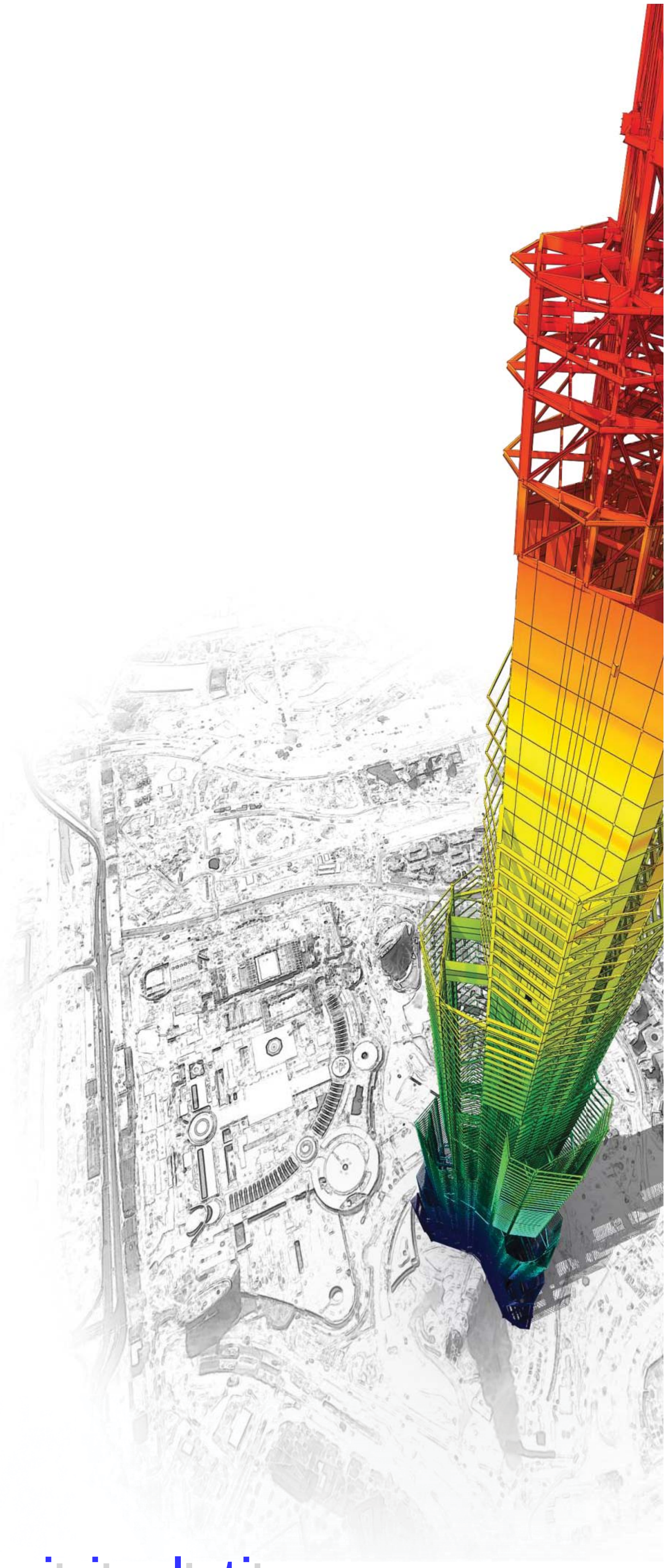
2014 MIDAS Technical Seminar

Challenges & Solution for Tall Building Design



MIDAS TECHNICAL SEMINAR

The Institution of Engineers (India, MSC)

November 15th, 2014 (Saturday)
09:30am - 05:30pm

About Seminar

The recent years have seen tremendous advances in High-Rise Building design in India. Structures such as the World One Tower (442m), Oasis Tower (372m) etc., have pushed the envelope like never before. As such, it becomes imperative for practicing engineers to be up to speed with all the latest developments in the field of High-Rise Buildings.

Today even the design codes have become demanding in terms of detailed and precise design. Understanding of behavior of structure and designing for safety brings in more concerns that should be addressed. Additionally, everything needs to be done quickly and efficiently. This creates a need for a powerful tool that addresses all the above issues. midas Gen has the strong ability to help engineers to perform modeling, analysis and design of structures. The software has been successfully applied to numerous projects thereby demonstrating creditability and stability.

This seminar will focus on familiarizing the structural analysis as well as design of buildings. midas Gen models are also compatible with major BIM tools which have gained a lot of importance recently.

Programs

Time	Sessions
9:30 - 10:00	**Registration**
10:00 - 10:30	Opening Remark
10:30 - 11:20	**Important criteria to be considered for high-rise building design report.** Prof. M. A. Chakrabarti, VJTI, (Former Member of High-rise Building Committee)
11:20 - 11:50	**Refreshment Break**
11:50 - 12:40	**Modeling Issues in high-rise buildings** Vinayak Naik, Sterling Engineering Consultancy Services
12:40 - 2:00	**Lunch break**
2:00 - 2:50	**Effect of wind loading on tall buildings** Prof. Tanuja Bandivadekar, SP College of Engineering
2:50 - 3:40	**Foundations for tall buildings** Jaydeep Wagh, Geocon International
3:40 - 4:00	**Refreshment Break**
4:00 - 4:50	**Column shortening analysis for high-rise building** Ravi Kiran Anne, MIDAS
4:50 - 5:30	**Introduction to MIDAS** - Introduction to midas Gen by Shayan Roy, MIDAS - Case studies by Raajesh Ladhad, Structural Concept
5:30 - 5:40	**Closing Remark & Lucky Draw**

Presenters

Speaker's Profile

The Institution of Engineers (India, MSC)

November 15th, 2014 (Saturday)
09:30am - 05:30pm

Modeling Issues in tall buildings

by Vinayak Naik, Sterling Engineering Consultancy Services Pvt. Ltd.

Vinayak Naik has designed number of ROB's, fly-overs *(a notable example is the Dadar T. T. flyover in Mumbai, Winner of Special Award by Indian Institute of Bridge Engineers for Year 2000),* mass housing projects, tall buildings, Loco car sheds for Railways (Precast / Prestressed), subways (a notable example is CST subway, Mumbai).

Important criteria to be considered for high-rise building design report.

by Professor M. A. Chakrabarti, VJTI
Former Member of High-rise Building Committee

Area of Interest
- Reliability engineering and system safety
- Non-linear dynamic analysis/system
- Self Repairable Concrete System

Effect of wind loading on tall buildings

by professor Tanuja Bandivadekar, SP College

Area of Interest
- Vibration control using passive dampers for structures
- Vibration control using passive dampers for bridges
- Multiple tuned mass dampers
- Mass excited structure, vibration control.
- Admixtures for high performance concrete.

Foundations for tall buildings

by Jaydeep D. Wagh, Geocon International

- Geotechnical consultant for over 10,000 projects across India.
- Completed projects in numerous countries of the world, including USA, Dubai, Nepal, Shri Lanka, and several countries in Africa.
- Geotechnical consultant for the tallest buildings in almost all metro cities of India, including Mumbai (6B+130 floors), Delhi (3B+85 floors), Kolkata (65 floors), Bangalore (3B+52 floors)(1997 - Present)

Case Studies using midas Gen

by Raajesh K. Ladhad, Structural Concept Design

Raajesh Ladhad has designed tall residential buildings, corporate office buildings, commercial buildings, and hospitals. He has personally trained and guided junior and senior engineers to develop structural designs and details in RC and steel structures.

Erection Engineering of high-rise building

- Column shortening analysis for high-rise building

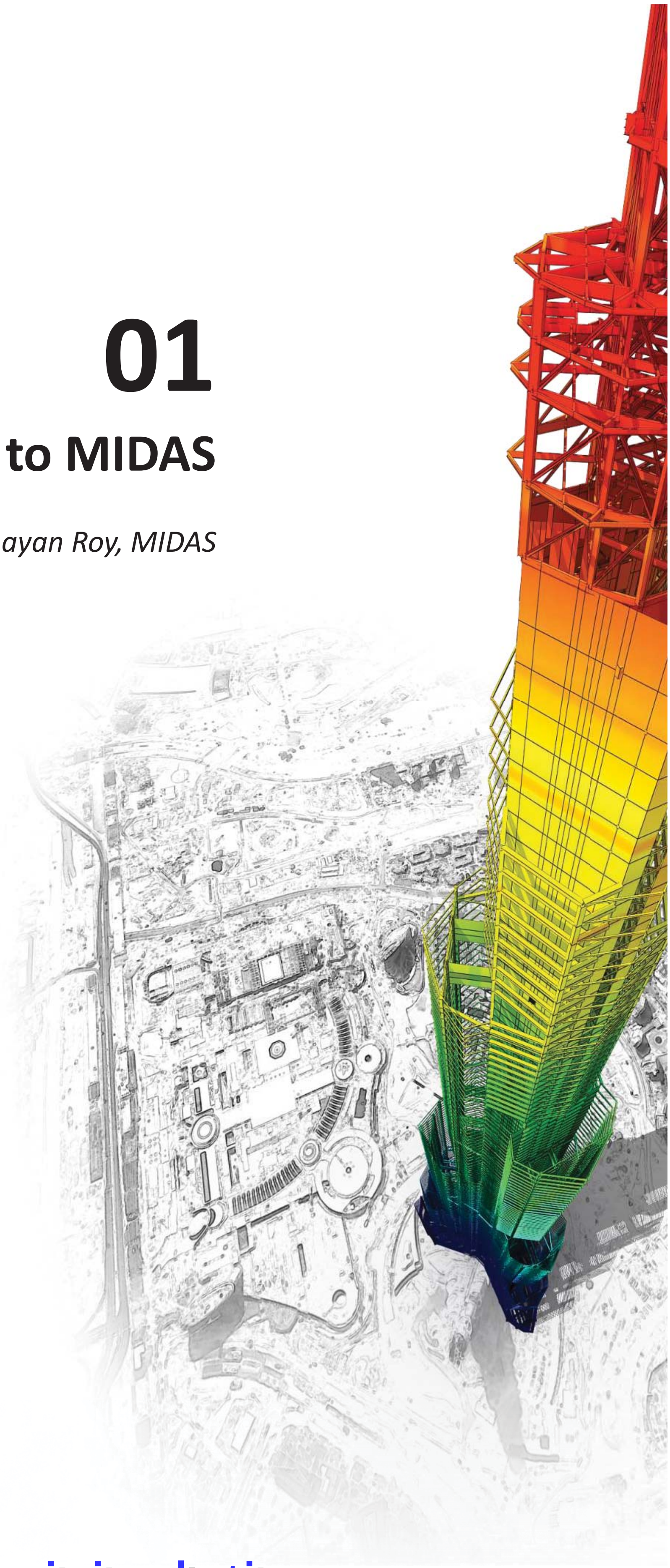
by Ravi Kiran Anne, MIDAS

Ravi Kiran is high-rise building design and finite element analysis specialist and technical director with 10 years' experience in high-rise buildings & infrastructure projects.

01

Introduction to MIDAS

Shayan Roy, MIDAS



MIDAS IT
Welcomes you to its
2014 Technical Seminar

Shayan Roy
MIDAS IT

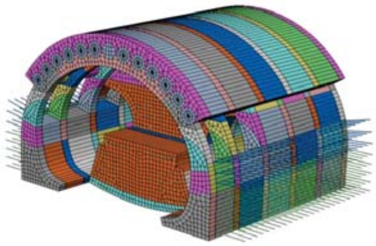
Contents

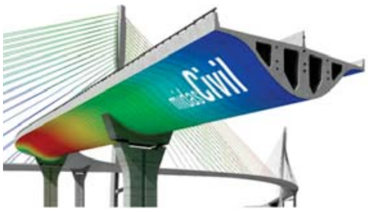
Objective

1. Creating Awareness about *MIDAS…*

2. Creating Awareness about the Latest & Powerful Technologies that MIDAS has…

3.Extending the Technologies & the Valued Tools to fulfill today's *ENGINEERING NEED in INDIA…*





About MIDAS

No. 1 Market Share
in Civil Engineering Software Solutions
450 Engineers & Professionals

120 *Distribution in over 120 Countries*
10,000 *Number of Clients*
30,000 *Number of Licenses Distributed Worldwide*

Global Network

MIDAS

Export to more than 90 countries worldwide through distributors in 28 countries

Retains the largest CAE market share

Business Areas

MIDAS

Engineering Consultancy	Software Developments
Bridge & Civil Structures	Bridge Engineering
Building & Plant Structures	Building Engineering
Geotechnical Analysis	Geotechnical Engineering
Mechanical Analysis	Mechanical Engineering

Business Areas

MIDAS

CAE Technology & Development

Engineering Consulting Service

Synergy

Optimal Solution for Practical Engineering

Product Line

MIDAS

Building & Structural Engineering	Bridge & Civil Engineering	Geotechnical & Tunnel Engineering	Mechanical Engineering
midas Gen *Integrated Design System for Building and General Structures*	**midas Civil** *Integrated Solution System for Bridge and Civil Structures*	**midas GTS** *2D / 3D Geotechnical and Tunnel analysis System*	**midas NFX** *Total Solutions for True Analysis-driven Design*
midas Design+ *Structural Component Design & Detailing*	**midas FEA** *Advanced Nonlinear and Detail Analysis System*	**SoilWorks** *Geotechnical Solutions for Practical Design*	**midas FX+** *General Pre & Post Processor for Finite Element Analysis*
midas DShop *Auto-Drafting Module to generate Structural Drawings and Bill of Materials*			

Introduction to midas Gen

Key Clients

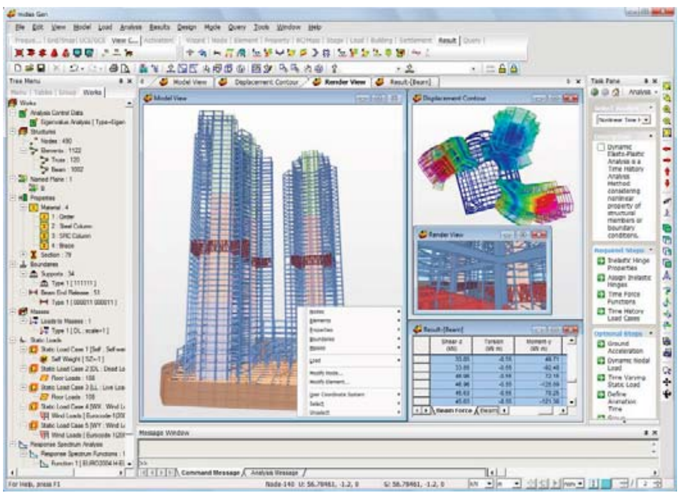
MIDAS

International			Indian	
ACKG	HDR	Parsons Brinckerhoff	AFCONS	Arun Gokhale & Associates
AECOM	HNTB	Ramboll Gruppen	CDM Smith	DCIPL
ARCADIS	Hyder	Royal Haskoning	CES	J+W Consultants
ARUP	Hyundai Engineering	SMEC	EGIS India	Milind Kulkarni
ATKINS	INGEROP	SNC-Lavalin International	L&T	Mahimatura
Beca Group	Italferr SpA	Thornton Tomasetti	Louis Berger Group	Nagarjuna Constructions
Bechtel	Jacobs	URS	Mott MacDonald	Navinnavare
Black & Veatch	Korea Power Engineering	WSP Group	Phiske Consultant	Satish Marathe Consultants
CH2M HILL	Langan		Pragati Consultants	Sunil Mutalik
COWI	Louis Berger Group		S.N.Bhobe & Associates	S.W.Mone & Associates
CTI Engineering	Michael Baker Corp.		STUP Consultants	Structus Consultants
Dar Al-Handasah	MMM Group		Shrikande Consultants	Vastec
DHV Group	Mott MacDonald		Tandon Consultants	
GHD	Mouchel		PWD, Navi Mumbai	
Golder Associates	MWH Global		RDSO	
Halcrow	Parsons		Western Railway	

Introduction to midas Gen

Structural Engineering

MIDAS



midas Gen

Burj Khalifa (UAE)

Kingdom Tower (Saudi Arabia)

Beijing Olympic (China)

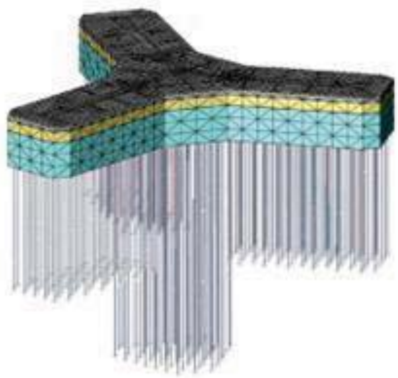
Structural Engineering

Application Areas

- ✓ All types of buildings (RC, Steel, Composite)
- ✓ Plant structures, Airport & Hangars
- ✓ Stadiums, arenas & gymnasiums
- ✓ Column shortening prediction and design
- ✓ Post-tension and pre-stressed concrete analysis
- ✓ Nonlinear seismic performance evaluation
- ✓ Structural safety checks through detail analysis

Tall Building Projects

Kingdom Tower (Saudi Arabia)
World's Tallest Building
Over 1,000 meters in height

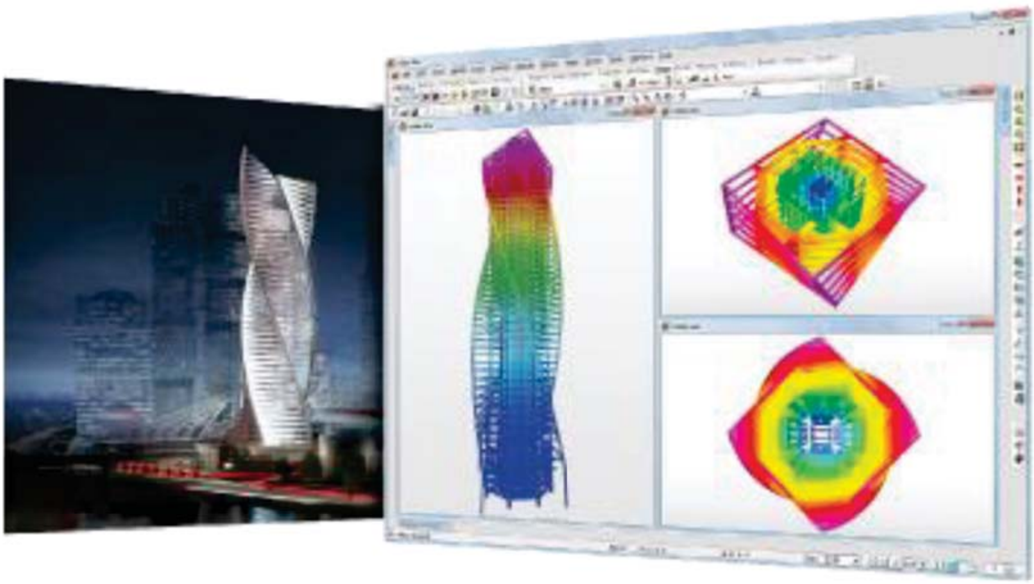


Tall Building Projects

Burj Khalifa (UAE)

The World's Tallest Building

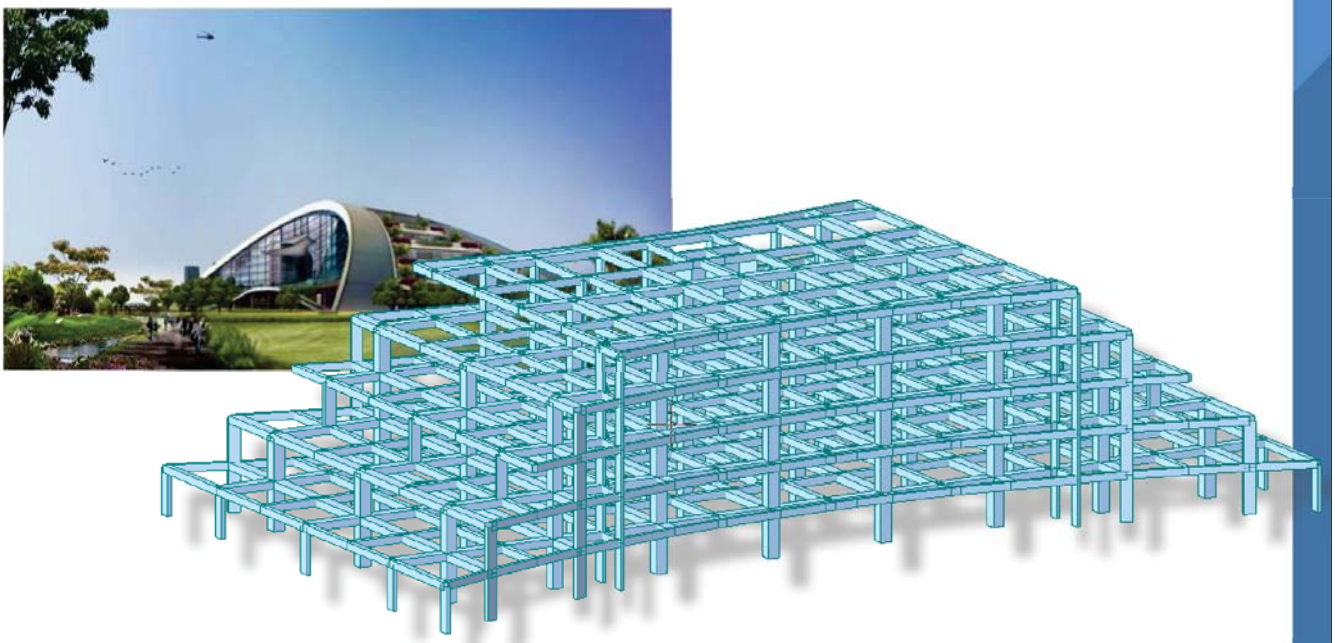
Tall Building Projects



Moscow City Palace Tower (Russia)

Twisting 46-story Building with Composite Columns

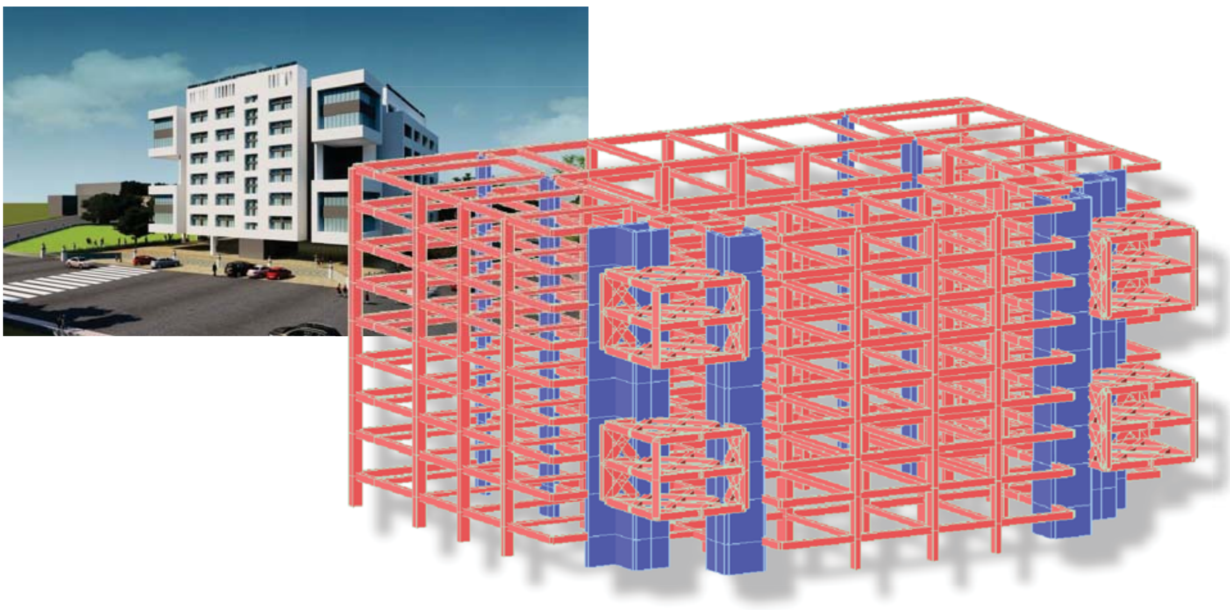
Tall Building Projects



Project Name: **ONGC Corporate Office, Navi Mumbai**
Designed by: **Structural Concept, Navi Mumbai**

Introduction to midas Gen

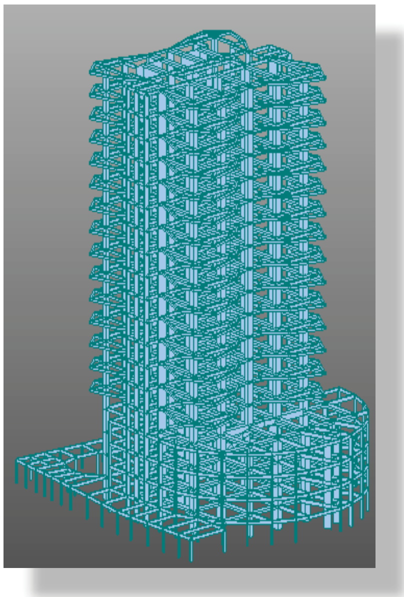
Tall Building Projects



Project Name: **Karnala *In*ternational School, Dronagiri**
Designed by: **Structural Concept, Navi Mumbai**

Introduction to midas Gen

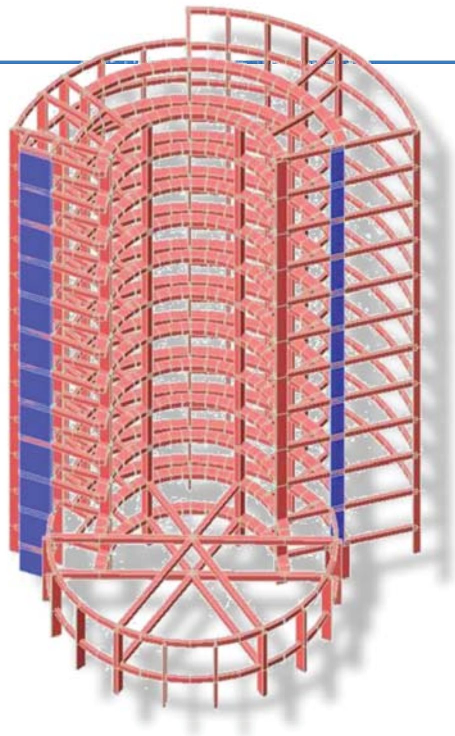
Tall Building Projects



Project Name: **Elerro Fiesta, Sanpada**
Designed by: **Structural Concept, Navi Mumbai**

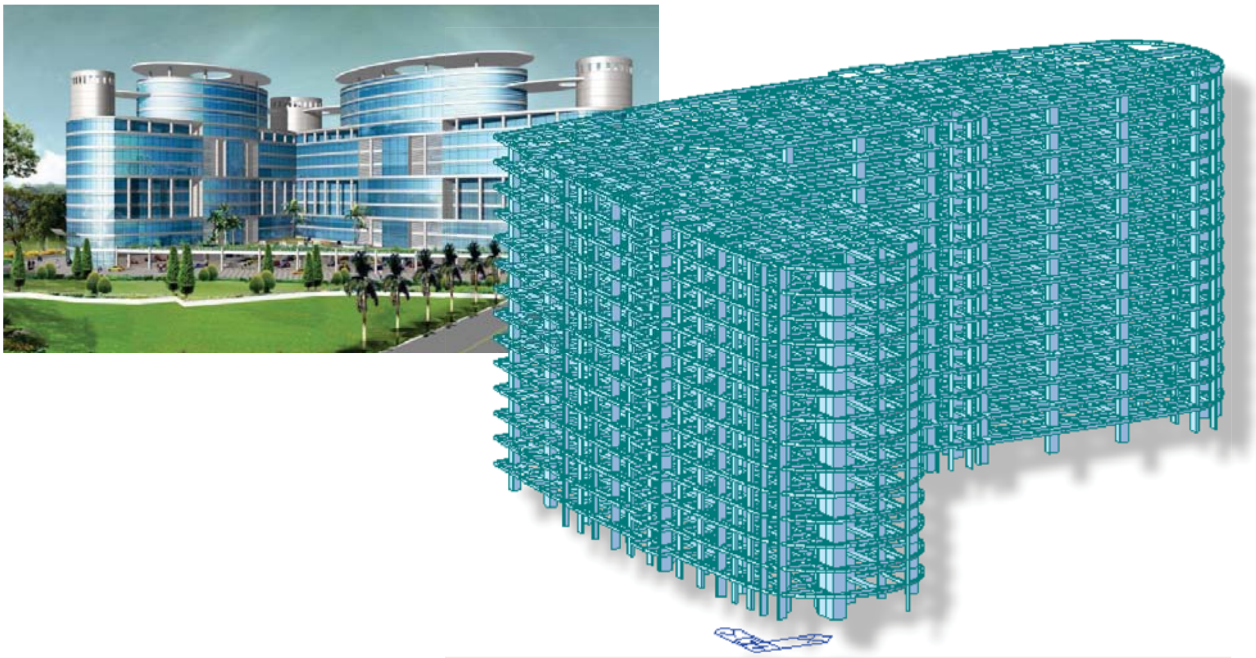
Introduction to midas Gen

Tall Building Projects



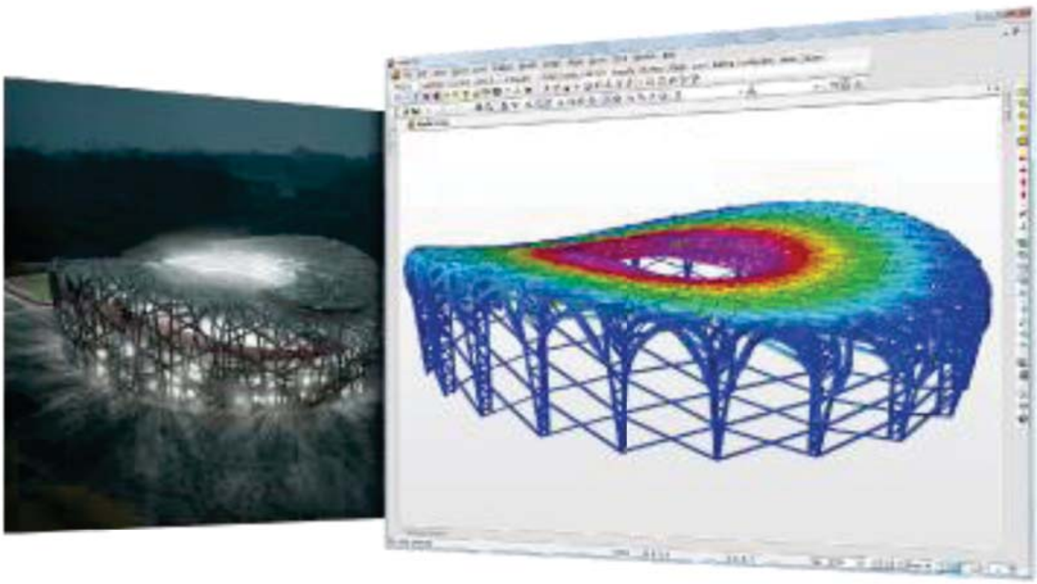
Project Name: **Marine Academy, Panvel**
Designed by: **Structural Concept, Navi Mumbai**

Tall Building Projects



Project Name: **Reliable Tech Park, Navi Mumbai**
Designed by: **Structural Concept, Navi Mumbai**

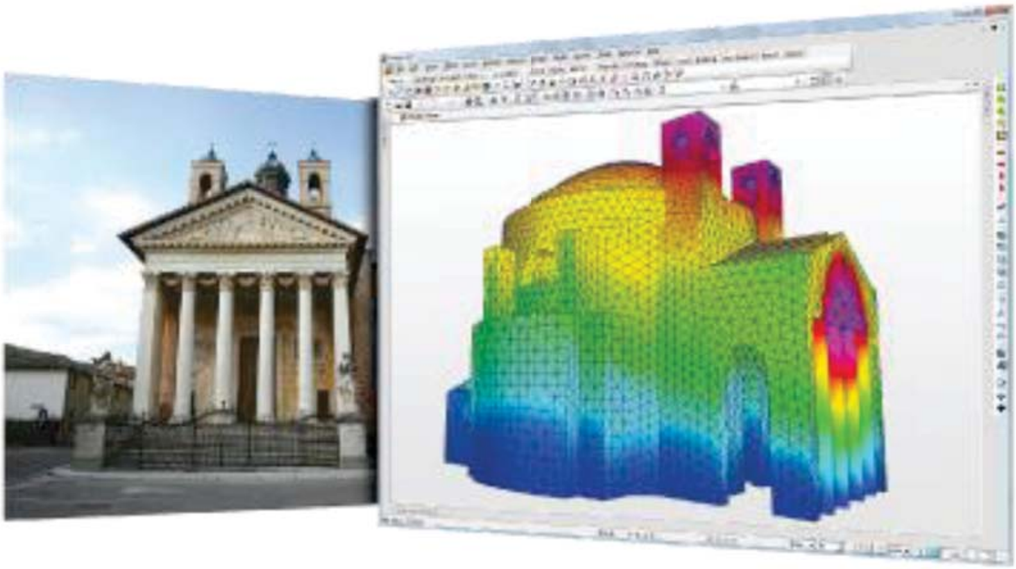
Speciality Projects



Beijing National Stadium (China)

Beijing Olympic Main Stadium

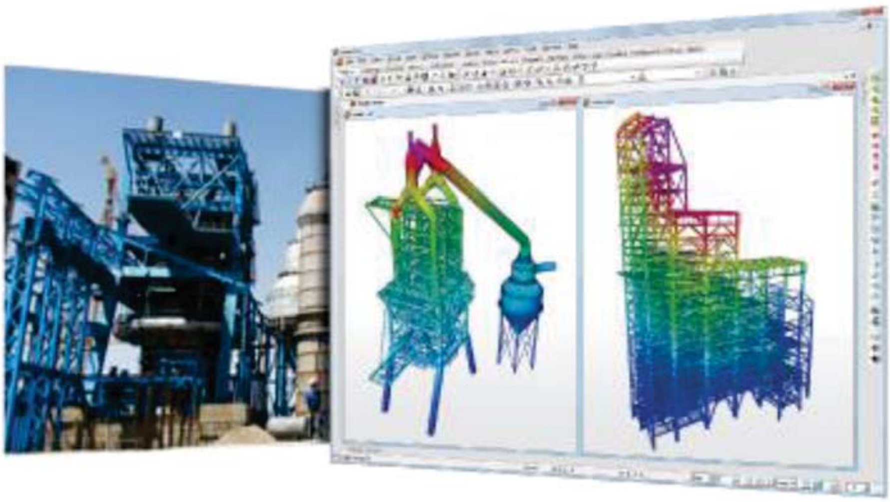
Speciality Projects



Temietto di Villa Barbaro (Italy)

