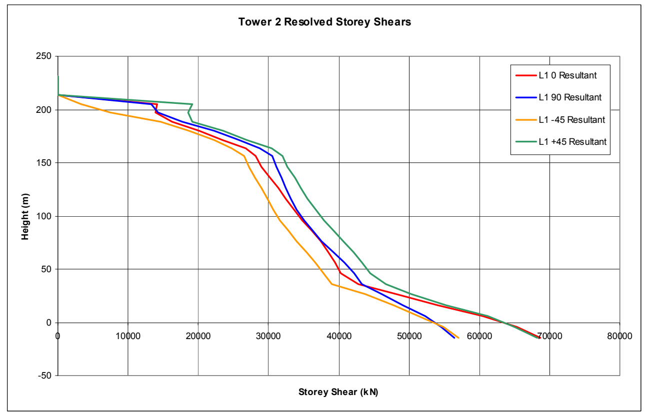


Figure 9.5f Resolved Storey Shears of Tower 2
图 9.5f 塔楼 2 组合层间剪力

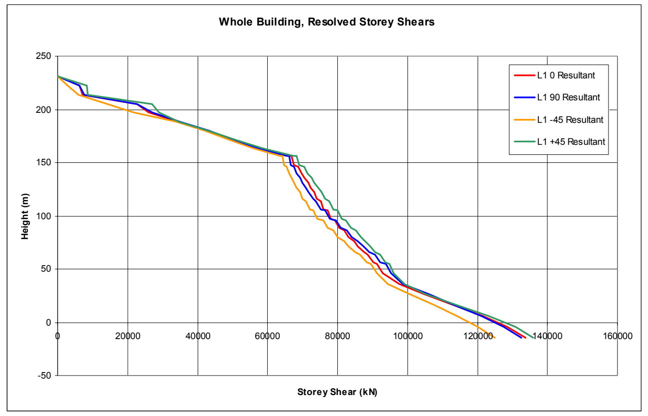


Figure 9.5g   Resolved Storey Shears of Whole Building
图 9.5g   整体结构的组合层间剪力

9.2.3 层间位移角

现将各侧向荷载产生的层间位移角图表示如下。最大层间位移角为 45° 方向地震作用下 X、Y 方向的组合值为 1/580，小于中国规范 1/300 的限值。

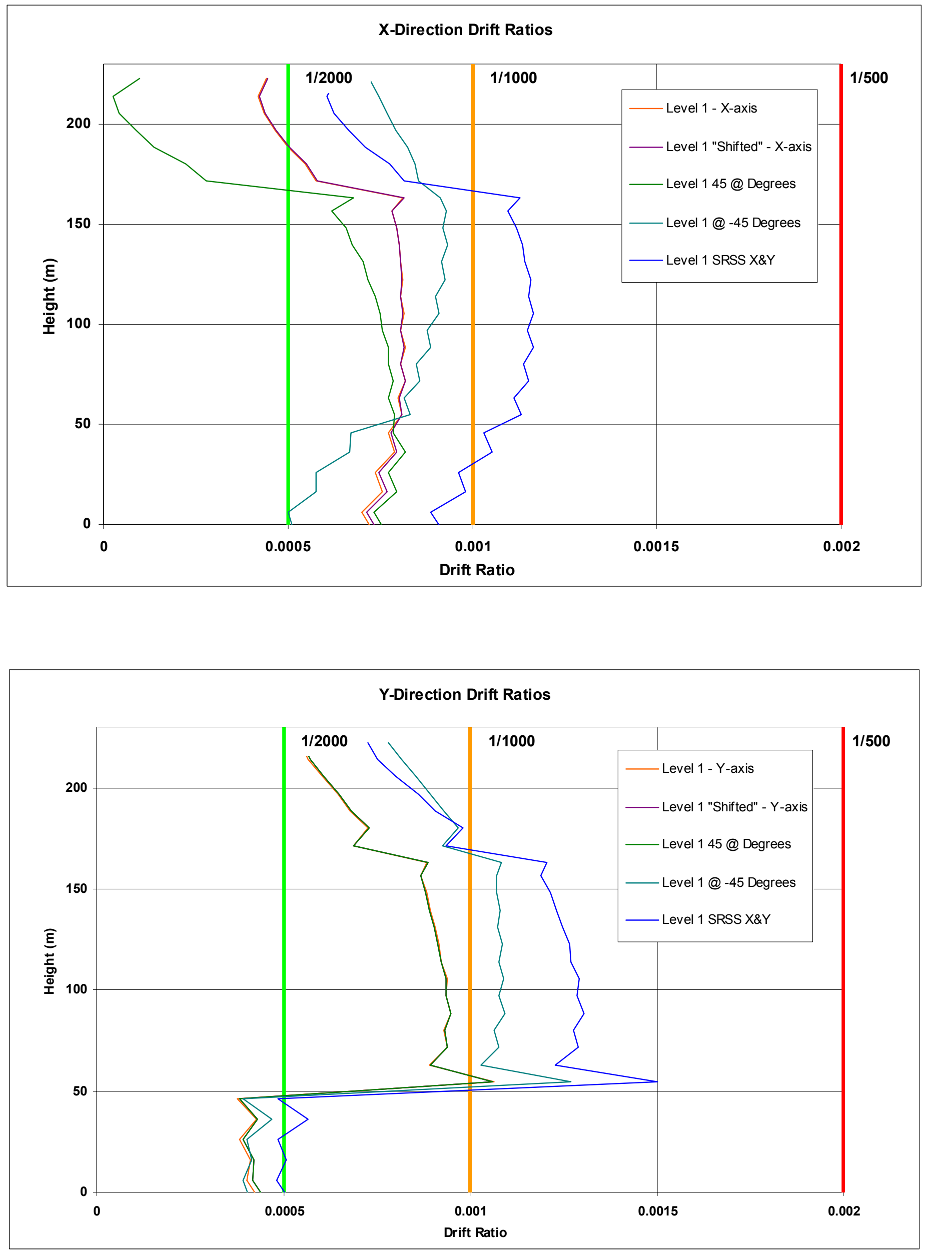


Figure 9.6 X, Y Dir. Drift Ratios under Seismic Load
图 9.6 地震作用下 X、Y 向层间位移角

9.2.3 Inter Storey Drift Ratio

The inter storey drift ratio plots for the various lateral load cases are shown below. In all cases the maximum inter story drift, even considering the most onerous case of resultant X-Y results from a 45 degree seismic event, are below the 1:300 limit set by the Chinese code. The maximum drift from any case and combination of level 1 seismic event is 1:580.

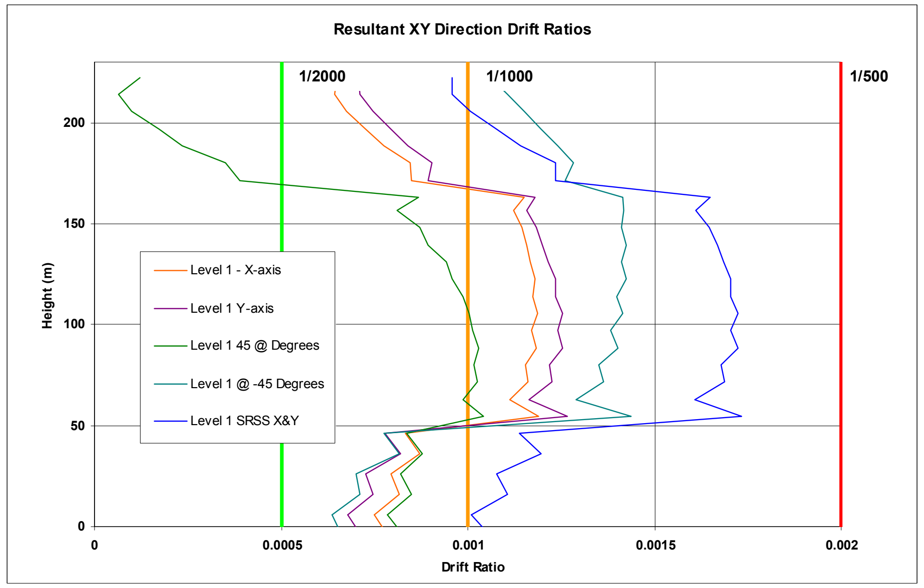


Figure 9.7 Resultant XY Director Drift Ratios
图 9.7 地震作用下层间位移角组合值

根据多遇竖向反应谱分析，悬臂部份的最大末端挠度小于 40 毫米。

通常处于正交风压下，位移角亦不超过 1：1500。随着进一步的设计，需要对高峰风速加速值影响下的使用性能作出深入研究。但是由于中央电视台主楼在风荷载下的内在刚度能可以预见，风荷载产生加速度应远小于舒适度要求限值。

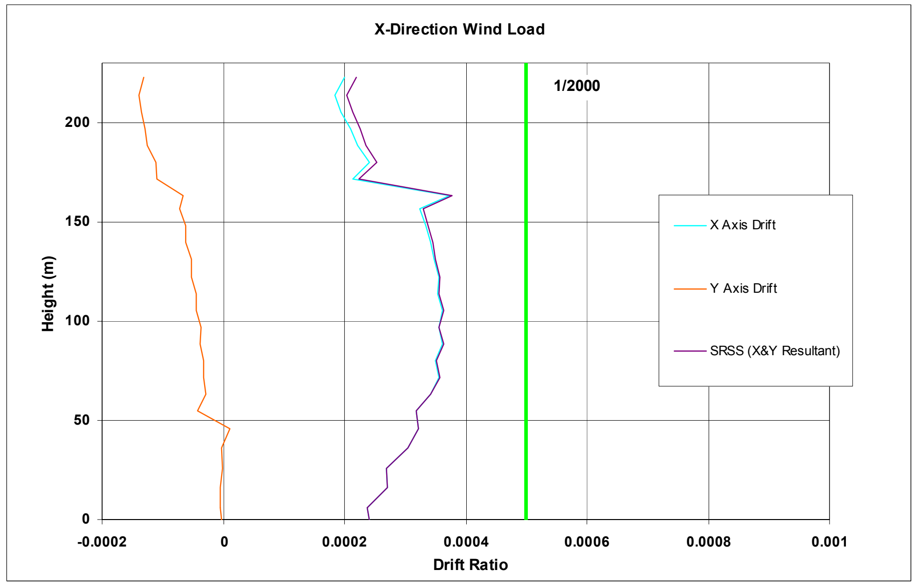


Figure 9.8a X Dir. Drift Ratios under Wind Load
图 9.8a 风荷载作用下 X 向层间位移角

Under Level 1 vertical response spectrum analysis the maximum vertical tip deflection of the overhang cantilever element is less than 40mm.

Typically under orthogonal wind loads the drift ratios are less than 1:1500. The serviceability issues with regard to peak wind acceleration values are to be studied in detail as the design proceeds but given the inherent stiffness of the CCTV building structure subjected to wind loading it is envisaged that wind accelerations will be well below the required comfort criteria.

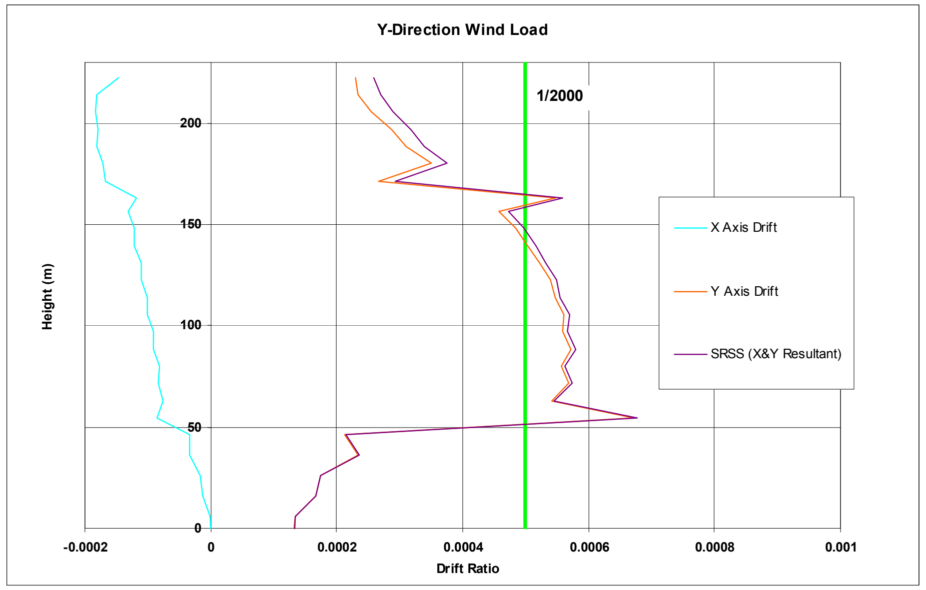


Figure 9.8b Y Dir. Drift Ratios under Wind Load
图 9.8b 风荷载作用下 Y 向层间位移角

9.2.4 构件应力验算

中央电视台主楼的主要稳定系统中各个结构构件均已经过验算，其设计承载力容量均按本报告第 4.6.1 节规定的设计荷载组合进行了比较。

验算结果列于以下表内，在现阶段，构件的重要性系数及列明于《结构设计依据》第 4.7 节的轴压比限值均不包括于引用比率内。为确定构件尺寸是否满足，应直接将相应系数及其限度与设计荷载值及设计承载力比进行比较。

请注意本报告提及的承载力比只代表现设计阶段的设计数值，但将于项目进入扩大初步设计的阶段，此承载力比将会因应最终设计需要作出优化及调整。

现将部份典型组合柱承载力比列于以下供参考。柱的大样与位置见 SD 图纸。

9.2.4 Elemental Stress Checks

All the structural elements within the primary CCTV tower stability system have been checked in terms of comparing their design capacity against the design load combinations as specified in section 4.6.1 of this report.

A summary of the results is given in the following table. It should be noted that at present the element importance factors and the axial stress ratio limits listed in section 4.7 of the Structural Design Brief are not included in the quoted ratios. The relevant factors and limits should be compared directly to the design load to design capacity ratios to determine element suitability.

It should be noted that the capacity ratios presented in this report represent a design that is in development. Over the EPD phase of the project the element design will be optimised to refine the capacity ratios as appropriate.

Some typical composite column capacity ratios are shown below. Refer to SD drawings for column details and locations.

Column Reference	Load Case 1	Load Case 1A	Load Case 2	Load Case 2A	Load Case 3	Load Case 4	Load Case 5	Load Case 6
1A	0.52	0.52	0.59	0.58	0.50	0.55	0.66	0.69
1	0.49	0.47	0.52	0.51	0.46	0.51	0.55	0.56
2	0.59	0.59	0.63	0.63	0.54	0.59	0.74	0.75
3	0.45	0.44	0.46	0.45	0.38	0.47	0.46	0.48
4	0.68	0.68	0.73	0.71	0.65	0.72	0.80	0.82
5	0.62	0.60	0.65	0.63	0.55	0.64	0.69	0.71
6	0.54	0.54	0.58	0.58	0.47	0.57	0.80	0.81

典型的承载力验算见图 9.9~9.10：

中央电视台主楼结构的部份斜撑构件目前仍有待优化。此部份优化方案将于扩大初步设计（EPD）阶段深入研究及设计。目前斜撑布置初步选择，为求满足本报告第 4.6.1 节提出的最不利荷载结合所产生的平均承载力比低于 1.0 的要求。因此，目前设计的总刚度应接近最终的设计。但是，目前部份少量构件有出现应力偏高的情况，为了反映这一点及作出调整，设计将修改作配合上述的要求。

图 9.10 所示为斜支撑构件的基本承载力的复核比率案例。根据结构设计依据第 4.7.4 节中表示，当考虑多遇地震组合下，以上比值尚应考虑抗震承载力调整系数 0.8。

A typical capacity check is shown in Figures 9.9 and 9.10.

The brace elements within the CCTV tower structure are currently not fully arranged in their optimum configuration and this is an area that will be developed in the EPD stage of the design. The current preliminary arrangement of bracing elements has been chosen so that the average capacity ratios are below 1.0 under the worst load combination from section 4.6.1. Hence the overall behaviour of the building in terms of global stiffness will be close to the final design. However a relatively small number of elements are clearly currently over-stressed and the design will be modified in the next iteration to reflect this.

A sample of typical brace elements is shown in Figure 9.10 with their basic capacity check ratios. It should noted that as described in section 4.7.4 of the Structural Design Brief an overstress factor of 0.8 can be applied to the above ratios for Level 1 seismic combinations.

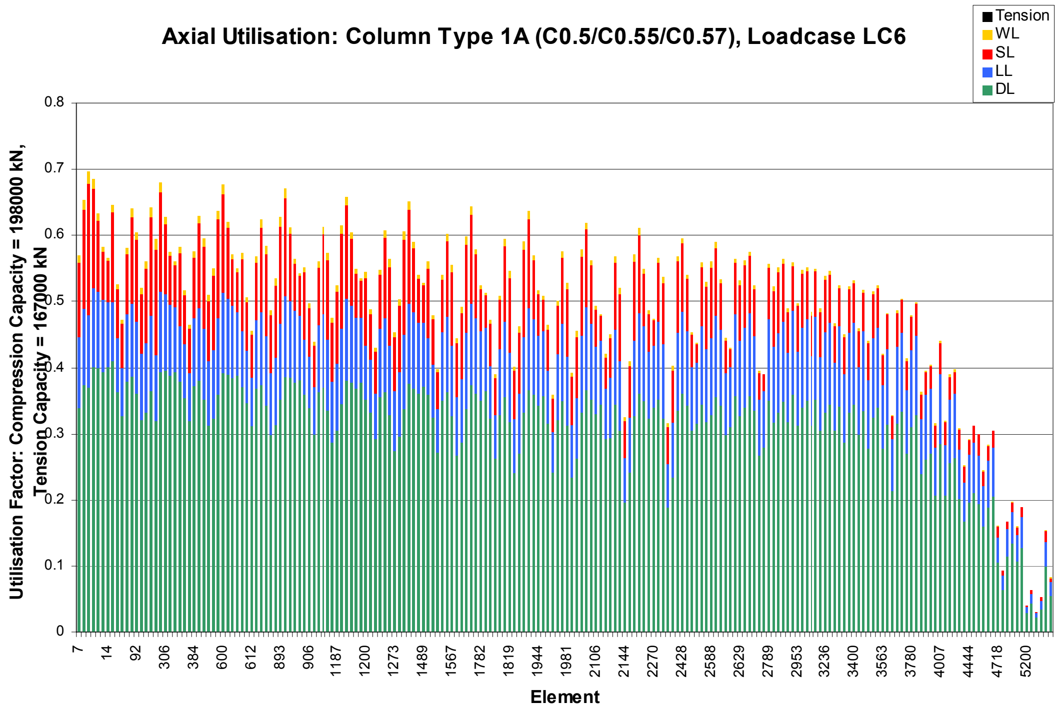


Figure 9.9 Column Capacity Ratios
图 9.9 柱荷载/承载力比

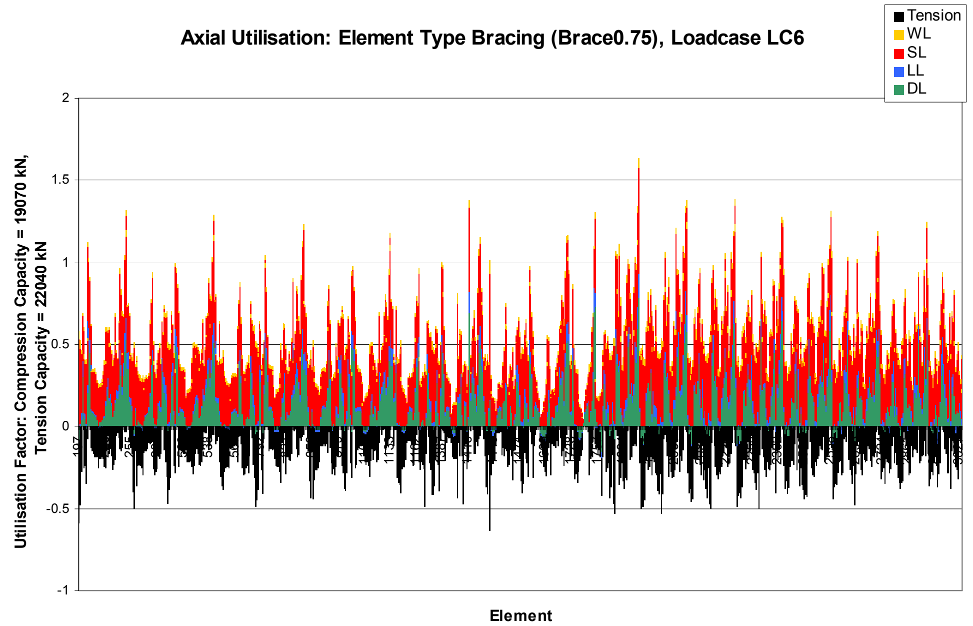


Figure 9.10 Brace Capacity Ratios
图 9.10 支撑荷载/承载力比

9.3 弹性分析结论

目前的分析结果表明，现中央电视台主楼设计表现出完全符合本报告第 4.6.1 节规定多遇地震下弹性荷载要求。同时，位移要求亦未超出指定标准，符合规范要求。各构件的应力水平一般均在承载力以内。

本报告列出的结果反映出现设计当前的进展情况。虽然于多遇地震下，整体及单独构件性能都能基本满足要求，但是为了提高结构性能和减少整个用钢量，随着设计进入更深入阶段及进行扩大初步设计（EPD），仍会持续调整及改良达到最终优化为目的。

9.3 Conclusions of Elastic Analysis

The analysis results to date illustrate that the current CCTV tower design performs reasonably well under the Level 1 elastic load requirements defined in section 4.6.1 of this report. Drift requirements are all within the given criteria and conform to code. Stress levels within the individual elements are generally below capacity.

The results given within this report represent a design in progress. The global and elemental performance to the Level 1 requirement although fairly satisfactory will now be further refined as the design progresses into the EPD stage of the project in order to improve the structural performance and reduce the overall steel weight.

10. 基础研究

10.1 介绍

这里选择双塔之中较高的塔楼 1 下的桩筏基础进行分析。分析中采用了奥雅纳开发的有限元分析软件 GSA 进行分析，此软件特别功能是能考虑结构与土壤互相作用下的分析及设计。

10.2 概述

8 米深蜂窝状的筏基被模拟为格形结构、当中直身墙身中心距离为 5 米，厚度为 2 米，如图 10.1 所示。顶板与底板厚度分别为 2.0 米和 2.5 米。于分析中桩被模拟设于各网格交点位置，桩直径为 1.2 米，长度为 55 米。由布置中可看出，桩筏基的位置与重力荷载中心已接近似同心，故其中心位置与塔楼的中心位置形成偏心。由上部结构模型分析结果取得的各荷载工况所产生的竖向力应用于地基模型中作分析之用。

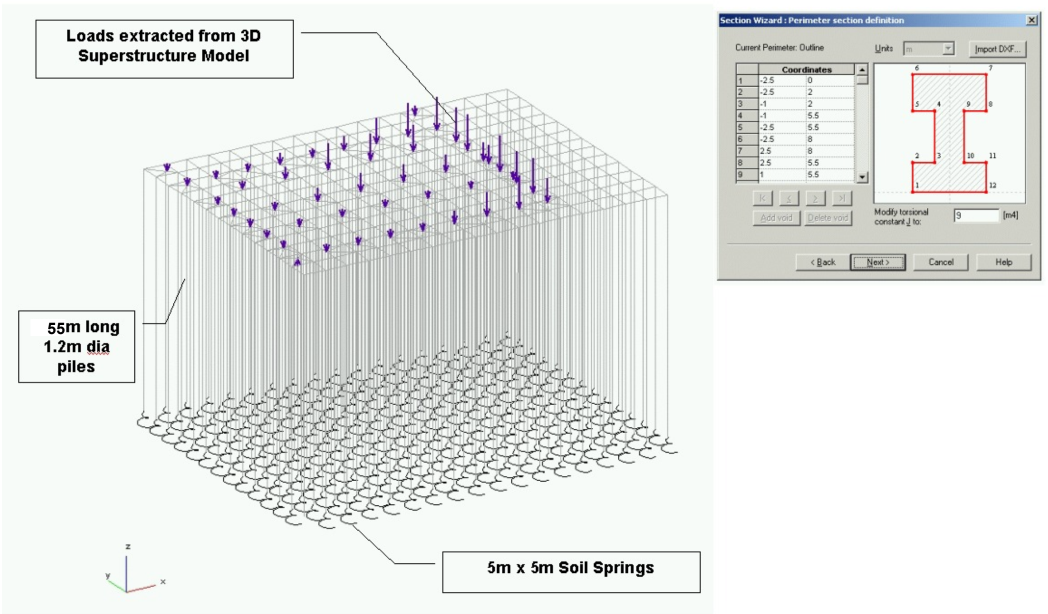


Figure 10.1 Pile Raft Model
图 10.1 桩筏基础模型

由于地震反应力是利用反应谱分析中产生的各项数值均为正数，如果将此荷载应用于筏基设计中，将不会反映筏基所承受的倾覆力矩作用。故将前两个振型产生的反应力放大至最大反力值以用作地震荷载。在罕遇地震的振型 1 与振型 2 的荷载表示如图 10.2。

10. FOUNDATION STUDY

10.1 Introduction

The preliminary foundation analysis described here has been performed on the piled raft supporting Tower 1, the taller of the two towers. Analysis has been performed using the Arup GSA finite element package, which includes an iterative structure-soil interaction capability.

10.2 General

The cellular raft was modelled as a grillage with an overall depth of 8m and orthogonal 2m ribs at 5m centres as shown in Figure 10.1. The top and bottom slabs are 2.5m and 2m thick respectively. Piles were modelled as 1.2m diameter, 55m long vertical sticks at each grid intersection. As can be seen, the piled raft was positioned to be approximately concentric with the gravity loads and its centre was therefore offset from the centre of the tower. Vertical reactions were extracted from the superstructure model for each analysis case and applied as loads to this model.

As the seismic reactions were generated from a response spectrum analysis, all values were positive and if applied as loads to the raft would not generate the appropriate overturning moments. The reactions from the first two modes of vibration were factored to match the peak response values and used to generate seismic loading. The loads for Modes 1 and 2 Seismic Level 3 are shown in Figure 10.2.

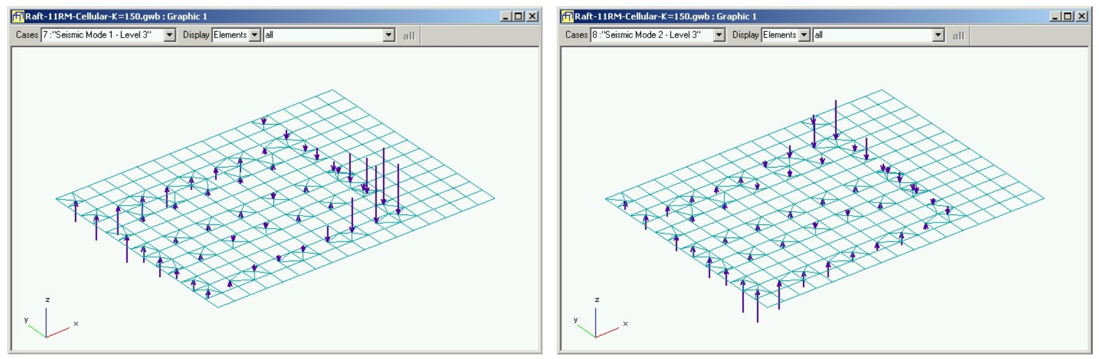


Figure 10.2 Loads under Seismic Level 3
图 10.2 罕遇地震下的荷载

10.3 分析

10.3.1 结构－土壤的互相作用

于模型的重力荷载分析中，最初每支桩均采用同样的弹簧刚度。然后将弹簧的反应力应用于地面以下 75 米深及 5 米正方土壤模型中。该土壤岩层模型记载于 5.5 节中的表内。本地基沉降计算是采用奥雅纳的 VDISP 软件，采用 Mindlin 法进行计算。各桩基底的荷载沉降关系是利用调整后的弹簧刚度进行计算。此方法重复计算直至最终沉降值收敛为止。

由分析结果得出的桩基荷载分布表示如图 10.3。

10.3.2 均布桩弹簧

各桩的总刚度由桩荷载除以筏板处相应沉降值得到。根据桩的平均刚度（150MN/米），采取相同弹簧支撑的排格筏基进行简化分析。图 10.4 表明，算出的桩荷载非常接近结构－土壤互相作用分析得到的荷载。因此，该简化模拟法可以应用于后续分析中。

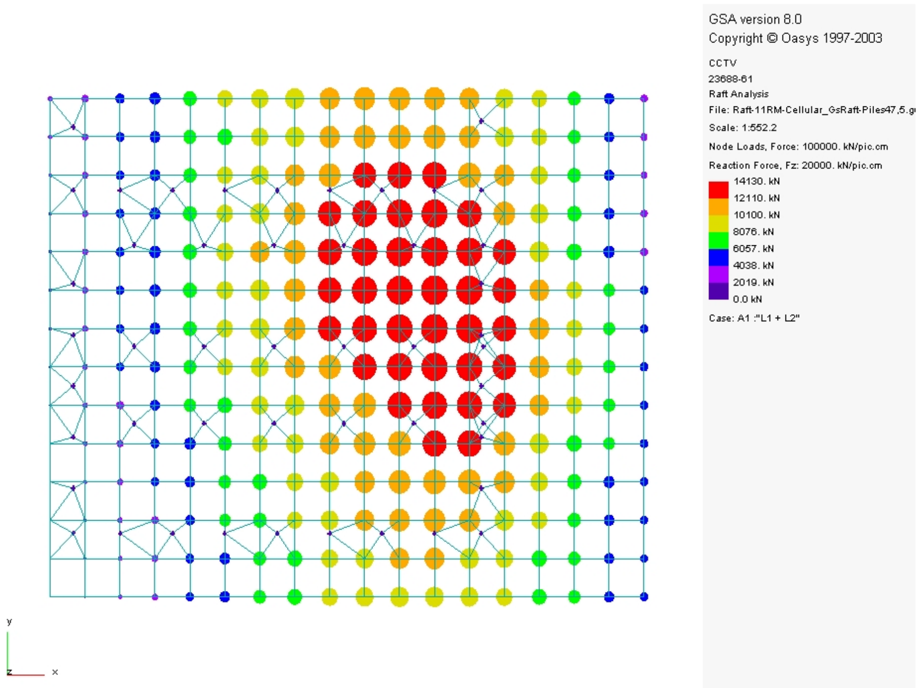


Figure 10.3 Pile Load Distribution by VDISP
图 10.3 桩荷载分布图 (VDISP 软件结果)

10.3 Analyses

10.3.1 Structure-soil Interaction

The model was initially analysed for gravity loading with uniform springs at the base of each pile. Reactions from these springs were then applied approximately 75m below ground to 5m square loaded areas in a soil model incorporating the strata described in the table of Section 5.5. Settlements were calculated by the Arup VDISP sub-program using the Mindlin method. The load settlement relationship at the base of each pile was used to calculate revised spring stiffness. This procedure was repeated until convergence was achieved.

The distribution of pile loads resulting from this analysis is shown below.

10.3.2 Uniform Pile Springs

An overall stiffness was calculated for each pile by dividing its load by its settlement at raft level. A simplified analysis was then performed with the raft grillage supported by equal springs based on the average stiffness of the piles (150MN/m). Figure 10.4 shows that the resulting pile loads are very close to those from the structure-soil interaction analysis. This simplified modelling was therefore used for subsequent analyses.

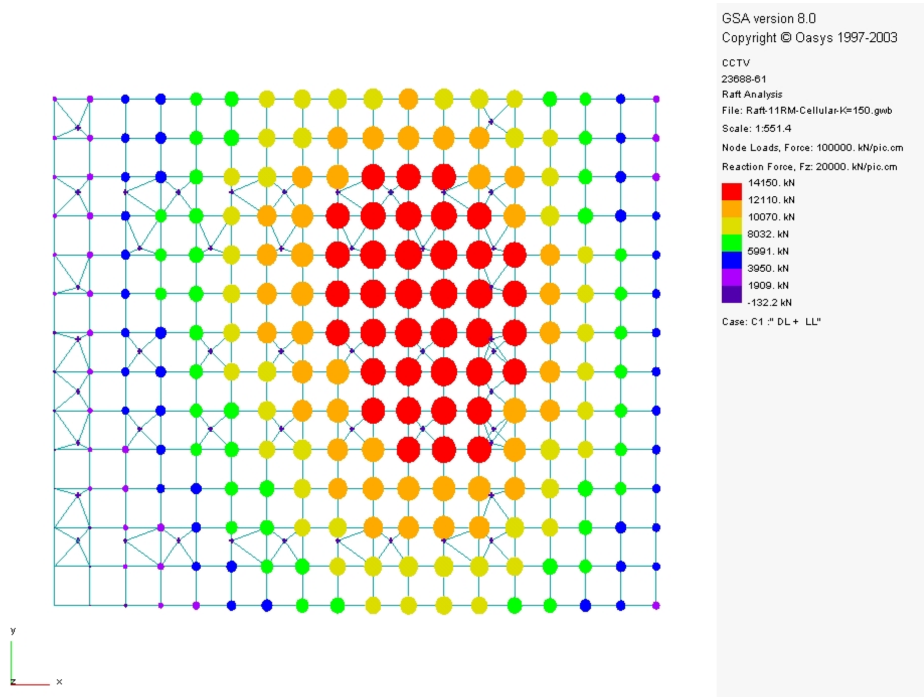


Figure 10.4 Pile Load Distribution (by simplified method)
图 10.4 桩荷载分布图 (简化方法结果)

10.3.3 桩承载力容量

单桩 (1.2 米直径) 承载力估计为 10MN。极限承载力估计为 20MN。但事实上，如果各桩发生超出承载力的情况，超出的荷载会重新分配到相邻桩上，只要荷载不超出混凝土桩身的承载能力即可。因此进行了迭代分析，通过减小过载桩的弹簧刚度，将桩的荷载限制定于 10MN。将桩荷载分布结果与相等弹簧模型进行比较，其弯矩和剪力图均显示于图 10.5。

10.3.3 Capacity Check

The working capacity of the 1.2m diameter piles has been estimated at 10MN for gravity loading and the ultimate capacity is estimated to be 20MN. The elastic analyses give maximum pile loads of 14.2MN. In practice, if individual piles are overloaded, they can redistribute the excess load to adjacent piles, provided the concrete shaft capacity is not exceeded. An iterative analysis has therefore been performed limiting the pile loads to 10MN by reducing the spring stiffness associated with overloaded piles. A comparison of the resulting pile load distribution with the equal spring model, together with bending moment and shear forces diagrams is included below (Figure 10.5).

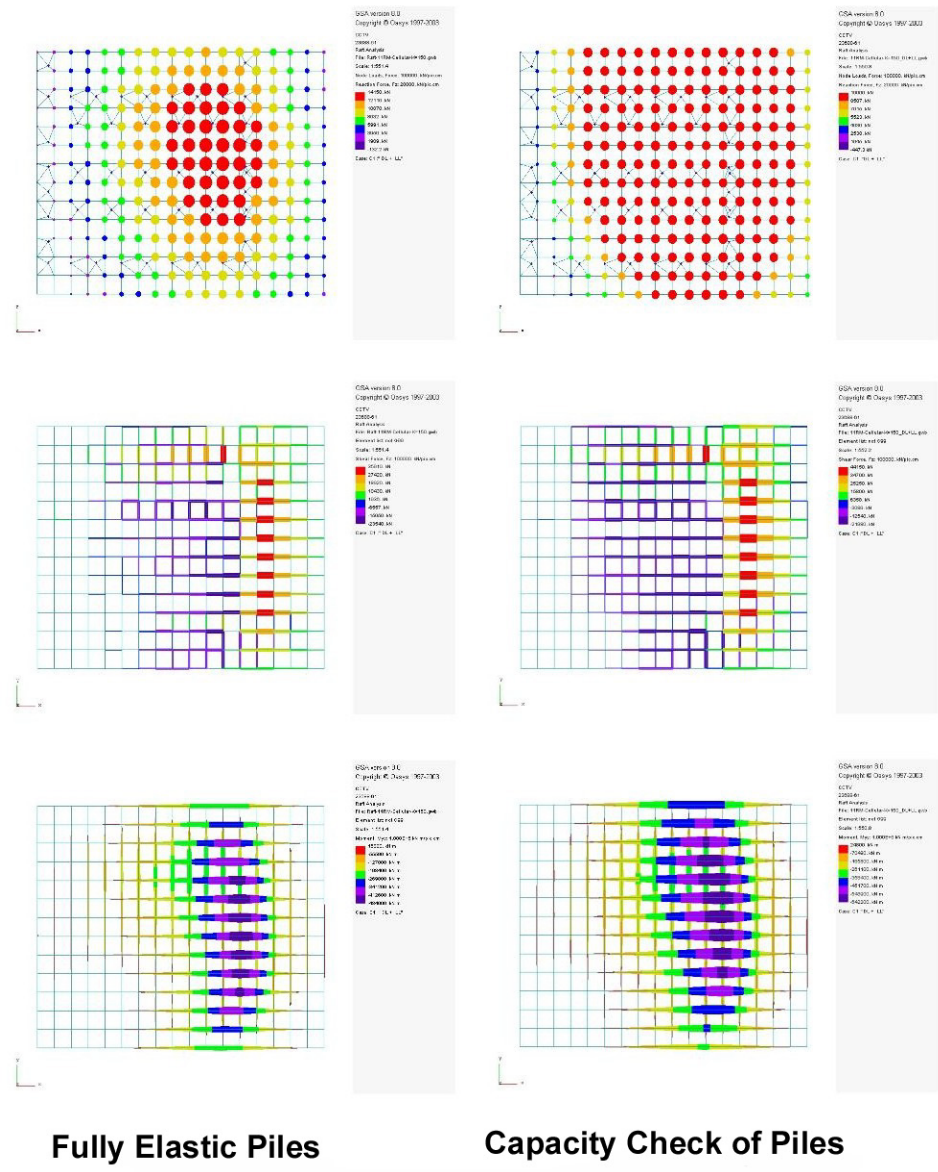


Figure 10.5 Pile Load Comparison
图 10.5 桩荷载比较

于桩承载力复核中，桩的荷载在桩群中重新分配，于最不利情形下进行桩承台设计。桩承载力复核必须满足与外荷载平衡的原则。

进一步的分析则考虑包括风及地震荷载的桩荷载组合值。下表列出了分析中采用的超限应力水平。所有工况的拉力已被限制于 1MN。

侧向荷载	应力超限幅度	容量 (MN)
风	20%	12
多遇地震	50%	15
罕遇地震	100%	20

The capacity check analysis which, redistribute the pile loads through the group, constitutes the most onerous case for the design of the pile cap. It should be noted that the capacity check of the piles satisfies equilibrium with regard to the applied loads.

Further analyses have been performed limiting pile loads for combinations including wind and seismic loading. The table below indicates the levels of over-stresses that have been taken for these analyses. Tension has been limited to 1MN for all cases.