Structural Evaluation of Vulnerable Historic

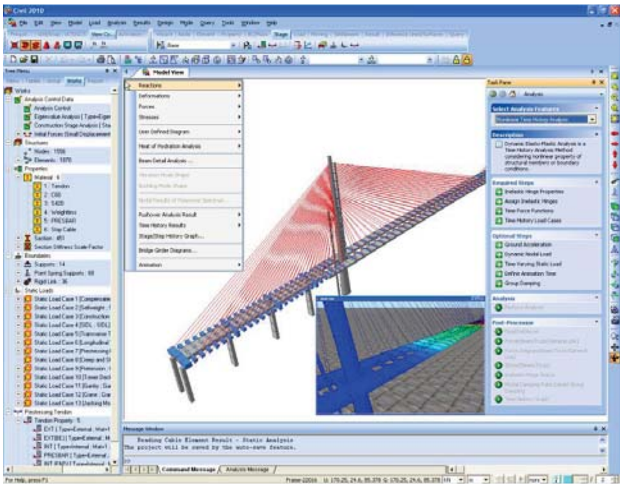
Plant Projects



Tavazon Steel Plant (Iran)

Steel Manufacture Plant

Bridge Engineering



midas **Civil**

Sutong Cable-stayed Br. (China)

Russky Island Br. (Russia)

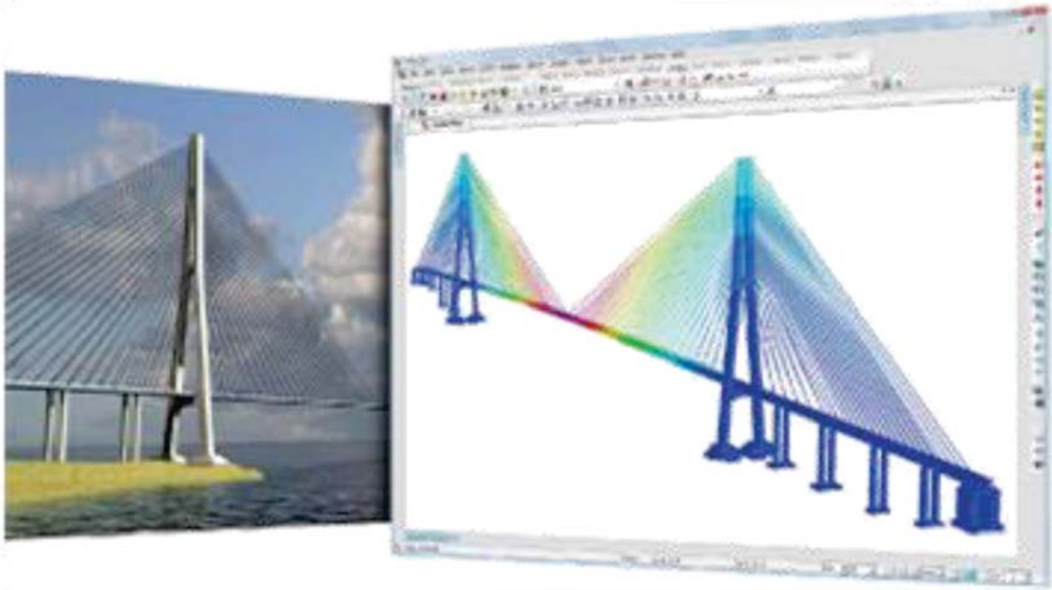
Sunda Strait Br. (Indonesia)

Bridge Engineering

Application Areas

- ✓ Conventional bridges (skewed slab, frame & culvert)
- ✓ Curved steel girders, composite, integral bridges & PC girder bridges
- ✓ Segmental post-tensioning (BCM, ILM, MSS & FSM)
- ✓ Cable stayed bridges & extradosed bridges
- ✓ Suspension bridges (Earth-anchored & Self-anchored)
- ✓ Fatigue check and seismic performance evaluation
- ✓ Wind evaluation (CFD analysis)

Cable Stayed Bridges



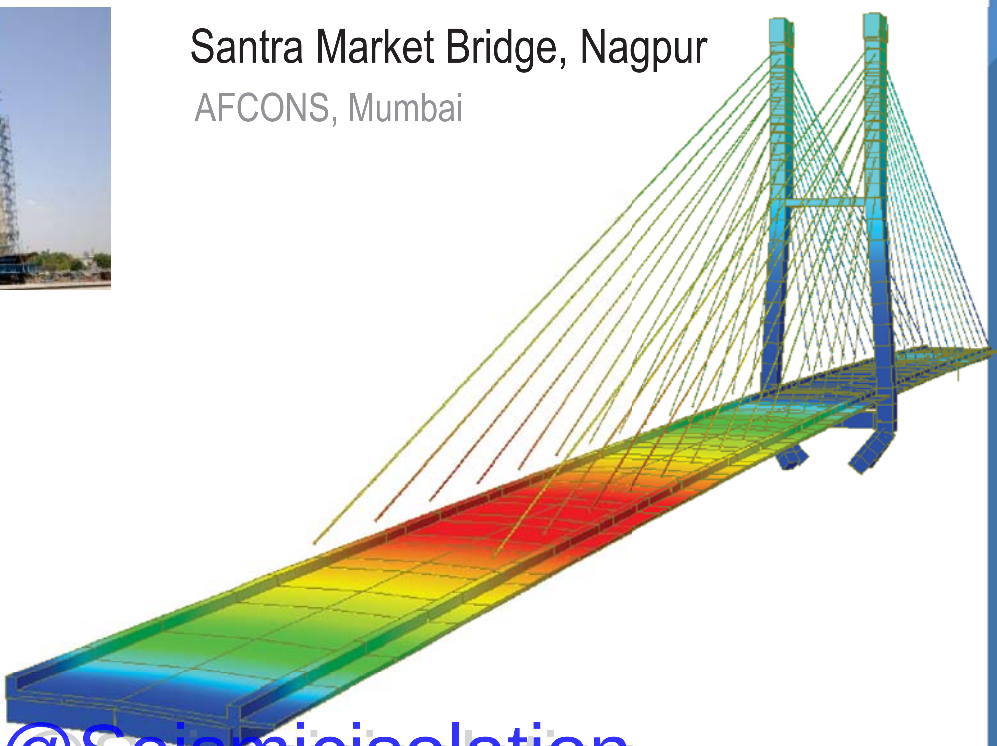
Russky Island Bridge (Russia)

The World's Longest & Tallest Cable Stayed Bridge

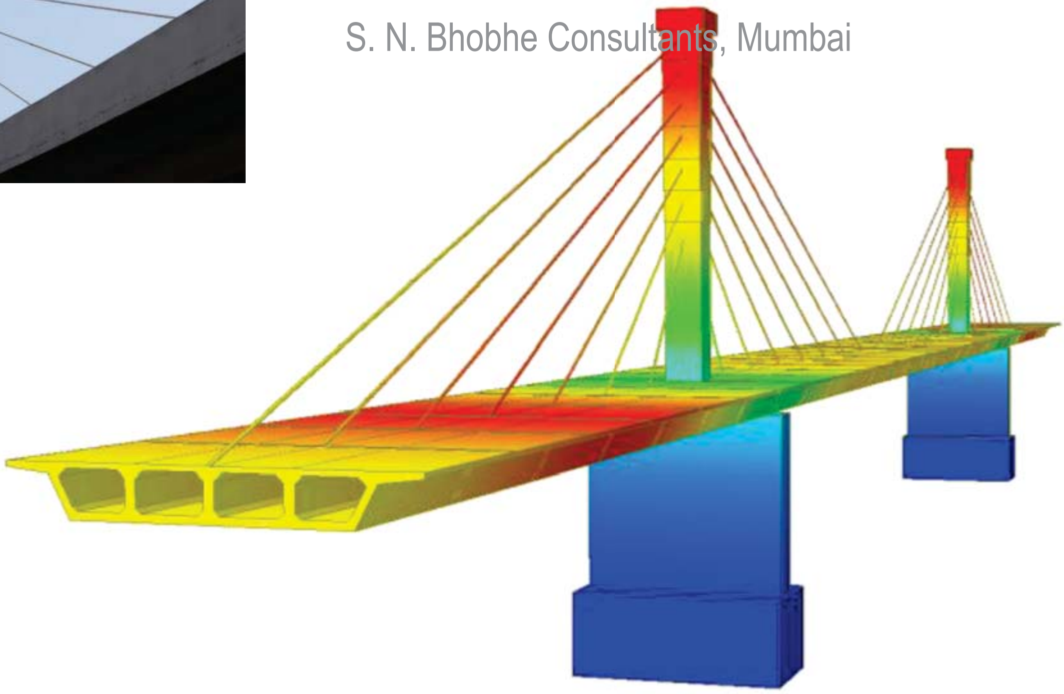
Cable Stayed Bridges

Santra Market Bridge, Nagpur

AFCONS, Mumbai



Cable Stayed Bridges



Tapi Cable Stayed Bridge, Surat

S. N. Bhobhe Consultants, Mumbai

Metro Rail

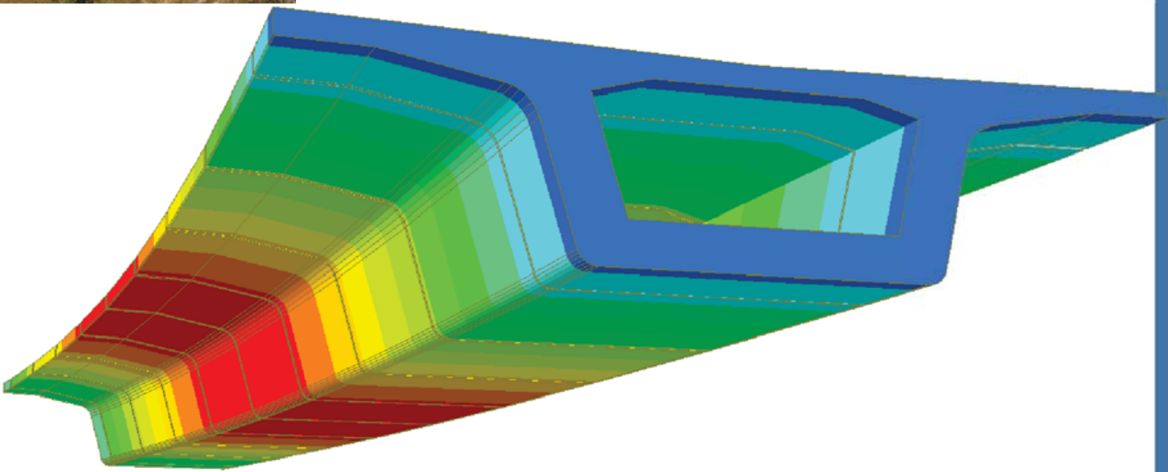
Hyderabad Metro

L&T Rambol, Chennai

Metro Rail

Navi Mumbai Metro

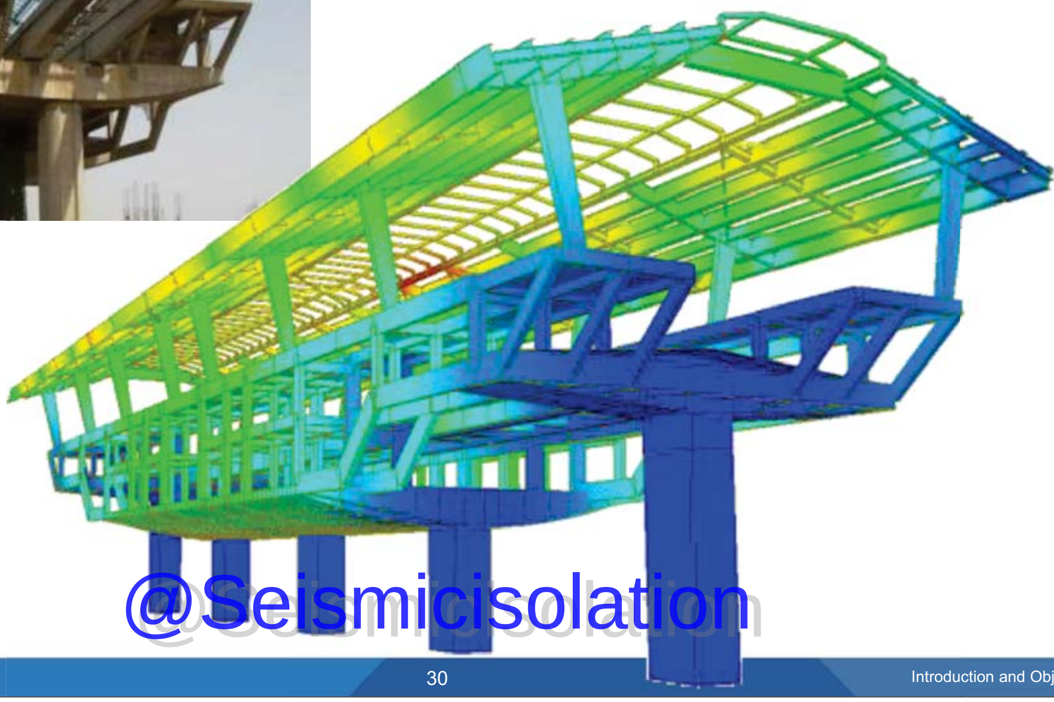
Louis Berger Group, Mumbai



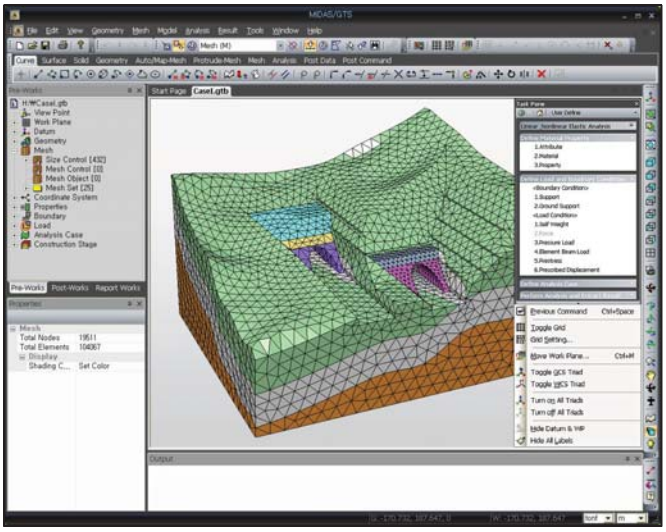
Special Structures

Mumbai Monorail Station

Louis Berger Group, Mumbai



Geotechnical Engineering



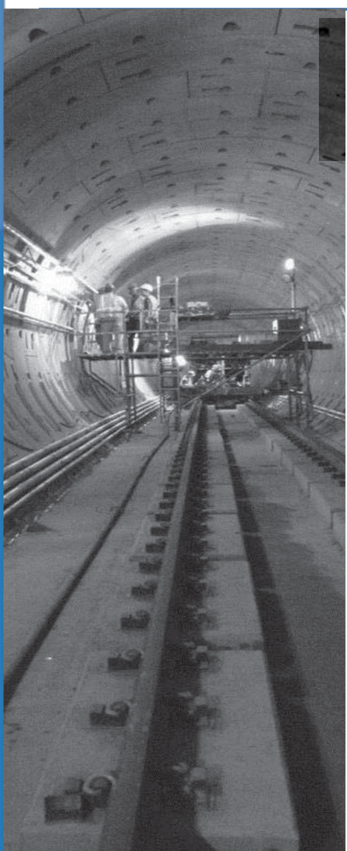
midas GTS

Kingdom Tower (Saudi Arabia)

New York Subway (USA)

King's Cross Station (UK)

Geotechical Engineering

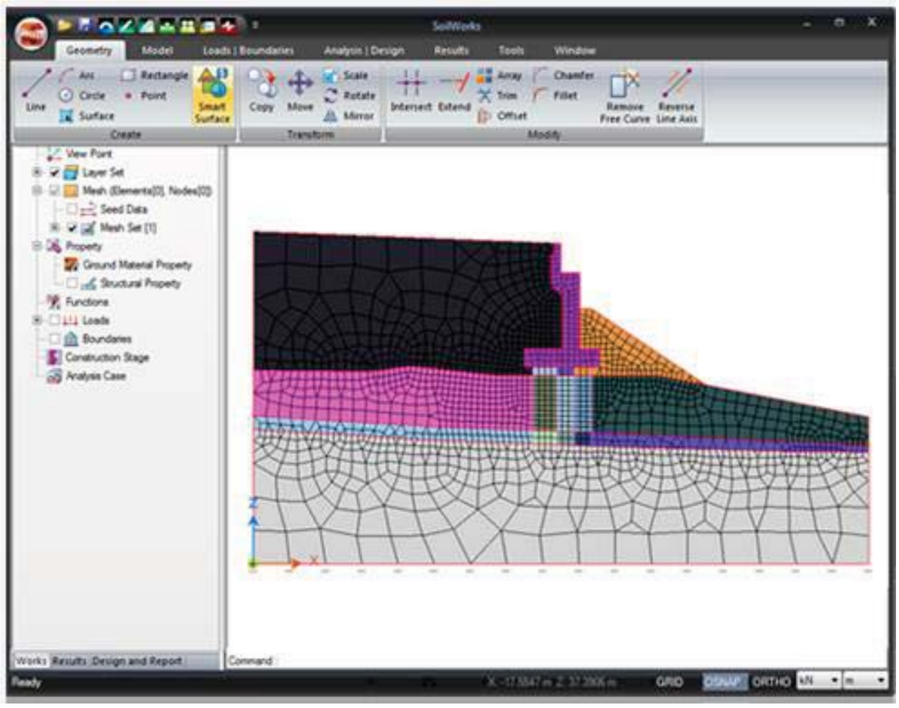


Application Areas

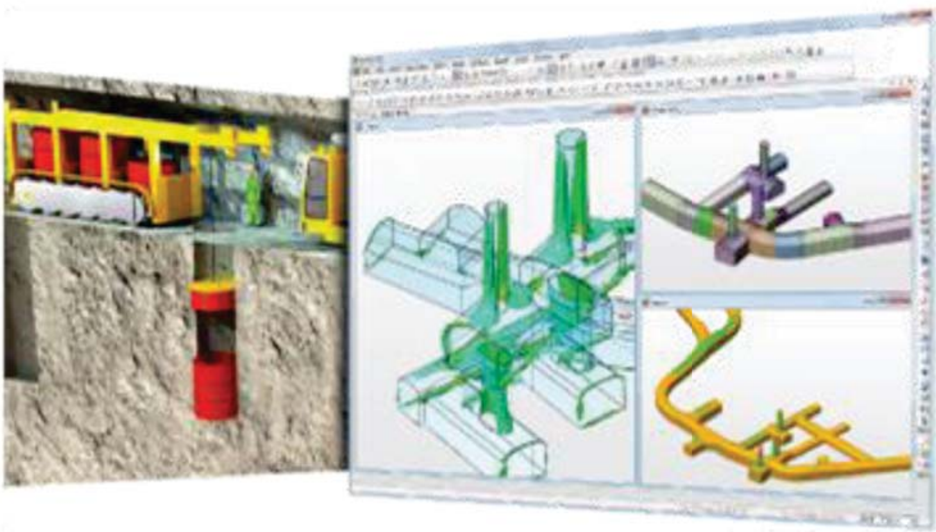
- ✓ Deep foundations & Soil-Structure Interaction
- ✓ Deep excavation and temporary structures
- ✓ Underground structures (subway & disposal facilities)
- ✓ Unconventional tunnel intersections
- ✓ Slope stability and embankments
- ✓ Groundwater Flow and Coupled Analyses
- ✓ Vibration analysis for earthquake & blasting

Geotechnical Engineering

SoilWorks



Tunnel & Underground Structures

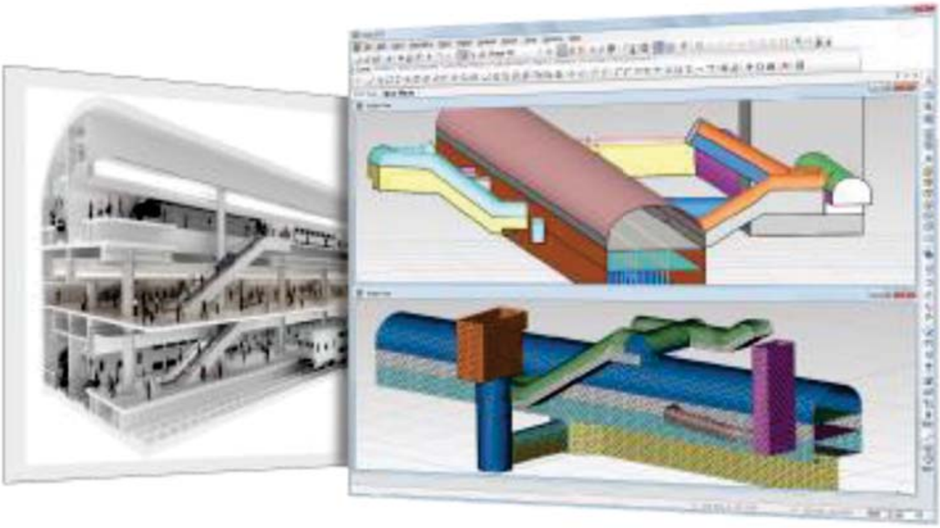


Posiva's ONKALO (Finland)

Nuclear Waste Disposal Facility

Introduction to midas Gen

Tunnel & Underground Structures



Trans-Hudson Express (U.S.A)

Stability Evaluation for Station Complex

Introduction to midas Gen

Excavations & Foundations

Dubai Tower (Qatar)

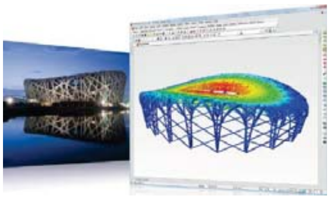
Piled-raft Foundation of 84-story Building

Introduction

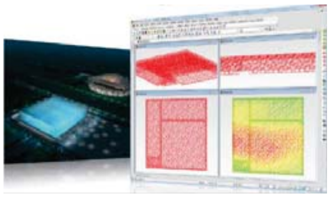
MIDAS

Specialty Structures Applications

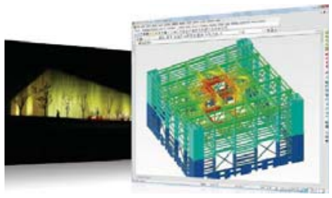
- → Stadiums
- → Power Plants
- → Hangar
- → Airport
- → Transmission Towers
- → Cranes
- → Pressure Vessels
- → Machine Structures
- → Underground Structures …



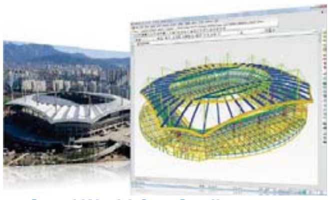
Beijing National Stadium



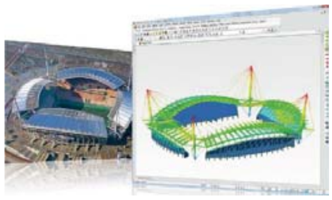
Beijing National Aquatic Center



Beijing Olympic Basketball Gymnasium

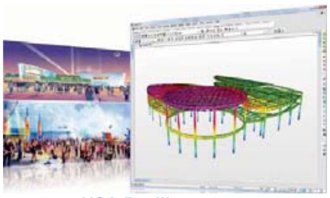


Seoul World Cup Stadium

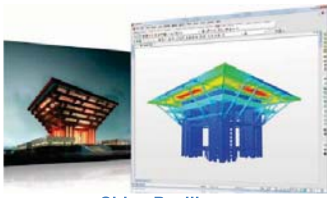


JeonJu World Cup Stadium

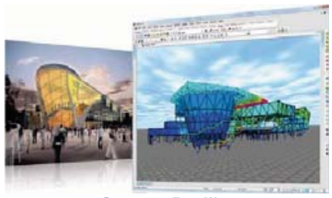
DeaJeon World Cup Stadium



USA Pavilion



China Pavilion



German Pavilion

Why midas Gen

MIDAS

Practical

Easy to Use

Software

Reliable

Good Support

Introduction to midas Gen

Reliable

1 Various Project Applications

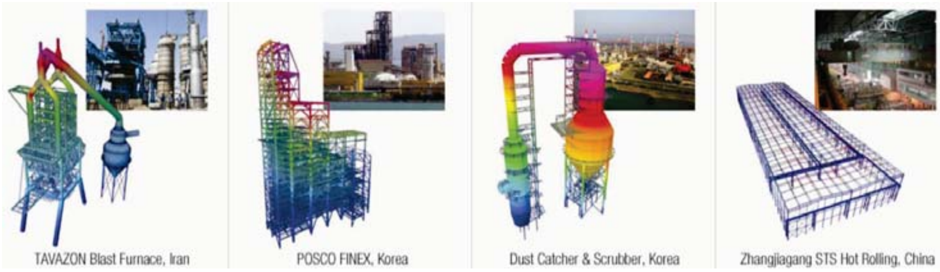
→ **50 countries, 6500 copies**

→ **Partial List of Client**

- URS Corp.
- Parsons Brinckerhoff
- TY LIN
- Ove Arup Gr.
- Jacobs Engineering
- RMJM
- Imbsen & Associates
- Michael Baker Jr.
- R.W. Armstrong and Associates
- Hewson Consulting Engineers Ltd
- Samsung Engg. & Construction
- POSCO Steel & Construction
- CALTRANS (California Dept. of Transportation)
- Oregon Dept. of Transportation
- Pennsylvania Dept. of Transportation
- US Army …

Spatial Structures

Specialty Structures



Introduction to midas Gen

Reliable

2 QA & QC System

→ **MQC System (midas Quality Control System)**

→ **Bug Reporting System**

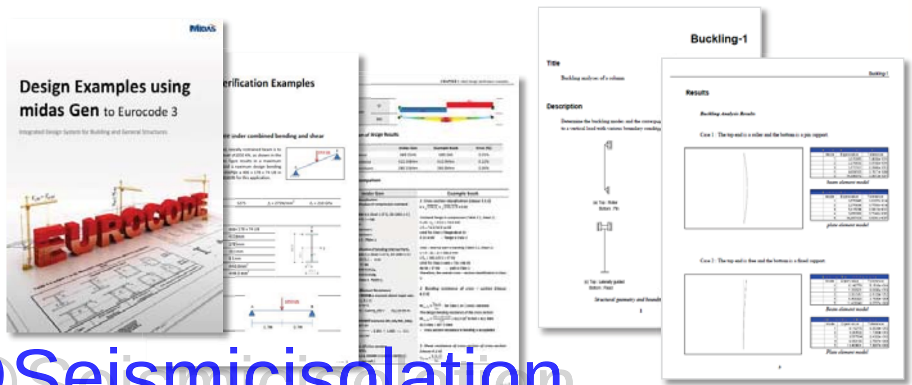
MQC System

Bug Reporting System

3 Verification Examples

→ **More than 100 Verification Examples**

→ **Design Verification Examples**



What Gen can Do

MIDAS

- **Static Analysis**
- **Dynamic Analysis**
  - Static Seismic Analysis
  - Response Spectrum Analysis
  - Time History Analysis
- **Geometric Nonlinear Analysis**
  - P-Delta Analysis
  - Large Displacement Analysis
- **Material Nonlinear Analysis**
  - Structural Masonry Analysis
- **Construction Stage Analysis**
  - Time Dependent Material
  - Column Shortening Analysis
- **Buckling Analysis**
- **Heat of Hydration Analysis**
  - With and Without Pipe Cooling
- **Boundary Change Analysis**
- **Boundary Nonlinear Analysis**
  - Damper, Isolator, Gap, Hook
- **Pushover Analysis**
  - RC, Steel, SRC, Masonry
- **Settlement Analysis**
- **Inelastic Time History Analysis**

What Gen can Do

MIDAS

RC Design	Steel Design	SRC Design
ACI318	AISC-LRFD	SSRC79
Eurocode 2, Eurocode 8	AISC-ASD	JGJ138
BS8110	AISI-CFSD	CECS28
IS:456 & IS:13920	Eurocode 3	AIJ-SRC
CSA-A23.3	BS5950	TWN-SRC
GB50010	IS:800 (1984 & 2007)	AIK-SRC
AIJ-WSD	CSA-S16-01	KSSC-CFT
TWN-USD	GBJ17, GB50017	**Footing Design**
AIK-USD, WSD	AIJ-ASD	ACI318
KSCE-USD	TWN-ASD, LSD	BS8110
KCI-USD	AIK-ASD, LSD, CFSD	
Slab Design	KSCE-ASD	
Eurocode 2	KSSC-ASD	

Thank You

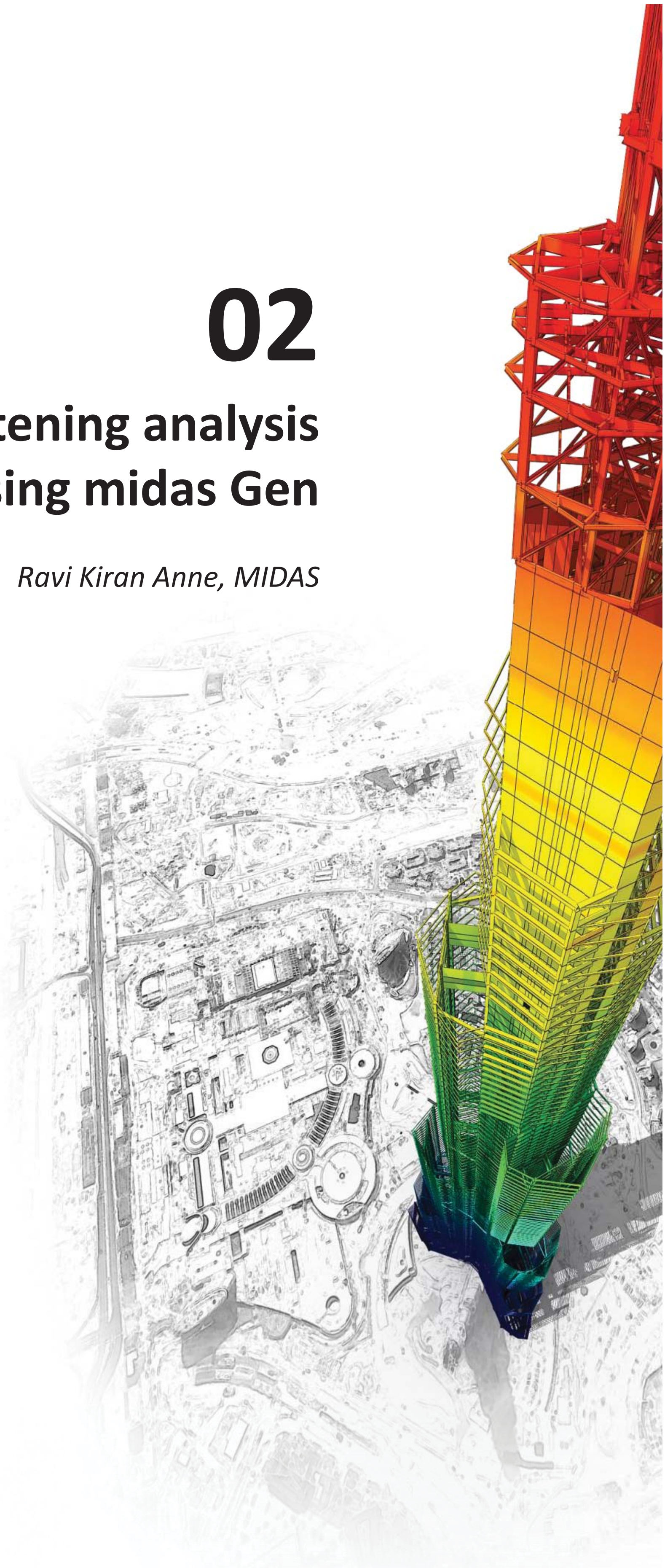
MIDAS

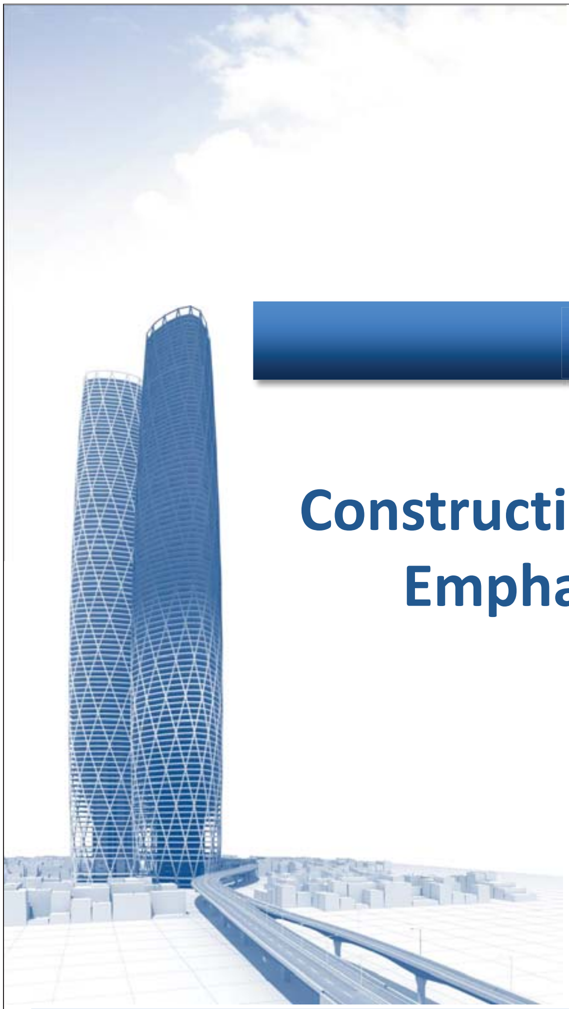
2014 MIDAS Technical Seminar
Challenges & Solution for Tall Building Design

02

Column shortening analysis for high rise building using midas Gen

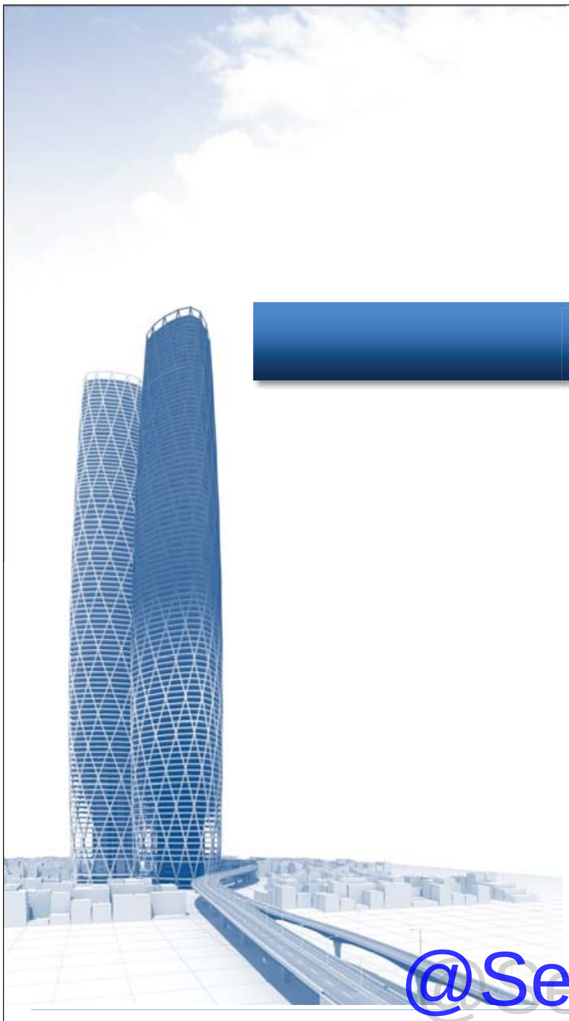
Ravi Kiran Anne, MIDAS





midas Gen – One Stop Solution for Building and General Structures

Construction Stage Analysis with Special Emphasis on Column Shortening

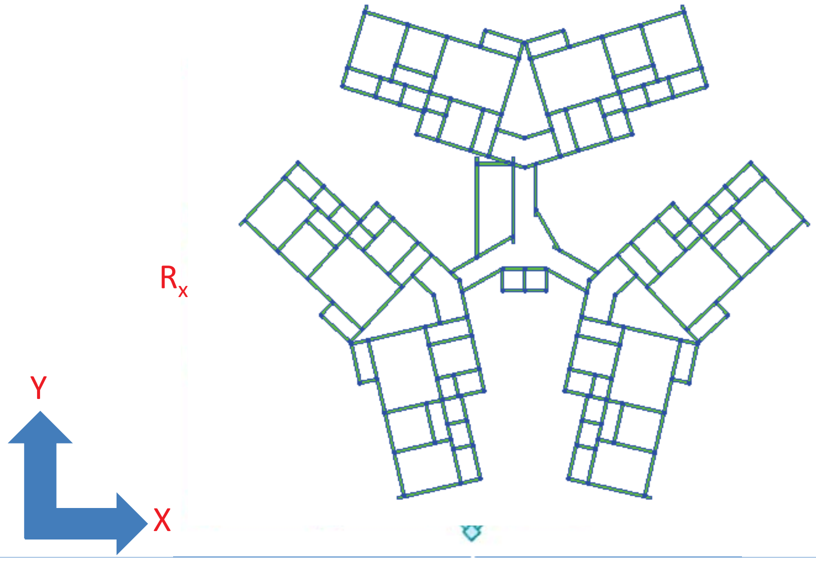


midas Gen – One Stop Solution for Building and General Structures

Contents

Response Spectrum Analysis

- In Irregular Structures, one directional response spectrum results may include a different direction's response.
- When it occurs, the base shear force from the response spectrum analysis is remarkably smaller than the base shear force calculated from static seismic analysis.
- This causes the scale factor to be very large, also causing an overestimation for the design.



Response Spectrum Analysis

➢ Principal axis

" Principal Axes of a building are generally two mutually perpendicular horizontal directions in a plan of a building along which the geometry of the building is oriented"

" Direction in which the seismic load has the largest influence on the structure."

➢ Ways to Find Principal axis

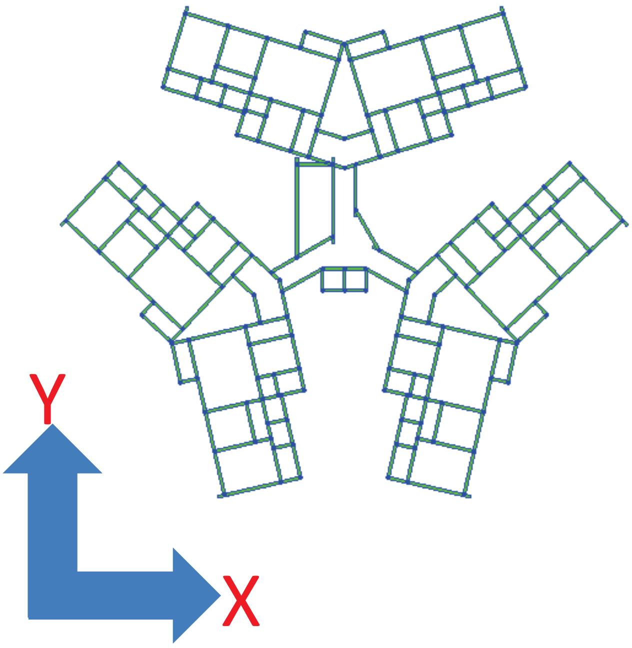
1. Establishment of the Reaction Direction of the 1st Mode to Principal Axis after Modal Analysis.
   E.L.Wilson. "Three-Dimensional Static and Dynamic Analysis of Structures", Computer and Structures, 2002.

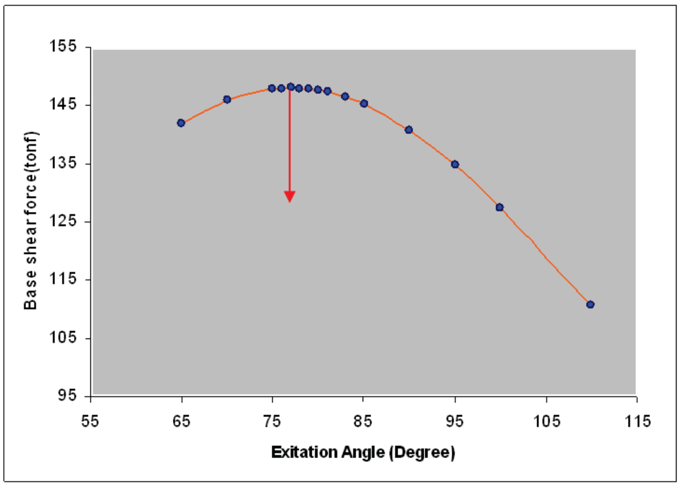
2. Finding the Critical Angle Using Modal Analysis Method's Fundamentals and CQC Theory, Trial and Error Method.
   O.A.Lopez and R. Torres. "The Critical Angle of Seismic Incidence and the Maximum Structure Response", EESD, 1997

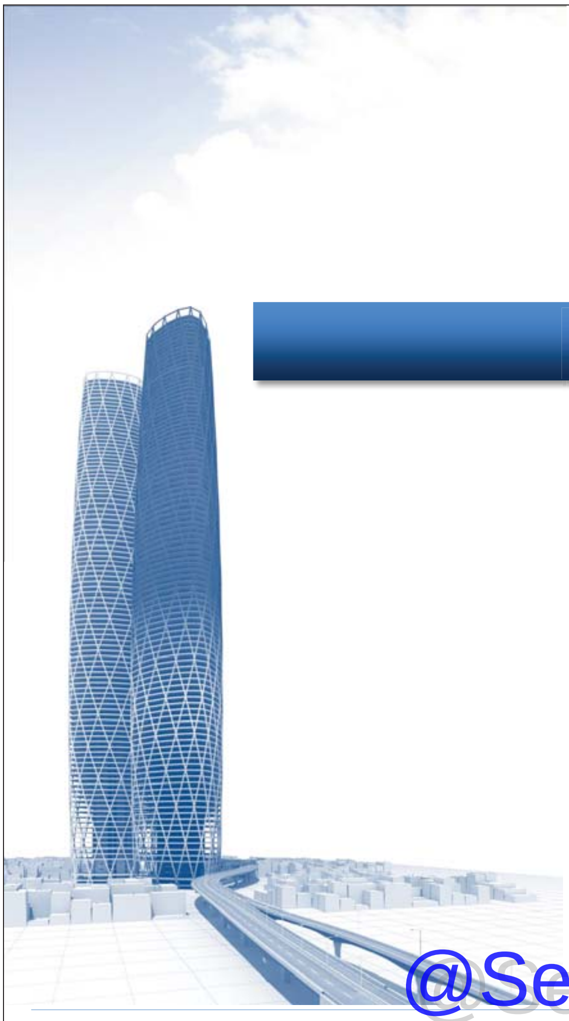
3. Trial and Error Method: Practical Approach.

Response Spectrum Analysis

➢Auto Search Principal axis







MIDAS

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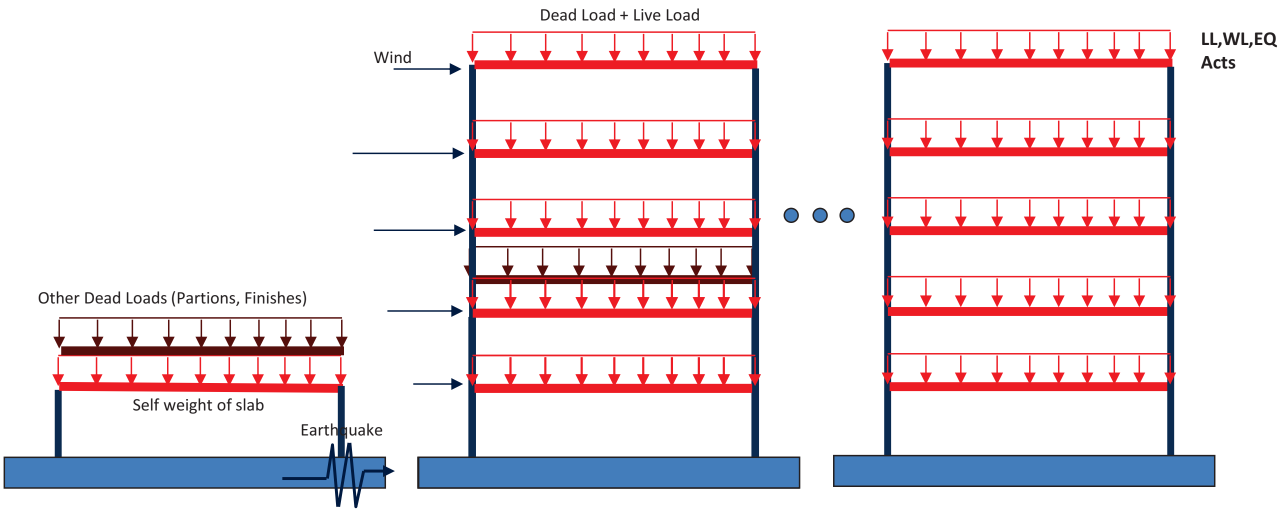
Why Construction Stage (CS) Analysis

- In general structures are analyzed assuming that the structure is built and loaded in a moment.
- Construction of structures is a time taking process and during this period Material Properties, Loads and Boundaries conditions may change.



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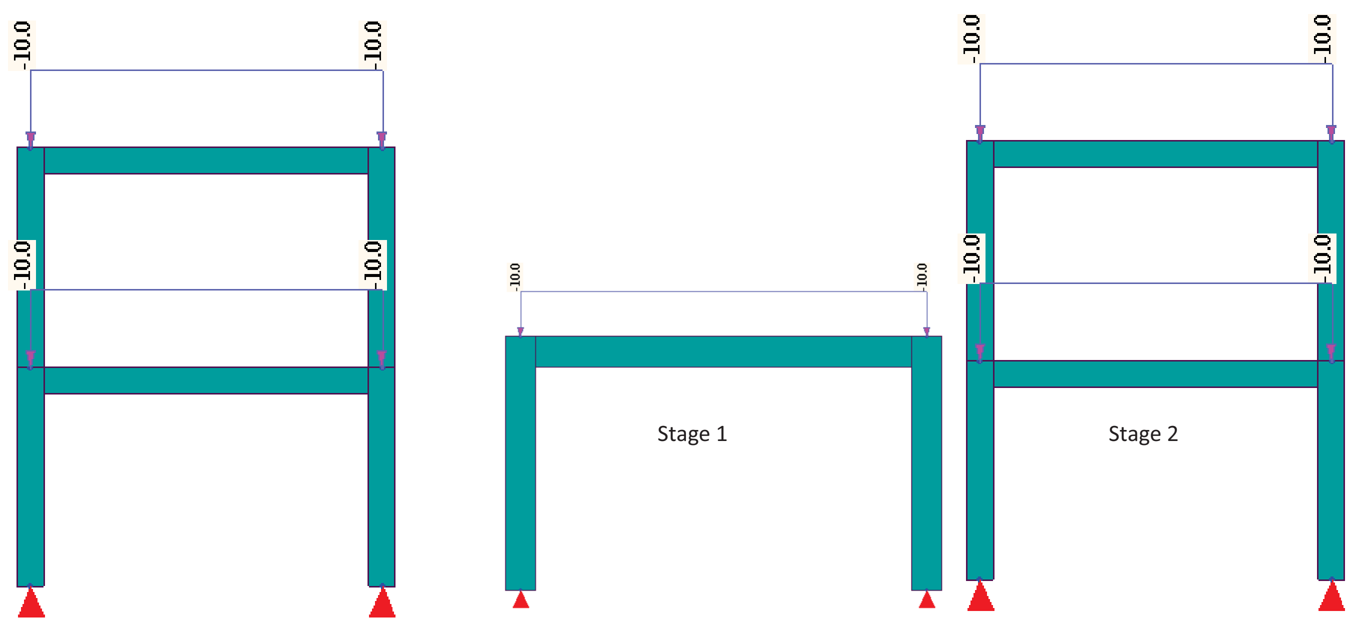
Construction Stage Analysis
One Stop Solution for Building and General Structures

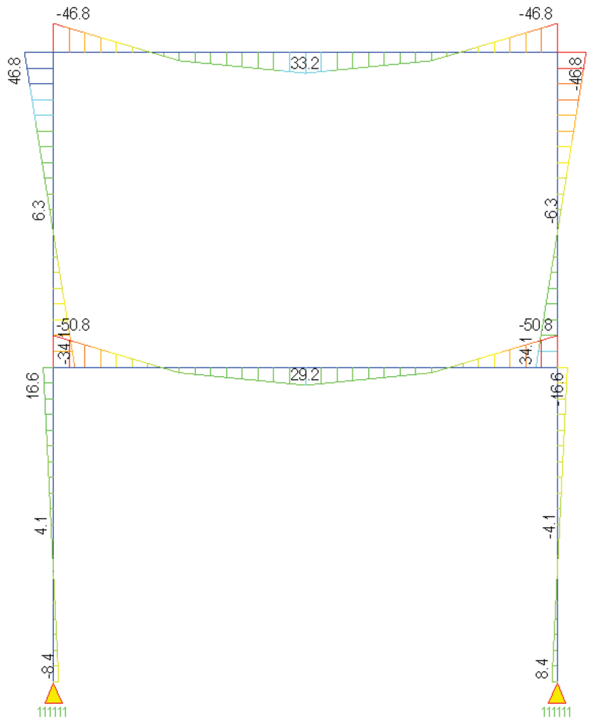
Conventional Analysis Vs. Construction Stage Analysis



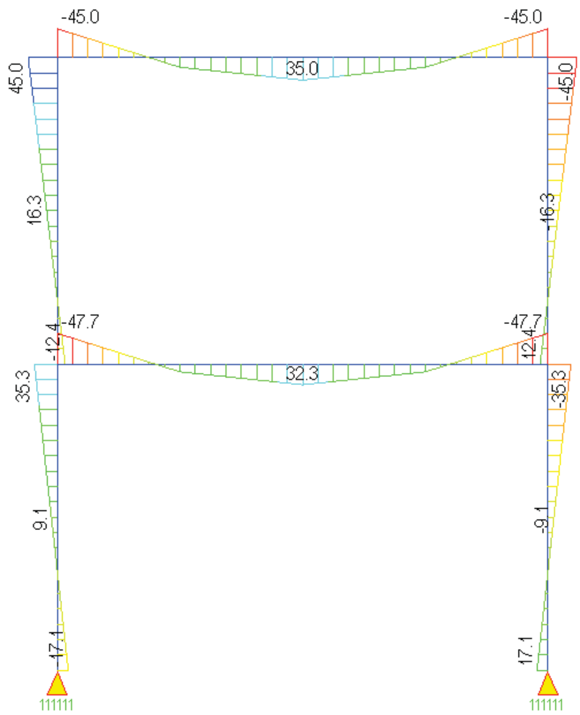
Case 1 – Conventional Analysis

Case 2 – CS Analysis

Conventional Analysis Vs. Construction Stage Analysis



Case 1 – Conventional Analysis



Case 2 – CS Analysis

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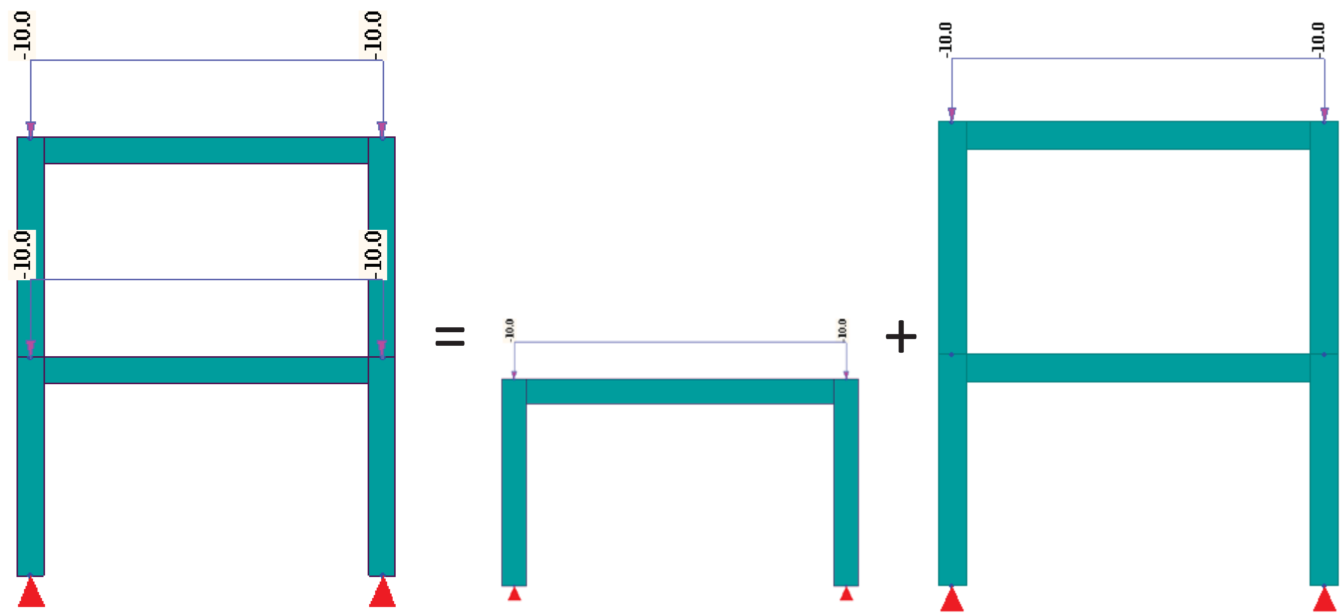
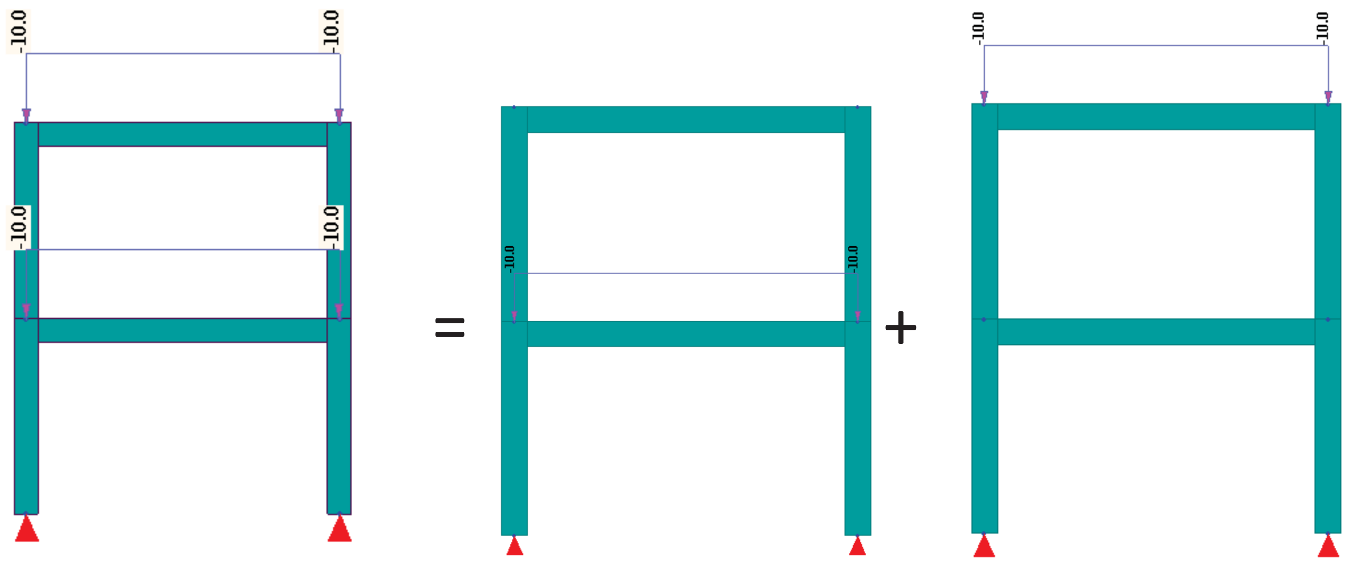
Conventional Analysis Vs. Construction Stage Analysis

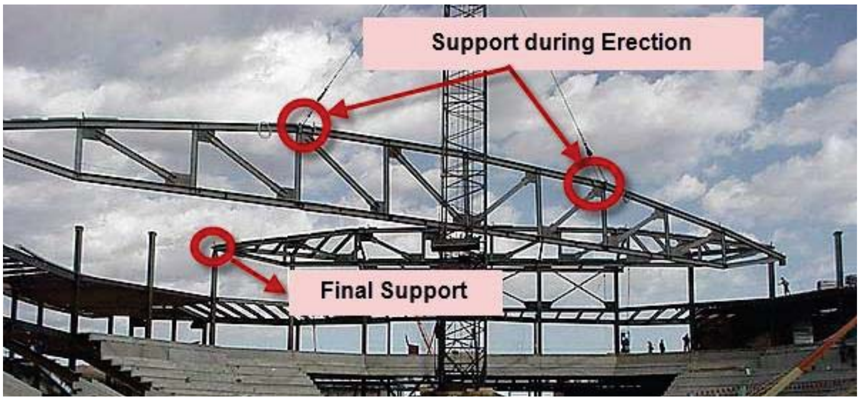


Conventional Analysis Vs. Construction Stage Analysis



Where CS Analysis is Required

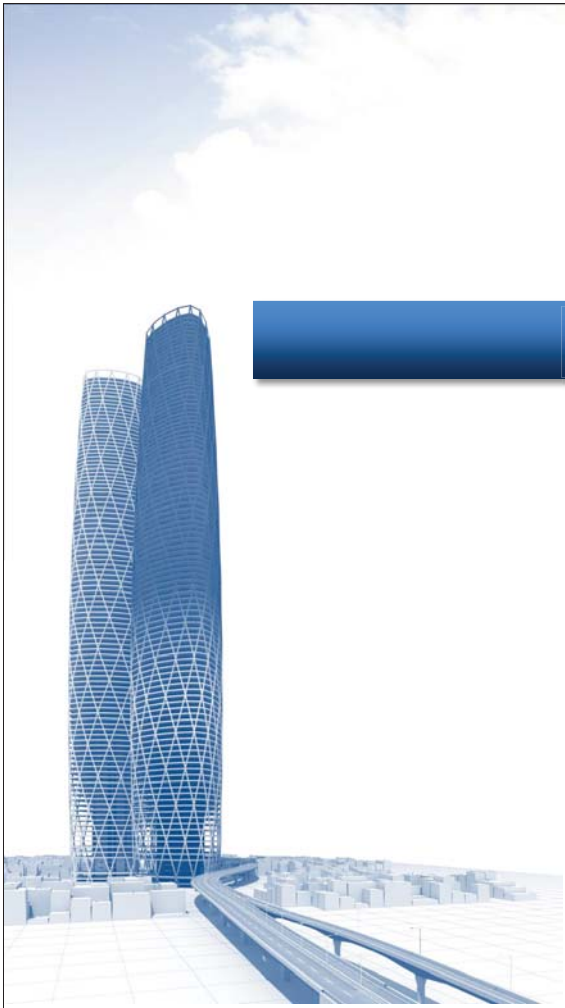
Long Span Trusses



Long Span Slabs, Beams constructed in multiple stages

Prestressed concrete Structures

CS analysis should be performed for all structures where there is a change in Support Conditions, Loading and varying material properties (Concrete).



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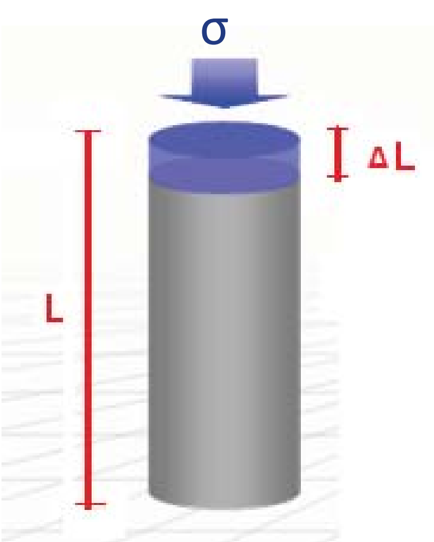
Construction Stage Analysis
Column Shortening & Related Issues
Effects of Column Shortening
Procedure for Accounting
Compensation at Site
Lotte World Tower Case Study
Q&A

Column Shortening and Related Issue

When any member is loaded with Axial Load, it undergoes axial deformation

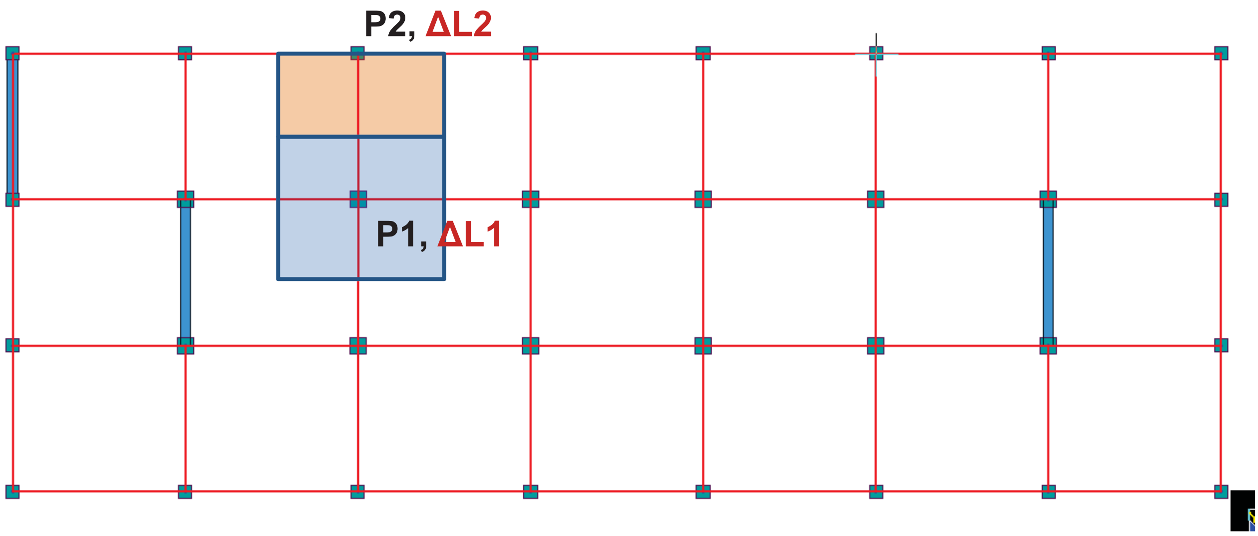


Why is this Important

$E = (\sigma / \varepsilon)$

$\Delta L = (PL/A\,E)$

Column Shortening and Related Issue



The differential shortening happening between the vertical members may cause additional forces and stress in Beams and Slabs

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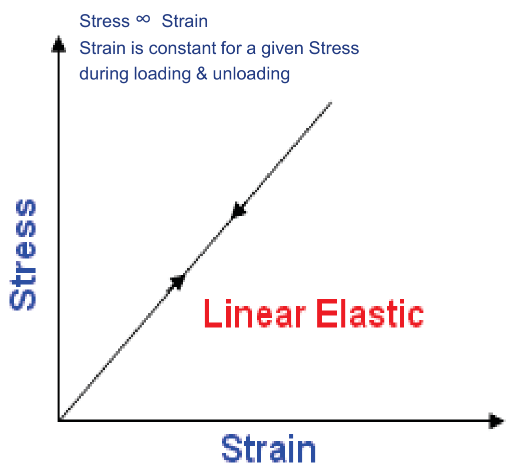
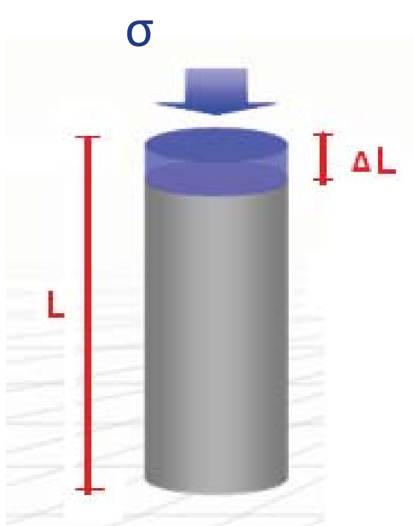
Column Shortening
One Stop Solution for Building and General Structures

Column Shortening and Related Issue

Steel Structures

- Linear elastic Behavior





$E = (\sigma / \varepsilon)$

$\Delta L = (PL / A\,E)$

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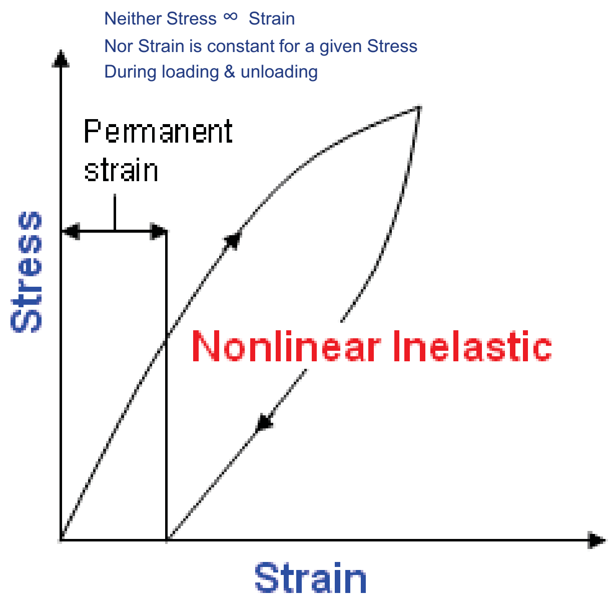
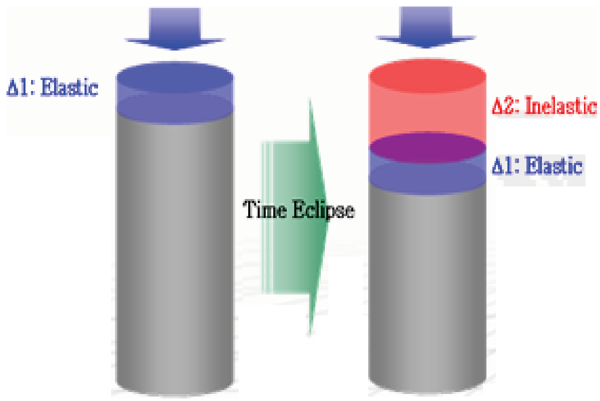
Column Shortening and Related Issue

Concrete Structures

- Nonlinear Inelastic Behavior
- But in general analysis and design behavior of concrete is treated as Linear Elastic Material



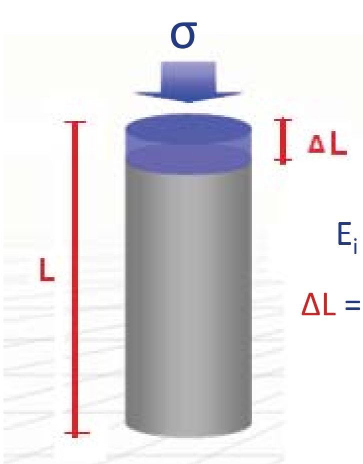
Elastic Strain + Inelastic Strain



Column Shortening and Related Issues

Concrete Structures

Elastic Shortening



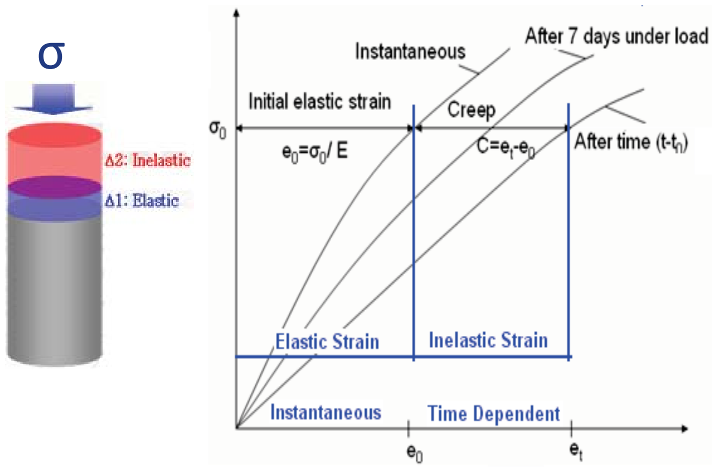
$E_i = (\sigma / \varepsilon)$

$\Delta L = (PL/A\, E_i)$

Modulus of Elasticity changes with time.