Lateral Loading	Overstress	Capacity (MN)
Wind	20%	12
Seismic Level 1	50%	15
Seismic Level 3	100%	20

10.4 设计包络

恒载、活载和罕遇地震产生的荷载标准值按 4.6.1 节描述的荷载组合进行分析及包络，以找出基础构件的剪应力和弯矩的最大值。以上组合在图 10.6 中表示。

凡是发生剪应力超出限值的情况，空穴位置可进行填充。

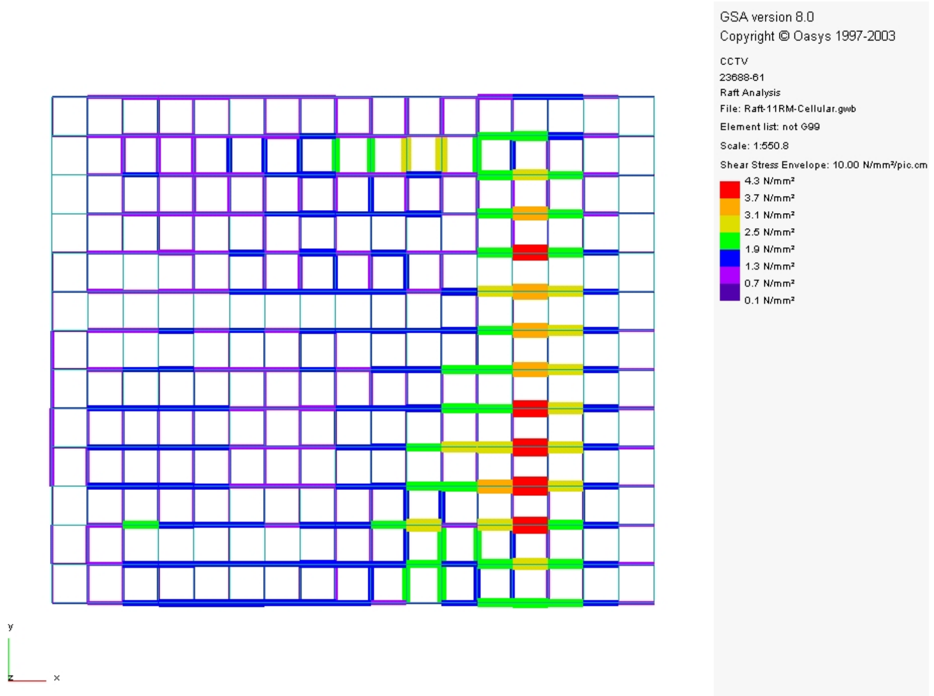


Figure 10.6a Shear Stress Envelope
图 10.6a 剪力包络图

10.4 Design Envelopes

The ultimate load combinations described in section 4.6.1 were created for all analyses and enveloped together with un-factored combinations of dead, live and seismic level 3 loads to give critical values of shear stress and bending moment for the raft elements. These are presented in the diagrams below (Figure 10.6).

Where shear stresses are excessive, voids may be filled.

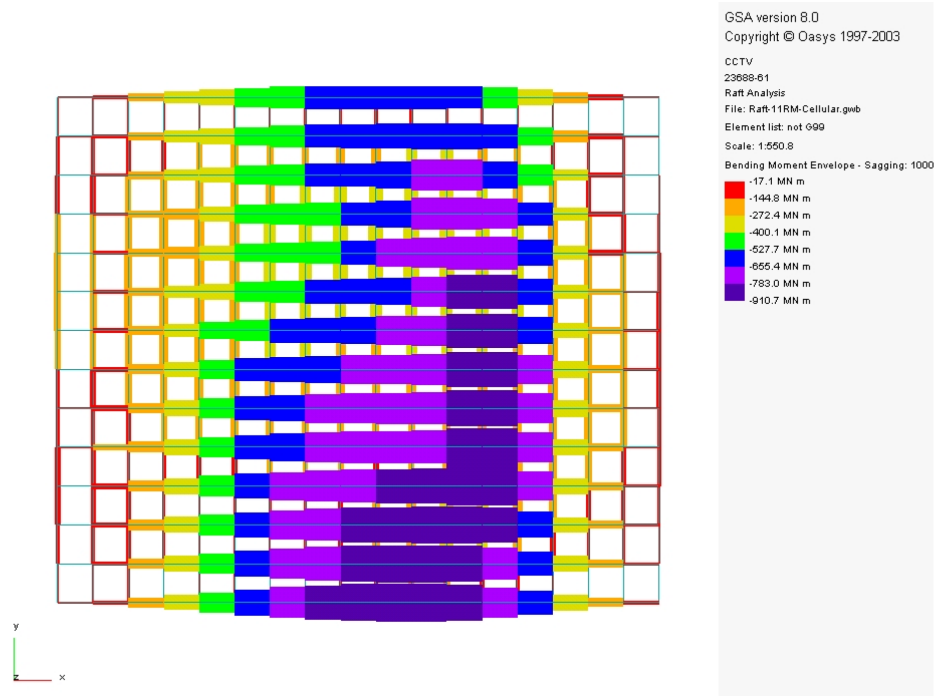


Figure 10.6b Bending Moment Envelope (Sagging)
图 10.6b 正弯矩包络图

相对于 910MNm 的最大弯矩，截面估算的正弯矩承载力值为 1300 MNm。

筏基的负弯矩不大。

The sagging capacity of the section has been estimated as 1300 MNm compared with a maximum moment of 910 MNm.

The raft does not attract significant hogging moments.

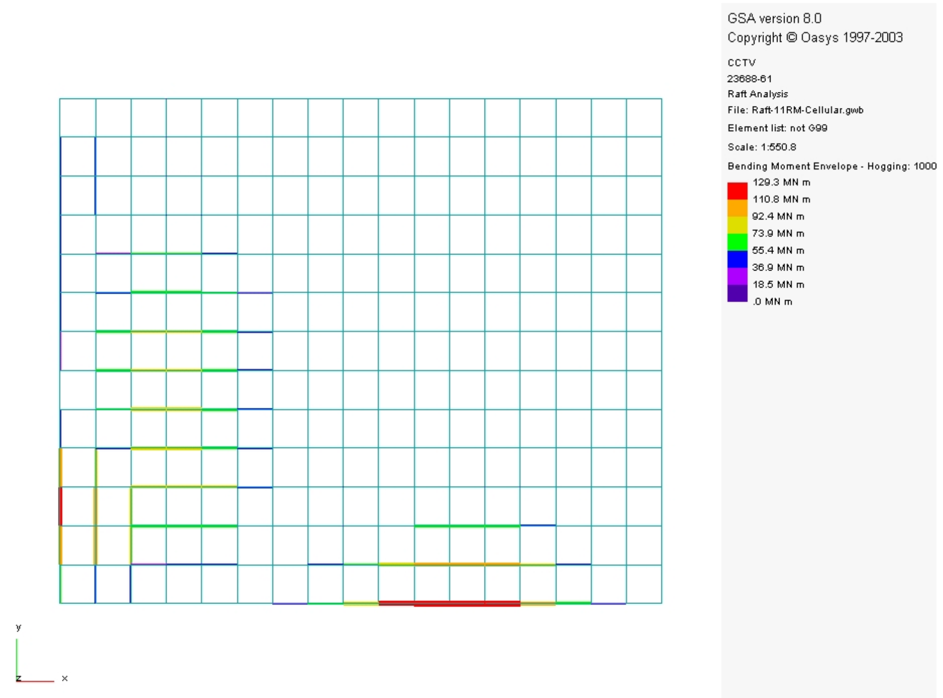


Figure 10.6c Bending Moment Envelope (Hogging)
图 10.6c 负弯矩包络图

11. 非线性地震反应分析与结构抗震性能评价

11.1 目的

本节介绍中央电视台新主楼结构在罕遇地震作用下的非线性反应分析方法、步骤、取得的结果及其抗震性能的初步评价。主要目的有两个：

1. 了解该结构屈服后阶段的弹塑性行为，当地面运动烈度高于设计常遇地震地面运动烈度时，该结构可能屈服从而进入弹塑性工作阶段。
2. 评价该结构的罕遇地震作用下的抗震性能，从而进一步判定该结构是否满足在罕遇地震作用下 (50 年超越概率约为 2%) 不倒塌的抗震设计目标。

根据中国建筑抗震设计规范 GB50011-2001，在罕遇地震作用下，结构允许出现严重结构性破坏，但应控制在规定范围内，以免倒塌。结构在非弹性工作阶段的定量地震反应参数，有必要通过非线性地震反应分析来确定并用来作为结构抗震性能评价的依据。

11.2 方法、步骤与参考文件

为达到罕遇地震作用下不倒塌的抗震设计目标，本节采用以抗震性能为基准的设计思想和以位移为基准的抗震设计方法。

结构丧失稳定以致倒塌是由于重力作用在有过大侧向变形后结构的几何状态所引起的。这种效应被广泛称作 “P-Δ” 效应。因此，达到防倒塌设计目标的中心思想是限制结构的最大层间位移角在规定的限值以内。GB50011-2001 规定应对罕遇地震作用下结构的弹塑性最大层间位移角进行了验算并要求限制结构的最大层间位移角在 1/50 以内 (对多层钢结构)。这项限值就是本节所采用的最大层间位移角可接受的限值。

限制结构的最大弹塑性层间位移角还不足以保证达到防倒塌的抗震设计目的。结构构件的破坏 (以结构构件的弹塑性变形来衡量) 也必须被限制在可接受的规定限值以内，以保证结构构件仍有能力承受地震结束后作用在结构上的重力荷载。

然而，GB50011-2001 并没有提供结构构件的弹塑性变形限值。因此，有必要参考其它被国际广泛应用的以抗震性能为基准的抗震设计指导文件。本节采用美国联邦紧急事务管理署 (FEMA) 第 356 号文件 (以下简称 FEMA356) “建筑抗震修复预标准及其说明” 所提供的结构构件弹塑性变形可接受限值，以及所建议的结构非线性地震分析方法与步骤。对钢结构构件如钢梁、钢柱及钢斜撑，弹塑性变形限值直接引用 FEMA356 所提供的限值。

11. NON-LINEAR SEISMIC RESPONSE ANALYSIS AND SEISMIC PERFORMANCE EVALUATION

11.1 Objectives

This section presents the non-linear seismic analysis methodology, procedure, results achieved and an initial evaluation of the seismic performance of the CCTV building structure under the design rare earthquake. The objectives are two folds:

1. Understanding the post-yielding inelastic behaviour of the CCTV building structure when it is subjected to earthquake ground excitations having intensities higher than that causing onset of yielding in this structure.
2. Evaluating the seismic performance of the structure in order to assess if it satisfies the requirements for achieving the collapse prevention seismic performance objective when subjected to the design rare earthquake corresponding to a probability of exceedance of 2% in 50 years.

In accordance with the provisions of the Chinese seismic building code (Ministry of Construction, GB 50011 – 2001 Code for Seismic Design of Buildings, Beijing, 2001), under the design rare earthquake, building structures are permitted to be severely damaged but collapse must be prevented. It is therefore expected that the CCTV structure is loaded well into the inelastic range when subjected to the design rare earthquake. Non-linear seismic response analysis becomes necessary in order to obtain quantitatively the structure's key seismic response parameters for the purpose of seismic performance evaluation.

11.2 Methodology, Design Procedure and Reference Documents

The performance-based seismic engineering methodology and the displacement-based seismic design procedure have been adopted in this section in order to achieve the collapse prevention seismic performance objective under the design rare earthquake.

The loss of stability and subsequent collapse of buildings are caused by gravity acting on the deformed geometry (configuration) of building structures, an effect generally termed the "P-Δ" effect. Hence, central to achieving the collapse prevention performance objective is to limit the inter-storey drifts to be within acceptable values. GB 50011 – 2001 requires design engineers to calculate the inter-storey drift ratios of structures under the design rare earthquake and to carry out explicit checking in order to ensure that the inter-storey drift ratios of all storeys be less than the acceptable limits, this being 1/50 (2%) for multi-storey steel structures. This is the building global seismic performance acceptance limit adopted for the collapse prevention objective for the CCTV building.

Limiting the inter-storey drifts alone is not sufficient to achieving the collapse prevention seismic performance objective under the design rare earthquake. The degree of damage to individual structural components (members), quantified by the inelastic deformations of structural components, must also be limited to acceptable values such that individual structural components are still capable of carrying the gravity force.

However, GB 50011 – 2001 does not provide any guidelines on acceptable limitations of inelastic deformations of structural components. Therefore, other performance-based seismic design documents need to be referenced for this purpose. For seismic design of the CCTV building, the US "Pre-standard and Commentary for the Seismic Rehabilitation of Buildings", commonly referred to as the FEMA 356 document, has been adopted as the reference document for performance-based seismic design methodology, analysis and design procedure, and component inelastic deformation acceptance criteria for steel members such as steel beams, columns and braces.

对于钢-钢筋混凝土组合构件，FEMA356 没有提供弹塑性变形限值的建议。因而必须通过其它方法来建立。在本节，这类构件的轴向压缩变形限值由约束混凝土的最大可用压变形决定。其最大轴向拉伸变形限值则采用 FEMA356 中建议的受拉钢构件的轴向受拉变形限值。这类构件弯曲塑性铰最大转角限值则通过截面弯矩-曲率分析来决定。

11.3 结构非线性地震反应分析模型

11.3.1 整体结构模型

为进行结构的非线性地震反应分析, 本节建立了一个三维结构整体分析模型，如图 11.1 所示。该结构模型模拟了外筒结构的每一个梁、每一个柱和每一个斜撑。内筒只承受重力荷载，并非抗侧力结构的一部份。故内筒在结构地震分析模型中被略去。结构整体模型从地表面以下 14.5 米处，桩基承台表面开始。位于桩基承台面的结点被假定为固定端，所有六个自由度被约束住。

楼板由弹性板-壳弹元模拟，同时考虑平面内刚度及平面外弯曲刚度。平面外弯曲刚度由楼板厚度及支撑楼板的钢梁截面尺寸决定。在本节，与梁-柱-斜撑结点在同一立面高度处的楼板直接由板壳弹元模拟。这样每两层的楼板有一层楼板由板壳单元直接模拟，另一层楼板则被略去。其相应的质量和重力荷载则被分配到其相邻的楼板单元中。

楼板单元材料的质量密度与重力荷载代表值相对应。每层楼板的重力荷载代表值都通过结构静力分析分配到支持该楼板的梁-柱-斜撑结点上。因而在本节的结构整体分析模型中，楼板的内部结点上并无重力荷载作用因而并无竖向位移。

从平面看，结构的前两个振型沿两个互相垂直的方向振动。结构的第一振型沿悬挑结构部份的尖端所指的方向振动。第二振型沿连接两个塔楼的直线方向振动，悬挑结构相当于一个横梁，两个塔楼相当于两个柱子，形成框架作用。在本节中，结构的总体坐标系中的水平轴 X 和 Y 是分别平行于结构的前两个振型的振动方向的。X 轴平行于第一振型的方向。Y 轴平行于第二振型的方向。

FEMA 356 does not provide deformation acceptance limits for composite steel reinforced concrete (SRC) components. Therefore, the deformation acceptance limits for composite SRC columns of the CCTV building need to be determined by other methods. In this report, the acceptable axial compression deformation limit is determined by the maximum usable compression strain of confined concrete and the acceptable plastic hinge rotation angles are determined by column cross section moment-curvature analysis, as described in detail in the next section.

11.3 Non-linear Structural Model for Seismic Response Analysis

11.3.1 Global structural model

A three-dimensional global structural model, as illustrated in Figure 11.1, has been built for the purpose of non-linear seismic response analysis. This model has explicitly modelled every column, every beam and every brace of the perimeter tube structures. The internal cores, which support gravity load only, have been neglected in this model for simplicity. The structural model starts from the pile cap level 14.5 m below the ground surface. Nodes at the pile cap level are assumed to be fully restrained in all six degrees of freedom.

Floor diaphragms are modelled explicitly by elastic shell elements having stiffness in both membrane and out-of plane bending, determined by floor slab thickness and cross section properties of steel floor beams. In this report, only floor diaphragms at elevation levels of beam-column-brace joints have been modelled explicitly. The mass and gravity load associated with floor diaphragms not explicitly modelled are equally distributed to floor diaphragms above and below. The material mass density of the floor shell elements corresponds to the gravity load representative value. No gravity force is applied to the floor shell element therefore they do not deflect vertically.

On plan, the first two free vibration modes of the CCTV building show the building vibrates along two perpendicular horizontal directions, corresponding to the results from the elastic analysis models. In the first mode, the building vibrates in the direction pointed to by the tip of the overhang building. In the second mode, the building vibrates along the direction that connects the two towers, forming a portal frame action with the two tower acting as columns and the overhang building acting as a beam. For the non-linear model the global horizontal axes X and Y are oriented such that the X-axis is parallel to the direction of the first mode and the Y-axis is parallel to the direction of the second mode which is in fact rotated through 45 degrees to the axes of the elastic analysis model. This revision to the axis system is performed to make post-processing the results simpler.

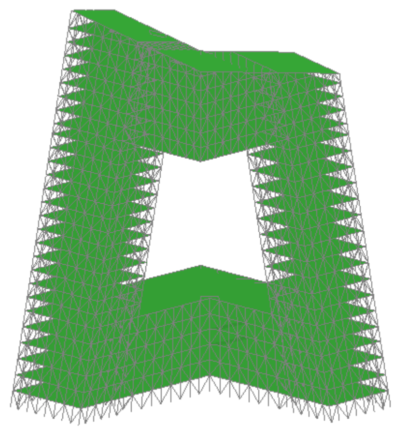


Figure 11.1 Three-Dimensional Structural Model for Linear Analysis
图 11.1 非线性分析结构模型

11.3.2 施工阶段的结构受力模拟

在施工阶段中，结构重力荷载由结构本身承受，不由临时结构支撑。再加上两个塔楼几何构造上是倾斜的, 在结构自重作用下两个塔楼产生变形，互相靠陇。结构在自重作用下的变形增加重力荷载作用下的倾覆力矩。因而增加底部柱的轴力。悬挑部份结构是在两个塔楼在自重作用下的变形发生完成后才开始施工建造。因此，考虑施工阶段时结构底部柱子的轴力比不考虑施工阶段，按结构完全施工完毕后的结构模型 (图 11.1) 进行分析得到的轴力要更为不利。在后一种情况下，悬挑结构的刚度减小两个塔楼在重力荷载下的变形并进而减小结构底部的倾覆力矩。

为了正确反应结构在整个施工过程中及施工完成后的受力状态，结构的总体分析模型考虑了结构在两个施工阶段的受力状态和对应的分析模型，如图 11.2a 和 11.2b 所示。结构在此两个阶段的受力状态分别由两个不同的静力分析确定。

11.3.2 Modelling of construction stages

As discussed in the elastic modelling section of the report the tower construction methods, given the inclined configuration of the two towers, must be taken into account in the analysis. The deformation of the two towers due to their lean during construction stages increases the overturning moment due to gravity load and therefore increases the column axial forces near the base of the towers. The overhang building will be built later after some of the deformation of the towers has already occurred. Therefore, column axial forces in the towers near the base considering this construction sequence will be more unfavourable than those calculated based on the completed structural model (Figure 11.1) without considering the staged construction. In the latter case, the stiffness of the overhang building reduces the deformation of the towers and hence reduces the overturning moment due to gravity.

In order to capture the true build-up of the member forces during construction stages, the global building structural model has considered a two-stage construction sequence, as illustrated in Figures 2a and 2b, respectively. Two static analyses are carried out corresponding to the two construction stages.