Inelastic Shortening

Creep Shortening.
Shrinkage Shortening.



Column Shortening and Related Issue

With increased height of structures the effect of column shortening (Elastic & Inelastic) take on added significance and need special consideration in design and construction.

Elastic Shortening of 80 Storey Steel Structure ~ 180 mm to 255 mm.

Elastic Shortening of 80 Storey Concrete Structure ~ 65 mm.

Total Shortening of 80 Storey Concrete Structure ~ 180 to 230 mm.

Inelastic Shortening ~ 1 to 3 times Elastic shortening.

Column Shortening and Related Issue

- Two basic prerequisites for accurately and efficiently predicting these effects are
  - ✓ Reliable Data for the creep and shrinkage characteristics of the particular concrete mix
  - ✓ Analytical procedures for the inclusion of these time effects in the design of structure.
- Some of the popular predictive methods for predicting creep and shrinkage strains are
  - ✓ ACI 209 -92
  - ✓ Bazant – Bewaja B3
  - ✓ CEB – FIP (1978, 1990)
  - ✓ PCA Method (Mark Fintel, S.K.Ghosh & Hal Iyengar)
  - ✓ GL 2000 (Gardner and Lockman)
  - ✓ Eurocode

Column Shortening and Related Issue

❖ The total strain at any time t may be expressed as the sum of the instantaneous, creep and shrinkage components:

$$\varepsilon(t) = \varepsilon_e(t) + \varepsilon_c(t) + \varepsilon_{sh}(t)$$

Where,

ε_e (t) = Instantaneous strain at time t,

ε_c (t) = Creep strain at time t,

ε_{sh} (t) = Shrinkage strain at time t.

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Column Shortening and Related Issue

❖ The instantaneous strain in concrete at any time t is expressed by

$$\varepsilon_e(t) = \frac{\sigma(t)}{E_c(t)}$$

Where,

σ (t) = stress at time t,

E_c(t) = Elastic modulus of concrete at time t, given by

$$E_c(t) = 5000\sqrt{f_{ct}}$$

f_{ct} = Compressive strength at any time t, given by

$$f_{ct} = \frac{t}{\alpha + \beta(t)} f_{c,28}$$

α & β are constants depending on Type of Cement & Type of Curing

Column Shortening and Related Issue

Inelastic Shortening = Creep + Shrinkage

Creep

Creep is time-dependent increment of strain under sustained stress.

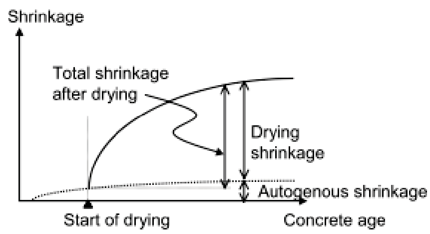
- Basic creep occurs under the condition of no moisture movement to and from the environment.
- Drying creep is the additional creep caused by drying.

Drying creep has its effect only during the initial period of load.

Shrinkage

Shrinkage is the time-dependant decrease in concrete volume compared with the original placement volume of concrete.

- Drying Shrinkage is due to moisture loss in concrete.
- Autogenous Shrinkage is caused by hydration of cement.
- Carbonation shrinkage results as the various cement hydration products are carbonated in the presence of CO



Column Shortening and Related Issue

Inelastic Shortening = Creep + Shrinkage

Creep

As per ACI 209R-92 the creep coefficients are predicted

as = $$v_t = \frac{t^{0.60}}{10 + t^{0.60}} v_u$$

Where,
t = time in days after loading.
v_u = Ultimate creep coefficient = 2.35 γ_c
γ_c = Product of applicable correction factors

Shrinkage

As per the ACI 209R-92, shrinkage can be predicted by

After 7 days for moisture cured concrete

$$(\varepsilon_{sh})_t = \frac{t}{35 + t}(\varepsilon_{sh})_u$$

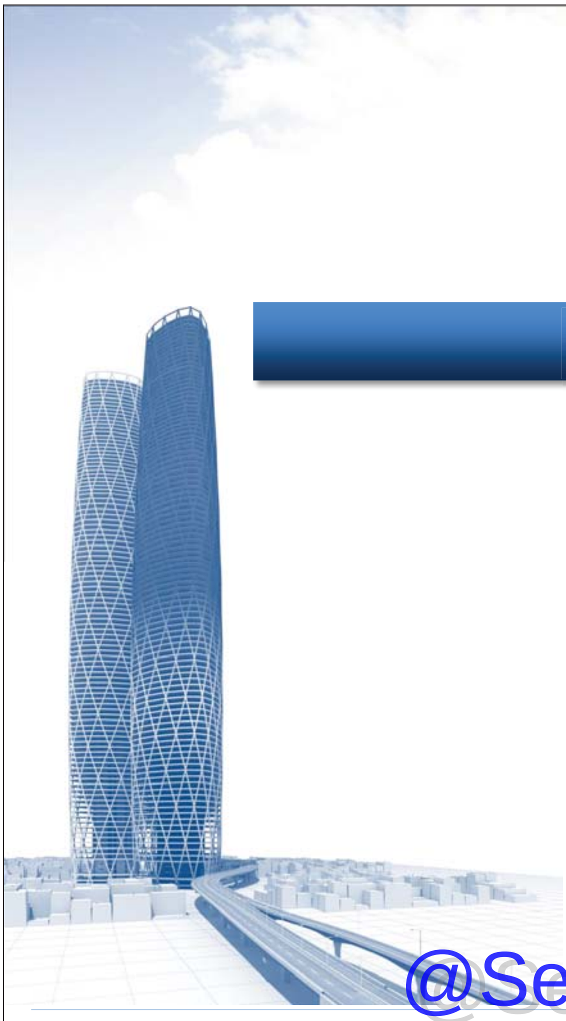
After 1-3 days for steam cured concrete

$$(\varepsilon_{sh})_t = \frac{t}{55 + t}(\varepsilon_{sh})_u$$

Where,
t = time in days after the end of Initial Curing
$(\varepsilon_{sh})_u$ = Ultimate Shrinkage Coefficient = 780 γ_{sh} x 10^{-6} m/m
γ_{sh} = Product of applicable correction factors

Column Shortening and Related Issue

Factors affecting the Creep & Shrinkage of Concrete		
Concrete (Creep & Shrinkage)	Concrete Composition	Cement Paste Content Water – Cement ratio Mixture Proportions Aggregate Characteristics Degrees of Compaction
	Initial Curing	Length of Initial Curing Curing Temperature Curing Humidity
Member Geometry and Environment (Creep & Shrinkage)	Environment	Concrete Temperature Concrete Water Content
	Geometry	Size and Shape
Loading (Creep Only)	Loading History	Concrete age at load Application During load Period Duration of unloading Period Number of load Cycles
	Stress Conditions	Type of Stress and distribution across the Section Stress/Strength Ratio



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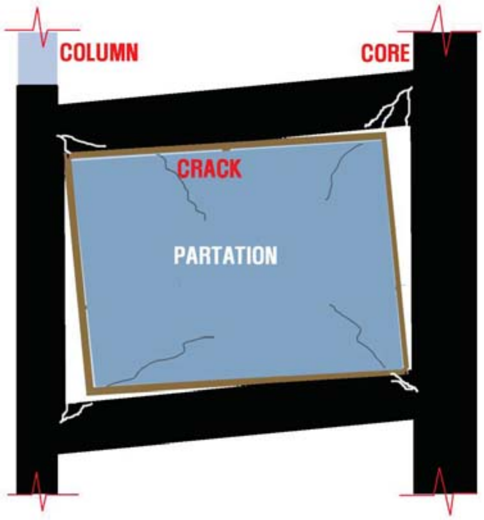
Effects of Column Shortening

❖ Absolute shortening is rarely of practical interest.

❖ Differential shortening between adjacent vertical elements is the most important factor for engineer.

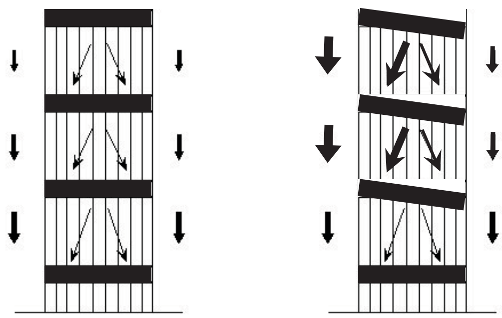
❖ Axial Shortening of vertical elements will not effect those elements very much, horizontal elements like beams and slabs and non structural elements are affected.



Effects of Column Shortening

Structural Effects

- Slabs may not be truly horizontal after some time.
- Beams could be subjected to higher bending moments.
- Load transfer.

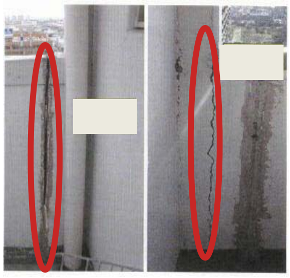


Non Structural Effects

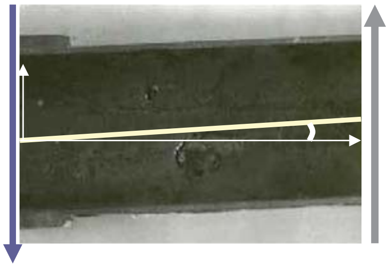
- Cracks in Partition Walls.
- Cracks in Staircases
- Deformation of Cladding.
- Mechanical Equipment.
- Architectural Finishes.
- Built in Furnishings.

These non structural elements are not intended to carry vertical loads and are therefore not subjected to shortening.

Effects of Column Shortening



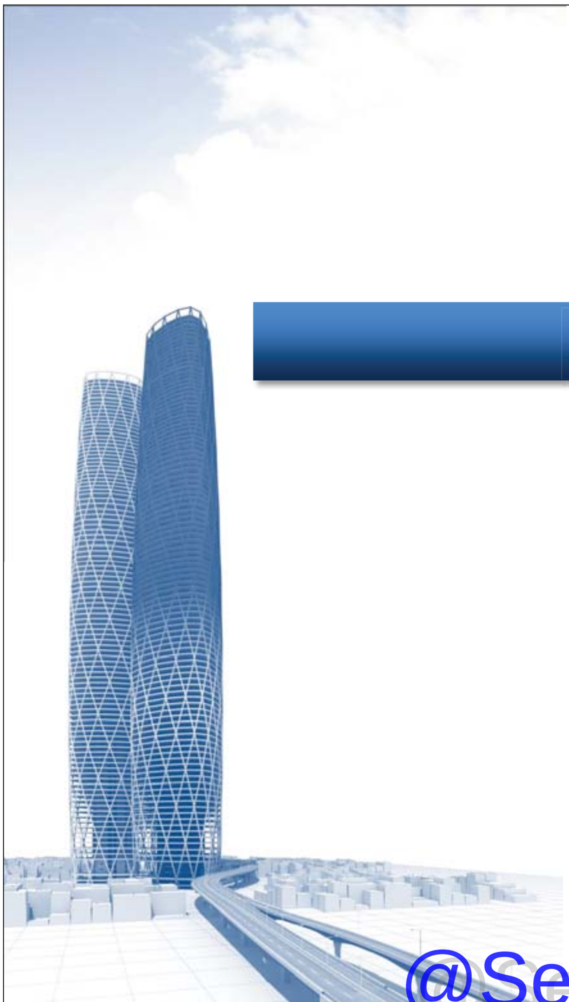
Deformation and breakage of Facades, windows & Parapet walls…



Reverse Inclination of Drainage Piping System

Deformation of Vertical Piping System

Deformation and breakage of internal partitions



MIDAS

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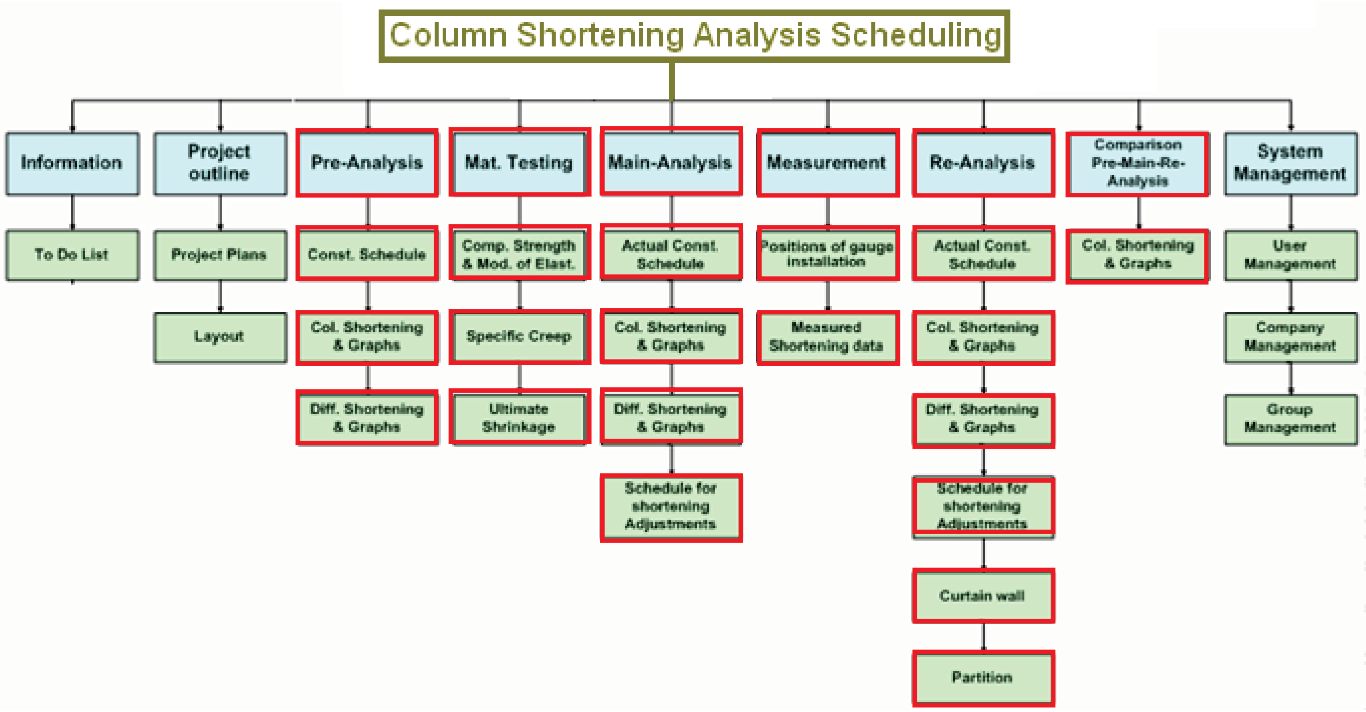
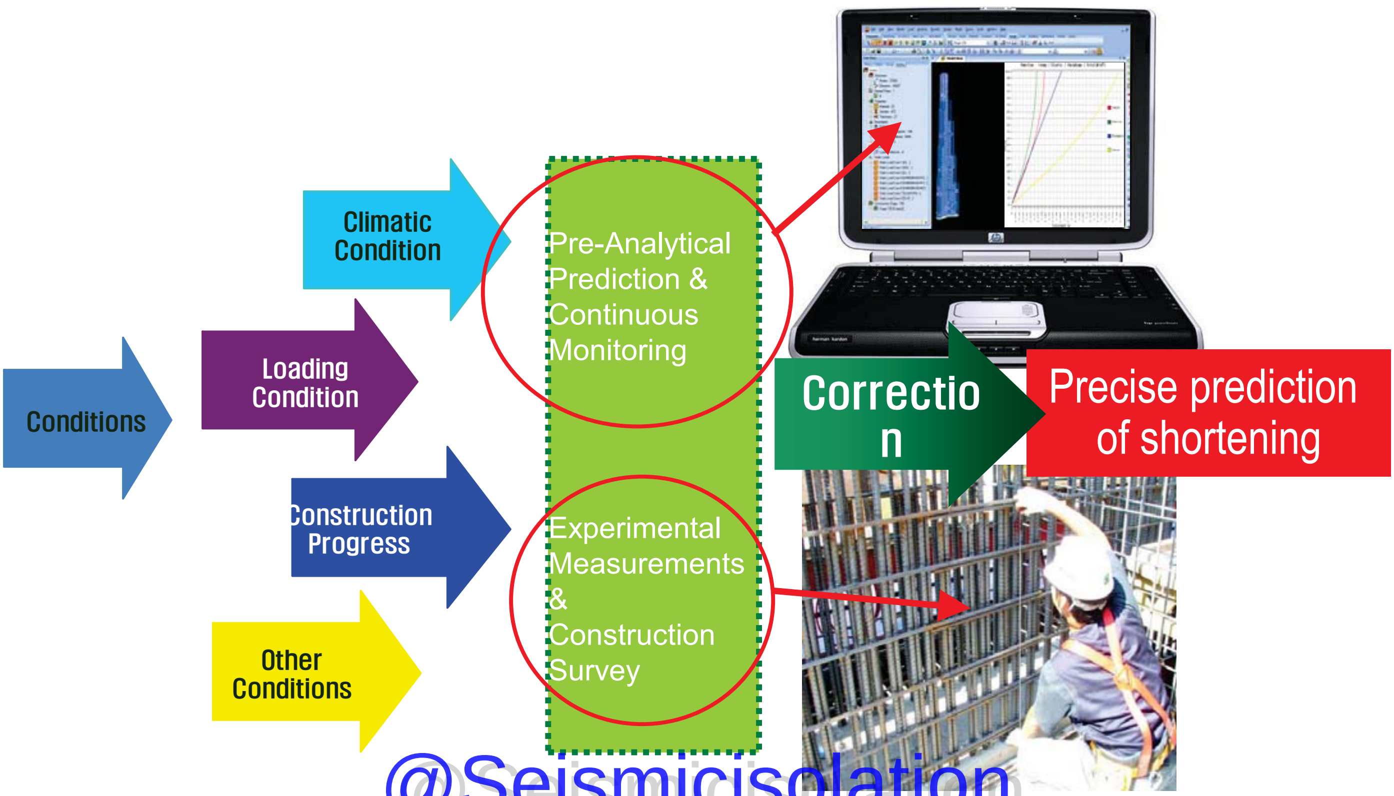
Procedure for Accounting Column Shortening



Procedure for Accounting Column Shortening



Procedure for Accounting Column Shortening

Analytical Measurement

Using Software or Manually
(Manual calculation is almost impossible)

Reflection of physical properties in calculation from material experiment:
Young's Modulus, Poisson's Ratio, Mean Compressive strength, Volume to Surface ratio, Shapes, sizes etc.

Reflection of effects of Climate on shortening:
Average Temperature , RH etc.

Construction Sequence:
Stage duration, Additional Steps, Member Age, Load activation age, Boundary activation age etc.

Reflection of the above effects on site master-schedule.

Method has Limitation

Feed Back

Experimental Measurement

Field Measurements

Installation of sensors or gages in members for determining the actual shortening.

Understanding and noting the following:
Curing procedure / Temperature,
Actual Shortening,
Change in Ambient Temperature (Important),
Actual Humidity,
Deviation from Defined Construction Stages,

Manipulation of factors in analytical Calculation, Re-Analysis…

Procedure for Accounting Column Shortening

Field Measurements

Determination of Installation location

Installation of Gauge

After Installation

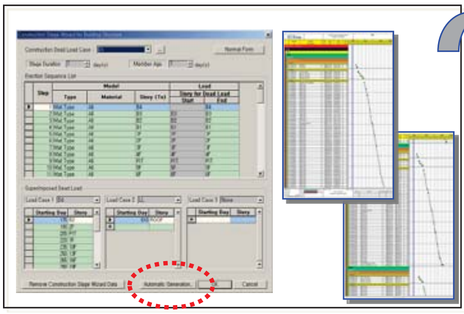
After Installation of Gauge

After Casting of Concrete

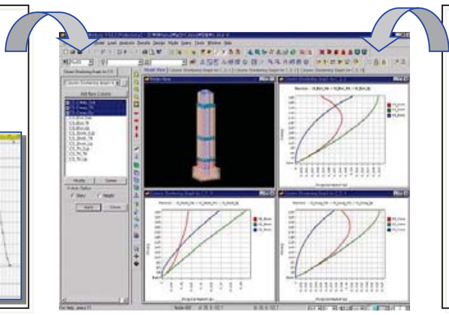
Field data collection

Procedure for Accounting Column Shortening

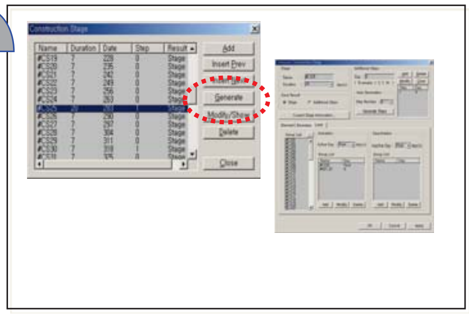
Engineering Re-Analysis



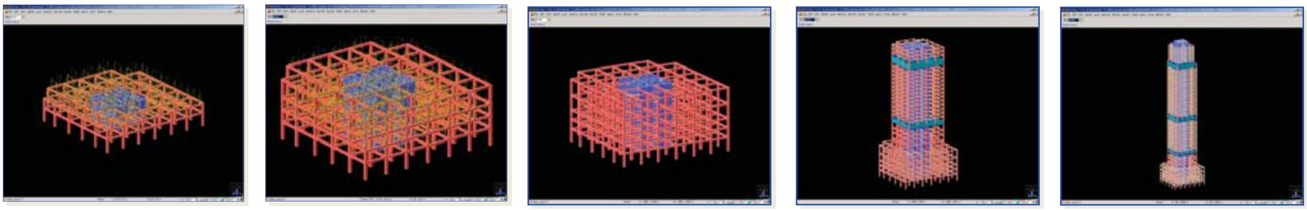
Manipulation of the construction stage as per site condition



Perform 3D CS Analysis



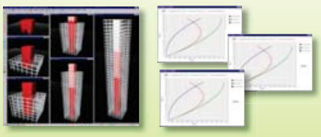
Construction stage of each group



Activation of Construction stages

Procedure for Accounting Column Shortening

Pre-Analysis



Preliminary Analysis

Material / Section Properties
Applied Load, Schedule

Design with Additional Force

Main Analysis, Construction & Re-AnalysiS

Material Experiment

- Compressive strength
- Modulus of elasticity
- Creep & Shrinkage

Main analysis

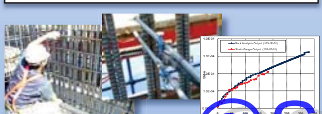
- Updating material properties from experiments
- Construction sequence considering the field condition

1st, 2nd, 3rd Re-Analysis

Suggestion of compensation and details for non-constructed part of structure

Measurement

Measurement of strain for Column & Wall



Final Report

Shortening, result from test, measurement Review

Applying Compensation to in-situ structure

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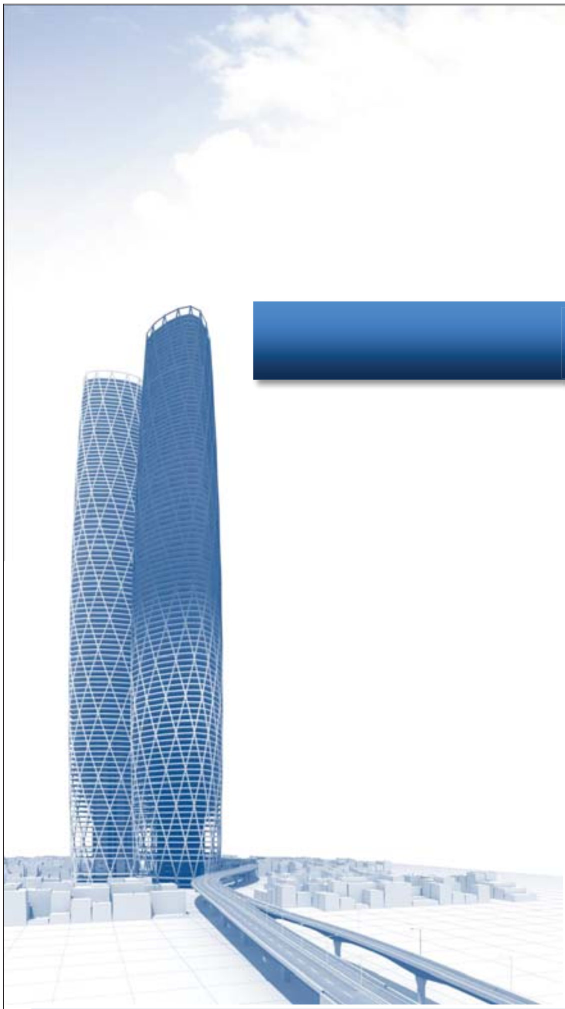

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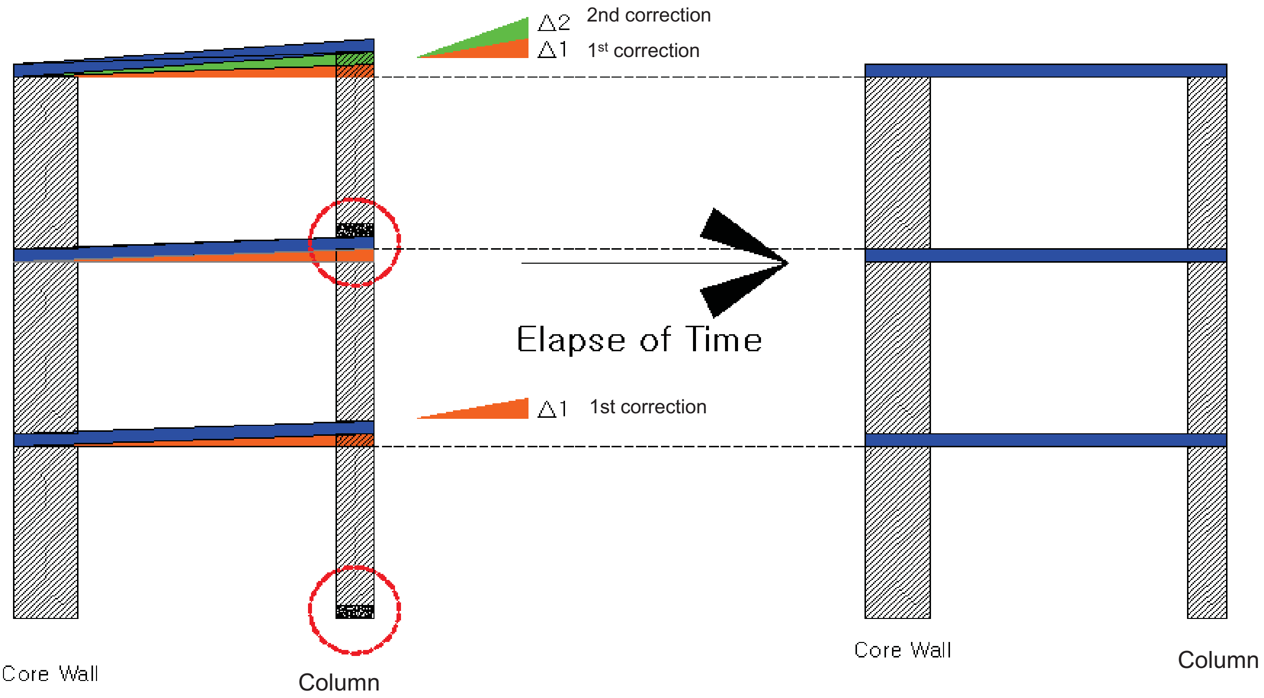
Compensation at Site



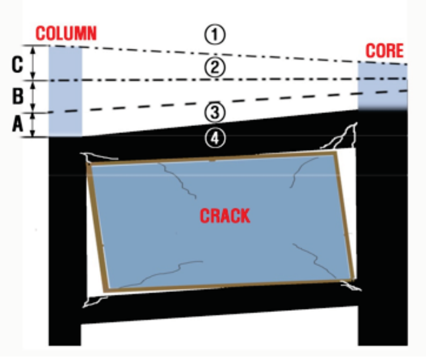
Movement Related to Construction Sequence

Depending on the stage of construction:

- **Pre-slab installation shortenings**
  - Shortenings taking place up to the time of slab installation
- **Post-slab installation shortenings**
  - Shortenings taking place after the time of slab installation

Depending on the construction material:

- **Reinforced Concrete Structure**
  - Pre-slab installation shortenings has no importance
  - Compensation by leveling the forms
  - Post-slab installation shortenings due to subsequent loads and creep/shrinkage
- **Steel Structure**
  - Columns are fabricated to exact length.
  - Attachments to support the slabs
  - Pre-slab installation shortenings need to be known.
  - Compensation for the summation of Pre-installation and Post-installation shortenings



① :Compensation

② : Design Level

③: Pre-slab Installation shortening

④: Post-slab Installation shortening

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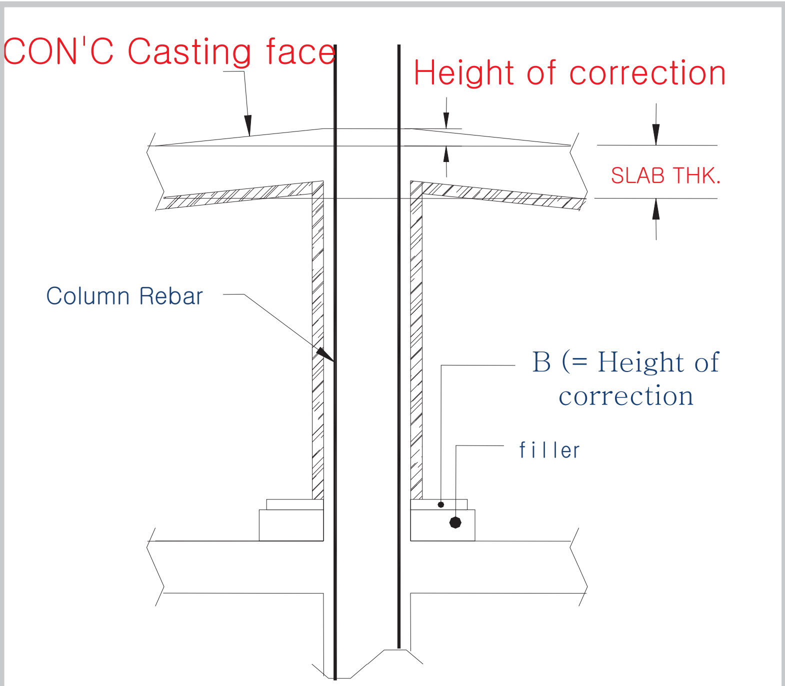
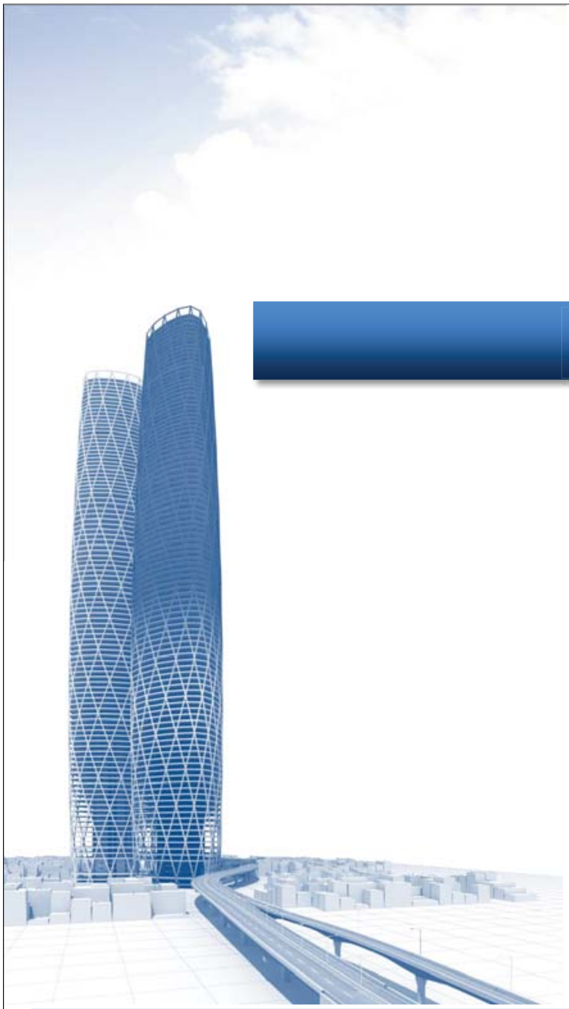
Compensation at Site



❑ The order of construction

installation of column forms
↓
insertion of FILLER
↓
insertion of correcting FILLER
↓
Installation of SLAB forms
↓
securing the thickness of slab.



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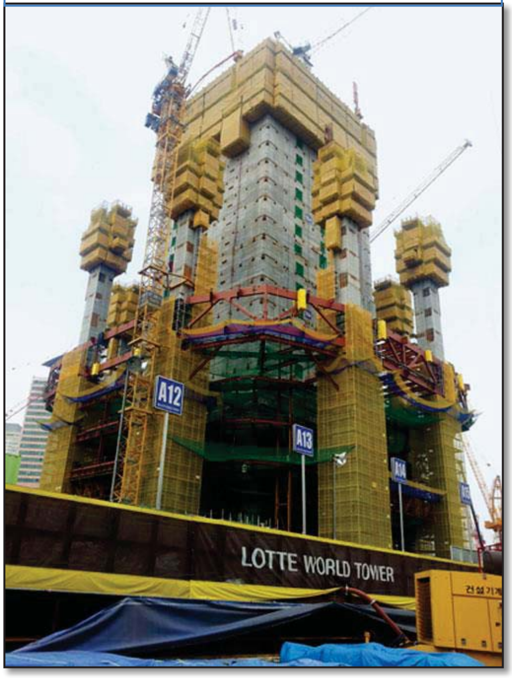
Overview

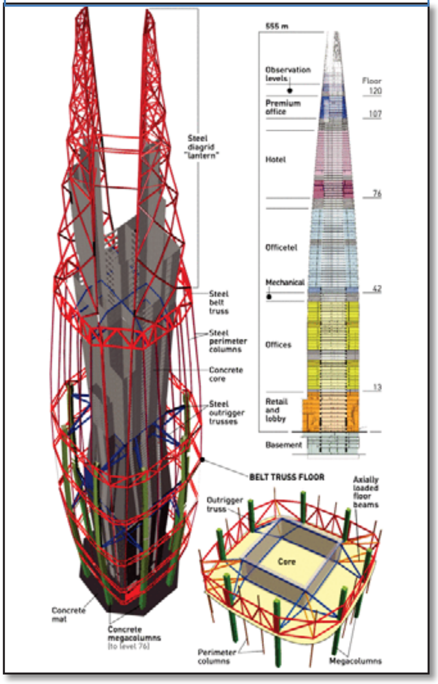
Lotte World Tower

Location	Jamsil, Seoul, South Korea.
Height	**Roof** – 554.6 m; **Antenna Spire** – 556 m
No. of Floors	123
Floor Area	304,081 m^2
Function / Usage	Office, Residential, Hotel, Observation Deck (497.6 m)
Structure Type	Reinforced Concrete + Steel
Lateral load resisting system	Core Wall + Outrigger Truss + Belt Truss
Foundation Type	Mat Foundation
Construction Period	March 2011 ~ 2015

Construction Site

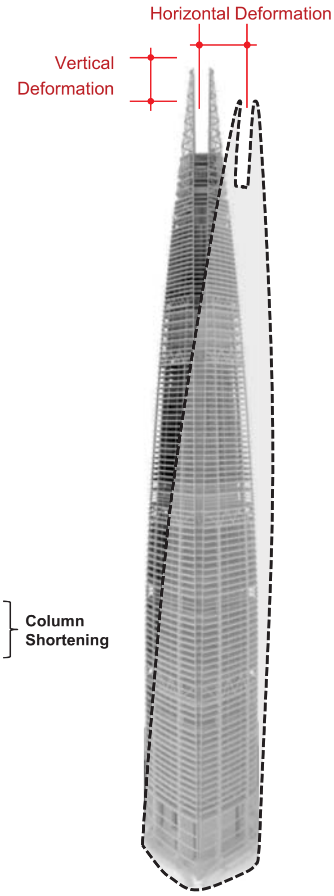


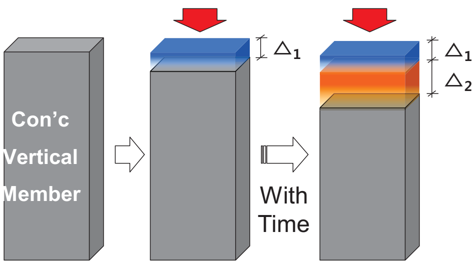
Structural Schematic



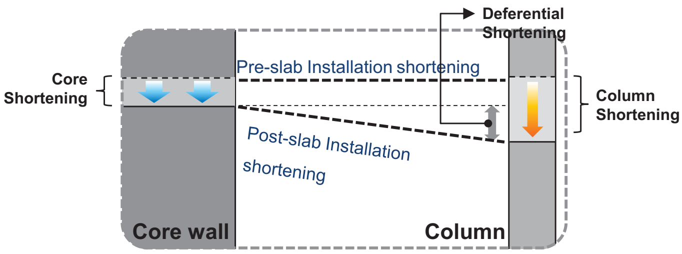
Overview

General	• Height 555m / 123 floors
Tower Deformation	• Deformation of the tower is a naturally occurring depending on material, construction method • Vertical Deformation: Vertical Shortening / Settlement / Construction Errors • Horizontal Deformation: Differential Shortening / Settlement Uneven load due to construction method Asymmetric floor plan / Construction errors





Initial Curing <Elastic> <Inelastic>

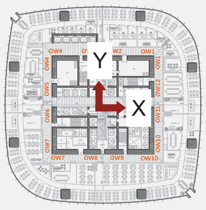


< Deferential Deformation >

Pre-Analysis - Deformations

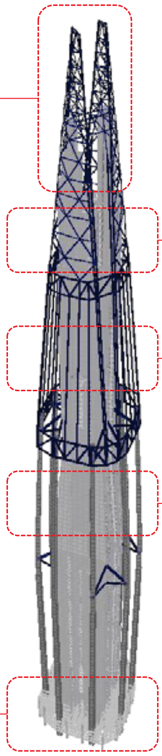
Lotte World Tower

Horizontal deformation

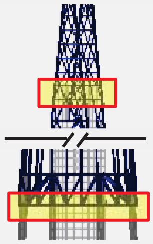


- **Prediction**
  - X dir: 27.2mm
  - Y dir: 115.5mm
- Safety check
  - → Elevator's rails
  - → Vertical Pipes

Lantern & Core

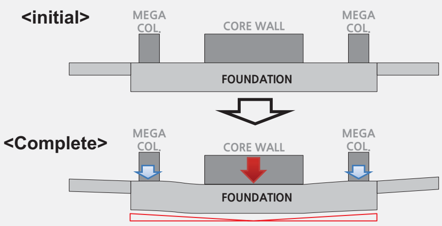


Vertical deformation



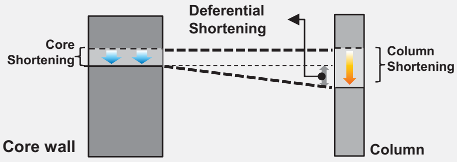
- Top of tower
  - ⇒ Steel Frame: 368.7 mm
  - ⇒ Core wall: 314.0 mm
- Top of mega column
  - ⇒ Mega Col: 297.8 mm
  - ⇒ Core wall: 232.8 mm

Differential settlement



⇒ Core wall settlement: 35mm
⇒ Column settlement: 16mm

Differential Shortening

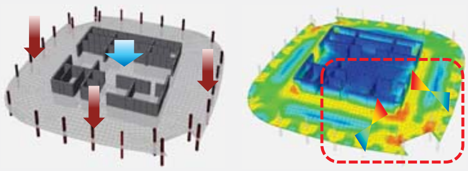


- Deferential shortening btw Core & Column
  - ⇒ Steel column: Max 55mm
  - ⇒ Mega column: Max 65mm

Pre-Analysis - Deformations

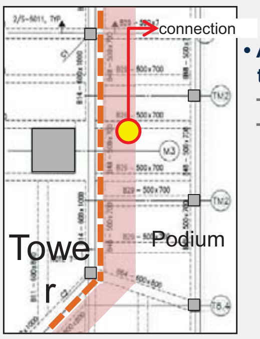
Lotte World Tower

Slab's additional stress

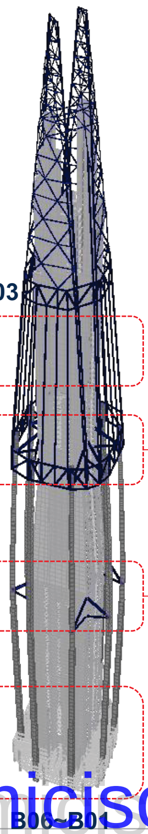


Differential Deformation btw Slab-Column
→ Slab has additional stress

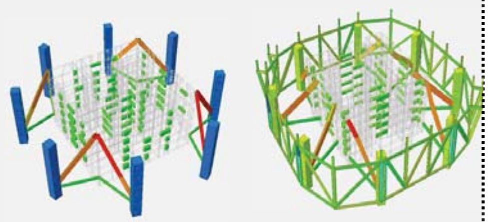
Podium's additional stress



- Additional stress btw tower & podium
  - → Max 100 ton.m
  - → Require Settlement Joint & Safety check



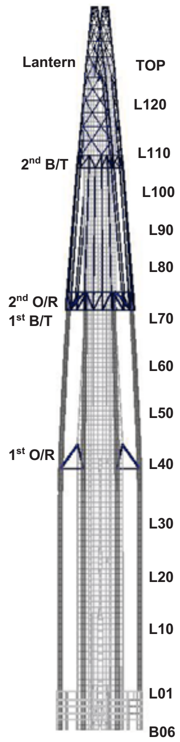
Outrigger's additional stress



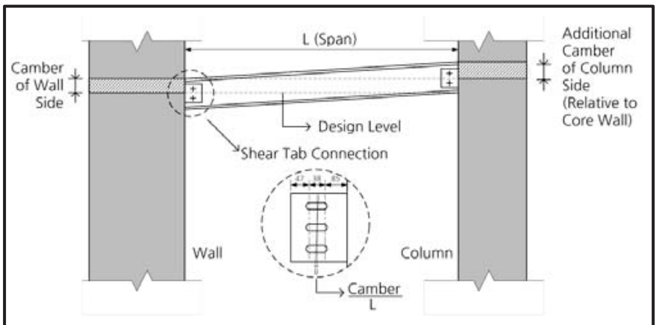
- Additional Stress without Delay Joint
  - ⇒ 1st outrigger (L39~L43): 3,600 tons
  - ⇒ 2nd outrigger (L72~L75): 4,700 tons
  - ⇒ required a delay joint installation
- Additional Stress with Delay Joint
  - ⇒ 1st outrigger (L39~L43): 1,700 tons
  - ⇒ 2nd outrigger (L72~L75): 2,000 tons

Pre-Analysis – Compensation

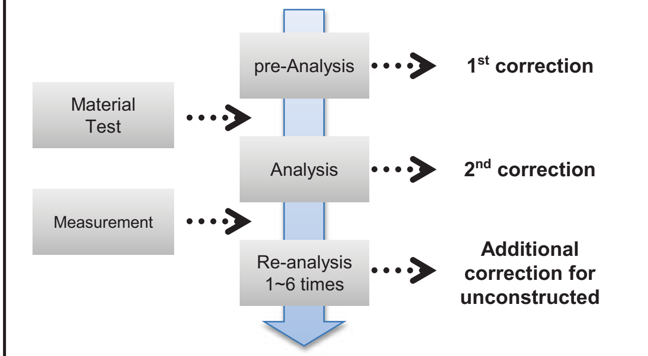
- **Core Wall:** Absolute correction for securing design level
- **Column:** Relative correction for deferential shortening



Floor	Core	Column
L106-L123	Design level+1mm	Steel columns
L76-L105	Design level+2mm	Steel columns
L72-L75	Design level+2mm	Core level+25mm
L69-L71	Design level+2mm	Core level+30mm
L66-L68	Design level+2mm	Core level+35mm
L63-L65	Design level+2mm	Core level+40mm
L60-L62	Design level+2mm	Core level+45mm
L57-L59	Design level+2mm	Core level+50mm
L37-L56	Design level+3mm	Core level+55mm
L54-L56	Design level+3mm	Core level+60mm
L34-L36	Design level+3mm	Core level+55mm
L31-L33	Design level+3mm	Core level+50mm
L28-L30	Design level+3mm	Core level+50mm
L25-L27	Design level+3mm	Core level+45mm
L22-L24	Design level+3mm	Core level+40mm
L19-L21	Design level+3mm	Core level+35mm
L16-L18	Design level+3mm	Core level+30mm
L13-L15	Design level+3mm	Core level+25mm
L10-L12	Design level+3mm	Core level+20mm
L7-L9	Design level+3mm	Core level+15mm
L4-L6	Design level+3mm	Core level+10mm
B6-L3	Design level+3mm	Core level+5mm



▲ Relative correction between core and column

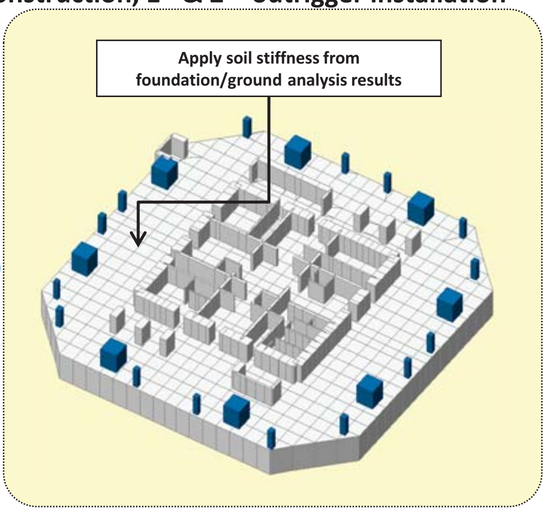


▲ correction due to measurement

Main Analysis & Re-Analysis

Analysis Condition and Assumption

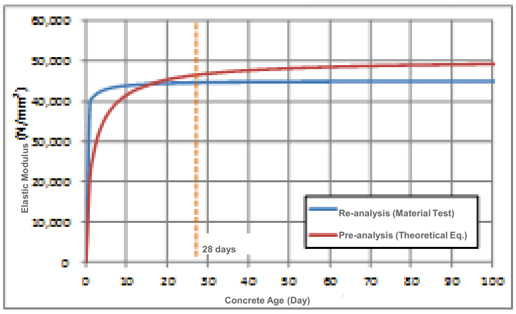
- **Analysis Tool: midas/GEN**
  - 3D Structural Analysis with changes of material properties
- **Material properties**
  - Regression analysis results from the material test data (6 month)
  - Comparing to pre-analysis results, 32~33% in creep deformation, 39~42% in shrinkage deformation
- **Outrigger Installation Condition: After completion of frame construction, 1st & 2nd outrigger installation**
- **Loading Condition**
  - Dead Load & 2nd Dead Load: 100%, Live Load: 50%
- **Environment: Average relative humidity 61.4%**
  - Relative humidity of average 5 years
- **Target period of shortening**
  - Safety verification: 100years after (≒ultimate shortening)
  - Service verification: 3years after (95% of ultimate shortening)



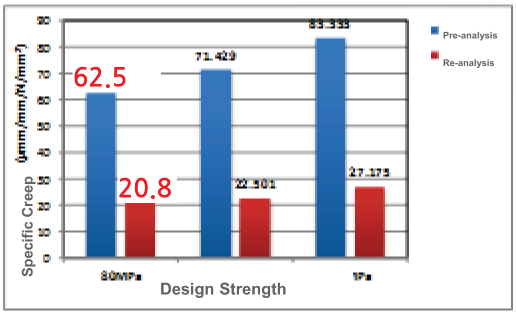
Lotte World Tower

Material Test Results

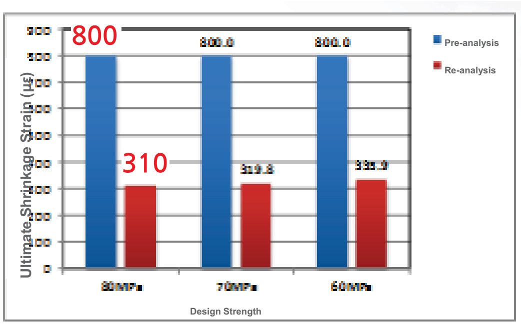
Material test results for re-analysis



<Comparison Graph of Elastic Modulus>



<Comparison Graph of Specific Creep>

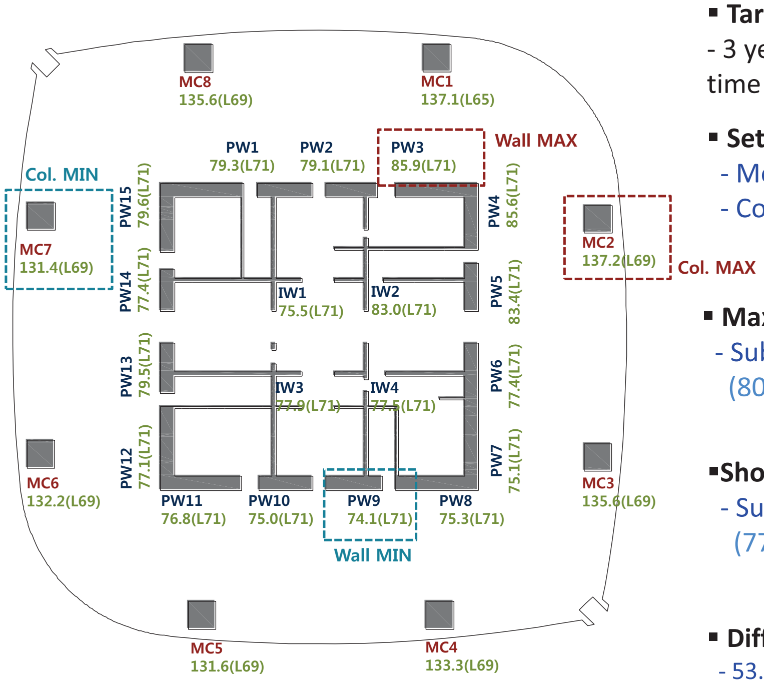


<Comparison Graph of Shrinkage>

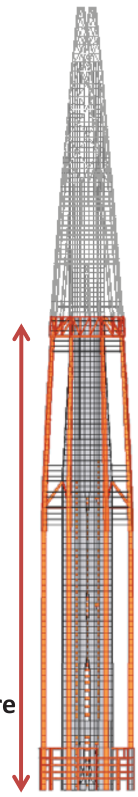
Lotte World Tower

Re-analysis Results

Shortening Results- 1-1. Mega Column Shortening (B06~L75)



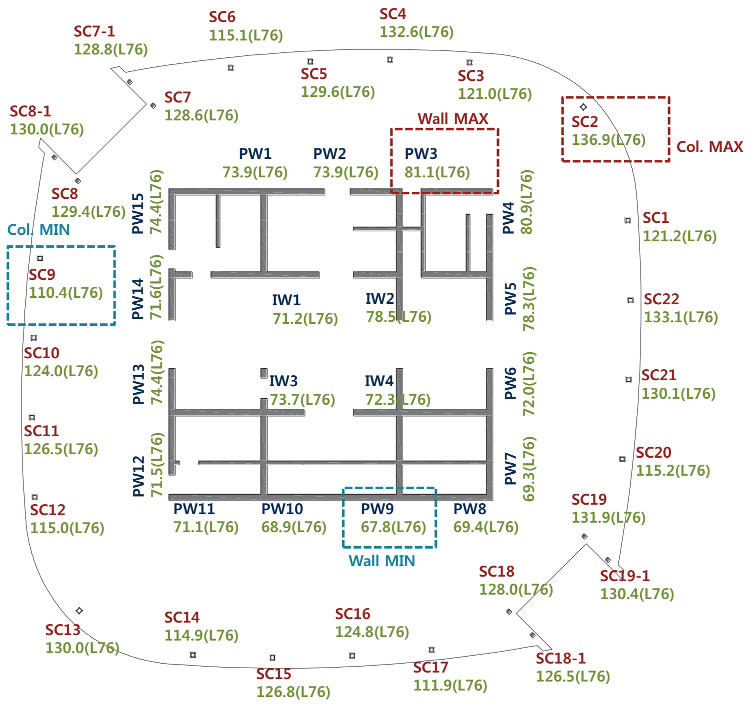
- **Target Period: 3years**
  - 3 years was determined as the optimal time of target serviceability application.
- **Settlement Shortening**
  - Mega column: 21.2~25.5mm (B6)
  - Core wall: 23.6~29.1mm (B6)
- **Maximum shortening of mega column**
  - SubTo: 131.4~137.2mm (L65, L69) (80~83% of pre-analysis)
- **Shortening of core walls**
  - SubTo: 74.1~85.9mm (L71) (77~78% of pre-analysis)
- **Differential shortening between column-core**
  - 53.1~60.9mm (L65)



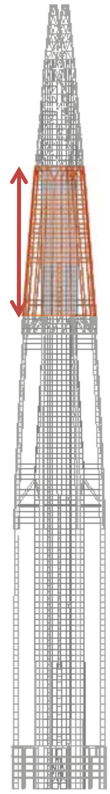
Lotte World Tower

Re-analysis Results

Shortening Results- 1-2. Steel Column Shortening(L76~L106)



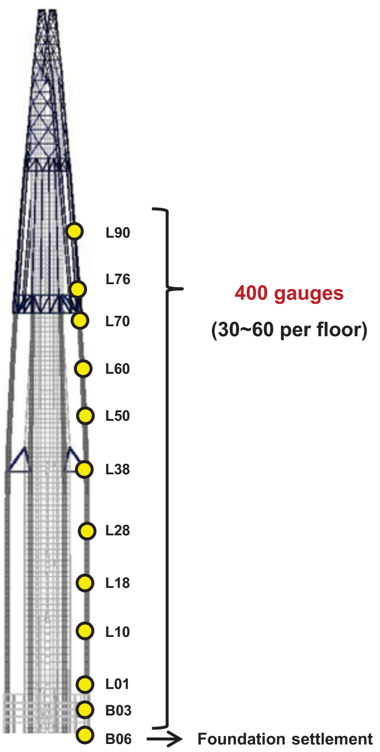
- **Target Period: 3years**
  - 3 years was determined as the optimal time of target serviceability application.
- **Maximum shortening of steel column**
  - SubTo: 110.4~136.9mm (L76) (80% of pre-analysis)
  - Total: 260.7~286.1mm (L76) (80% of pre-analysis)
- **Shortening of core walls**
  - SubTo: 67.8~81.0mm (L76) (65~70% of pre-analysis)
  - Total: 162.9~213.6mm (L76) (67~70% of pre-analysis)
- **Differential shortening between Column-core**
  - 40.1~44.5mm (L76)

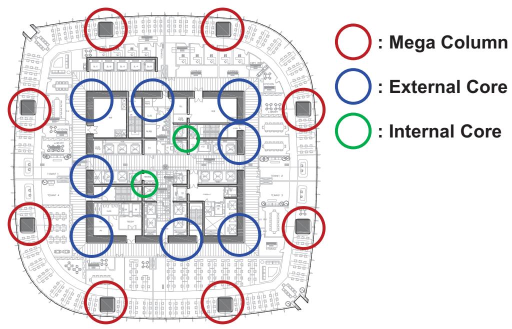


MIDAS

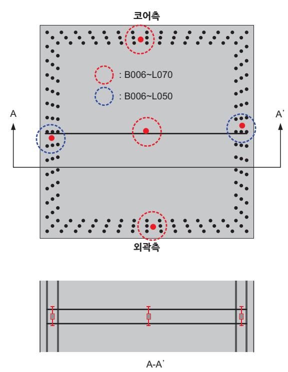
Lotte World Tower

Vertical Shortening Measurement

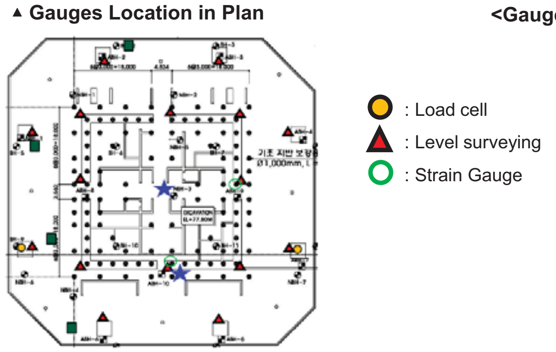




▲ Gauges Location in Plan



<Gauge location in Mega Columns>



▲ Gauges Location of settlement

MIDAS

Lotte World Tower

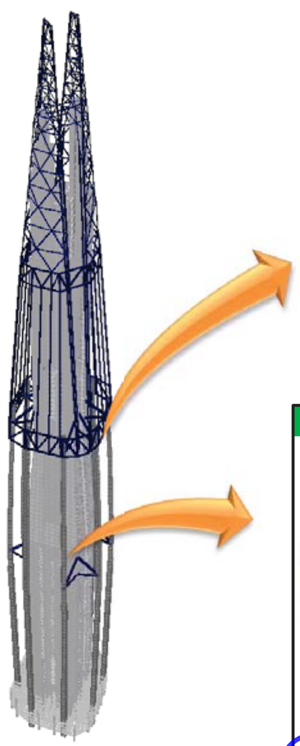
Re-analysis Results

Compensation due to core and column differential shortening

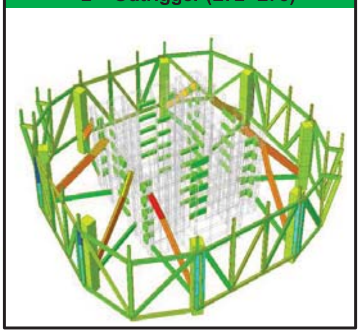
Floor	Core	Column
L120-L123	Design level+2mm	Design level+25mm
L113-119	Design level+3mm	Design level+30mm
L107-112	Design level+3mm	Design level+35mm
L103-106	Design level+4mm	Design level+45mm
L100-102	Design level+4mm	Design level+50mm
…	…	…
L30-32	Design level+4mm	Design level+80mm
L28-29	Design level+5mm	Design level+75mm
L23-27	Design level+4mm	Design level+70mm
L22-22	Design level+3mm	Design level+65mm
L19-21	Design level+3mm	Design level+60mm
L18-18	Design level+3mm	Design level+55mm
L14-17	Design level+3mm	Design level+50mm
L13-13	Design level+3mm	Design level+45mm
L10-12	Design level+2mm	Design level+40mm
L8-9	Design level+2mm	Design level+35mm
L6-7	Design level+2mm	Design level+25mm
L5-5	Design level+2mm	Design level+20mm
LB6-4	-	-

Lotte World Tower

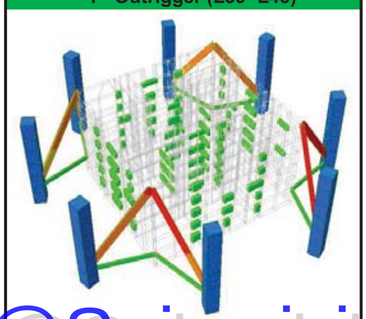
Outrigger Structural Safety issues and alternatives proposed



▲ 2nd Outrigger (L72~L75)



▲ 1st Outrigger (L39~L43)



Effect & Safety Measure

- **Additional stress due to differential shortening between core & column**
- **Provide outrigger delay joint**

① Steel Outrigger Delay Joint

② Steel Outrigger Adjustment Joint

(Securing safety under construction)

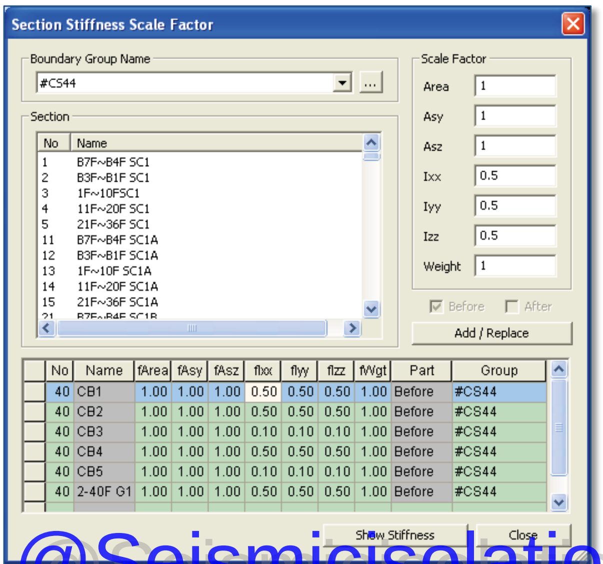
MIDAS

Midas Gen – One Stop Solution for Building and General Structures

Contents

Material Stiffness Changes for Cracked Sections

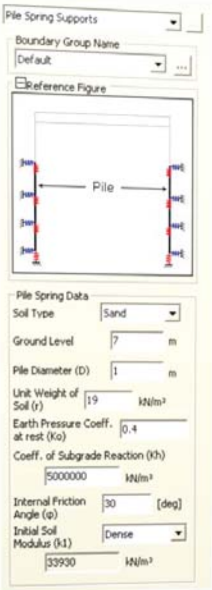
- Specific stiffness of specific member types may be reduced such as the case where the flexural stiffness of lintel beams and walls may require reduction to reflect cracked sections of concrete.
- Section stiffness scale factors can be included in boundary groups for construction stage analysis. The scale factors are also applied to composite sections for construction stages.



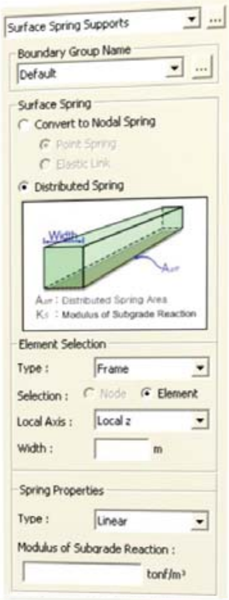
No	Name	fArea	fAsy	fAsz	fIxx	fIyy	fIzz	fWgt	Part	Group
40	CB1	1.00	1.00	1.00	0.50	0.50	0.50	1.00	Before	#CS44
40	CB2	1.00	1.00	1.00	0.50	0.50	0.50	1.00	Before	#CS44
40	CB3	1.00	1.00	1.00	0.10	0.10	0.10	1.00	Before	#CS44
40	CB4	1.00	1.00	1.00	0.50	0.50	0.50	1.00	Before	#CS44
40	CB5	1.00	1.00	1.00	0.10	0.10	0.10	1.00	Before	#CS44
40	2-40F G1	1.00	1.00	1.00	0.50	0.50	0.50	1.00	Before	#CS44

Spring Supports for Soil Interaction

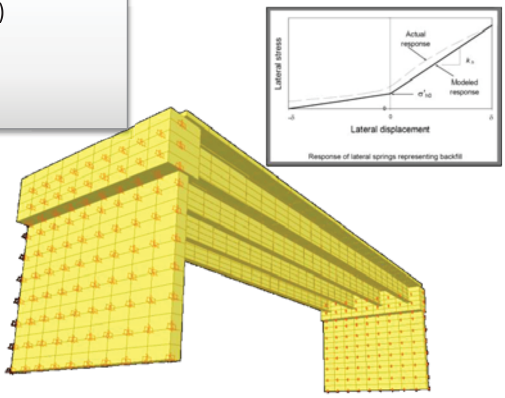
- Point Spring Support (Linear, Comp.-only, Tens.-only, and Multi-linear type)
- Surface Spring Support (Nodal Spring, and Distributed Spring)
- Springs can be activated / deactivated during construction stage analysis.