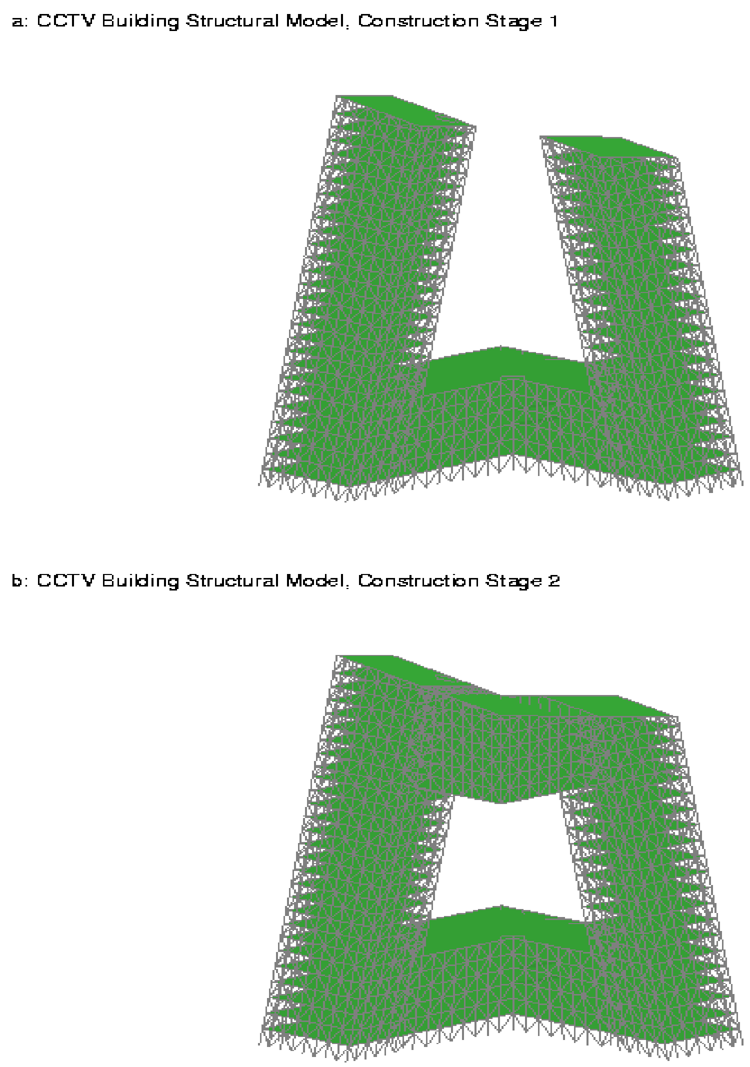


Figure 11.2 Structural Model for Construction Stages Analysis under Gravity Load
图 11.2 重力荷载下考虑施工过程的结构模型

本节采用一种将“死”单元“击活”的技巧来进行结构施工阶段的结构分析，模拟结构在整个施工阶段过程中的刚度和重力荷载的变化。结构整体分析模型按图 11.2b 建立，包括全部结构的结点和单元。然后，在进行第一阶段结构重力荷载作用下的受力分析时，在第二施工阶段才建造完成的单元被指定为“死”单元。这些“死”单元没有刚度，也没有重力荷载作用。当第二施工阶段受力分析开始时，这些“死”单元才被“击活”恢复其应有的刚度和作用在这些单元上的重力荷载。此时，结构在第一施工阶段的变形已经完成。这些被“击活”的单元建立在变形后结构的几何状态 (结点坐标) 上。由于所采用的有限元分析软件是大变形有限元分析软件，静力平衡建立在变形后结构的几何状态 (结点坐标) 上，重力荷载产生的 P-Δ 效应被自动考虑。结构构件在第二施工阶段的重力荷载作用下的内力与第一阶段重力荷载产生的内力相加，因而准确地模拟在整个施工阶段过程中结构构件内力的变化过程。

11.3.3 结构构件的模拟及其可接受的弹塑性变形限值

11.3.3.1 钢梁

钢梁由梁单元模拟。在中心支撑钢框架结构中，例如中央电视台新主楼结构，抗震设计意图是主要由斜撑消耗能量。梁和柱尽可能在弹性范围工作，不作为主要耗能构件。在中央电视台新主楼外筒结构中，每一个梁都和柱及斜撑相连接，形成桁架作用。因而在地震力作用下，梁以轴向受力为主，弯矩是次要效应。因此，本节将假设梁为弹性，因为楼板平面内的刚度大，钢梁轴向屈服的可能性很小。另一方面，钢梁弯曲屈服的可能性也很小，因为弯曲只是钢梁单元的次要效应。

11.3.3.2 钢-钢筋混凝土组合柱

钢-钢筋混凝土组合柱由梁单元模拟。在中央电视台新主楼结构中，这些柱主要以轴向受压与受拉为主，弯曲变形是次要效应。虽然设计意图是主要由斜撑屈服(屈曲)耗能。柱子在罕遇地震作用下也有可能轴向受压或受拉屈服。在本节，钢-钢筋混凝土组合柱由非线性梁-柱单元模拟，允许在柱的两端形成压-弯或拉-弯塑性铰。

钢-钢筋混凝土组合柱由轴向钢板、轴向主筋、横向约束箍筋和混凝土组成。其截面轴向弹性有效刚度值，EA，由其等效截面弹性轴向刚度计算确定，考虑截面各种材料的实际面积和各种材料的弹性模型。截面对两个主轴的抗弯有效刚度值，EI，则通过截面弯矩-曲率分析计算决定。在进行截面弯矩-曲率分析时，考虑了各柱在重力荷载代表值作用下的轴力值，截面弯矩-曲率分析同时考虑弯矩和轴力的作用。截面对两个主轴的弯矩-轴力相互作用屈服曲线 (N-M 曲线) 也通过截面弯矩-曲率分析确定。

弯矩-曲率分析采用纤维模型，截面被离散化为许多小面积称作纤维。每一个纵向主筋是一个纤维。同时采用平截面假定及材料单向应力-应变曲线。弯矩-曲率分析由位移控制，假定截面曲率从零开始单向逐步增加。由截面曲率计算出每个纤维的应变，再根据每个纤维材料的单向应力-应变曲线确定每个纤维中的正应力。最后通过截面应力积分求得截面的弯矩和轴力值。

组合柱中钢板和主筋的材料为软钢，其拉-压应力-应变曲线假设为理想弹塑性双线型，忽略应变硬化。材料屈服强度取其对应的材料设计强度值。软钢的极限拉应变限值假定为 15%。

A technique that turns “dummy” elements to “active” elements has been employed in this study to simulate the evolution of structural stiffness and gravity load during the two construction stages. The global structural model is built as shown in Figure 11.2b, including all elements and nodes of the completed structure. However, those elements constructed in stage 2 are initially assigned as “dummy” elements that have neither stiffness, nor gravity during the stage 1 analysis. At the beginning of the stage 2 analysis, these elements become “alive” by recovering their full stiffness and gravity from the deformed geometry (configuration) established by the stage 1 gravity load analysis. Member forces obtained in the stage 2 analysis are added to those obtained in the stage 1 analysis, thus accurately simulating the build-up of member forces during the construction process.

11.3.3 Structural component (member) modelling and acceptable inelastic deformation limitations

11.3.3.1 Steel beams

Steel beams are modelled by beam elements. In concentrically braced frames such as the perimeter structure of the CCTV building, the intension is to dissipate energy by the diagonal braces, not by beams and columns. Furthermore, in the perimeter structure of the CCTV building, every beam is connected to columns and braces. As a result, under seismic action the beams act primarily axially like truss members with bending as a secondary action. Hence, beams are modelled as elastic members since they are unlikely to yield axially due to the high in-plane stiffness of the floor slabs. Furthermore, they are unlikely to yield in bending either because bending is a secondary action in the beams.

11.3.3.2 Composite steel reinforced concrete columns

Composite steel reinforced concrete (SRC) columns are modelled as beam-column elements. They act primarily axially in compression and tension, with bending as a secondary action. Although it is not the design intension to dissipate energy in columns, yielding of columns in compression and tension may occur under the action of the design rare earthquake. In this report, the composite SRC columns are modelled as non-linear beam-column elements that permit axial-flexure plastic hinges to form at both ends of each column.

Composite SRC columns consist of longitudinal steel plates, longitudinal reinforcement bars, transverse confining reinforcement hoops and concrete. The effective cross section axial stiffness, EA, of this type of component is calculated considering the actual area of steel plates, longitudinal reinforcement bars and concrete together with their respective material modulus of elasticity. The effective cross section flexural stiffness, EI, about both principal axes of a column section is calculated by moment-curvature analysis with the axial force due to the representative value of the gravity load applied to the section being analysed. The axial force - bending moment (N - M) interaction yielding curves about the two principal axes of a column section are also calculated by moment-curvature analysis.

Moment-curvature analysis employs the fibre model, with a cross section being discretised to many small areas termed fibres. Each longitudinal reinforcement bar is a fibre. In moment-curvature analysis, a plane cross section is assumed to remain a plane cross section. Material uni-axial stress-strain curves are employed directly in determining the normal stress in each fibre from strain values, which in turn are related to the monotonically increasing value of curvature.

An elastic-perfectly-plastic stress-strain relationship has been assumed for mild steel plates and reinforcements. The yielding strength values were taken as the respective material design strength. The limiting (fracture) tensile strain in the stress-strain curves is assumed to be 15%.

混凝土的单向应力-应变曲线取为由 Mander 和 Priestley 推荐的曲线 (Mander, J. B, and Priestley, M. J. N., “Observed Stress – Strain Behaviour of Confined Concrete”, Journal of Structural Engineering, ASCE, Vol. 114, No. 8, pp1827 – 1849, 1988) . 对非约束混凝土，其应力-应变曲线的最大压应力值取为其设计强度值，27.5MPa (C60 混凝土)。其最大可用压应变值取为 0.004。对约束混凝土，本节考虑了横向约束效应对强度和最大可用压变值的增加。强度的增加考虑为 30%。最大可用压应变值考虑为 0.015。这两项数值都是比较保守的下限值，比较容易达到。混凝土的抗拉强度被忽略不计。

弯矩-曲率分析采用计算机程序 XTRACT (http://www.imbsen.com) 进行计算。图 11.3 表示一个组合柱截面的纤维模型。该截面对两个主轴的 N-M 相互作用屈服曲线示于图 11.4 中。其对两个主轴的弯矩-曲率曲线绘于图 11.5 中，与重力荷载代表值作用下的柱轴力值相对应。图 11.4 与 11.5 所示曲线作为非线性动力有限元分析数据，用来决定梁-柱单元的轴力-双向弯曲相互作用屈服曲面及屈服后弯矩-塑性铰转角恢复力曲线。

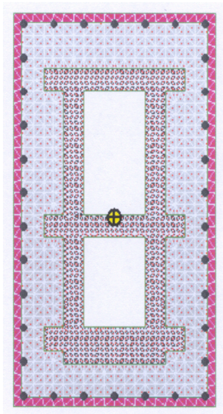
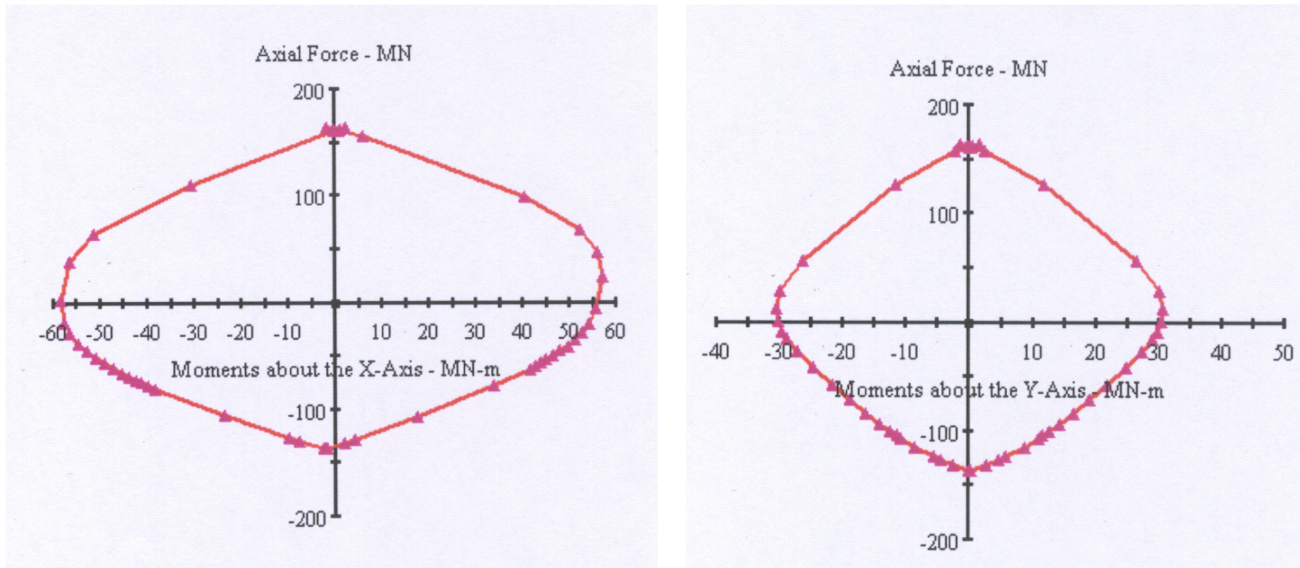


Figure 11.3 Fibre model of a steel SRC column cross section
图 11.3　典型 SRC 柱截面的纤维模型

The Mander - Priestley concrete uni-axial stress-strain curve (Mander, J. B, and Priestley, M. J. N., “Observed Stress – Strain Behaviour of Confined Concrete”, Journal of Structural Engineering, ASCE, Vol. 114, No. 8, pp1827 – 1849, 1988.) has been employed in this report. For unconfined concrete, the maximum stress in the stress – strain curve is taken as its design strength (27.5 MPa) and the maximum usable compression strain is taken as 0.004. For confined concrete, the increase in strength and maximum usable compression strain due to confinement has been considered. The increase in compression strength is assumed to be 30% and the maximum usable compression strain is assumed to be 0.015. Both are realistic lower bound values for well-confined concrete and are therefore conservative assumptions. Tensile strength of concrete has been neglected.

Moment curvature analysis of cross sections was carried out employing the computer software XTRACT (http://www.imbsen.com). Figure 11.3 illustrates the fibre model of a composite SRC column cross section. Its N – M interaction curves about both principal axes are shown in Figure 11.4. The moment – curvature relationships of this section about the two principal axes are shown in Figure 11.5, corresponding to the compression axial force under the representative value of the gravity load. Curves shown in Figures 11.4 and 11.5 are input to the non-linear finite element analysis software for determining the axial force – bi-axial bending moment interaction yielding surface and the post-yielding bending moment – plastic hinge rotation angle relationship.



a) N-M Interaction Curve about the Major Axis　　b) N-M Interaction Curve about the Minor Axis

Figure 11.4 N-M Interaction Curves about the Two Principal Axes
图 11.4　绕两主轴的轴力-弯矩曲线

截面弯矩-曲率分析同时确定了组合柱截面最大弯曲变形能力，也就是柱最大可接受曲率限值。根据 FEMA356 的建议，假设塑性铰有效长度为截面受弯高度的一半。截面最大塑性铰转角可接受限值可根据最大曲率可接受限值计算确定。对每一种不同的组合柱截面，其重力荷载代表值作用下最不利受压柱截面的可接受塑性铰转角限值大约是 0.0065 弧度。该值由约束混凝土最大可用压应变值确定 (本节假定为 0.015)。

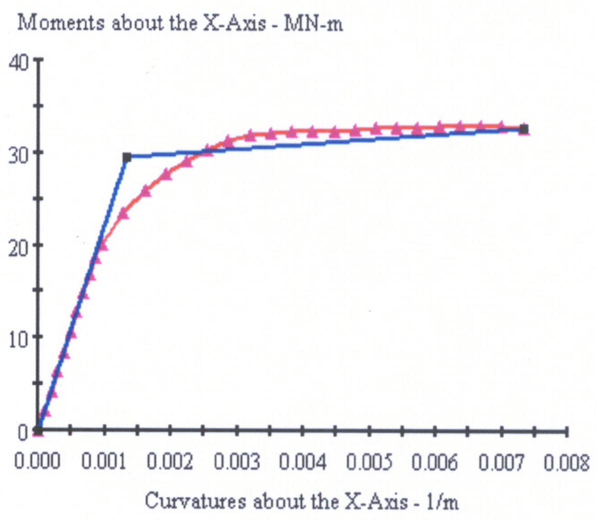
组合柱也可能因受压或受拉屈服。其屈服后轴力-轴向应变关系曲线取为理想弹塑性双线型。组合柱受压最大可接受弹塑性变形限值取决于约束混凝土的最大可用压应变值，本节取为 0.015。因此，本节将与防倒塌抗震设计目标对应的组合柱受压最大可接受弹塑性应变限值定为 0.015。其最大可接受弹塑性拉应变限值则采用 FEMA356 对受拉构件的建议，定为受拉屈应变的 9 倍，为 0.013。

压-弯成拉-弯塑性铰可在组合柱的两端出现。该类塑性铰出现的条件是，在地震反应分析过程中，柱端轴力值和柱端双向弯矩值的组合位于柱轴力-双向弯矩相互作用屈服曲面上。该曲面在三维空间像一个橄榄球，由柱的对两个主轴的两条 N-M 相互作用屈服曲线确定。塑性铰塑性变形的计算基于相关塑性流动法则。每个塑性铰有三个塑性变形分量: 一个轴向塑性应变分量和两个分别对两个主轴的塑性铰转角分量。根据相关塑性流动法则，塑性铰的塑性变形向量与塑性铰屈服曲面相垂直。

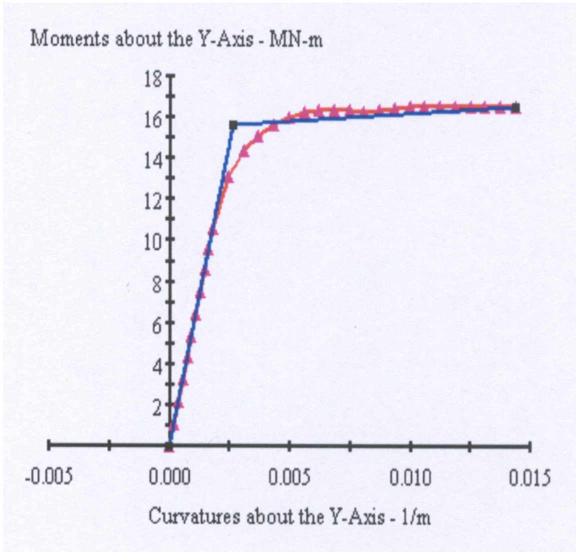
The moment – curvature relationships establish the limiting values of column bending deformation, namely the maximum curvature deformation capacity. Adopting the recommendation in FEMA 356 and assuming that the effective plastic hinge length is equal to half the cross section depth, the maximum curvature deformation capacity can be converted to the maximum plastic hinge rotation capacity. For each different steel SRC column cross-section configuration and geometry, the maximum plastic hinge rotation capacity of the column subjected to the highest compression force is approximately 0.0065 radians, limited by the maximum usable compression strain of confined concrete (0.015).

The steel SRC columns may yield axially in compression or tension. The post-yielding axial force – axial strain relationship is assumed to be elastic-perfectly-plastic. The maximum inelastic deformation capacity of these columns in compression is governed by the maximum usable compression strain of confined concrete, assumed to be 0.015 in this study. Therefore, the maximum acceptable steel SRC column compression strain is 0.015 for the collapse prevention performance objective. The maximum inelastic deformation capacity of these columns in tension for the collapse prevention performance objective is adopted from FEMA 356 for tension components, being 9 times the tensile yielding deformation (tensile axial strain 0.013).

Axial – flexure plastic hinges may form and develop plastic deformations at both ends of the steel SRC columns if values of column axial force and bi-axial bending moment during the seismic response analysis reach the axial force – bi-axial bending moment interaction yielding surface. This yielding surface looks like a rugby ball in three-dimensional space and is determined by the two N – M interaction curves about the two principal axes as shown in Figures 11. 5. An associative plastic flow rule is adopted for calculating the three components of plastic deformation of an axial – flexural plastic hinge, including the axial plastic strain, the plastic rotation angles about the two principal axes. According to the associated plastic flow rule, the vector of plastic deformation of a plastic hinge is normal to the yielding surface.



a) Moment – Curvature Curve about the Major Axis



b) Moment – Curvature Curve about the Minor Axis

Figure 11.5 Moment - Curvature Curves about the Two Principal Axes
图 11.5 绕两主轴的弯矩曲率曲线

11.3.3.3 钢柱

中央电视台新主楼结构的钢柱都是短柱，其非线性有限元模拟方法与钢-钢筋混凝土组合柱相同。钢柱对两个主轴的弯矩-轴力相互作用屈服曲线和弯矩-曲率曲线都由弯矩-曲率分析决定。由于桁架作用，钢柱以轴向受力为主，与斜撑相似。本节对钢柱与抗倒塌抗震设计目标对应的可接受轴向塑性变形限值采用 FEMA356 所建议的拉、压构件 (斜撑) 的弹塑性变形限值。可接受压缩变形限值为受压屈服 (屈曲) 轴向变形值的 7 倍。可接受受拉变形限值为受拉屈服变形的 9 倍。因此，为达到防倒塌抗震设计目标，钢柱的受压最大可接受应变限值为 0.01，其受拉最大可接受应变限值为 0.013。

11.3.3.4 钢斜撑

钢斜撑是中央电视台新主楼结构的关键构件。结构抗震设计意图是斜撑在地震作用下屈服 (屈曲) 以消耗能量。该结构中的斜撑都是短粗斜撑。其长细比小于 40。钢斜撑截面紧凑，为塑性截面。其翼缘板自由外伸宽度与厚度之比，以及腹板计算高度与厚度比都较小，可有效防止局部失稳。这类短粗斜撑在反复受压加载条件下具有好的延性，强度和刚度退化不明显，如图 11.6 所示。该图绘出了一个短粗斜撑典型的在反复拉压加载下的轴力-轴向变形滞回曲线。本节将钢斜撑模拟为梁单元，承受轴向拉、压作用。斜撑的轴向抗拉强度取决于材料的抗拉屈服强度和截面积。其抗压强度由弹塑性屈曲强度决定，大约为抗拉强度的 90%左右。

11.3.3.3 Steel columns

Steel columns in the CCTV building are short (stocky) columns and are modelled in the same manner as steel SRC columns. N – M interaction curves and moment – curvature relationships about the two principal axes are again computed by moment – curvature analysis of steel sections using the fibre model. Because steel columns act primarily axially in the CCTV building, they are regarded as axially loaded components like steel braces. The axial deformation acceptance limits for the collapse prevention performance objective are adopted from those for steel braces listed in FEMA 356. The acceptable limit in compression shortening is 7 times the yielding (buckling) axial compression shortening and the acceptable limit in tension elongation is 9 times the yielding tension elongation, resulting in an acceptable axial compression strain value of 0.01 and an acceptable axial tensile strain value of 0.013.