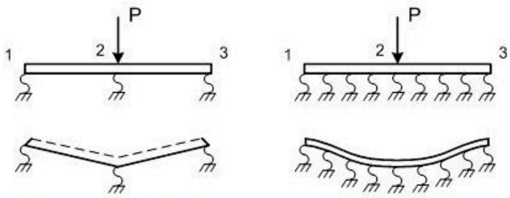
[Pile Spring Support]



[Surface Spring Support]



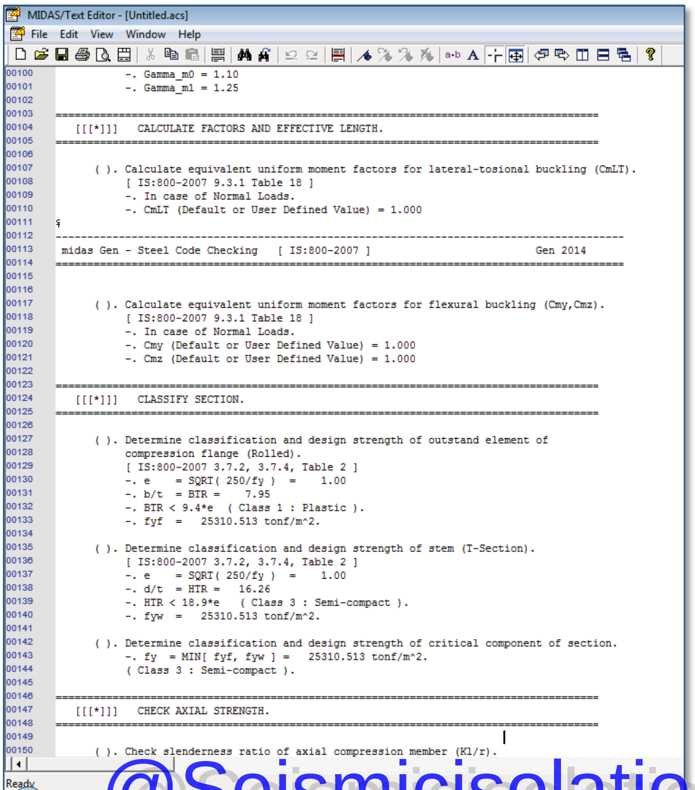
[Nonlinear point spring support]

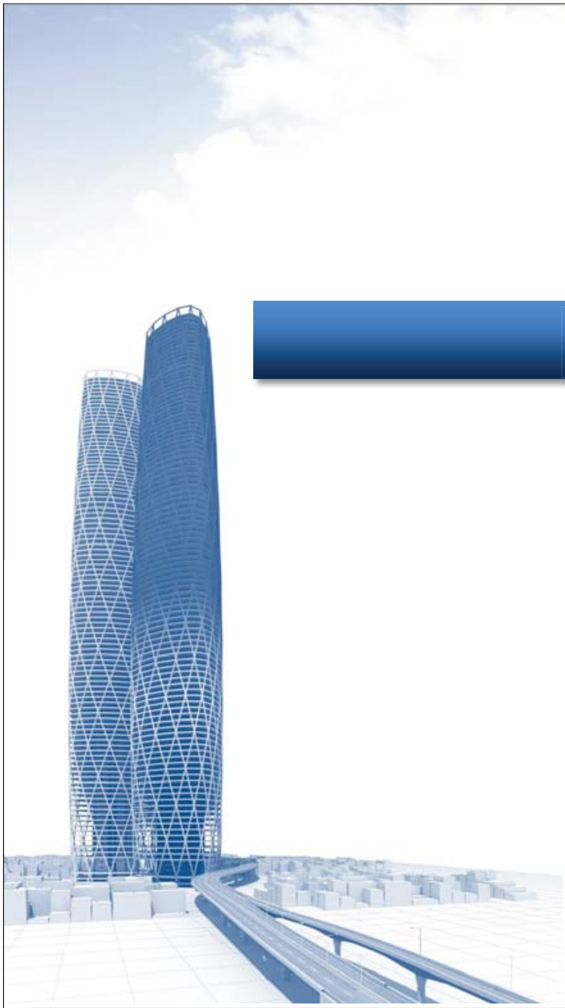


(a) Convert to Nodal Spring (b) Distributed Spring

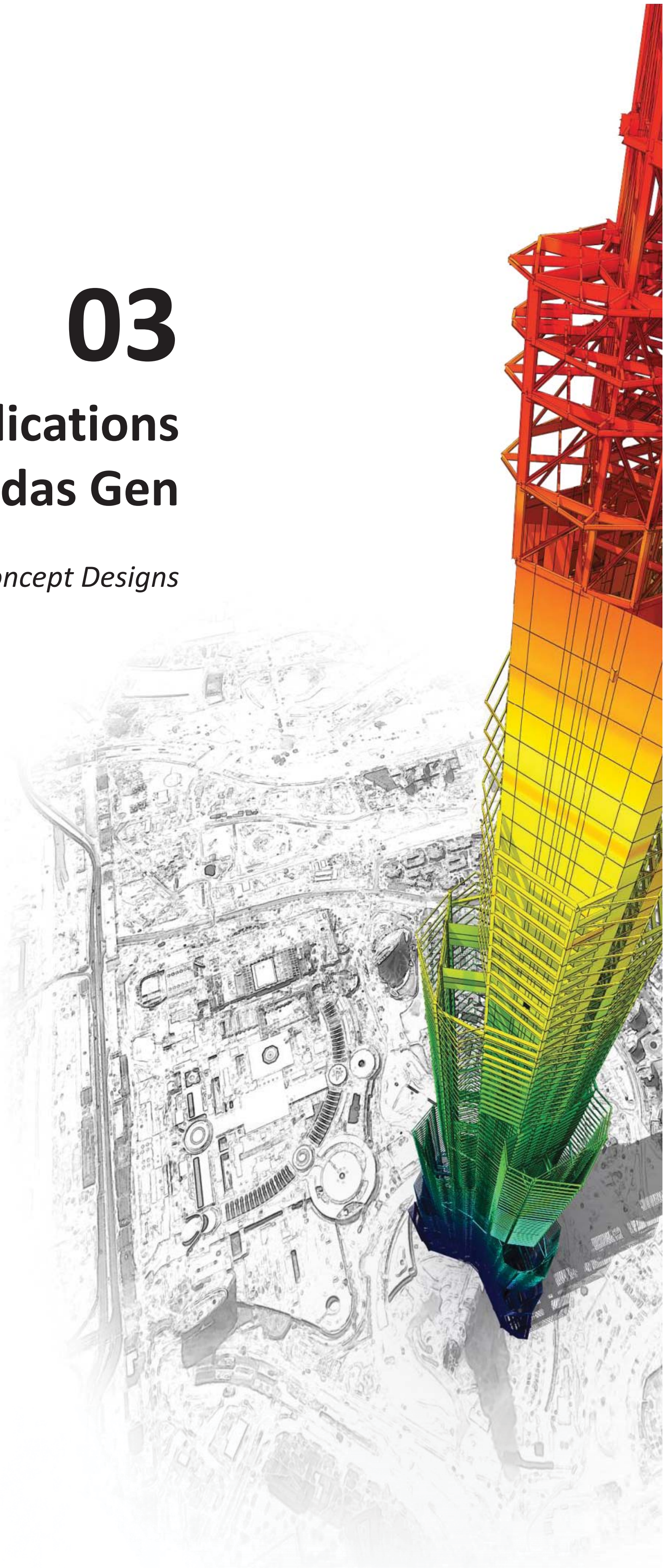
[Nodal Spring and Distributed Spring]

Detailed Design Reports





Q & A



03

Project Applications using midas Gen

Raajesh Ladhad, Structural Concept Designs

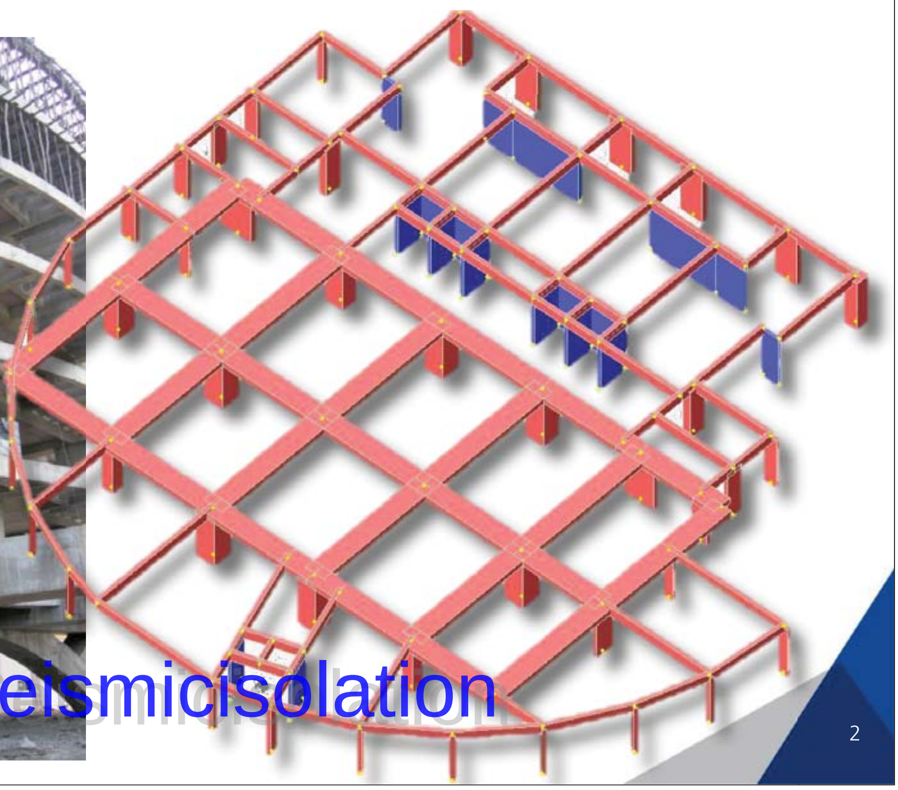
Project Applications using midas Gen

- Challenges & Solutions of Tall Building Design -

Raajesh K. Ladhad
Structural Concept Designs Pvt., Ltd.

Elerro Fiesta, Sanpada, 2012

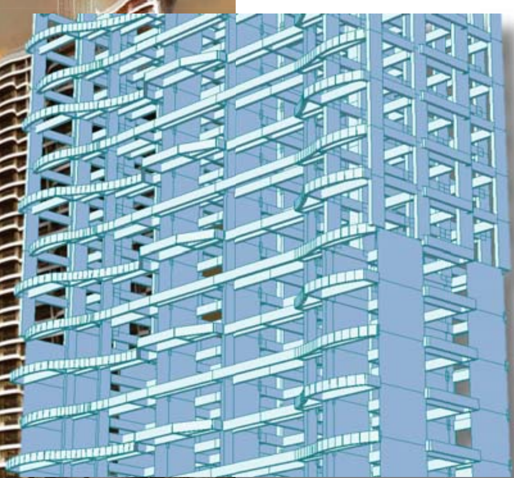
Typical 14 upper floor resting on floating column from An Beam spanning 25 meter

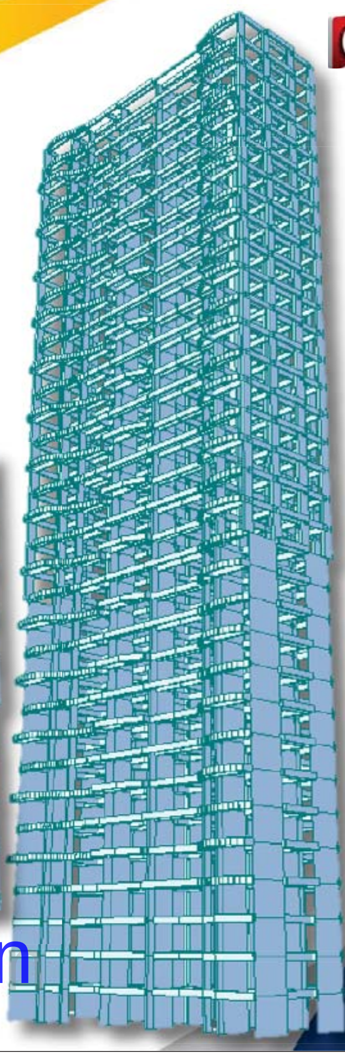


Marine Academy, Panvel, 2010

Pot 5, Sector 11, Ghansoli

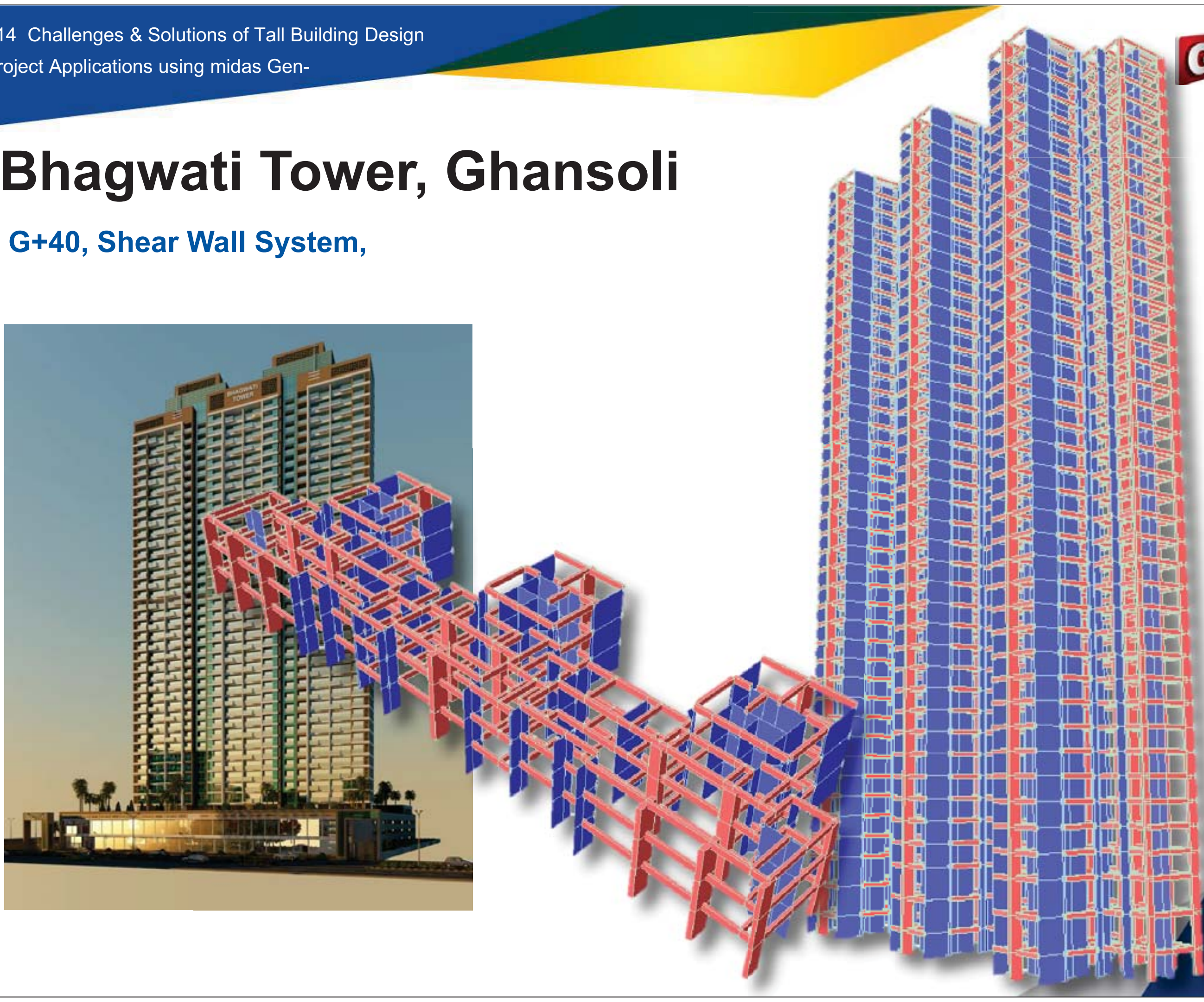
G+34, Shear Wall System



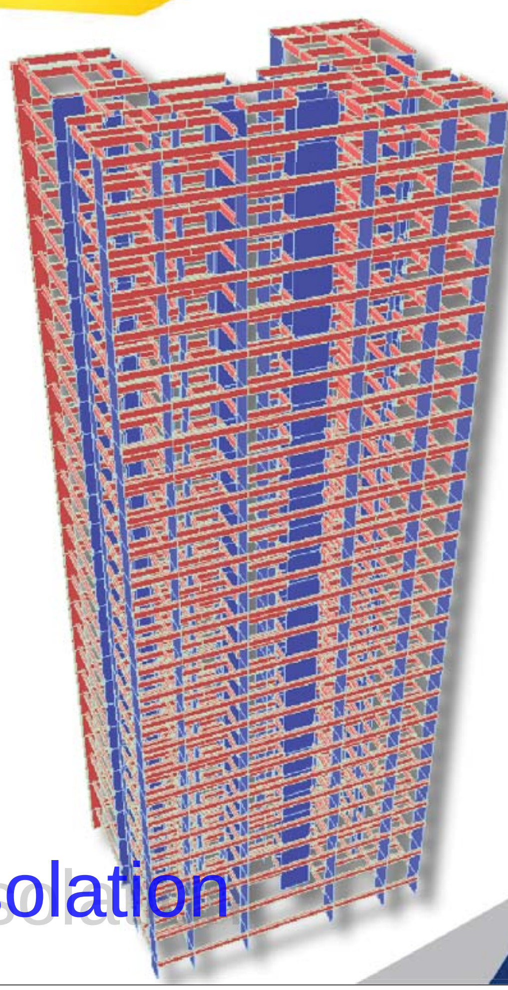


Bhagwati Tower, Ghansoli

G+40, Shear Wall System,

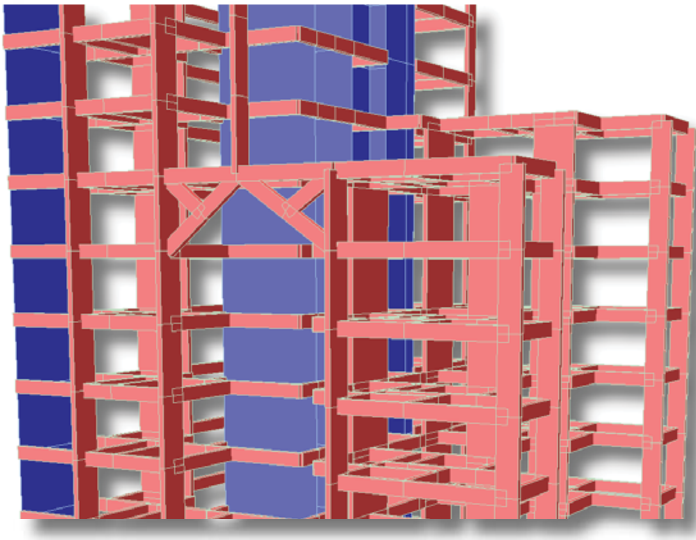


Kalyan Manek Colony,



Kesar group Matunga,

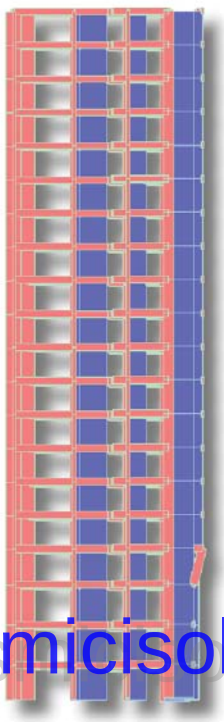
G+20, Shear Wall and partial Braced

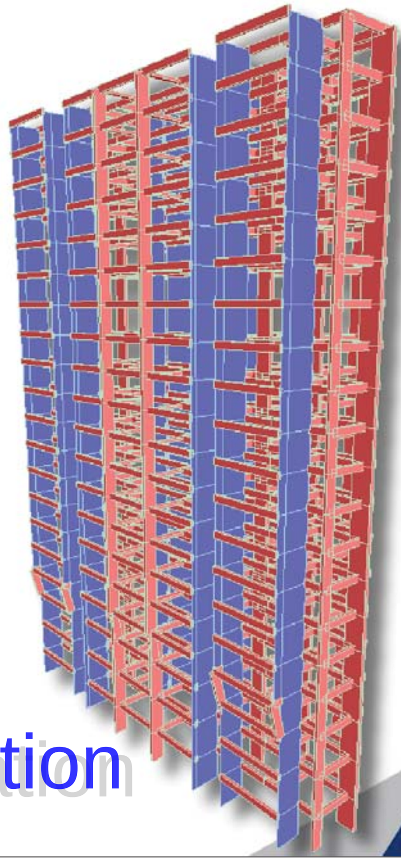


2014 Challenges & Solutions of Tall Building Design
- Project Applications using midas Gen-

Dimension Paradise Group, Kharghar

G+20, Shear Wall and partial Braced





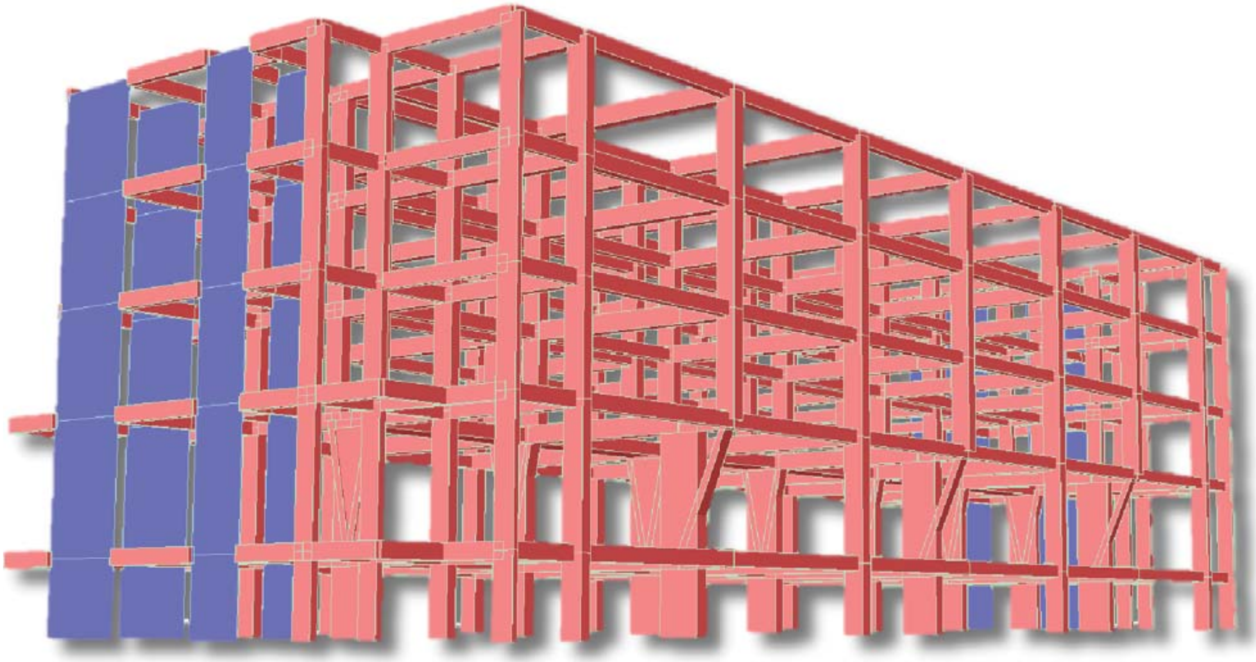
Sai World City, Kharghar

G+45, Shear Wall

Plot No. 22, Sector 4, Sanpada

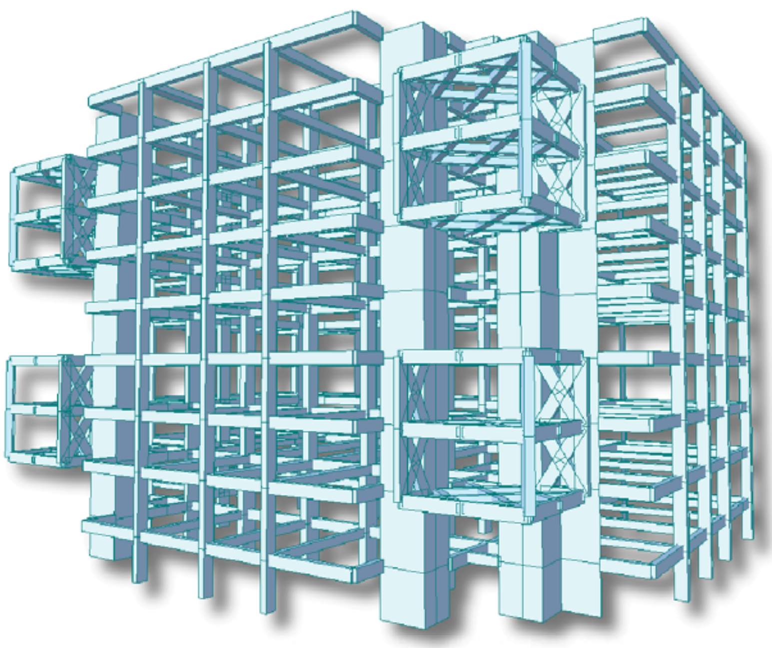
G+16, Shear Wall

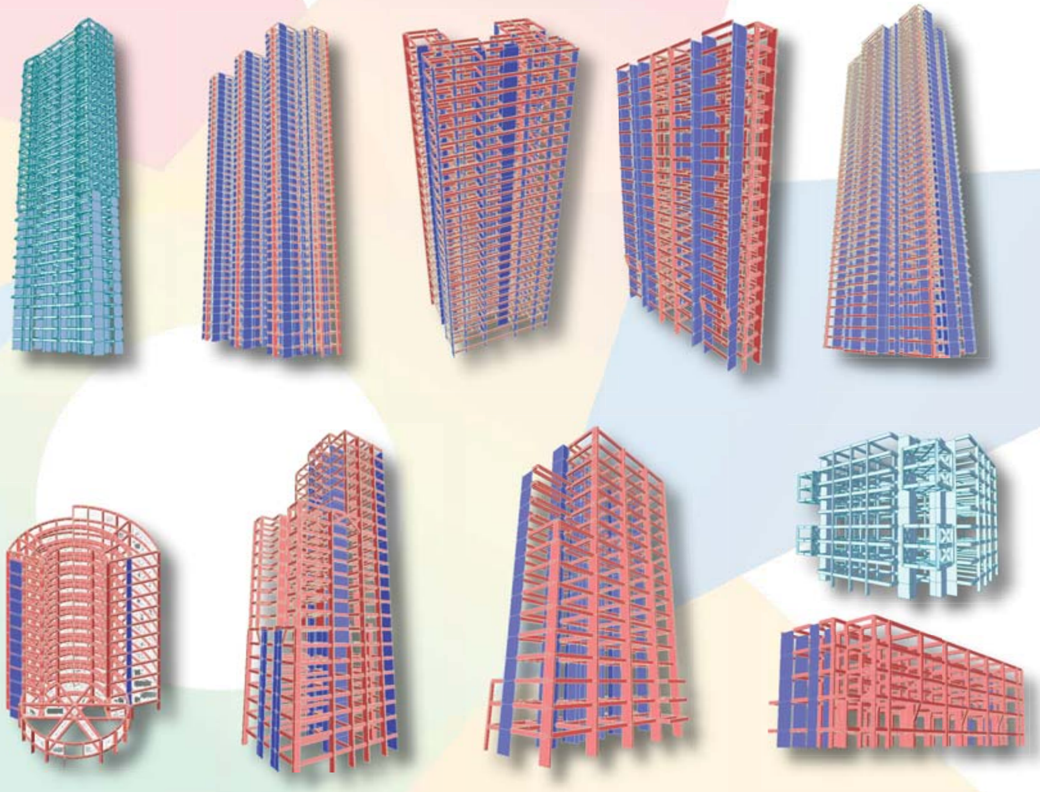
K12 Taloja, Y column



2014 Challenges & Solutions of Tall Building Design
- Project Applications using midas Gen-

Karnar School, Dronagiri, 2014





Thank you

Raajesh K. Ladhad
Structural Concept Designs Pvt., Ltd.

04

Important criteria to be considered for tall building design report

Prof. M. A. Chakrabarti, VJTI

k7664913 www.fotosearch.com

TALL BUILDING DESIGN

DR.M.A.CHAKRABARTI
Professor
Structural Engineering Department, VJTI

TALL BUILDING- HISTORY

- Park Row Building, New York, 1899 30 storeys
- 102 storeyed Empire State Building 1931, New York
- Latest Bhurj Khalifa, Dubai
- Many more to come

WHY DIFFERENT?

- Not only high gravity loads
- Lateral loads are important
- Evolution of structural systems to resist lateral loads
- Aerodynamic forms and shapes for better performance
- Damping to reduce drifts

SEISMIC LOADS

- Unified approach everywhere
- Three earthquake levels to be examined
- E1 frequent , low intensity, return period 72 years
- E2 lesser frequent, medium intensity, return period 475 years
- E3 least frequent, high intensity, return period 2475 years
- Elastic response spectrum
- Accelerograms available or can be generated for a site

FAILURE MODES

- Experiments upto failure of tall building models on shake table have shown that
- Elastoplastic deformation and energy dissipation takes place before collapse
- During collapse there is rigid body movement, structural element fracture and contact and collision of structural fragments

ASEISMIC DESIGN

- Seismic performance objectives
- Analysis and design requirements
- Classification of buildings
- High rise and low to medium rise
- Height measured from lowest ground level
- Exclude basements completely underground

PERFORMANCE LEVELS

- Immediate Occupancy-Minimum damage
- Life safety – Controlled damage
- Collapse prevention – Extensive damage
- The regions in between these levels are performance ranges
- Graph of strength v/s deformation

PERFORMANCE OBJECTIVES

Building Occupancy Class	Earthquake Level E1	Earthquake Level E2	Earthquake Level E3
Normal	IO	LS	CP
Special	-	IO	LS

PLANNING REQUIREMENTS

- Building and structural systems
- Structural simplicity
- Uniformity, symmetry and redundancy
- Adequate resistance and stiffness
- Similar resistance and stiffness in both main directions
- Adequate torsional resistance and stiffness
- Diaphragm action
- Proper foundation design

PLANNING REQUIREMENTS

- Compliance of regularity requirements in plan and elevation
- Torsional irregularity
- Floor discontinuities
- Projections in plan
- Interstorey strength irregularity
- Interstorey stiffness irregularity
- Discontinuity of vertical structural elements

PLANNING REQUIREMENTS

- Primary seismic members
- Secondary seismic members

SELECTION OF ANALYSIS

- Multimode response spectrum analysis
- Linear response history method
- Nonlinear response history method
- Consideration of vertical component of earthquake
- Consideration of seismic forces on basements
- Directional combination of simultaneous earthquake loads

LIMIT STATE COLLAPSE

- Strength verification – Capacity design
- Avoid any brittle or sudden failure from ocurring
- Structural elements
- Relevant non structural elements
- Connections
- Load combinations
- Second order effects

LIMIT STATE SERVICEABILITY

- Damage limitation
- Limitation of storey drifts
- Seismic joints
- Proper detailing for ductility
- Proper schemes of splicing reinforcement
- Nonstructural elements

PERFORMANCE BASED DESIGN

- Design stage 1-A
- Design stage 1-B
- Design stage 2
- Design stage 3
- Applicable to non structural elements also

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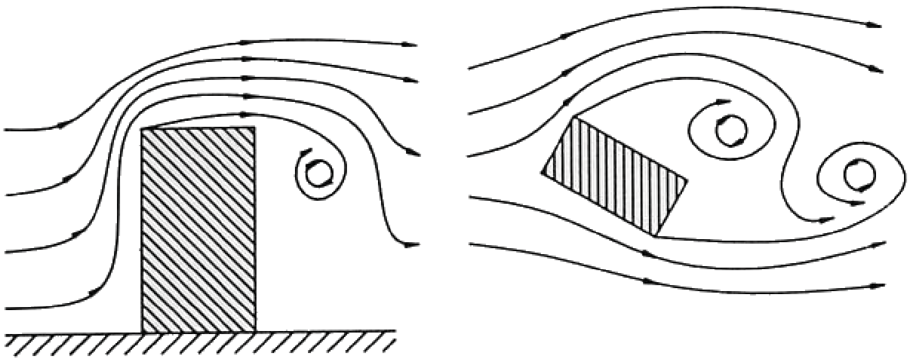
PHENOMENON OF WIND

- Complex phenomenon
- Composed of numerous eddies of different sizes and rotational characteristics
- Eddies give wind its gusty character
- Gustiness with interaction with surface features
- Average wind speed over a time period of 10 minutes increases with height while gustiness reduces with height

PHENOMENON OF WIND

- Wind vector is the sum of mean vector component (static part) and a dynamic or turbulent component
- Dynamic wind loads depend on size of eddies
- Large ones whose dimensions are comparable with those of the structure give pressures as they envelop the structure
- Small ones result in pressures on various parts of the structure that are uncorrelated with the distance of separation
- Tall and slender structures respond dynamically to the effects of wind

EDDIES



(a) Elevation (b) Plan

Figure 1: Generation of eddies.

FAILURE O TACOMMA NARROWS

REASONS OF FAILURE

1. "It is very improbable that resonance with alternating vortices plays an important role in the oscillations of suspension bridges. First, it was found that there is no sharp correlation between wind velocity and oscillation frequency such as is required in case of resonance with vortices whose frequency depends on the wind velocity. Secondly, there is no evidence for the formation of alternating vortices at a cross section similar to that used in the Tacoma Bridge, at least as long as the structure is not oscillating. It seems that it is more correct to say that the vortex formation and frequency is determined by the oscillation of the structure than that the oscillatory motion is induced by the vortex formation."
Source: Ammann, O.H., T. Von Karman, and G.B. Woodruff. "The Failure of the Tacoma Narrows Bridge." Report to the Federal Works Agency. Washington, DC (March 28, 1941).

REASONS OF FAILURE

2. "The primary cause of the collapse lies in the general proportions of the bridge and the type of stiffening girders and floor. The ratio of the width of the bridge to the length of the main span was so much smaller and the vertical stiffness was so much less than those of previously constructed bridges that forces heretofore not considered became dominant."
Source: Paine, C., et al. "The Failure of the Suspension Bridge Over Tacoma Narrows." Report to the Narrows Bridge
Loss Committee (June 26, 1941).

REASONS OF FAILURE

3. "Once any small undulation of the bridge is started, the resultant effect of a wind tends to cause a building up of vertical undulations. There is a tendency for the undulations to change to a twisting motion, until the torsional oscillations reach destructive proportions."
Source: Steinman, David B., and Sara Ruth Watson. *Bridges and Their Builders.* New York: Putnam's Sons, 1941.

REASONS OF FAILURE

4."The experimental results described in a (1942) report indicated rather definitely that the motions were a result of vortex shedding."
Source: *Aerodynamic Stability of Suspension Bridges.* Univ. of Washington Engineering Experiment Station Bulletin (Seattle, WA) 1.16 (1952).

REASONS OF FAILURE

5. "Summing up the whole bizarre accident, Galloping Gertie tore itself to pieces, because of two characteristics: 1) It was a long, narrow, shallow, and therefore very flexible structure standing in a wind ridden valley; 2) Its stiffening support was a solid girder, which, combined with a solid floor, produced a cross section peculiarly vulnerable to aerodynamic effects."
Source: Gies, Joseph. *Bridges and Men.* Garden City, NY: Doubleday, 1963.

REASONS OF FAILURE

6. "Aerodynamic instability was responsible for the failure of the Tacoma Narrows Bridge in 1940. The magnitude of the oscillations depends on the structure shape, natural frequency, and damping. The oscillations are caused by the periodic shedding of vortices on the leeward side of the structure, a vortex being shed first from the upper section and then the lower section."
Source: Houghton, E.L., and N.B. Carruthers. *Wind Forces on Buildings and Structures: An Introduction*. New York: John Wiley and Sons, Inc., 1976.

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DYNAMIC PHENOMENA OF WIND

- Buffeting
- Vortex Shedding
- Galloping
- Flutter
- Ovalling

BUFFETING

- The buffeting is defined as the wind-induced vibration in wind turbulence that generated by unsteady fluctuating forces as origin of the random ones due to wind fluctuations.
- Random Vibration Problem
- The purpose of buffeting analysis is that prediction or estimation of total buffeting response of structures (Displacements, Sectional forces: Shear force, bending and torsional moments)

VORTEX SHEDDING

- **Look carefully at a flagpole** or streetlight on a windy day and you may see the structure oscillating in the breeze. Imagine the phenomenon scaled to the height of an urban skyscraper and you can appreciate that at a minimum, life for the inhabitants on the upper floors would be uncomfortable; and should the building fail due to the forces exerted on it, life would be in peril.
- How, then, do the designers of tall buildings mitigate against the effect of winds that routinely have velocities of 50–150 km/hr near the tops of tall city buildings?
- The key phenomena that building engineers need to worry about are the vortices—swirling flows of air—that form on the sides of a building as the wind blows by it and the forces that arise as those vortices form and subsequently detach from the building.