11.3.3.4 Steel braces

Steel braces are key components in the CCTV building structure, with the design intension being to dissipate energy in these components. The braces in the CCTV building are stocky, with slenderness ratios less than 40, and consist of plastic sections. This type of brace behaves in a ductile manner with insignificant strength and stiffness degradation under cyclic axial load as shown in Figure 11.6 for a typical stocky brace. In this report, the braces are modelled as beam elements acting axially in compression and tension. Their tensile axial strength is equal to the material tensile yielding stress multiplied by the cross section area. Their compression axial strength is determined by the inelastic buckling strength, being approximately 90% of the axial tensile strength.

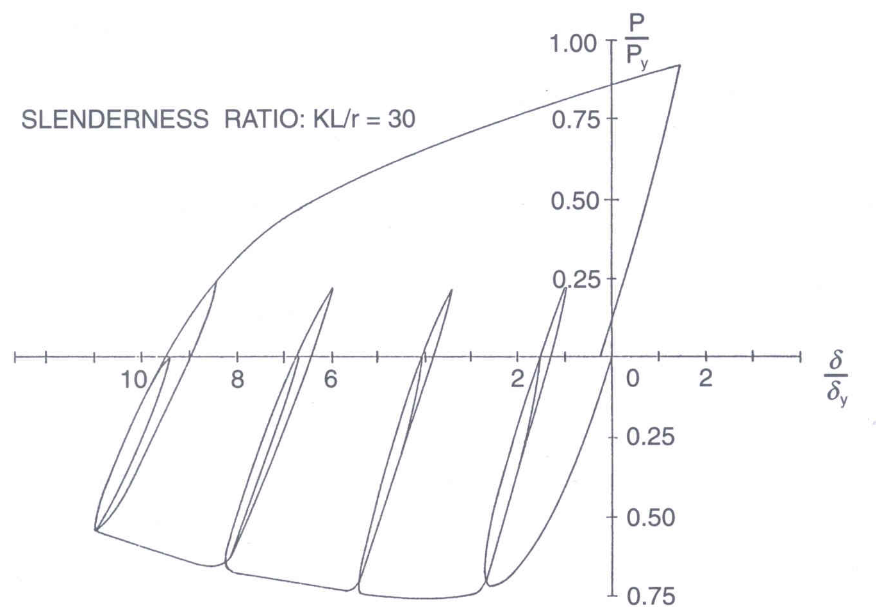


Figure 11.6 Typical Axial Force – Axial Deformation Hysteresis Loops of Stocky Braces under Cyclic Axial Load (Bruneau, M., Uang, C.- M., and Whittaker, A., Ductile Design of Steel Structures, McGraw-Hill, 1998)
图 11.6 典型短支撑在周期性轴力下的轴力-变形恢复力曲线

The maximum slenderness ratio of any primary brace is currently less than 30.

如图 11.6 所示，当轴向压缩变形不大于斜撑屈曲变形的 7 倍左右时，斜撑并无明显强度和刚度退化。因此本节采用理想弹塑性双线型恢复力曲线来描述钢斜撑轴力-轴向应变关系曲线。其轴向压缩与轴向拉伸变形最大可接受限值采用 FEMA356 中建议的与防倒塌对应的斜撑弹塑性变形限值 (FEMA356 中表 5-7)。轴向压缩变形最大可接受限值为轴向屈曲变形值的 7 倍。轴向拉伸变形最大可接受限值为受拉屈服变形的 9 倍。因此，本节采用的斜撑最大可接受轴向压应变为 0.0094，最大可接受拉应变值为 0.013。

11.3.4 质量

结构的总质量及其分布与结构的重力荷载代表值对应。结构非线性分析模型的总质量是 387,000 吨。

11.3.5 阻尼

根据 GB50011-2001 的建议，在罕遇地震作用下，高层钢结构的阻尼比取 0.05。该组尼比在频域范围 0.2 Hz 至 10 Hz 内不变。

11.3.6 非线性分析软件与 P-Δ 效应

结构的非线性地震反应分析采用大型通用非线性动力有限元分析软件 LS-DYNA 进行计算 (Hallquist, Johnn O., LS-DYNA 理论手册，Livermore Software Technology Corporation, 1998 年 5 月． LS-DYNA 用户手册, 1999 年 5 月)。该非线性有限元软件采用大位移有限元格式，同时考虑几何非线性与材料非线性。结构的动力平衡方程建立在结构变形后的几何状态 (结点坐标) 上。因此，P-Δ 效应被自动考虑。

As Figure 11.6 suggests, a bi-linear elastic-perfectly-plastic relationship has been employed to describe the post-tension yielding and the post-buckling compression axial force - axial deformation relationship of braces in the CCTV building. The acceptable limits on axial shortening and axial elongation for the collapse prevention performance objective are adopted from FEMA 356 (Table 5-7). The axial shortening limit is 7 times the axial deformation at the buckling axial force and the axial elongation limit is 9 times the axial deformation at the tensile yielding axial force, resulting in an acceptable axial compression strain value of 0.0094 and an acceptable axial tensile strain of 0.013.

11.3.4 Mass

The total mass of the structure and its distribution correspond to the representative value of the structure's gravity load. The total mass of the non-linear structural model is approximately 400,000 tonnes.

11.3.5 Damping

Under the rare earthquake, damping in the structure is assumed to be 5% of critical. This constant damping ratio is applied to the structure within the frequency range from 0.2 Hz to 10 Hz.

11.3.6 Non-linear Analysis software and consideration of P-Δ effect

The non-linear seismic analysis of the CCTV building structure under the rare design earthquake is carried out employing a general purpose non-linear dynamic finite element software LS-DYNA (Hallquist, John O., LS-DYNA Theoretical Manual, Livermore Software Technology Corporation, May 1998. Livermore Software Technology Corporation, LS-DYNA Keyword User's Manual, Version 950, May 1999). This non-linear finite element analysis software adopts a large displacement finite element formulation, considering both geometric and material non-linear behaviour of structures. The dynamic equilibrium of the structure under the gravity and seismic actions is established based on the deformed geometry (configuration) of the structure. Hence, P-Δ effect is considered automatically.

11.4 地震输入

在本报告完成时，工程地震危险性分析，工程场地地震反应分析及地震波时程曲线选择报告尚未正式送交给奥雅纳工程顾问公司。与设计罕遇地震 (50 年超越概率 2%) 相对应的地震影响系数曲线取为 GB50011-2001 规定的北京地区 (8 度区)，第一组，III 类场地的地震影响系数曲线。该曲线作为本节罕遇地震反应谱曲线。地震影响系数曲线最大值 α_{max} 等于 0.9。场地特征周期 T_g 的取值增加 0.05 秒，取为 0.5 秒。该反应谱示于图 11.7 中的绿色曲线，代表罕遇地震时地面运动输入。在罕遇地震作用下，结构的屋顶相对位移及抗震性能点的确定都基于该反应谱曲线。

在非线性地震时程反应分析中，本节假设地震波输入为单向。所采用的地震波与罕遇地震反应谱相符。该地震波由中国建筑科学研究院抗震所提供。图 11.7 示出目标谱 (绿线) 与该地震时程曲线的反应谱 (红线) 的比较。图 11.8 绘出该地震波的时程曲线。从图 11.7 可看到，在长周期区段，反应谱曲线低于目标谱曲线，因而偏于不安全。在以后的工作中，所采用的地震时程曲线应与目标谱相符得更好。这可以通过修正地震波的频率成份得以实现。另外，与由场地土地震反应分析确定的场地地表反应谱相符的地震时程曲线也将在将来的工作中用作地震输入。

11.4 Earthquake Input

At the time of writing this report, the report on the probabilistic site-specific seismic hazard assessment, site soil response analysis and the selection of earthquake acceleration time histories has not yet been issued to Arup. For the design rare earthquake having a 2% probability of exceedance in 50 years, the seismic influence coefficient curve specified in GB 50011 – 2001 for Beijing (Zone 8), group 1 design earthquake and site soil type III has been adopted as the design response spectrum. The maximum value of the seismic influence coefficient curve α_{max} is 0.9. The ground characteristic period, T_g, has been increased by 0.05 seconds to 0.50 seconds. This code design spectrum representing the design rare earthquake ground motion input is shown as the green curve in Figure 11.7 and is used to calculate the roof target displacement and hence to determine the performance point in the non-linear static pushover analysis.

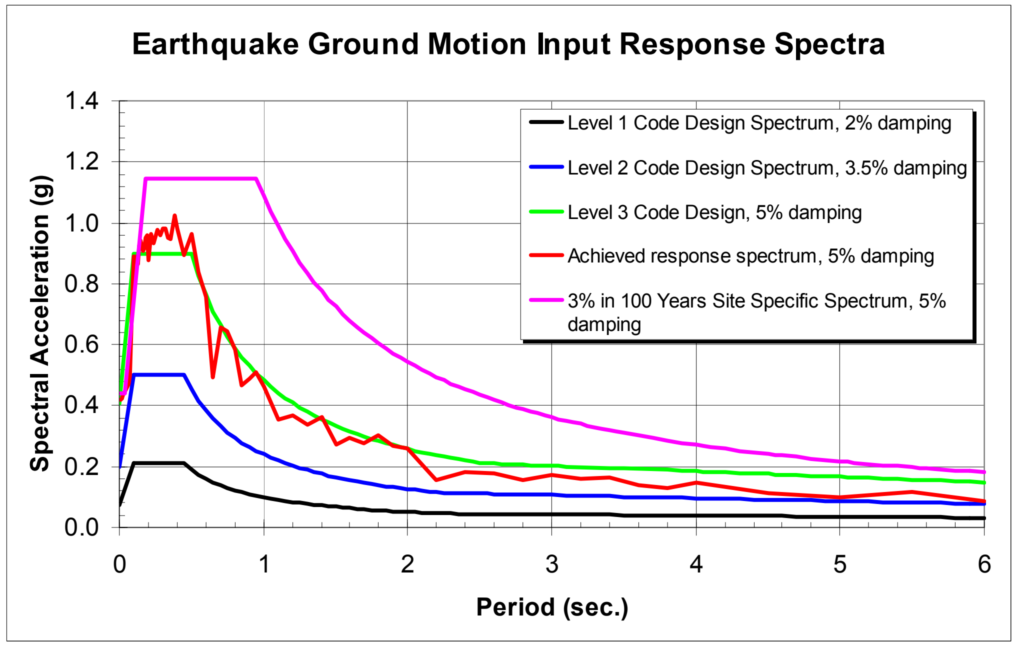


Figure 11.7 Earthquake Ground Motion Input Response Spectral Curves
图 11.7 地震作用反应谱

In the non-linear seismic time history analysis, a uni-axial artificial earthquake acceleration time history compatible with the above described response spectrum is used as earthquake input. This time history was provided by the Institute of Earthquake Engineering of China Academy of Building Research. Figure 11.7 shows a comparison between the target code design spectrum (green curve) and the achieved response spectrum (read curve) of the artificial earthquake acceleration time history. Figure 11.8 shows the acceleration time history. It can be noticed that in the long period range, the achieved response spectrum curve falls below the target code spectrum and is therefore unconservative. In future work, a close match between the code target and the achieved response spectrum curves will be obtained by adjusting the frequency content of the artificial earthquake time history. Furthermore, earthquake time histories compatible with the site-specific ground surface response spectrum curve will also be used as input when they become available.

Also shown in Figure 11.7 are the response spectra corresponding to the design frequent earthquake (black curve, Level 1 design earthquake having a probability of exceedance of 63% in 50 years), the design intensity earthquake (blue curve, Level 2 earthquake having a probability of exceedance of 10% in 50 years), and finally the rare earthquake (pink curve) having a probability of exceedance of 3% in 100 years, respectively. The latter two curves have also been used to calculate their corresponding roof target displacements and hence to determine their corresponding performance points.

11.5 非线性静力分析

11.5.1 非线性静力分析的目的与局限性

非线性静力分析是一种简化的非线性地震反应分析方法。严格来讲，这种方法仅适用于侧向刚度较大的短周期结构。该类结构的地震反应主要由基本振型决定。对于较为柔软，周期较长的高层建筑结构，非线性静力分析方法并非很适用。后一类结构的地震反应受高振型的影响很显着。非线性静力分析方法中所假设的水平作用力的竖向分布很难准确反应结构非线性地震反应过程中高振型的影响。

因此，中央电视台新主楼结构的抗震性能评价主要以非线性地震时程分析的结果为依据。非线性静力分析的结果只作补充和参考。

11.5 Non-linear Static Pushover Analysis

11.5.1 Objective and limitation of pushover analysis

Pushover analysis is a simplified non-linear seismic analysis procedure. Strictly speaking, pushover analysis is only suitable for relatively stiff structures in the short to intermediate period range in which the fundamental mode dominates the structure’s dynamic seismic response. For flexible, long period structures such as the CCTV building, limitations exist with respect to the ability of this simplified non-linear analysis procedure to account for the effect of higher modes. Difficulties arise in assuming a pattern or a few possible patterns for the vertical distribution of the applied horizontal force that can accurately account for the effect of higher modes in flexible structures.

As a result, the seismic performance evaluation of the CCTV structure under the design rare earthquake will be primarily based on results obtained from the non-linear seismic time history analyses with those obtained from the pushover analysis serving as supplementary results.

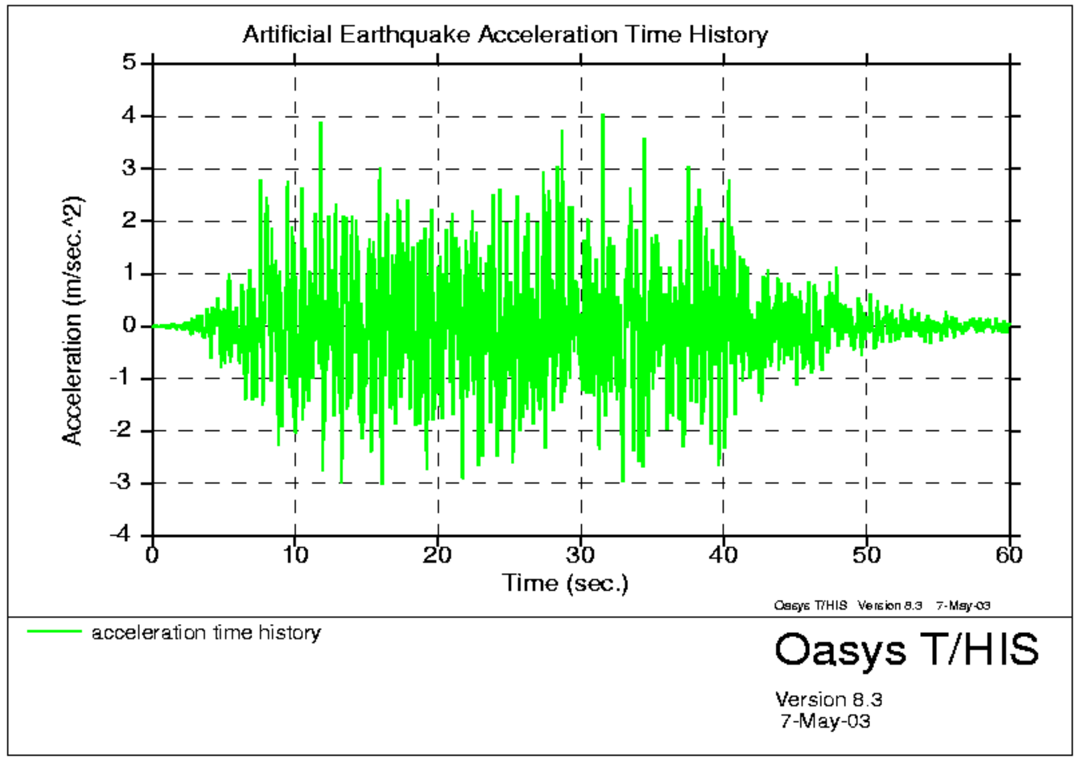


Figure 11.8 Artificial Earthquake Acceleration Time History Compatible with the Code Target Spectrum for the Design Rare Earthquake
图 11.8 与规范要求的设计大震对应的人工时程波

11.5.2 加载顺序与水平作用力的竖向分布

在非线性静力分析中，所有由桩承台支持的结点都假设为固定端，所有 6 个自由度的位移为零。作用在结构上的竖向力与重力荷载代表值对应。由内筒所承受的那部份重力荷载代表值在本节中略去。本节作用在外筒结构的重力荷载与外筒所承受的那部份重力荷载代表值相对应。作用在结构上的水平荷载是一组作用在楼板高度处的水平力。竖向与水平荷载分三步施加于结构上，如下所述：

第一步： 施加第一施工阶段的重力荷载。该项重力荷载与结构第一施工阶段分析模型，图 11.2a，所对应，取不变荷载值的 75%。

第二步： 维持第一步所施加的重力荷载不变。施加第二施工阶段的重力荷载。后者包括与第一施工阶段分析模型 (图 11.2a) 所对应的余下的 25% 的不变荷载与可变荷载总值的 50%，加上与第二施工阶段建造的悬挑部份结构对应的全部不变荷载值与可变荷载总值的 50%。第二施工阶段所建造的结构构件与相应的重力荷载由第一施工阶段完成后的变形后的结构所支持。由外筒所承受的总重力荷载值在第二施工阶段完成后为 1,868MN。

第三步： 维持第一步与第二步所施加的重力荷载不变。水平作用力值从零开始逐步增加，每次增加一个小的增量。随着非线性静力分析的进行，监视屋顶在水平作用力方向的水平位移。当屋顶水平位移超过预见的水平水移值时，非线性静力分析在人工干预下结束。

本节进行了两个非线性静力分析。在第一个非线性静力分析中，水平作用力沿结构第一型的方向 (结构整体坐标 X 方向) 作用。在第二个分析中，水平力沿第二振型的方向 (结构整体坐标 Y 方向) 作用。

在本节，水平作用力的竖向分布假设为均均分布。这是 FEMA356 所建议的几种分布中的一种. 这种分布模式相当于对结构施加一个水平方向的重力场，其水平重力加速度从零开始逐步增加一个小量。这是 LS-DYNA 非线性有限元分析软件所采用的水平力加载方法。

11.5.3 非线性静力分析结果小结与结构抗震性能评价

11.5.3.1 结构的能力曲线与性能点

图 11.9 绘出屋顶水平位移-结构基底剪力关系曲线。两条曲线分别对应于在前两个振型方向分别进行的非线性静力分析。这类关系曲线通常被称为结构的能力曲线。

在罕遇地震作用下屋顶的水平弹塑性相对位移，通常称为屋顶目标位移，由 FEMA356 所建议的位移系数发计算决定。该方法首先计算屋顶的弹性水平位移，以结构的自振周期，振型以及设计反应谱作为计算基本参数。屋顶水平弹塑性位移 (目标位移) 值然后通过三个修正系数 C_1、C_2和 C_3计算得到。这三个参数分别考虑非线性反应，不饱满的 (收缩型) 构件弹塑性恢复力滞回曲线及动力 P-Δ 效应对弹塑性位移的影响。根据位移系数法计算出沿第一振型进行非线性静力推覆分析时的屋顶弹塑性水平相对位移为 1.2 米，与第二振型对应的屋顶水平位移值则为 0.87 米。这两个位移值就是结构在罕遇地震 (50 年超越概率 2%) 作用下的屋顶目标位移值。

11.5.2 Loading sequence and the vertical distribution of applied horizontal force

In pushover analysis, all nodes supported on the pile cap are assumed to be fully restrained in all six degrees-of-freedom. The applied vertical load corresponds to the part of the gravity load representative value that is supported by the perimeter structure. The applied lateral load is a set of lateral forces at the levels of floor diaphragms. The loads are applied to the structure is three load steps:

Step 1: Apply construction stage 1 gravity load. This consists 75% of the dead load associated with the construction stage 1 structural model as shown in Figure 11.2a.

Step 2: Maintain the load applied in Step 1 constant and apply the construction stage 2 gravity load. The latter consists of the remaining 25% of the dead load and 50% of the full live load applied to the stage 1 structural model plus the full dead load and 50% of the full live load applied to the overhang building which is constructed in stage 2. The structural components constructed in stage 2 and the stage 2 gravity load are added to the deformed geometry (configuration) of the structure at the end of Step 1. The total gravity load supported by the exterior structure at the end of Stage 2 is 1,868 MN.

Step 3: Maintain the gravity loads applied in Steps 1 and 2 constant. Apply the lateral load monotonically in many small load steps. Monitor the roof horizontal displacement along the direction of the applied lateral load. Pushover analysis terminates when the roof horizontal displacement exceeds the expected target roof displacement value.

Two independent pushover analyses are carried out. In the first, the lateral load is applied along the direction of the structure's first free vibration mode (global X direction). In the second, the lateral forces are applied along the direction of the second mode (global Y direction).

In this study, the pattern of the vertical distribution of the applied lateral forces is assumed to be uniform, being one of a few recommended patterns in FEMA 356. Therefore, the application of the lateral forces in the pushover analysis is equivalent to applying a horizontal gravitational field to the structure with a slowly and monotonically increasing horizontal gravitational acceleration.

11.5.3 Summary of results and performance evaluation

11.5.3.1 Capacity curve and the performance point

Figure 11.9 presents the roof displacement – base shear force curves for pushover analyses in the first and the second mode, respectively. These curves are commonly referred to as the structure's capacity curve.

The inelastic roof displacement under the design rare earthquake, commonly referred to as the target roof displacement, is calculated by employing the displacement coefficient method suggested in FEMA 356. The elastic roof displacement is calculated based on the period, the mode shape of the structure in the direction of the pushover and the design spectrum curve shown in Figure 11.7. The inelastic roof displacement (target displacement) values are then calculated using three modification coefficients, C_1, C_2 and C_3, to consider the effects of non-linear response, pinched hysteretic shape and dynamic P-Δ. In accordance with the displacement coefficient method, the roof target displacements are calculated as 1.2 m when pushing along the first mode and 0.87 m when pushing along the second mode under the design rare earthquake (2% probability in 50 years).

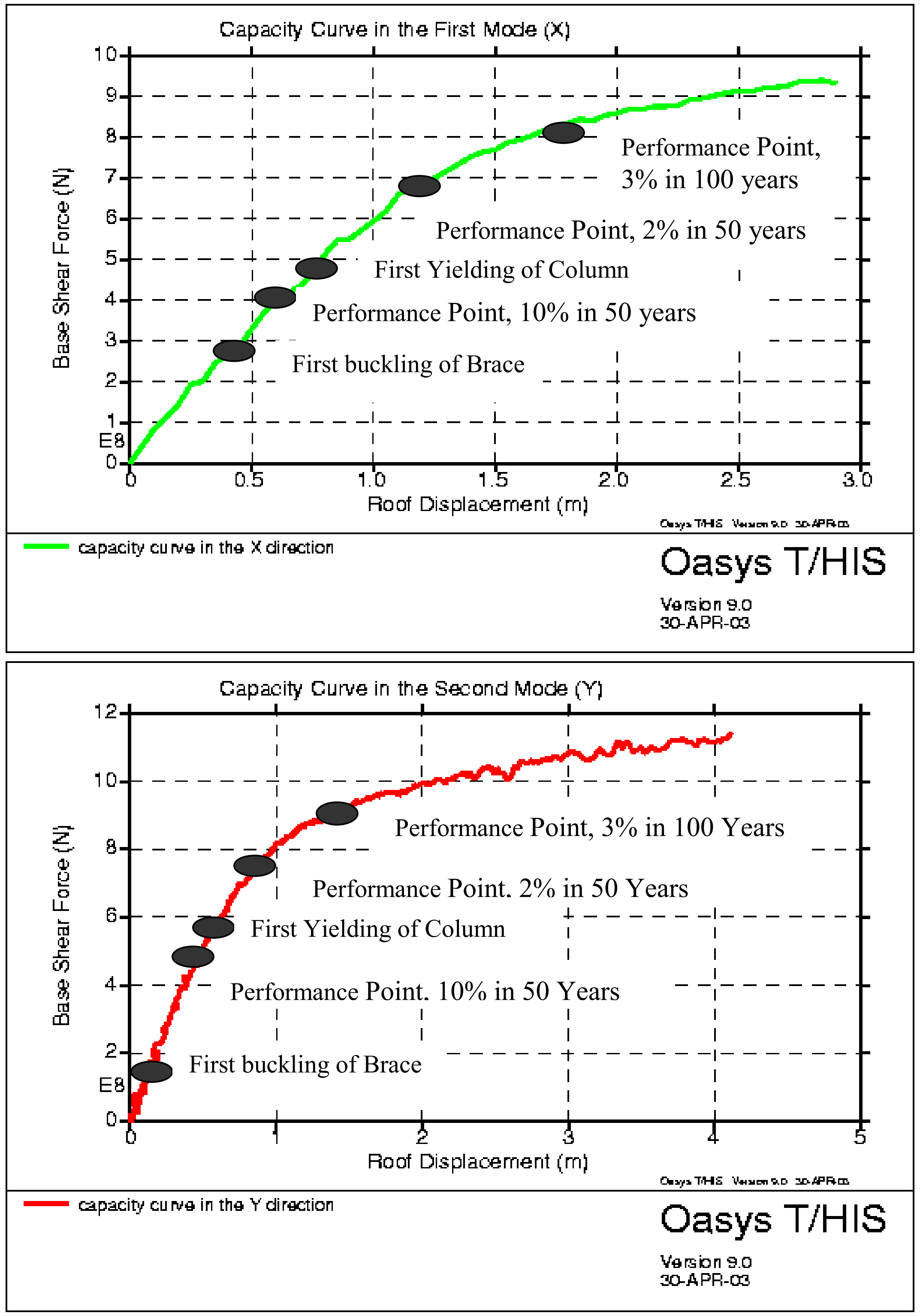


Figure 11.9 Capacity Curves of the Structure along the two Principal Axes of the Structure
图 11.9 结构在两个主轴方向的能力曲线

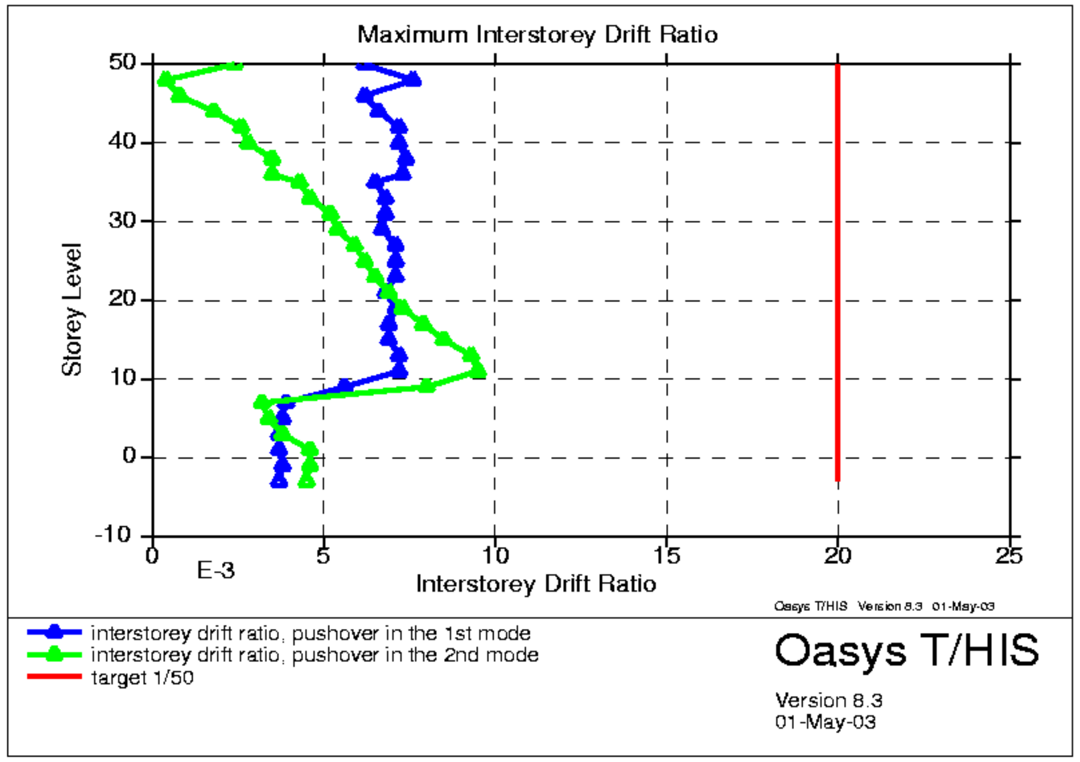


Figure 11.10 Maximum Interstorey Drift Ratio at the Performance Points, Non-linear Pushover Analysis
图 11.10 非线性推覆分析下结构位于能力点时的最大层间位移角

屋顶目标位移值确定抗震性能点在结构能力曲线上的位置，如图 11.9 所示，与罕遇地震 (50 年超越概率 2%) 抗震性能点对应的基底剪力值分别是 690MN (结构重力荷载代表值的 18%，X 方向) 和 770MN (Y 方向，结构重力荷载代表值的 20%)。

与第二水准烈度 (基本烈度，50 年超越概率 10%，图 11.7 中的蓝线) 对应的两个抗震性能点，以及与 100 年超越概率为 3%的罕遇地震相对应的两个抗震性能点也用相同方法求得并示于图 11.9 中。在第二水准烈度地震作用下，屋顶在两个方向的弹塑性水平位移值分别是 0.63 米和 0.47 米。相应的基底剪力值分别是 394MN 和 465MN。在 100 年超越概率为 3%的罕遇地震作用下，屋顶水平弹塑性位移值分别是 1.76 米和 1.46 米，相应的基底剪力值分别是 820MN 和 920MN。

在结构能力曲线上与斜撑开始屈曲以及与柱开始受压屈服相对应的点也被找到并示于图 11.9 中。当非线性静力推覆分析沿 Y 向进行时，与斜撑开始屈曲对应的基底剪力值是 170MN。该基底剪力值高于常遇地震作用下的弹性基底剪力值 150MN。这个结果显示目前结构设计方案可以达到 “小震不坏” 的抗震设计目标。

图 11.9 也表明与基本烈度对应的两个抗震性能点都低于与柱开始受压屈服对应的两个点。因此，在基本烈度地震作用下，柱尚未受压屈。这个结果表明，在中震作用下，虽然有斜撑发生屈曲和结构性破坏，柱子尚未出现结构性破坏。可认为结构可以达到 “中震可修” 的抗震设计目标。

图 11.10 表示与两个抗震性能点 (50 年超越概率 2% 的罕遇地震) 对应的层间最大位移角沿结构高度的变化曲线。在结构的所有各层最大层间位移角中，最大值稍低于 1%。该值是 GB50011-2001 规定的可接受最大层间位移角 1/50 的一半。

从图 11.10 可以观察到，最大层间位移角在平台结构到两个塔形结构的转换部位变化较大。这是由于平台结构的层间侧向刚度明显大于两个塔形结构的层间侧向刚度。

图 11.9 和图 11.10 显示，在设计罕遇地震 (50 年超越概率 2%) 作用下，整体结构仍然具有明显的强度和变形能力储备。结构整体仍具有多余的强度和变形能力来抵抗烈度更高的地震输入最后达到临界倒塌的破坏极限状态。因此，可以初步判定结构整体可以达到 “大震不倒” 的抗震设计目标。不仅如此，在 100 年超越概率为 3%的罕遇地震作用下，结构的抗震性能仍可实现 “大震不倒” 的抗震设计目标，如图 11.9 中所对应的两个抗震性能点所示。

结构构件的破坏程度由最大轴向变形来定量评估，如图 11.11 至图 11.14 所示。这四张图绘出了斜撑和柱子与抗震性能点 (罕遇地震 50 年超越概率 2%) 对应的最大轴向应变云斑图。图中深蓝色代表构件受压屈曲或屈服，粉红色代表构件受拉屈服。从图 11.11 至图 11.14 可以观察到，斜撑和柱子的轴向最大压应变为 0.004 左右，最大拉应变为 0.005 左右。这些轴向变形最大值都远小于其对应的最大可接受限值，见 10.3.3.2 节至 10.3.3.4 节。因此结构构件的抗震性能也满足罕遇地震作用下不倒塌的抗震设计目标。

The values of the target roof displacements determine the locations of the performance points on the capacity curves, as shown in Figure 11.9. Corresponding to the performance points, the base shear force is 690 MN (18% of the building's gravity load representative value) and 770 MN (20%), respectively when pushing along the first two modes.

The performance points corresponding to the design intensity earthquake (Level 2 earthquake having a 10% probability of exceedance in 50 years) and the rare earthquake having a probability of exceedance of 3% in 100 years have also been determined, as shown in Figure 11.9, employing the same method. Under the Level 2 earthquake, the roof target displacements are 0.63 m and 0.47 m and the base shear force values are 394 MN and 465 MN, respectively when pushing in the first and the seconds mode. Under the rare earthquake having a 3% probability of exceedance in 100 years, the roof target displacements are 1.76 m and 1.46 m, and the base shear force values are 820 MN and 920 MN, respectively.

The points corresponding to the onset of brace buckling and column compression yielding are also identified on the capacity curves in Figure 11.9. Braces start to buckle at a total base shear force of 170 MN when pushing in the direction of the second mode (global Y direction). This value is higher than the elastic base shear force, 150 MN, induced by the design frequent earthquake having a probability of exceedance of 63% in 50 years. This result suggest that the present structural design is capable of achieving the no structural damage performance objective under the action of the design frequent earthquake.

It can also be noticed from Figure 11.9 that the performance points corresponding to the design intensity earthquake (Level 2 earthquake) are lower than the points corresponding to the first yielding of columns in compression. Hence, it may be concluded that under the design intensity earthquake, columns do not yield in compression. As a result, the structure may be considered reparable, although structural damage has occurred due to brace buckling.

Figure 11.10 presents the maximum interstorey drift ratios corresponding to the two performance points under the design rare earthquake. The highest value of all interstorey drift ratios is just under 1%, being approximately half of the acceptable limit on the interstorey drift ratio, 1/50.

It can be noticed from Figure 11.10 that at the transition level from the Podium to the towers, there is a large difference in the maximum interstorey drift ratios. This is due to the large difference in interstorey lateral stiffness in the Podium and in the tower.

Figures 11.9 and 11.10 indicate that under the design rare earthquake, the global structure still possesses significant additional strength and deformation capacities before it is pushed to a limit state at which the structure is on the verge of collapsing. As a result, it may be concluded that the global structure is capable of achieving the collapse prevention seismic performance objective under the design rare earthquake. Furthermore, the above conclusions are still valid even when the structure is subjected to the rare earthquake having a probability of exceedance of 3% in 100 years as demonstrated by the respective two performance points in Figure 11.9.

At the local structural component level, Figures 11.11 to 11.14 illustrate the total axial deformation in the format of total axial strain contour plots of braces and columns. The total axial strain of braces and columns range from approximately 0.004 in compression to approximately 0.005 in tension. These values are well within the acceptable limits established in Sections 10.3.3.2 to 10.3.3.4. As a result, the performance of the structural components satisfies requirements for the collapse prevention performance objective under the design rare earthquake.

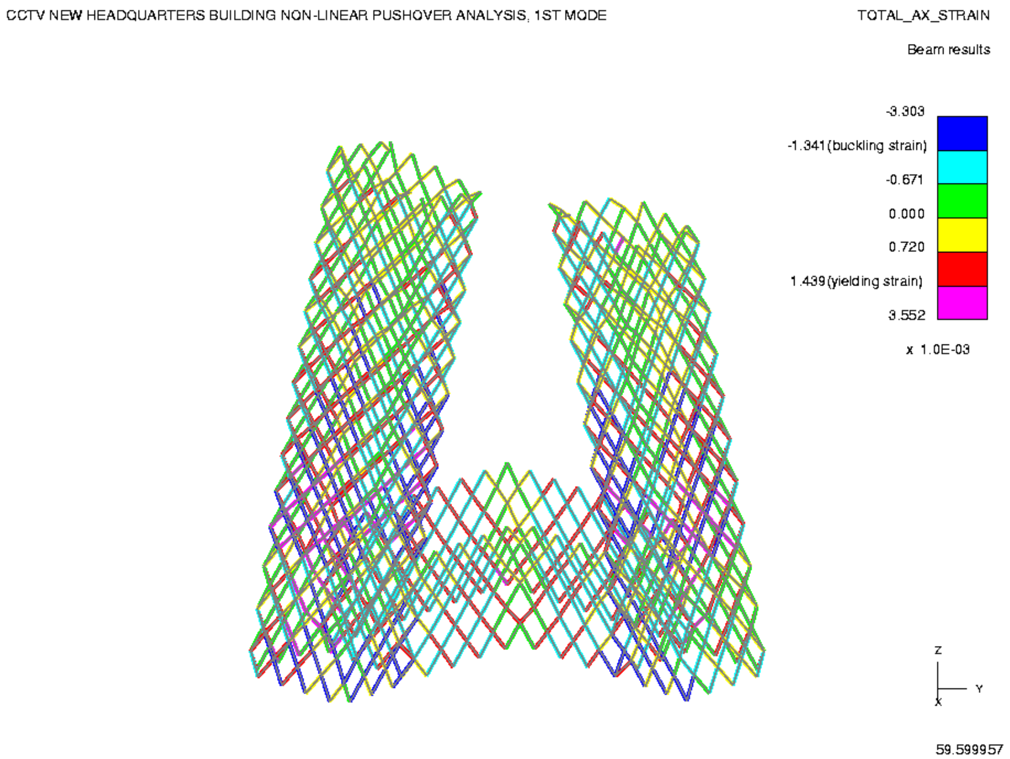


Figure 11.11 Total Axial Strain of Braces, Pushover in the First Mode (X)
图 11.11 以第一周期作静力推覆下支撑的轴向总应变 (X)