VORTEX SHEDDING

- **Vortices** can form coherently on the sides of a building buffeted by steady winds, exert alternating forces on the structure (black arrows), and, once detached, form a so-called Kármán street downwind of the building. If the coherent vortex shedding is not mitigated, the resulting forces on the building can grow dangerously large.
- Best way to handle is to stop coherence by confusing the wind

GALLOPING

- Galloping is transverse oscillations of some structures due to the development of aerodynamic forces which are in phase with the motion. It is characterized by the progressively increasing amplitude of transverse vibration with increase of wind speed
- Normal phenomenon in Tacoma Narrows Bridge

FLUTTER

- Flutter is unstable oscillatory motion of a structure due to coupling between aerodynamic force and elastic deformation of the structure. Perhaps the' most common form is oscillatory motion due to combined bending and torsion. Long span suspension bridge decks or any member of a structure with large values of d/t (where d is the depth of a structure or structural member parallel to wind stream and t is the least lateral dimension of a member) are prone to low speed flutter.

OVALLING

- Thin walled structures with open ends at one or both ends such as oil storage tanks,and natural draught cooling towers in which the ratio of the diameter of minimum lateral dimension to the wall thickness is of the order of 100 or more, are prone to ovalling oscillations. These oscillations are characterized by periodic radial deformation of the hollow structure

IS WIND LOAD STATIC ONLY?

DESIGN AGAINST WIND

- Three basic wind effects
- Environmental wind studies
- Wind loads for facades
- Wind loads for structures

DESIGN CRITERIA

- Stability against overturning, uplift/sliding of whole structure
- Strength
- Serviceability
- Control of sways, interstorey drifts
- Control of sway accelerations

WIND CODES

- Static analysis methods
- Dynamic analysis methods
- Dynamic analysis methods must be used for buildings with both height/breadth ratio >5 and a first mode frequency < 1 hertz

STATIC ANALYSIS

- Quasi steady approximation
- Easy to use

DYNAMIC EFFECTS

- Flow pattern of wind around a building complicated due to
- Distortion of mean flow
- Flow separation
- Fluctuation of vortices
- Development of wake
- Large aerodynamic loads act on structural system
- Intense localized fluctuating forces act onm façade
- Building vibrates in rectilinear and torsional modes
- Amplitudes are dependent on aerodynamic forces and dynamic characteristics of building

ALONG WIND FORCES

- Due to buffeting
- Separation into mean and fluctuating components
- Basis of gust factor approach
- This is accurate if wind flow is not affected by presence of neighbouring tall buildings

ACROSS WIND FORCES

- Due to vortex shedding
- Due to incident turbulence producing varying lift and drag forces and pitching moments on a structure over a wide band of frequencies
- Due to displacement dependent excitations like galloping, flutter and lock in
- Formulae are available to gauge these forces
- IS875 Part 3 Review and Draft of revisions

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WIND DRIFT DESIGN

- Performance Objectives
- Limit damage to cladding on façade, partitions and interior finishes
- Reduce effects of motion perceptibility
- Limit the P-Delta or secondary loading effects
- Performance Criteria
- Racking Drift
- Chord Drift

COMFORT CRITERIA

- Based on lateral accelerations
- Provision of ductility

CONTROL OF DRIFTS

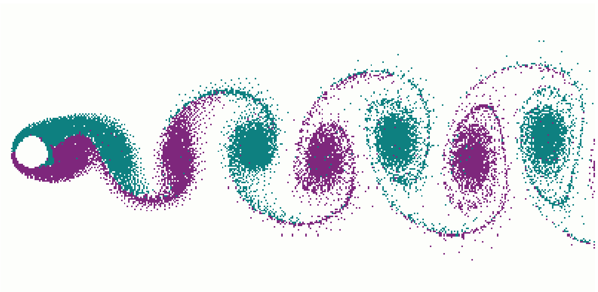
- Suitable damping mechanisms
- Suitable shape of buildings
- Confuse the wind flow
- Provide reduced size as one moves higher
- Provide openings for free wind flow

FUTURE TRENDS

- Assumptions closer to real behaviour
- Detailing to satisfy the assumed behaviour
- Performance based design philosophy
- Reduced maintenance costs
- Use of sophisticated means of analysis and design
- Simulation using CFD software

INTERFERENCE EFFECTS

- Karman's vortex street
- Forms only at critical velocity ranges



Buildings of similar size located in close proximity to proposed tall building can cause large increases in cross wind responses

AWARENESS IS REQUIRED

- Design philosophy
- Codes of practices
- Codes of foreign countries
- Good representation of Design Basis Reports
- Disemination of knowledge by seminars
- Proper use of computer software
- Peer review in design office

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THANK YOU

2014 MIDAS Technical Seminar
Challenges & Solution for Tall Building Design

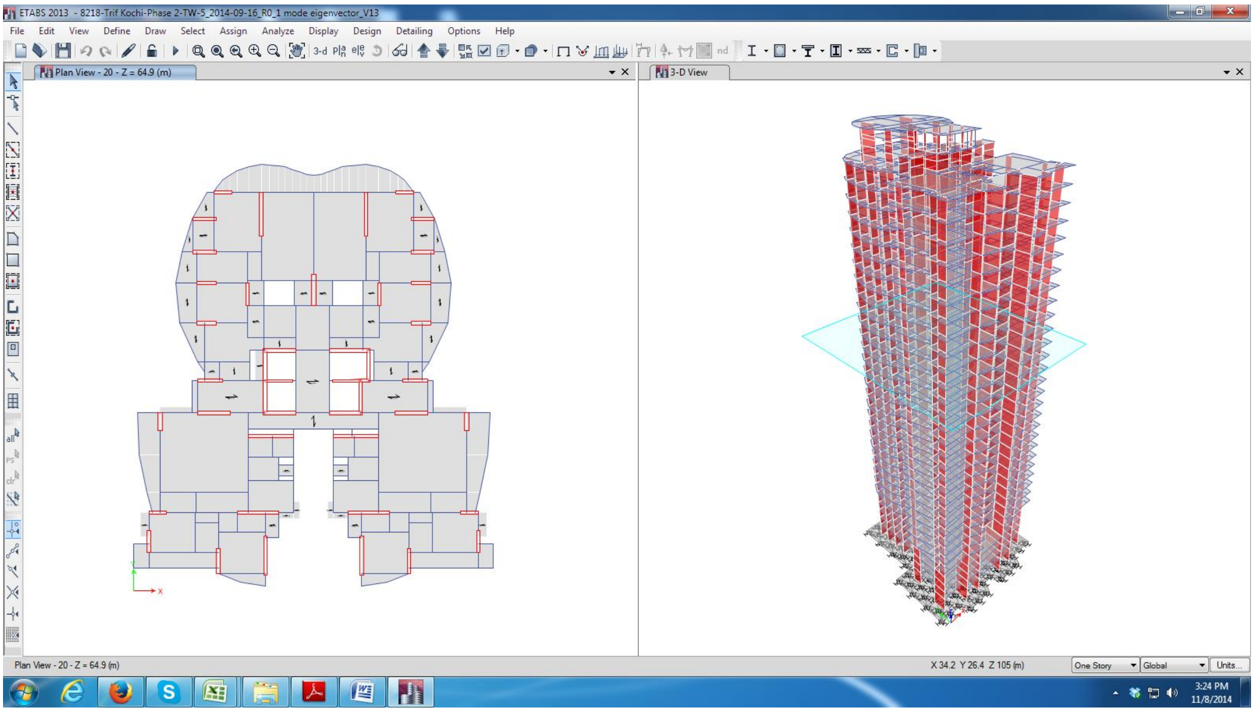
05

Modeling issues in high rise building

Vinayak Naik, Sterling Engineering consultancy Services pvt., ltd

Study to investigate the contribution of Torsional Fundamental mode.

Typical floor plan and 3D view of structure used for the exercise.



This structure exhibits a Torsional Fundamental mode. Two response spectrum analyses were done first, using only one mode i.e. only the Fundamental mode and the other using 20 modes for modal superposition. The second analysis captured more than 90% of the total seismic mass. The scale factor in both cases was 1.

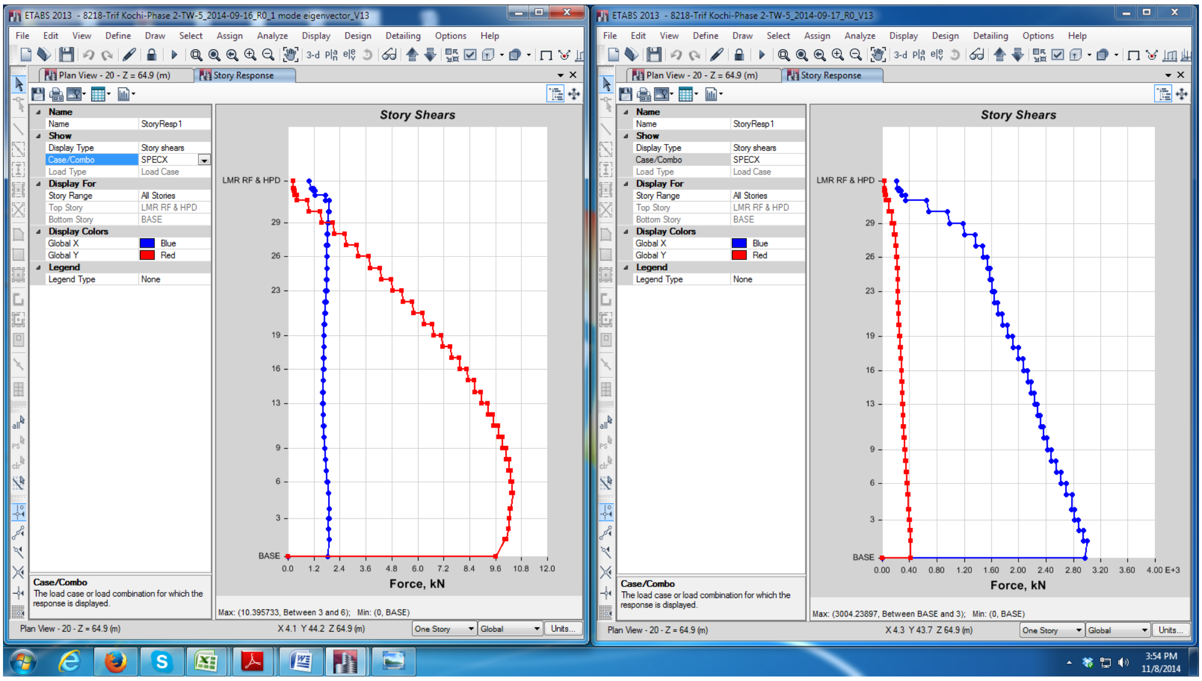
Single mode case mass participation

Mode	Period sec	UX	UY	RZ
1	4.977	0.0005	0.0152	0.5853

20 mode case mass participation

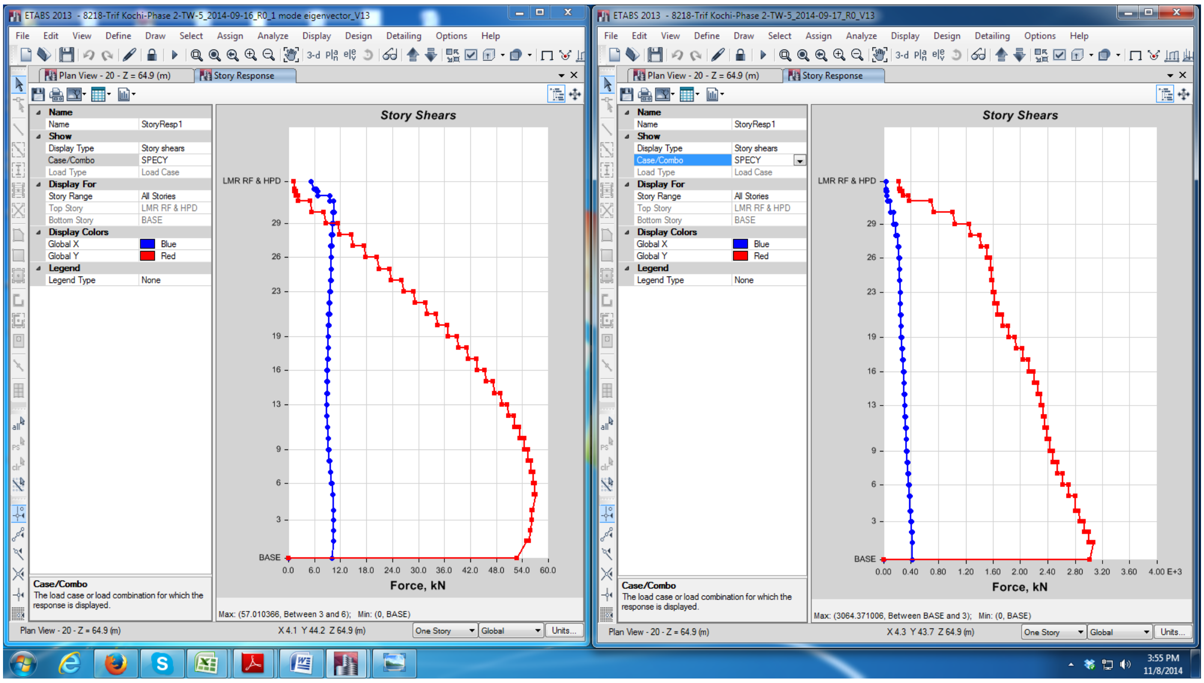
TABLE: Modal Participating Mass Ratios					
Case	Mode	Period sec	UX	UY	RZ
Modal	1	4.977	0.0005	0.0152	0.5853
Modal	2	4.689	0.0143	0.5996	0.0155
Modal	3	4.304	0.6133	0.0134	0.0007

Comparison of Base shear for SPECX for one mode and 20 mode analyses



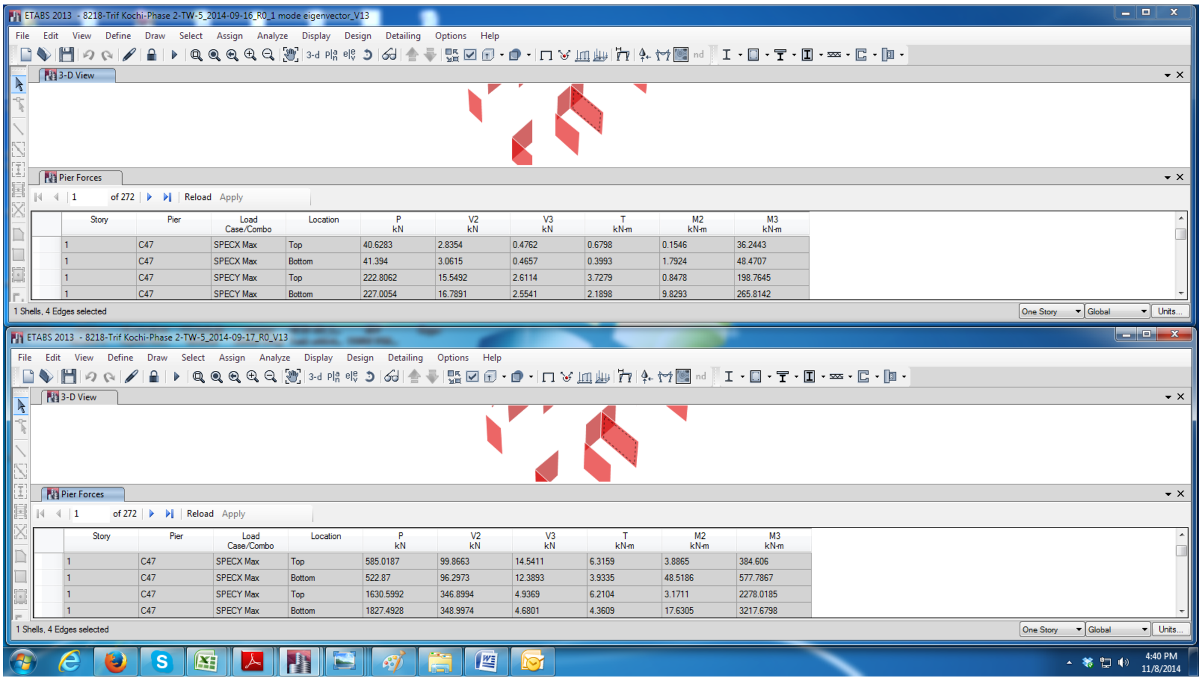
Base shear in X direction = 10.5kN **Base shear in X direction = 3004.3kN**

Comparison of Base shear for SPECY for one mode and 20 mode analyses



Base shear in Y direction = 57kN **Base shear in Y direction = 3064.4kN**

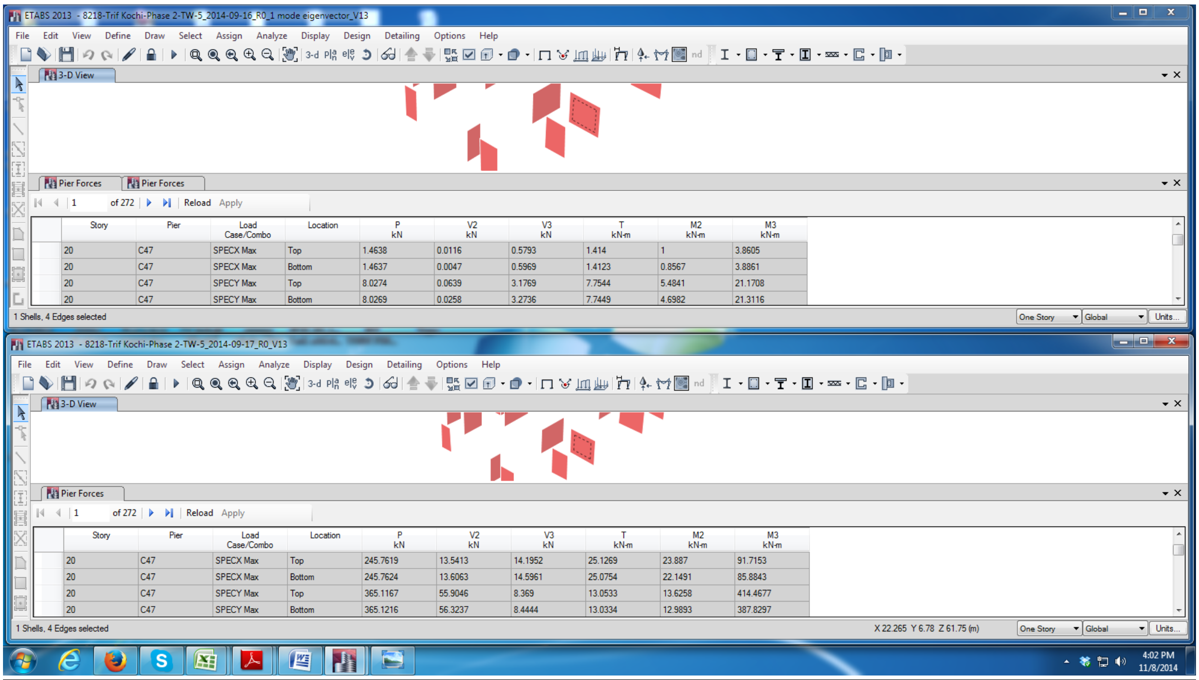
Comparison Forces in Wall C47 on Level 1



Story	Pier	Load Case/Combo	Location	P kN	V2 kN	V3 kN	T kN-m	M2 kN-m	M3 kN-m
1	C47	SPECX Max	Top	40.6283	2.8354	0.4762	0.6798	0.1546	36.2443
1	C47	SPECX Max	Bottom	41.394	3.0615	0.4657	0.3993	1.7924	48.4707
1	C47	SPECY Max	Top	222.8062	15.5492	2.6114	3.7279	0.8478	198.7645
1	C47	SPECY Max	Bottom	227.0054	16.7891	2.5541	2.1898	9.8293	265.8142

Story	Pier	Load Case/Combo	Location	P kN	V2 kN	V3 kN	T kN-m	M2 kN-m	M3 kN-m
1	C47	SPECX Max	Top	585.0187	99.8663	14.5411	6.3159	3.8865	384.606
1	C47	SPECX Max	Bottom	522.87	96.2973	12.3893	3.9335	48.5186	577.7867
1	C47	SPECY Max	Top	1630.5992	346.8994	4.9369	6.2104	3.1711	2278.0185
1	C47	SPECY Max	Bottom	1827.4928	348.9974	4.6801	4.3609	17.6305	3217.6798

Comparison Forces in Wall C47 on Level 20



Story	Pier	Load Case/Combo	Location	P kN	V2 kN	V3 kN	T kN-m	M2 kN-m	M3 kN-m
20	C47	SPECX Max	Top	1.4638	0.0116	0.5793	1.414	1	3.8605
20	C47	SPECX Max	Bottom	1.4637	0.0047	0.5969	1.4123	0.8567	3.8861
20	C47	SPECY Max	Top	8.0274	0.0639	3.1769	7.7544	5.4841	21.1708
20	C47	SPECY Max	Bottom	8.0269	0.0258	3.2736	7.7449	4.6982	21.3116

Story	Pier	Load Case/Combo	Location	P kN	V2 kN	V3 kN	T kN-m	M2 kN-m	M3 kN-m
20	C47	SPECX Max	Top	245.7619	13.5413	14.1952	25.1269	23.887	91.7153
20	C47	SPECX Max	Bottom	245.7624	13.6063	14.5961	25.0754	22.1491	85.8843
20	C47	SPECY Max	Top	365.1167	55.9046	8.369	13.0533	13.6258	414.4677
20	C47	SPECY Max	Bottom	365.1216	56.3237	8.4444	13.0334	12.9893	387.8297

Conclusion:

I. Base shears due to First mode in X direction and Y direction are less than 2% of the 20 mode Base shears.

II. The forces in a randomly chosen wall are 8-10% for Level 1 and 5-6% for Level 20 of the forces obtained from 20 mode analysis.

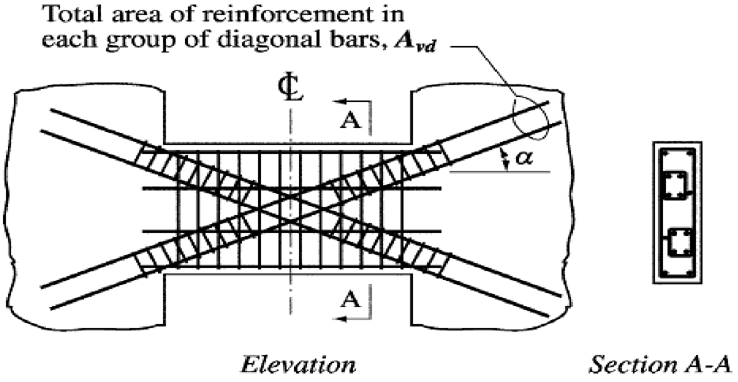
Thus, the First (Fundamental) mode does not play a major role in the response of the structure indicating that the contribution of the higher modes is significant.

According to FEMA-356 (Cl. 2.4.2.1), higher mode effects shall be considered significant if the shear in any story resulting from the modal analysis considering modes required to obtain 90% mass participation exceeds 130% of the corresponding story shear considering only the first mode response.

Measures to control deflection under Lateral loads.

a) Increasing grade of concrete for Columns and shear walls.
b) Providing flanges to walls or dumbbell type of columns wherever possible.
c) Coupling RC shear walls.

Individual RC shear walls have a very low lateral resistance since they act as cantilevers. Coupling the shear walls with RC beams changes the cantilever behavior to a frame behavior. This greatly reduces the deflection. Coupling beams are usually subjected to high levels of shears from lateral load. As a result, it is most commonly observed that the coupling beams fail in shear. The most common practice is to reduce the flexural stiffness of these beams. This results in the reduction of shear in the beams. However, the overall stiffness of the frame decreases resulting in greater lateral deflections. One way to overcome this difficulty is to use diagonal reinforcement as per IS 13920 : 1993 which specifies that the entire earthquake induced shear and flexure shall, preferably, be resisted by diagonal reinforcement. However this clause makes no mention of the permissible limit of shear stress. This may give rise to an impression that if we use this type of detailing, we are not bound by the maximum permissible shear stress specified in Table 20 (IS 456 : 1993). International codes, however, put a cap on the maximum shear stress, even if this type of diagonal reinforcement is adopted. For example, in ACI 318, this value is 0.742(f'c)^0.5 (f'c is characteristic cube strength N/mm^2).



Coupling beam with diagonally oriented reinforcement

Avd = total area of reinf. in each group of diagonal bars in a diagonally reinforced coupling beam

For M40 this value is 4.69 N/mm^2. Also note that it is grade dependent. For grade M70 it is 6.21 N/mm^2. We may therefore, use such international codes in this case.

It is preferable to use the same grade of concrete for the coupling beams as is used for shear walls which is usually high. This simplifies the construction since during the casting of the slabs, the concreting of walls and the coupling beams can be done simultaneously.

However if one wishes to restrict to IS code strictly, then we are limited by the 4 N/mm^2 cap which is often found to be insufficient.

Either way, in cases where the shear stress exceeds the permissible limit, one may use structural steel beams. These have a high shear resistance. Good literature is available on the net for the design of these beams. For these beams to be effective, they have to be provided with an embedment length. This depends on the grade of concrete used for the walls and the thickness of the walls. The embedment lengths could vary from 1.5m to even greater than 3m. Such depths are not available sometimes. In such cases a composite steel column may be used in the boundary element zone to transmit the shear and bending moments to the walls.

Use of coupling beams using diagonal reinforcement or structural steel beams, poses problems wherever jump form type of construction is used. Even if the construction is conventional, the steel beams interfere with the reinforcement detailing. This problem also needs careful attention.

d) Using Outriggers.

An outrigger is a stiff beam that connects the shear walls to exterior columns. When the structure is subjected to lateral forces, the outrigger and the columns resist the rotation of the core and thus significantly reduce the lateral deflection and base moment, which would have arisen in a free core.

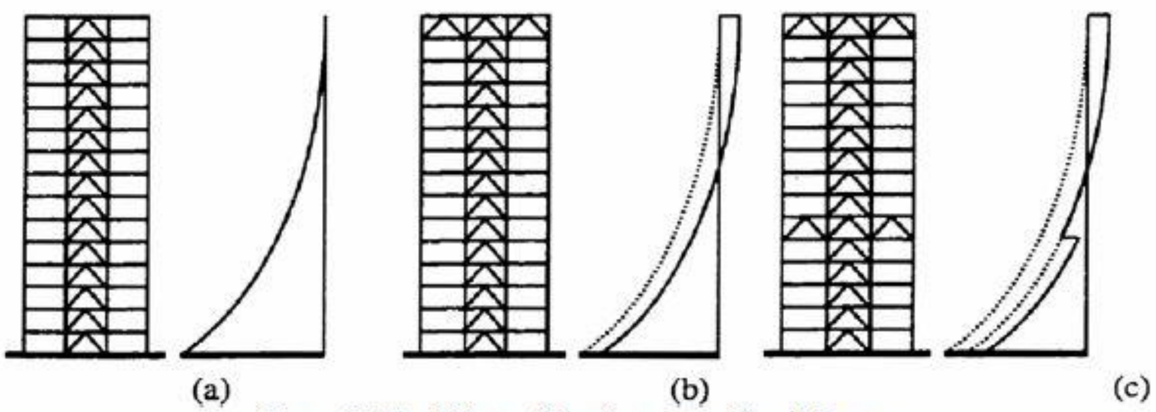


Figure 2. The Effect of Outriggers on Core Moment

Smith and Coull (1991) studied the optimum location of outriggers by considering hypothetical structures whose outriggers were flexural rigid. They found that a single outrigger in a one-outrigger system should be located at approximately half height of the building, that the outriggers in a two-outrigger system should be located roughly at one-third and two-thirds height, and that in a three-outrigger system they should be at approximately one-quarter, one-half, and three-quarters height, and so on. Generally for the optimum performance of an n-outrigger structure, the outriggers should be placed at the l/(n+l), 2/(n+l), up to the n/(n+l) height locations.

The Smith and Coull study found that the reduction in core base bending moment is approximately 58%, 70%, 77% and 81% for one-outrigger, two-outrigger, three-outrigger and four-outrigger structures, respectively. Unexpectedly, contrary to a traditional location for outriggers, they found that it is structurally inefficient to locate an outrigger at the top of a building. In an optimally arranged outrigger system, the moment carried by any one outrigger is approximately 58% of that carried by the outrigger below. However, if an additional outrigger is placed at the top of the building, it carries a moment that is roughly only 13% of that carried by the outrigger below, which clearly shows the inefficiency of this outrigger location.

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There are certain important points one has to understand whenever outriggers are used. It is common practice to analyze the structure as a whole. In structures in which outriggers are used, this poses a problem. The problem is that the outriggers act as deep cantilevers. As a result, the supporting columns start acting as suspenders instead of load bearing columns. Therefore it is necessary to use sequential construction analysis to offset at least the effect of self-wt of the structure below. Though this does reduce the effect, it may not be fully neutralized because the super-imposed dead load and live load are effective on the whole structure. Coupled with the lateral loads, this leads to very high shear forces in the outriggers. This results in the shear stresses exceeding the maximum permissible limits. A strut and tie model could be used in such cases. This is easier said than done. The high steel percentages required for the struts and ties, detailing problems, demanding anchorages requirement etc. are quite challenging. Jump form type of construction proves a big hindrance. Structural steel trusses could be used instead though they pose other constructional problems in connection to the RCC elements, erection problems etc.. However outrigger steel trusses have a unique advantage over RC outrigger girders or trusses. The steel diagonals of the steel trusses can be connected using delayed construction joints. This almost eliminates the extra shear that is induced due to superimposed dead load and live load which in turn lessens the demand on the outrigger trusses resulting in a beneficial design.

e) Use of composite frames

The easiest way to increase the stiffness of the structure is to increase the thicknesses of shear walls. This usually encounters a stiff resistance from the architect and the client. One way to achieve greater stiffness without increasing the thickness or in fact reducing the thickness of the walls could be to use steel encased walls. As is obvious, this shall have its own share of problems in construction besides being disproportionately expensive. Another way could be to embed structural steel braced frames in RC walls. A recent study done by us indicated that the stiffness of a wall increased by about 25% but the costs almost doubled.

f) Reduction of the Torsional behavior of the structure.

It is quite easy to understand that the Torsional behavior of the structure causes the outermost frames to deflect more. The deflection of these frames governs the design of structures. The structural engineer should interact with the Architect in the concept stage to minimize such problems which ultimately result in a costlier design.

Wind loads :

Wind loads play a major role in the design of high rise structures. In terms of designing a structure for lateral wind loads the following basic design criteria need to be satisfied.

a) Stability
b) Strength design
c) Seviceabilty
d) Comfort

For RC structures, the strength design loads are based on 50 year return period using 2% damping. The serviceability design loads are based on 20 year return period using 1.5% damping. Acceleration values are determined using 1 year and 10 year return period using 1% damping. Note that we use 2% damping for strength design instead of 5% damping used in earthquake conditions. This is because the structure is in the elastic range under wind loads unlike the earthquake condition wherein the structure has to perform in the non-linear range.

For tall buildings gust wind loads based on the Wind code IS:875 (Part 3) - 1987 are used to do the preliminary design in the schematic stage. The preliminary design is usually governed by the serviceability limits i.e. maximum tip deflection and permissible drift. One could use the 20 year return period loads for this purpose but it is advisable to use the 50 year return period loads instead. This would mean that we are over designing the structural stiffness by about 25-30%. The reason why this is still recommended, is because, the wind tunnel results usually result in higher overturning base moments to the tune of about 40% or more than the gust wind based loads, even if the base shears match. This is because the resultant of the wind tunnel loads acts at much higher level compared to the wind gust loads.

A design-wise workable model at this stage is a model with the preliminary sizes for shear walls, columns, beams and slabs. This model does not reflect the greater stiffness which is available but not captured adequately. This happens for the following reason. The beams in this model are line elements and not 2D elements. The slabs are usually membrane elements and not shell elements. Once we convert the beams to 2D and membrane slabs to shell elements, the stiffness of the structure increases by about 15-35% more or less. This results in lesser Fundamental time period, lesser tip deflection and lesser wind tunnel forces.

It is this model that is sent to the Wind tunnel engineer. It is always advisable to do an initial desktop study and check the model for these loads because we can then tune up the model if so required before a wind tunnel design is carried out. This may help in eliminating or reducing the multiple runs of wind tunnel design.

In a recent exercise for a structure, the beam model deflection was 720mm for 50 year gust wind loads. It increased to 940mm (30% more) under desktop study loads based on 50 year return period. The deflection for the 2D beam model under the desktop loads was 610mm (35% less). Considering that this deflection will reduce by 25% for the 20 year loads, the expected deflection is 458mm. The permissible limit is 440mm. The analysis was done using serviceability property modifiers recommended by ACI 318.

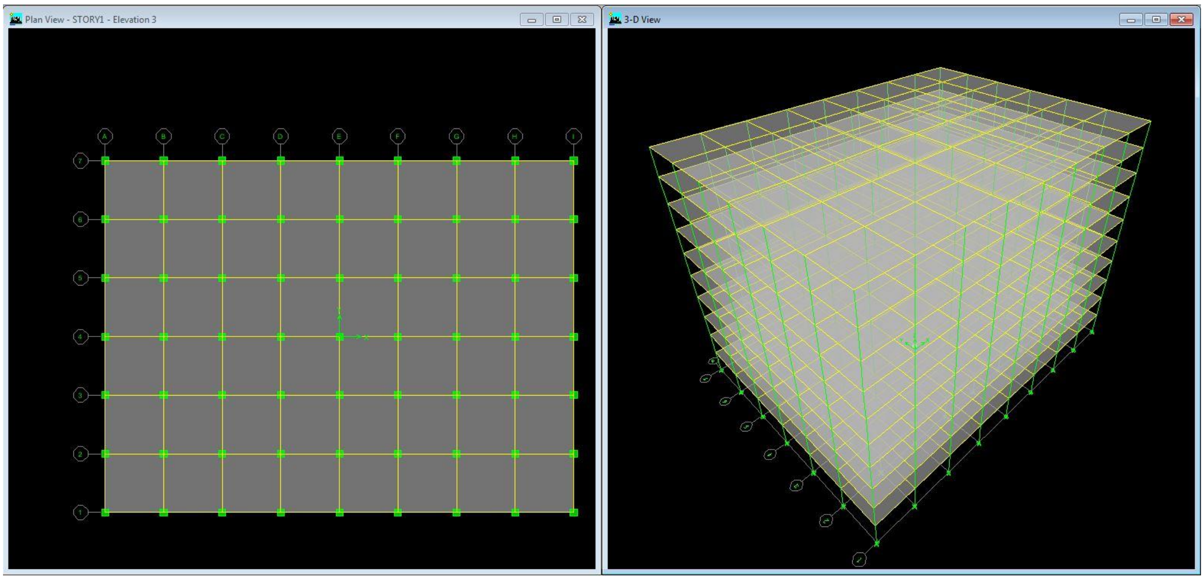
Selection of axes plays an important role in the design for Wind loads. This is illustrated using the following example. The structure considered for the study is a fictitious space frame

structure. The first model has conventional set of orthogonal axes, the second is rotated w.r.t Global axes.
The study is done to compare forces in corner columns, in the two models, due to wind forces acting in the Global X & Y directions. The wind forces are generated by Etabs based on the diaphragm widths (obstructed area).