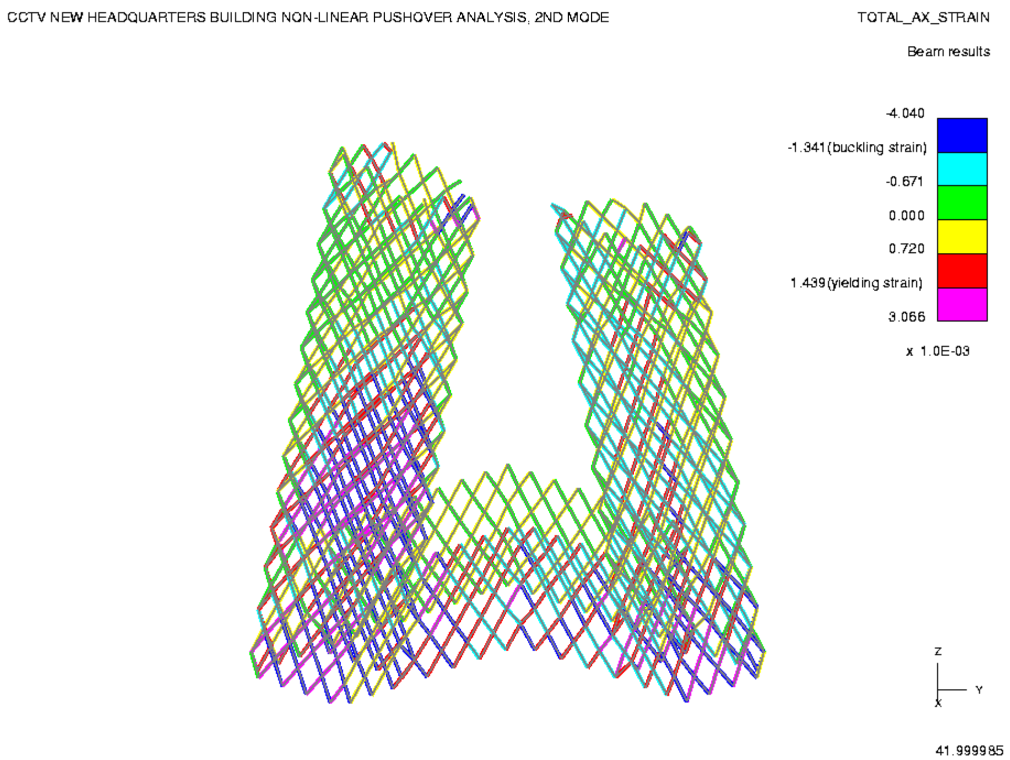


Figure 11.12 Total Axial Strain of Braces, Pushover in the Second Mode (Y)
图 11.12 以第二周期作静力推覆下支撑的轴向总应变 (Y)

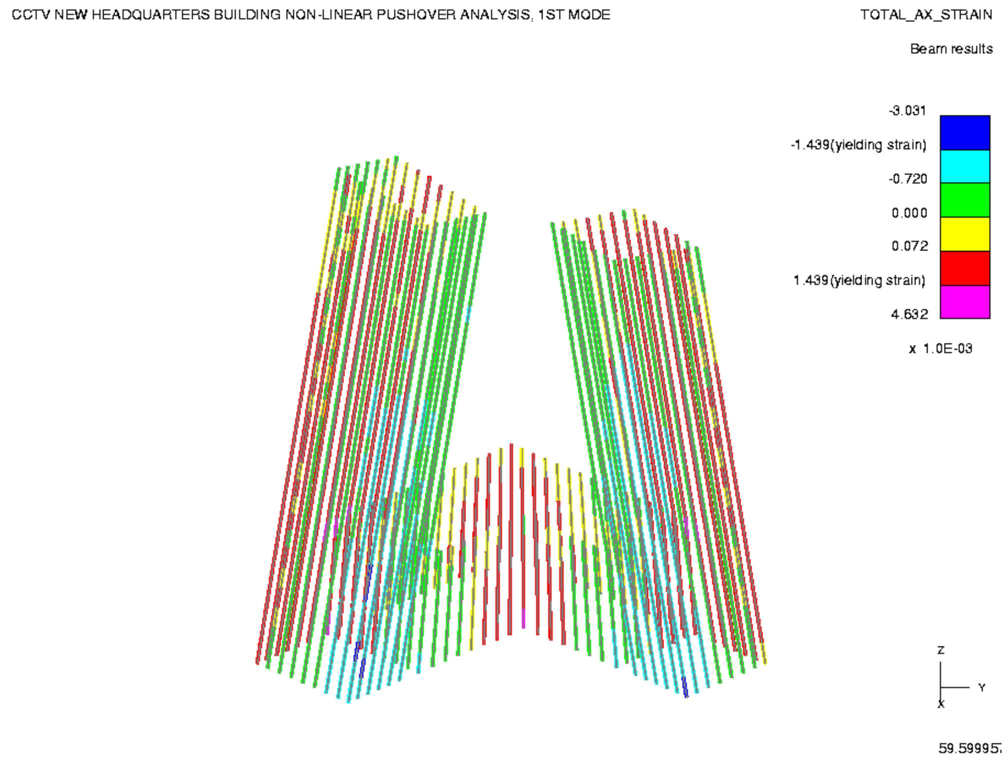


Figure 11.13 Total Axial Strain of Columns, Pushover in the First Mode (X)
图 11.13 以第一周期作静力推覆下柱的轴向总应变 (X)

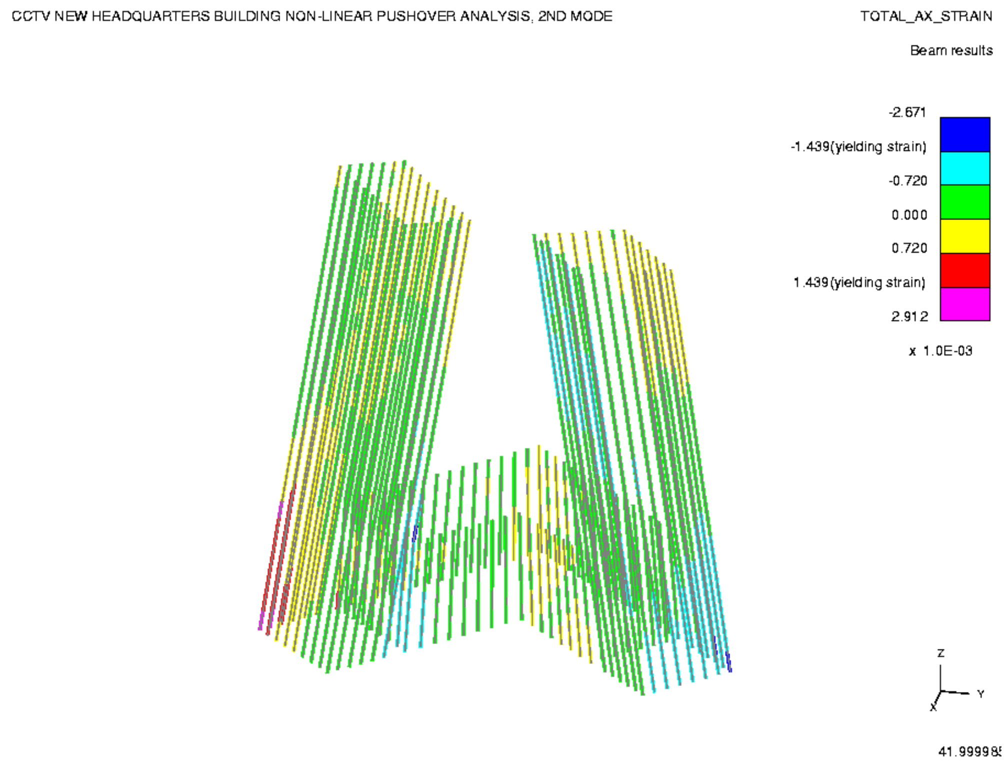


Figure 11.14 Total Axial Strain of Columns, Pushover in the 2nd Mode (Y)
图 11.14 以第二周期作静力推覆下柱的轴向总应变 (Y)

11.6 非线性动力时程分析

非线性动力时程分析是进行结构非线性地震反应分析最完善的方法。这种动力分析方法去除了非线性静力分析方法的局限性，可以准确地体现高振型的影响。在中央电视台新主楼的结构设计过程中，抗震性能评价主要以非线性动力时程分析所得结果为依据。

11.6.1 加载顺序与地震波的输入

GB50011-2001 要求使用不少于 3 组地震波输入，其中要有两组真实地震记录和一组人工地震记录。在完成本报告时，所需要的地震纪录尚未送交给奥雅纳。因此在本节，图 10.8 所示的单一地震记录被用来作为两个非线性动力时程分析的地震波输入。在第一个时程分析中，该地震波沿结构第一振型的方向 (结构整体坐标 X 向) 作用。在第二个时程分析中，该地震波沿结构第二振型方向 (Y 向) 作用。

同样，作用在结构上的重力荷载代表值与地震波分三步施加。前两步与 10.5.2 节所述相同。在第三步，地震沿 X 向或 Y 向作用在桩基承台所支持的结点上。

11.6 Non-linear Dynamic Time History Analysis

Non-linear dynamic time history analysis is the most adequate and comprehensive analysis procedure to evaluate the non-linear seismic response of structures. It removes the limitation inherent in the static pushover analysis as discussed previously. Therefore, in the design of the CCTV building, seismic performance evaluation under the design rare earthquake will be primarily based on results obtained from the non-linear dynamic time history analysis.

11.6.1 Loading sequence and the application of the ground motion time history

The Chinese seismic building code GB 50011 – 2001 requires that no less than three sets of earthquake records be used when carrying out dynamic time history analysis, with no less than two sets of real earthquake records and at least one set of spectrum compatible artificial records. However, at the time of writing this report, these records have not yet been provided to Arup. Therefore in this report, a single artificial earthquake acceleration time history described in Section 10.4 is used in two independent analyses. In the first, the time history is applied along the direction of the first mode (global X). In the second, it is applied along the direction of the second mode (global Y).

Again, a three-step loading sequence is followed. The first two steps are the same as those described in Section 10.5.2 in the non-linear pushover analysis. In the third step, the earthquake acceleration time history plotted in Figure 11.8 is applied to the nodes supported on the pile cap, along the global X or Y directions respectively in the two independent analyses.

11.6.2 非线性时程分析结果小结与结构抗震性能评价

图 11.15 至图 11.18 绘出两个非线性动力时程分析所得屋顶水平相对位移与基底剪力时程曲线。当地震波沿第一振型方向 (X 向) 作用时，屋顶最大水平位移值是 1.24 米，最大基底剪力值是 650MN。这两个结果与非线性静力推覆分析结果 1.2 米和 690MN 很接近。当地震波作用于第二振型方向 (Y 向) 时，最大屋顶水平相对位移值是 0.96 米，最大基底剪力值是 745MN。同时，这两个结果与非线性静力推覆分析结果 0.87 米和 770MN 也很接近。

图 11.19 和图 11.20 绘出动力时程分析全过程中各层层间相对位移角的最大值，并与其可接受限值 1/50 作比较。从这两个图中可看出，所有各层的最大层间位移角都小于 1/100，显示与临界倒塌破坏极限状态相比较，结构整体仍具备较大的变形能力储备。因此，在整体结构水平，结构的抗震性能似乎要优于“大震不倒”的抗震设计目标。

在结构构件水平，每一个斜撑和每一个柱子的变形时程曲线都由有限元分析后处理软件得到。每一个构件的最大轴向变形值都从其变形时程曲线上找出。对每一类构件，其最大变形值小结于下表中。该表同时也绘出了 11.3.3 节所建立的与防倒塌对应的构件变形最大可接受限值，以便比较并进行构件抗震性能评价。可以从该表明显观察到，所有构件的最大变形都远小于其相对应的最大可接受限值。因此，所有结构构件的破坏程度都在允许范围内，并具有较大安全储备。所以，结构构件在罕遇地震作用下的表现也表明结构的抗震性能要优于“大震不倒”的抗震设计目标的要求。

11.6.2 Summary of results and performance evaluation

Figures 11.15 to 11.18 present the time histories of roof displacement and base shear force. In the analysis case when the earthquake ground motion time history is applied along the direction of the first mode (global X), the maximum roof displacement is 1.24 m and the maximum base shear force is 650 MN. These results are close to 1.2 m and 690 MN obtained from the non-linear pushover analysis. In the case when the earthquake time history is applied along the direction of the second mode (Y), the maximum roof displacement is 0.96 m and the maximum base shear force is 745 MN, again close to 0.87 m and 770 MN obtained from the non-linear pushover analysis.

Figures 11.19 and 11.20 present the maximum interstorey drift ratios compared to the acceptance criteria of 1/50. The maximum interstorey drift ratios are all less than 1%, indicating that the structure still has a large reserve deformation capacity before it is shaken to the limit state being on the verge of collapse. As a result, at the global structural level, the structure seems to perform better than the collapse prevention objective under the design rare earthquake.

At the component level, the deformation time histories of every brace and every column have been obtained by a finite element analysis post-processor. The maximum deformation values of each type of component are then identified from the time histories and are summarised in the table below. Also listed in this table are the deformation acceptance limitations for each type of component corresponding to the collapse prevention performance objective. It can clearly be seen that the maximum component deformations are much smaller than their respective acceptable limits. Therefore, the degree of damage to the components also seems to suggest that the structure performs better than satisfying the requirements to achieve the collapse prevention performance objective under the action of the design rare earthquake.

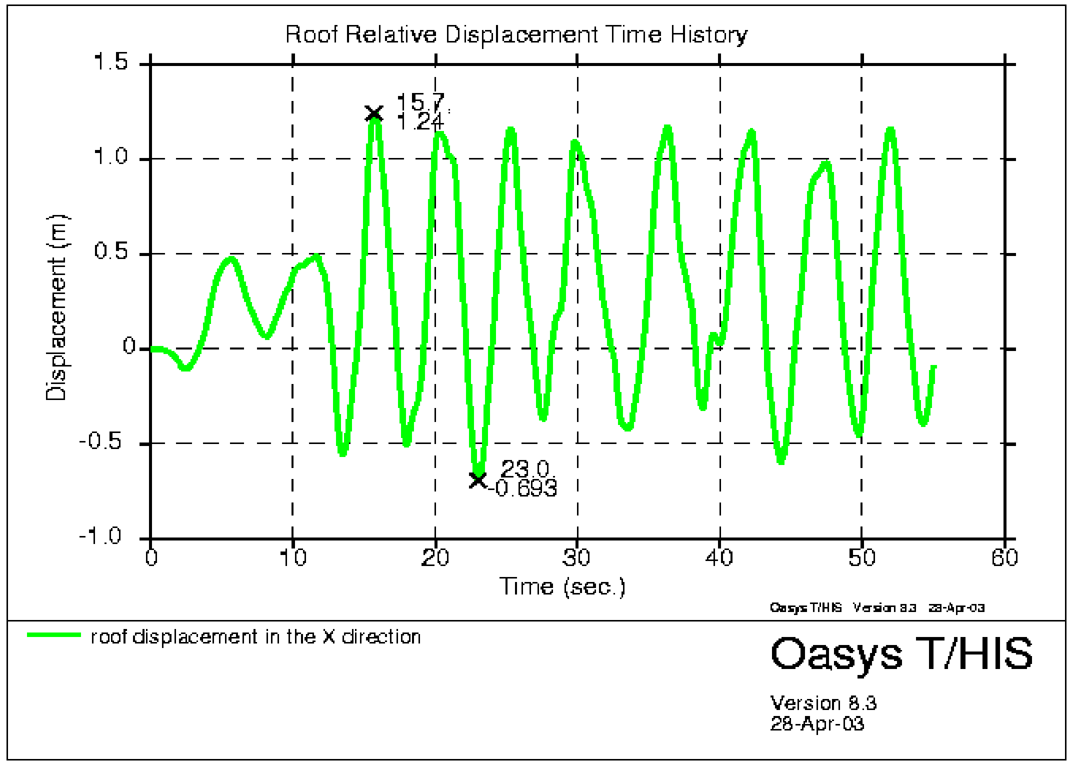


Figure 11.15 Roof Relative Displacement Time History along the X Direction
图 11.15 X 向屋顶相对位移时程曲线

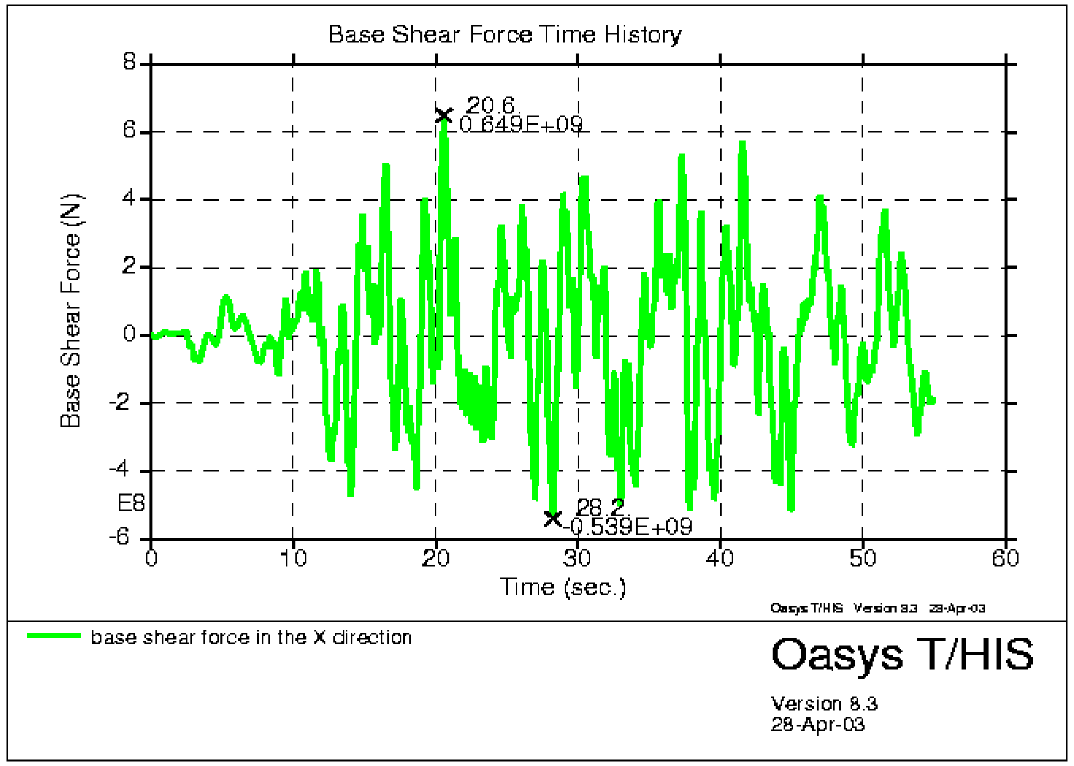


Figure 11.16 Base Shear Force Time History along the X Direction
图 11.16 X 向基底剪力时程曲线

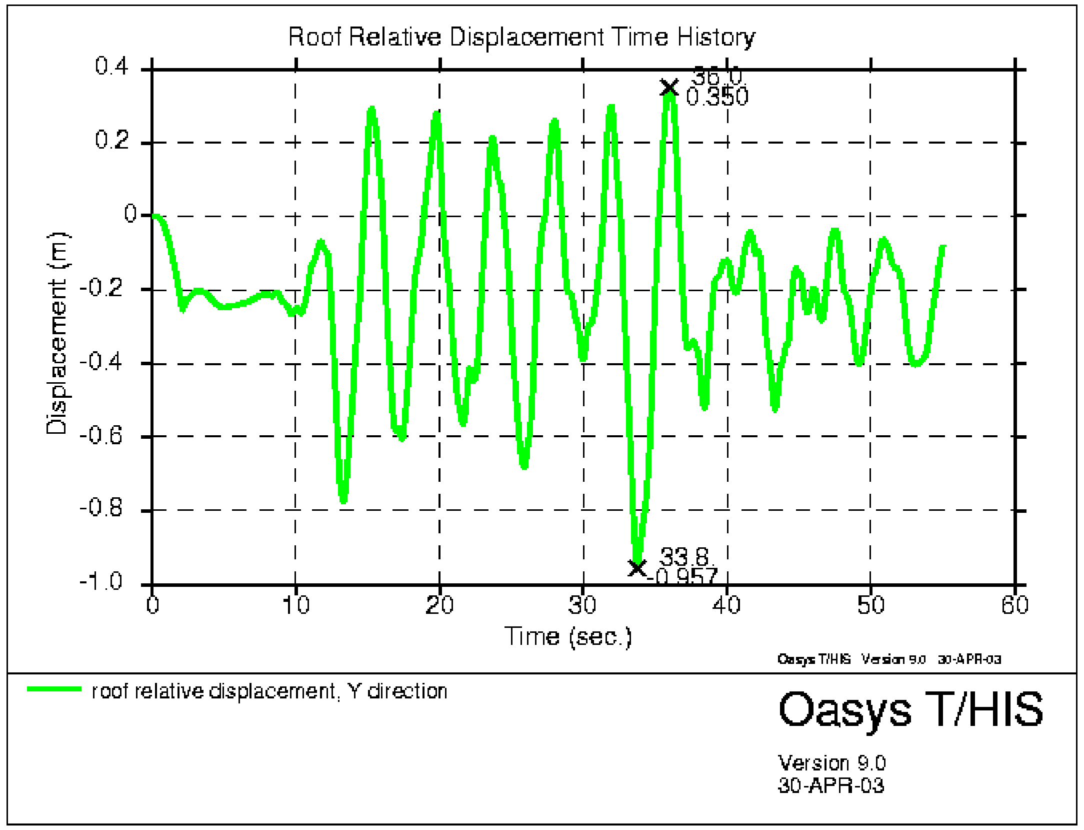


Figure 11.17 Roof Relative Displacement Time History in the Y Direction
图 11.17 Y 向屋顶相对位移时程曲线

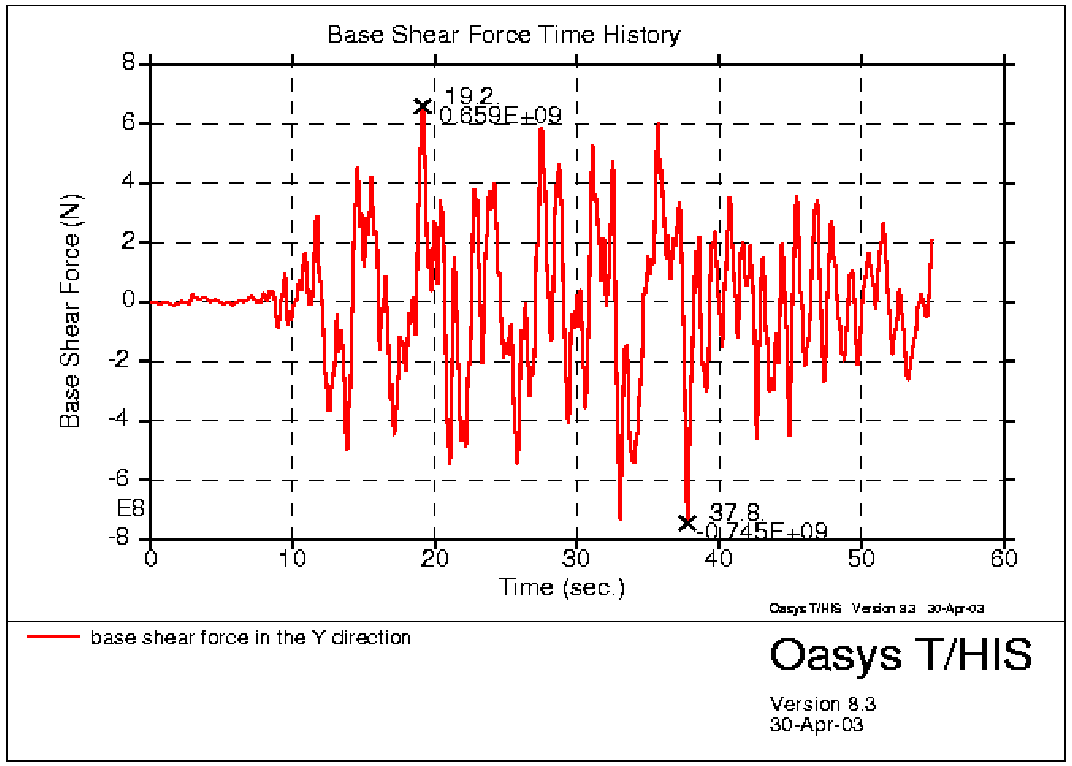


Figure 11.18 Base Shear Force Time History along the Y Direction
图 11.18 Y 向基底剪力时程曲线

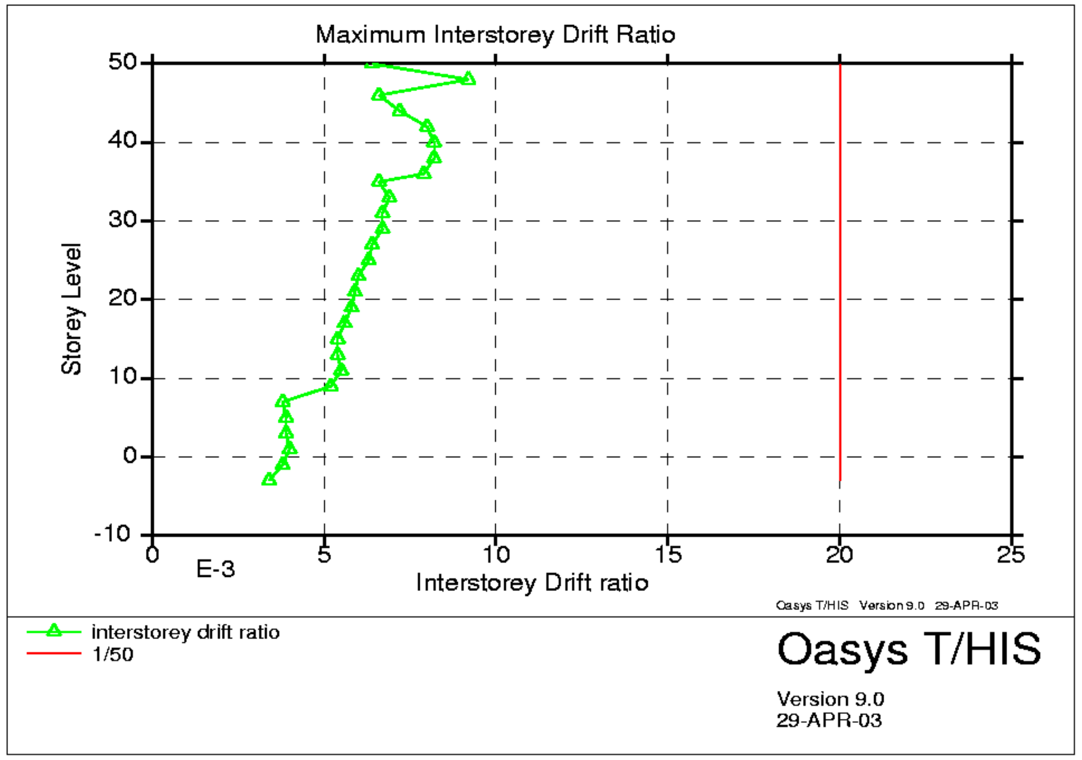


Figure 11.19 Maximum Interstorey Drift Ratio along the X Direction
图 11.19 X 向最大层间位移角

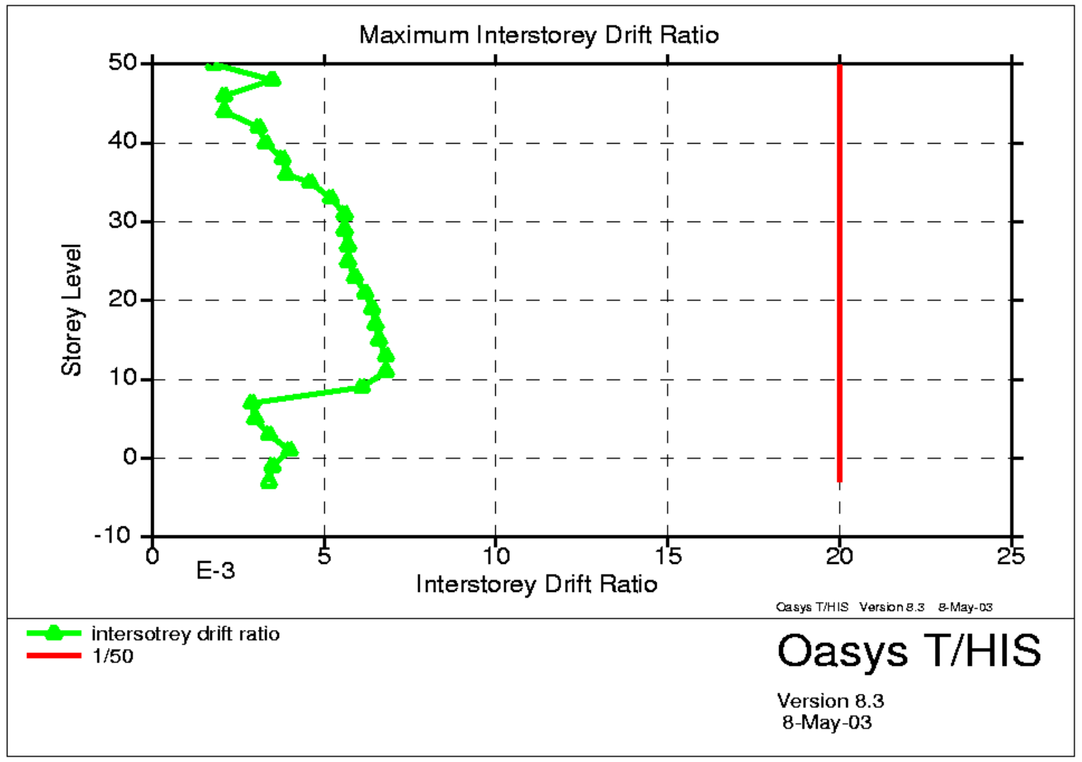


Figure 11.20 Maximum Interstorey Drift Ratio along the Y Direction
图 11.20 Y 向最大层间位移角

斜撑和柱最大变形小结

构件	变形参数	地震波沿 X 向	地震波沿 Y 向	防倒塌可接受大变形限值
斜撑	最大轴向压缩应变	0.003	0.0033	受压屈曲轴向变形的 7 倍，轴向压应变 0.0094
斜撑	最大轴向拉伸应变	0.003	0.0024	受拉屈服应变的 9 倍，轴向拉应变 0.013
钢-钢筋混凝土组合柱	最大轴向压缩应变	0.0026	0.0024	约束混凝土最大可用压应变 0.015
钢-钢筋混凝土组合柱	最大轴向拉伸应变	0.0057	0.002	受拉轴向屈服应变的 9 倍，轴向拉应变 0.013
钢柱	最大轴向压缩应变	0.0013	0.0013	受压屈曲轴向应变的 7 倍，轴向压应变 0.01
钢柱	最大轴向拉伸应变	0.0012	0.001	受拉屈服变形的 9 倍，轴向拉应变 0.013

Summary of Brace and Column Deformation

Component	Deformation Parameter	Earthquake Time History Applied along X	Earthquake Time History Applied along Y	Acceptable Limit for Collapse Prevention Performance Objective
Brace	Maximum Axial Shortening (Compression) Strain	0.003	0.0033	7 times the buckling axial deformation, axial compression strain 0.0094
Brace	Maximum Axial Elongation (Tension) Strain	0.003	0.0024	9 times the tensile yielding deformation, axial tensile strain 0.013
Steel SRC Column	Maximum Axial Compression Strain	0.0026	0.0024	Maximum usable compression Strain of confined concrete = 0.015
Steel SRC Column	Maximum Axial Tension Strain	0.0057	0.002	9 times the tensile yielding deformation, axial tensile strain 0.013
Steel Column	Maximum Axial Compression Strain	0.0013	0.0013	7 times the yielding axial compression Strain 0.01
Steel Column	Maximum Axial Tension Strain	0.0012	0.001	9 times the tensile yielding deformation, axial tensile strain 0.013

11.7 初步结论

根据本节非线性地震反应分析所得结果以及结构的抗震性能评价，可得出以下初步结论：

1. 目前采用的结构体系和方案可满足常遇地震作件下不出现结构性破坏的要求。非线性静力分析的结果显示，与斜撑开始受压屈曲对应的基底剪力值大于常遇地震下的弹性基底剪力值。这表明结构在常遇地震作用下的抗震性能稍优于规范 GB50011-2001 规定的最低最求。
2. 在中震作用下 (50 年超越概率 10%)，结构的柱子尚未屈服，有部份斜撑已因受压屈曲。因此，出现结构性破坏但结构的破坏程度得到有效控制，可认为达到了“中震可修”的抗震设计目标。
3. 在罕遇地震作用下 (50 年超越概率 2%)，结构的抗震性能满足防倒塌的抗震设计目标。结构整体和结构各个构件的最大弹塑性变形都远小于相应的可接受最大弹塑性变形限值。非线性静力推覆分析与非线性动力时程分析的结果都表明，在罕遇地震作用下，结构整体和各个结构构件仍具有明显的强度和变形能力安全储备。表明结构的抗震性能优于规范 GB50011-2001 规定的防倒塌的最低要求。不仅如此，在 100 年超越概率为 3%的罕遇地震作用下以上结论仍然成立。

11.7 Preliminary Conclusions

On the basis of results obtained from the non-linear seismic response analyses and performance evaluation, the following preliminary conclusions may be drawn:

1. The present structural concept and scheme satisfies requirements to achieve the no structural damage performance objective when subjected to the action of the design frequent earthquake. Non-linear pushover analyses indicate that the base shear force corresponding to first yielding in the structure is greater than the elastic base shear force under the design frequent earthquake, suggesting that the structure performs better than the code minimum requirement under the design frequent earthquake.
2. Under the action of the design intensity earthquake having a 10% of probability of exceedance in 50 years, columns do not yield in compression although brace buckling has occurred. Therefore, the degree of damage to the structure is under control and may be considered repairable.
3. The present structural concept and scheme satisfies requirements to achieve the collapse prevention performance objective when subjected to the action of the design rare earthquake both at the global structural level and at the local structural component level. Results obtained from both the non-linear pushover analyses and the non-linear dynamic time history analyses indicate that under the design rare earthquake, the structure still has a large safety margin of strength and deformation capacities, suggesting that the structure is capable of performing better than the code minimum requirement under the design rare earthquake. This conclusion is still valid even when the structure is subjected to a rare earthquake ground motion input having a 3% probability of exceedance in 100 years.

12. 下一阶段工作的建议

本报告所描述的结构设计工作还只是初步阶段. 在下一阶段, 通过专家评审并根据专家们的意见还要进行大量工作以改进结构性能和改进经济效易. 由于时间限制, 本报告所进行的结构非线性地震反应分析及抗震性能评价也只是初步的工作。在下一阶段随着结构设计工作的深入, 尚需进行更为详细的非线性分析, 进一步改进结构的性能. 对下一阶段工作的建议包括以下几个方面：

12.1 对斜撑进行重新归化布置

对斜撑位置和尺寸进行详细研究以提高斜撑在地震作用下的工作效率.

12.2 结构构件的非线性模拟

斜撑是关键性构件。它们是主要耗能的构件。准确模拟钢斜撑屈曲后的轴力-轴向变形恢复力滞回曲线对非线性结构地震反应分析至关重要。本节采用理想弹塑性双线型模型来模拟短粗斜撑屈曲后的轴力-轴向变形恢复力曲线。这项假设应进一步加以证实。除了本节所列出的实验结果外，尚需对单个斜撑受压屈曲后的行为进行非线性有限元分析，得出短粗斜撑屈曲后的轴力-轴向变形恢复力滞回曲线的理论解。

钢-钢筋混凝土组合柱的非线性模拟也需进一步改进，特别是约束混凝土的应力-应变曲线。其强度和最大可用压应变值的增加尚应根据构件的详细设计计算确定。

12.3 非线性静力推覆分析

水平作用推覆力的竖向分布需作进一步改进以更好反应高振型的效应。FEMA356 所建议的另两种分布模式将在下一步工作中采用，分述如下：

1. 水平推覆力的竖向分布由线弹性地震反应谱分析得到的层间剪力确定。反应谱分析应包括足够的振型数以便各振型有效质量的总和不小于结构总质量的 90%。根据 FEMA356 的建议，对基本周基大于 1 秒的建筑，应采用这种模式。
2. 自适应的竖向分布。在非线性静力推覆分析过程中，对水平作用力的竖向分布进行调整，使得水平力的竖向分布模式与结构的水平变形形状成比例。

12.4 非线性动力时程分析

地震波输入应采用三向输入，有两个水平分量和一个竖向分量。此外，地震波的反应谱曲线应与目标谱曲线拟合得更好。

12.5 应用场地地震记录与场地反应谱

当场地地震记录与场地反应谱数据送交奥雅纳后, 下一阶段的工作将应用这些数据作为地震输入进行线性与非线性地震分析.

12. FURTHER WORK

Work carried out to date and described in this report is preliminary. Through the EPD phase of the project more work will be done to further study and improve the CCTV tower design from both a performance and efficiency perspective. To date time has only allowed for a preliminary non-linear seismic response analysis and performance evaluation to be undertaken. Future work will involve more detailed analysis in parallel with possible design developments to improve the overall performance of the building. Some of these proposed studies are summarised below:

12.1 Re-evaluation of the bracing arrangements

The arrangement and assignment of the bracing elements will be studied in detail both in the level 1 elastic design and non-linear level 3 performance checks to achieve more efficiency.

12.2 Non-linear modelling of components

Braces are critical components, being the primary source of energy dissipation. Accurate modelling of the post-buckling axial force – axial deformation relationship of the braces is essential in non-linear seismic analysis. Presently, an elastic-perfectly-plastic bi-linear curve is used to model the post-buckling behaviour of the braces because they are classified as stocky braces. This assumption needs to be substantiated or modified by a separate study in order to establish this relationship.

The modelling of steel SRC columns also needs to be improved, particularly the assumed stress – strain curve for confined concrete. The increase in strength and the maximum usable compression strain will be determined once a detailed component design is available.

12.3 Non-linear pushover analysis

The pattern of vertical distribution of the applied lateral pushover force must be improved to better account for the effect of higher modes. Recommendations in FEMA 356 will be adopted in future work. The following two patterns will be considered:

- A vertical distribution proportional to the storey shear distribution calculated by the response spectrum analysis that includes sufficient modes to capture at least 90% of the building's total participating mass. According to FEMA 356, this pattern should be used for buildings with a fundamental period exceeding 1.0 second.
- An adaptive distribution that changes the vertical distribution of lateral forces as the structure develops inter-storey drifts during the pushover. The pattern is changed during the pushover to be proportional to the deflected shape of the structure.

12.4 Non-linear dynamic time history analysis

Tri-axial earthquake time history input, having two horizontal components and one vertical component, will be used in future work. Moreover, the match between the target and the achieved response spectrum curves will need to be improved to remove unconservatism.

12.5 Site specific time history data

Following the receipt of the Site Specific Hazard Assessment report from the seismic bureau more site specific time history records and response spectra will be adopted for the non-linear performance level checks.

12.6 更有效地耗能机制

这项工作包括两个方面。第一，减少目前结构设计的用钢量并优化斜撑截面尺寸 (目前所有斜撑截面相同)。减少结构用钢量可以降低结构的侧向刚度，起到降低地震水平力的作用。优化斜撑截面尺寸可以更有效地消耗能量并同时满足常遇地震作用下不出现结构破坏的抗震设计要求。

第二方面，有必要探讨其它概念和实施以增加耗能的效率。这方面的工作可包括被动耗能装置的应用。被动耗能装置在较为柔软的结构中更为有效。本节非线性地震反应分析得到的结构最大层间位移角大约是可接受限值的一半左右，表明目前的结构方案过于刚硬。尚有余地降低结构刚度以便减少用钢量，降低水平地震力并提高能耗效率。

12.6 More effective energy dissipation

As discussed the size and distribution of the steel braces needs to be optimised in order to more effectively dissipate energy while at the same time achieving the no damage performance objective under the design frequent earthquake.

Other concepts and implementations to increase the effectiveness of energy dissipation may be investigated. These may include application of special passive energy dissipation devices.

Energy dissipation is more effective if the structure is more flexible. The interstorey drift ratios obtained from the non-linear seismic analyses are approximately 50% of the acceptable limit. Therefore, it seems that the present structure is unnecessarily stiff. There is scope to increase the flexibility of the structure in order to increase the effectiveness of energy dissipation.

附录 A1 SD 图纸

A1 SCHEME DESIGN (SD) DRAWINGS

附录 A2　结构设计依据

A2 STRUCTURAL DESIGN BRIEF

附录 **A3** 钢混凝土组合柱刚度/承载容量及组合详图

A3 COMPOSITE COLUMN PROPERTIES AND ARRANGEMENT