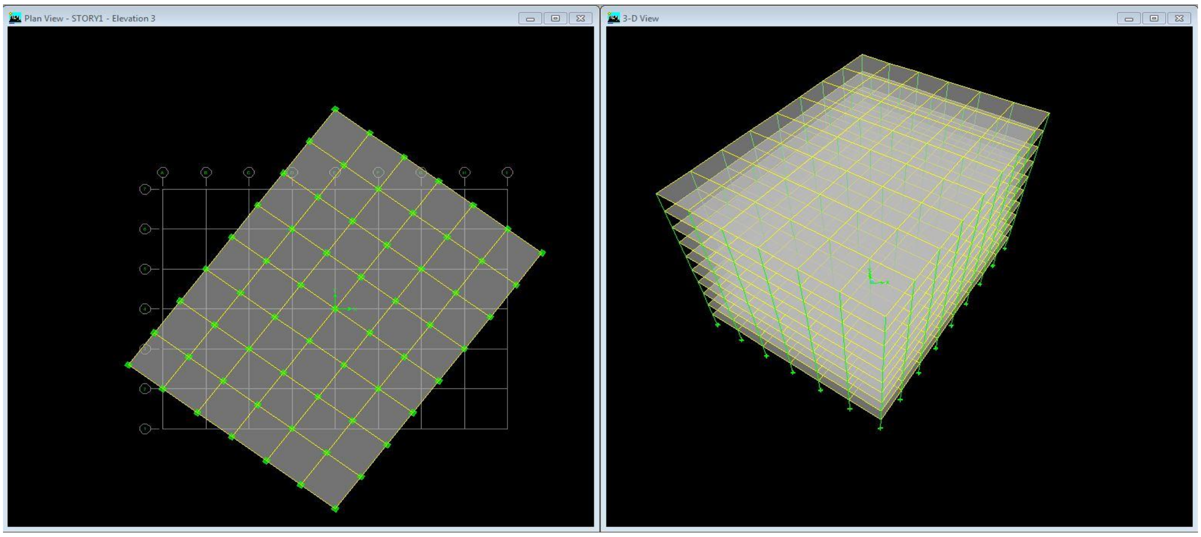
The structural data assumed is as follows,

1. Levels 10
2. Plan dimensions 30mx40m
3. Bay widths in both directions : 5m
4. Fl ht. : 3m
5. Cols. : 600x600
6. Beam depths 300x700
7. Slab depth 200
8. Grade of concrete : M40

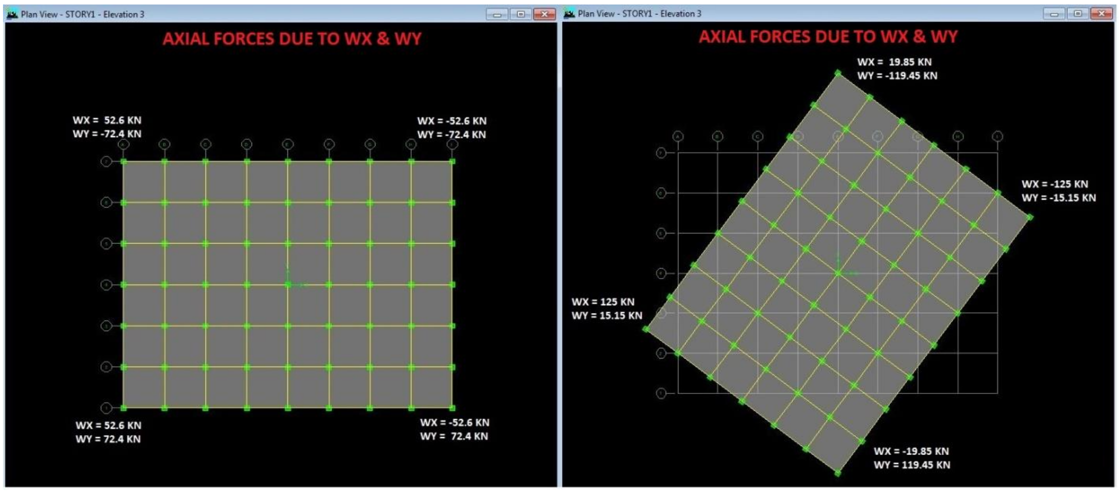
Model 1



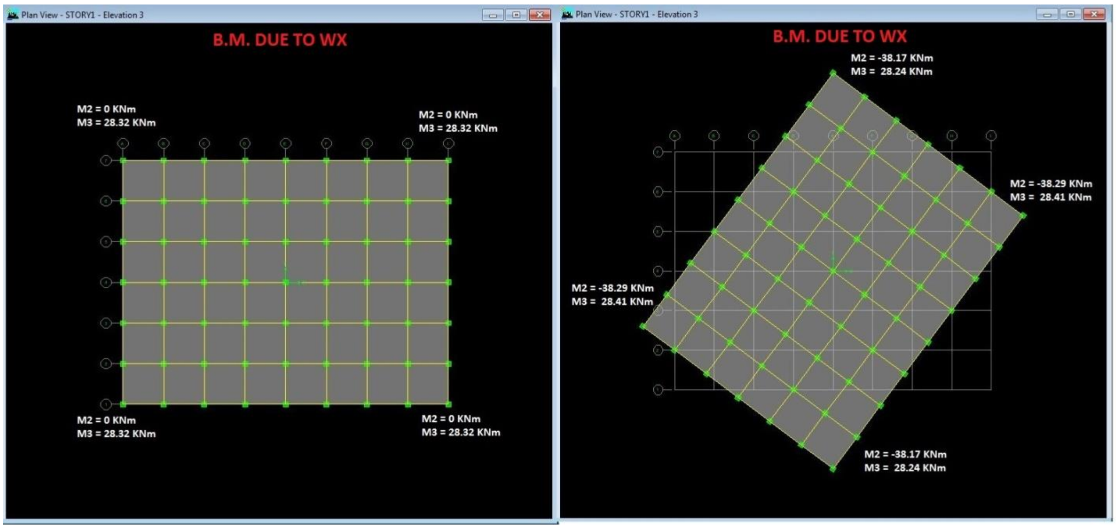
Model 2



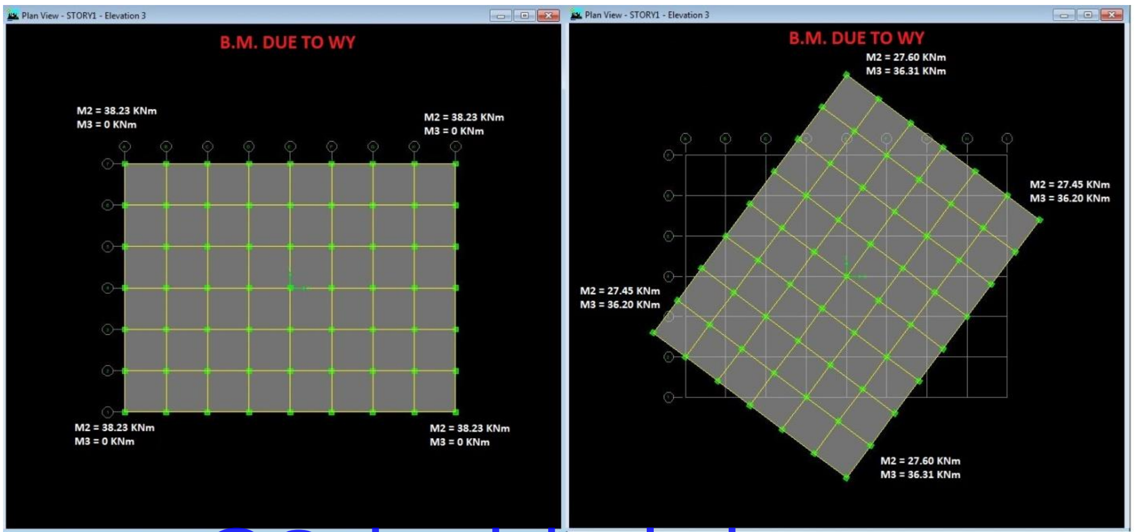
Comparison of axial forces due to WX and WY



Comparison B.M.'s due to WX



Comparison B.M.'s due to WY



As can be seen from the results, the forces arising from Model 2 (skew axes) are greater. This means, the conventional choice of axes will not result in maximum (governing) forces. A quick review of the forces indicates that both the Wind load cases should be combined 100% simultaneously to get the design forces.

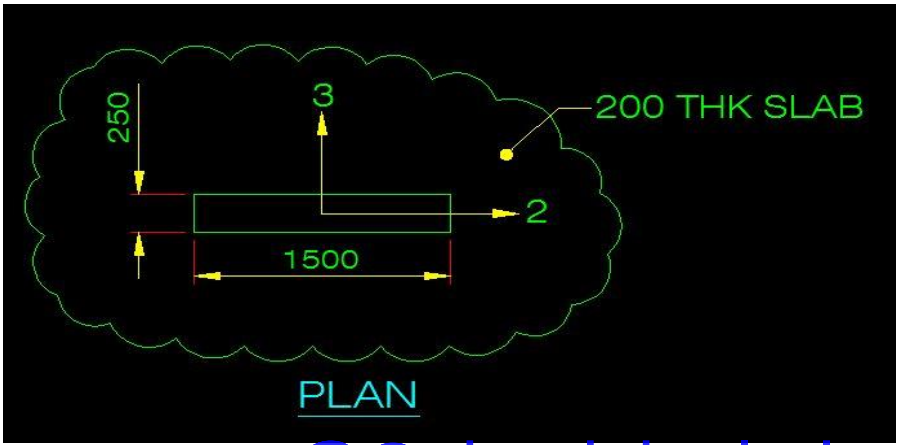
Time period for a 3D structure with podiums and basements

It is stipulated by IS 1893 (Part 1): 2002, that Buildings and portions thereof shall be designed and constructed, to resist the effects of minimum design lateral force calculated as per empirical formulae specified in Cl. 7.6.
This poses a problem for buildings with podiums and basements as the footprint of each part varies. The usual practice is to adopt the maximum X and Y dimensions and calculate the empirical period. This often results in high base shears.
One way which I suggest is as follows,

i. Calculate the typical floor stiffness, typical podium floor stiffness and typical basement floor stiffness by isolating each level.
ii. Note the deflection for a unit load used to calculate the stiffness of each these floors.
iii. Using the deflection and considering each of these storeys as cantilevers, calculate I (moment of inertia).
iv. Determine corresponding square section dimensions.
v. Note floor weight of each type.
vi. Construct a weightless cantilever model for the whole structure using the respective section dimensions for the respective floors.
vii. Lump the respective floor loads at each level.
viii. Perform a free vibration analysis and determine the Fundamental period T1.
ix. Perform similar analysis for another model considering only typical floors at all levels and note the Fundamental period T2.
x. Calculate the empirical period for a structure having typical floor footprint Tf.
xi. Then Ts= empirical period for the whole structure= Tf * T1/T2

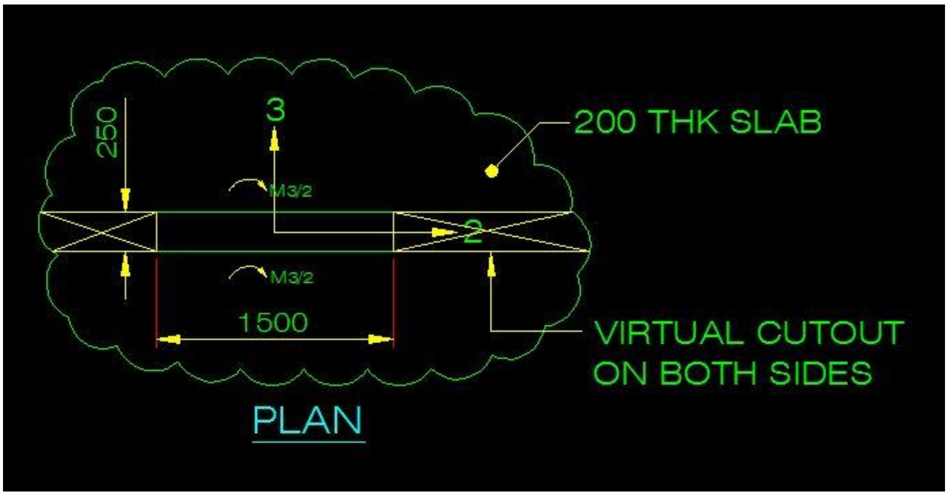
Punching or one way shear :
Consider the following data for an internal column in a 3D Etabs model with flat plates for floor slabs.

Column size = 250mmx1500mm , Effective slab thickness = 160mm
Concrete: M40
P = 440kN, M3 unbalanced = 518 kN-m, M2 neglected.



A) Using punching shear stress formulae, punching shear stress = 18.53 kg/cm^2
Allowable punching stress with shear reinforcement = 10.05 x 1.5 = 15.08 kg/cm^2
∴ Critical section fails in punching or two way shear.

B) Considering only two parallel long sides contributing in transferring shear and unbalanced moment,

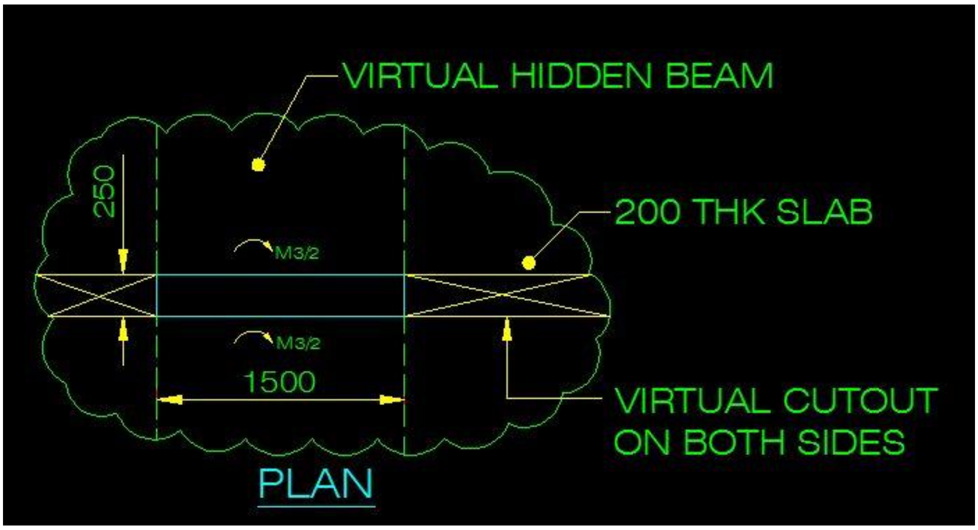


Punching shear stress = 33.73 kg/cm2 (using revised polar moment of inertia).
Allowable punching stress with shear reinforcement = 10.05 x 1.5 = 15.08 kg/cm2
∴ Critical section fails in punching or two way shear.

However, one can view this problem, also as a one way shear problem, since the transfer of shear on both sides is via a one way shear action.

In this case, permissible one way Shear = 40 kg/cm2 (maximum for M40)
∴ from a punching shear point of view the section fails. However if we consider the shear transfer as a one-way shear transfer, the section works, since 33.73 kg/cm2 < 40 kg/cm2. ∴ section can be designed with shear reinforcement.

C) Calculation for one-way shear stress based on equivalent shear in lieu of Torsion.



P = 440kN, M3 unbalanced = 518 kN-m (Torsion from beam 1500*200)
Shear on each side = 220 kN.
Torsion on each side = 518/2 = 259 kN
∴ Equivalent shear force due to torsion = (259*1.6)/1.5 = 276.2 kN
Total Shear on each face = 220 + 276 = 496.2 kN
∴ Shear Stress on each face = (496.2*100)/(150*16) = 20.68 kg/cm2 < 40 kg/cm2 ∴ section can be designed with shear reinforcement.

Some important modeling points

1. Property modifiers :
   ACI 318 recommends the following property modifiers for strength design,
   Beams.. 0.35Ig
   Columns... 0.70Ig
   Walls—Uncracked............................... 0.70Ig
   —Cracked 0.35Ig
   Flat plates and flat slabs 0.25Ig

   For serviceability design the values given above are multiplied 1.43 times.

   It is not necessary to use these two sets for all analysis as it becomes cumbersome. Moreover, since the factor is the same for all elements, the distribution of forces at joints is same for both sets under most conditions. Therefore it is common to use the property modifiers for serviceability limits for analysis since they pertain to deflections and are appropriate for the dynamic analysis for Wind loads. A few instances where this may not work is,
   a) for walls and columns having fixity at base.
   b) For P delta analysis
   c) For the instance when the program calculated time period is less than the empirical formula based period.

2. Walls and spandrel meshing.
   It is a good practice to mesh walls. A 3x3 mesh will do for most walls. 4x3 or 5x3 mesh can be used for longer walls. A 1x6 mesh is preferred for narrow walls. The meshing helps in instances where the there is bending in the weaker directions of walls.
   It is a must to mesh spandrels which otherwise would not be able to capture the bending of beams and will result in unnaturally siff beams. Usually 1x6 mesh is okay for beam spandrels. For deep spandrels or floor deep spandrels a 6x6 mesh should be okay. It is important to assign the equivalent stiffness to the spandrel which is used for a beam line element e.g. if Torsion = 1E-10, I22=0.5, I33=0.15 are applied to a beam, F11=0.5, F22=0.15, F12=0.15, M11=0.5 ,M22=0.5, M12=!E-10, V13=0.5, V23=0.5 should be applied to an equivalent spandrel.

3. Slab modeling
   Slabs can be modeled as membrane or shell elements. A membrane element should preferably a 4 sided element. Membrane elements transfer loads to supporting elements. They possess in-plane stiffness but no flexural stiffness. They should not be meshed. Slabs modeled as shell elements possess both in-plane and flexural stiffness. They are usually used in instances where the slabs contribute to the flexural stiffness of the structure e.g. flat slabs. It is mandatory to mesh slab shell elements. For load transfer a very coarse mesh of 3mx3m may be used. For capturing appropriate stiffness 1mx1m mesh may be required.

4. Floor deep girders should have semi-rigid diaphragms at both levels.

5. Transfer girders- Sequential construction analysis is essential.

REFERENCES :

Stafford Smith B, Coull A. (1991). Tall Building Structures, Wiley, New York.

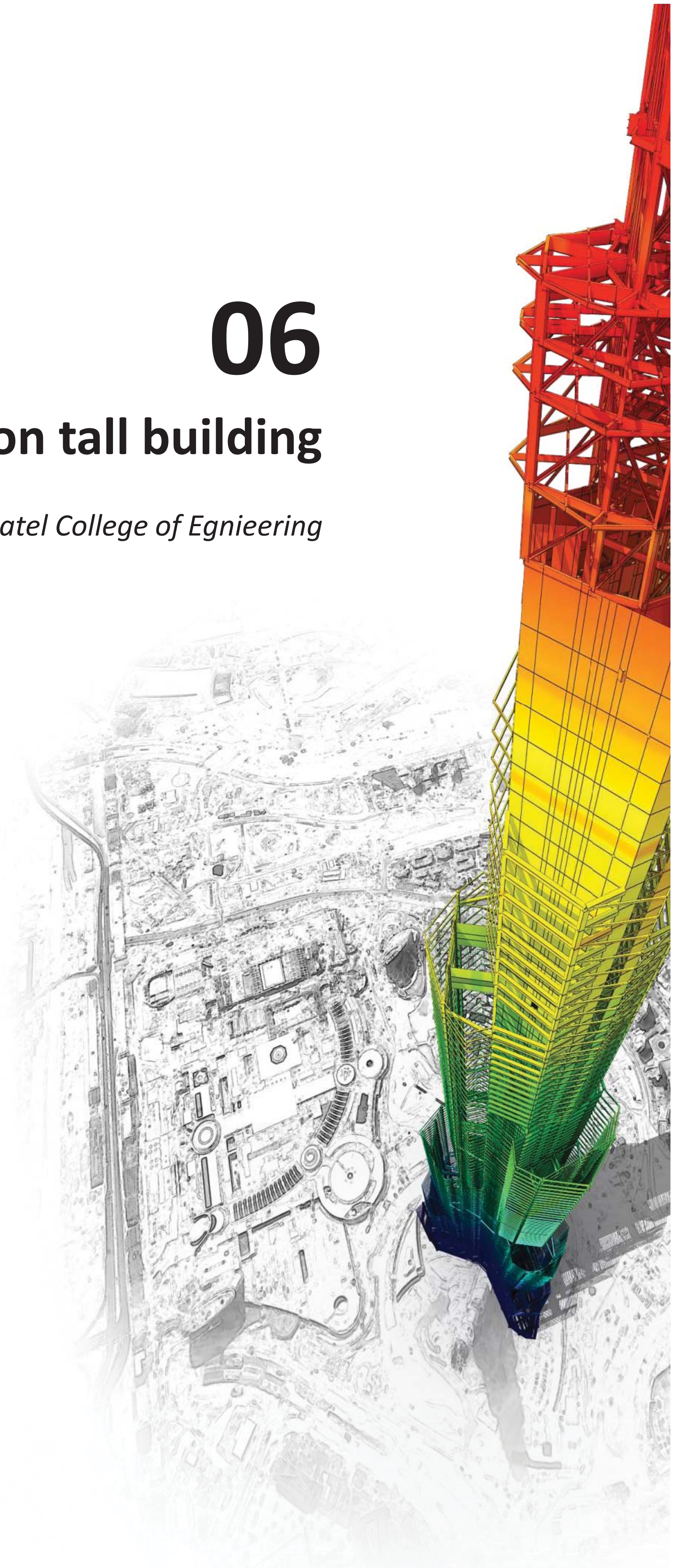
2014 MIDAS Technical Seminar

Challenges & Solution for Tall Building Design

06

Effect of wind loading on tall building

Prof. Tanuja Bandivadekar, Sardar Patel College of Egnieering



Challenges & Solutions for Tall Building Design

Effect of wind loading on tall buildings

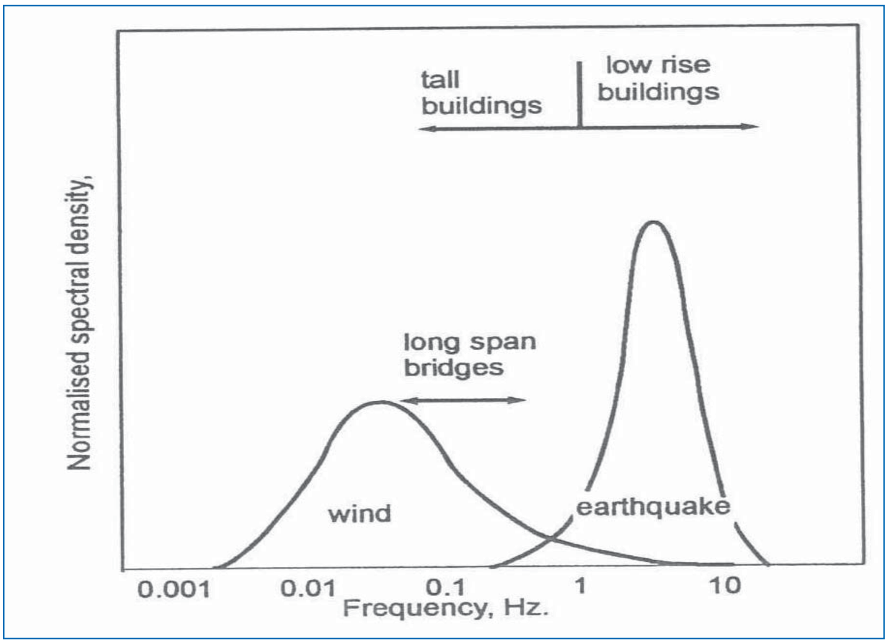
15th November 2014

PROFESSOR TANUJA BANDIVADEKAR
SARDAR PATEL COLLEGE OF ENGINEERING

- IS 875 : PART 3 code provisions
- Static and Dynamic methods of analysis
- Gust Factor
- Case study for two buildings :
  Wind load analysis for
  38 floors and 87 floors using IS code method.
- Wind tunnel test and results comparison
- CFD analysis and result comparison.

Necessity of Wind Engineering

- Wind load is usually governing than Seismic load for tall buildings.
- As the time period increases, Sa/g value decreases, and at the same time, as the structure becomes slender, wind forces becomes critical.
- Wind overturning moment will typically increase as height ^ 3, but the elastic seismic base moment is unlikely to increase at more than h^1.25.
- The design criteria w.r.t wind is to be strength based as well as Serviceability based.

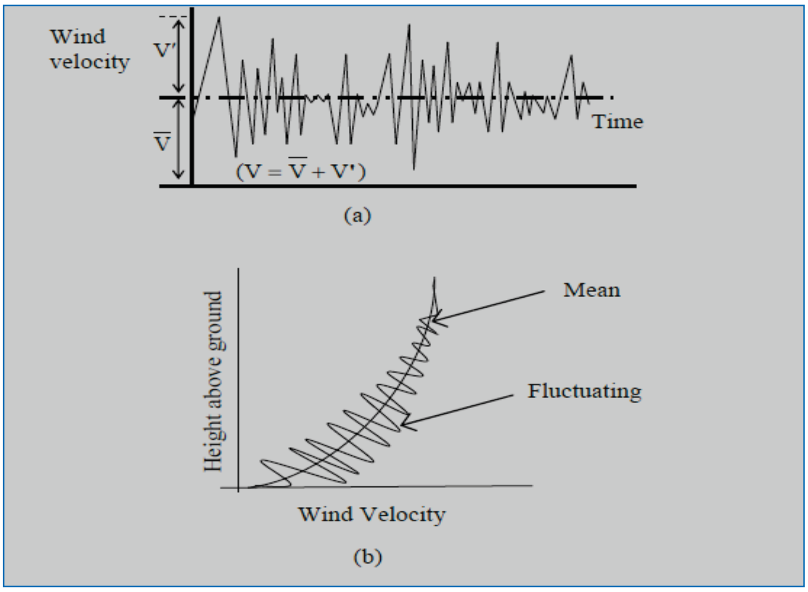


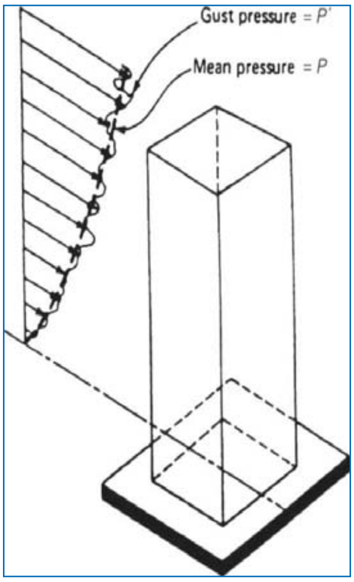
. DYNAMIC EXCITATION FREQUENCIES OF WIND AND EARTHQUAKE

Wind Load Effects

- There are 3 different types of effects on structures. Static, Dynamic and Aero Dynamic.
- When the structure deflects much in response to wind loads, then last two effects are to be considered.
- Aero elastic forces are substantial only if the structure is too slender like h/b >10, or too tall above 500m, or too light and tapered with steel as the medium of construction.
- So dynamic effects are to be studied in detail.

Wind : A randomly varying dynamic phenomenon

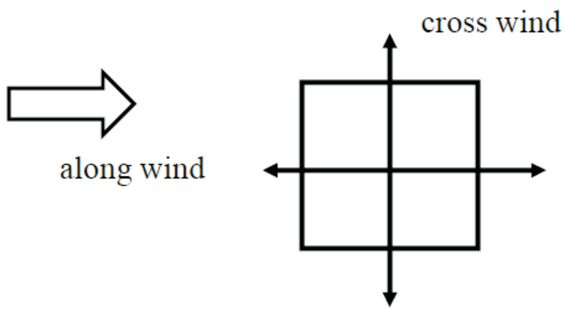


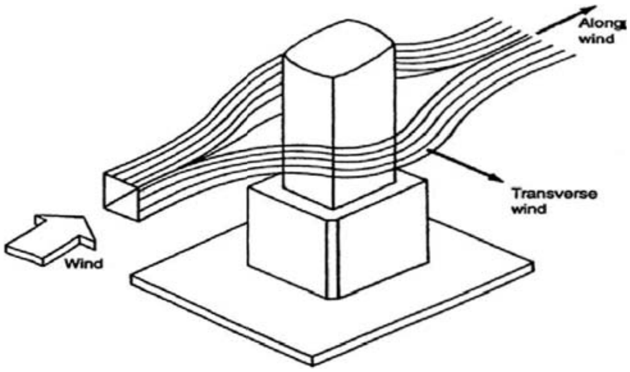


Gust : A positive or negative departure of wind speed from its mean value, lasting for not more than, say, 2 minutes over **a specified interval** of time.

Tall buildings

- Overall loading and dynamic response

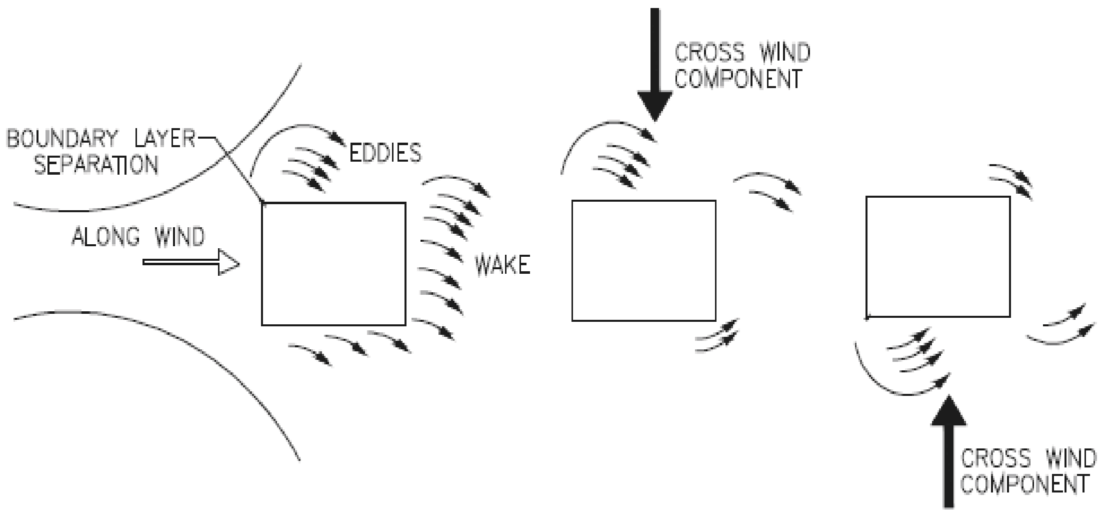




Cross-wind vibrations are usually greater than

along-wind vibrations for buildings of heights greater than 100m (330 feet)

The fig. below indicates how the vortices generate and thus create the across wind load components.



Dynamic Wind Effects

Wind Induced Oscillations

- Galloping
- Flutter
- Ovalling

Effects on Buildings

- Overturning effects
- Shearing effects
- Torsion effects
- Dynamically fluctuating loads on the overall building structure.

Wind Induced Oscillations

Galloping is transverse oscillations of some structures due to the development of aerodynamic forces which are in phase with the motion.

Flutter is unstable oscillatory motion of a structure due to coupling between aerodynamic force and elastic deformation of the structure.

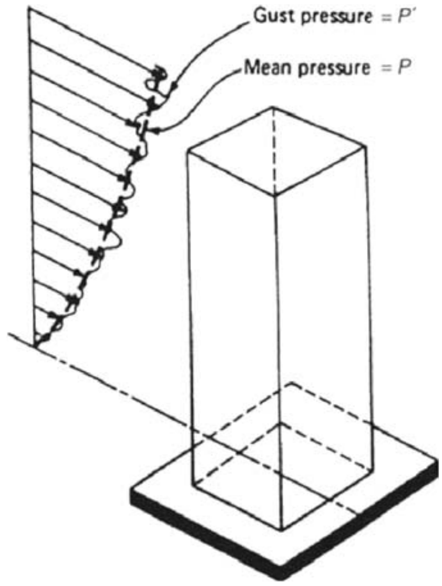
Ovalling: Thin walled structures with open ends at one or both ends such as oil storage tanks, and natural draught cooling towers are prone to ovalling oscillations.

The dynamic components which causes oscillations:

1. Gust 2. Vortex Shedding 3. Buffeting

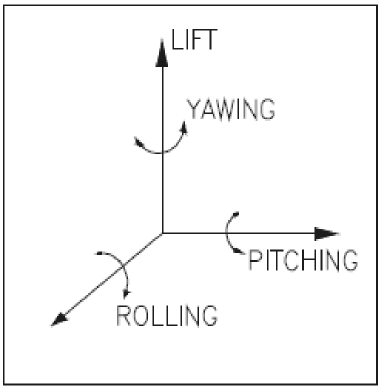
1 Gust. A positive or negative departure of wind speed from its mean value, lasting for not more than, say, 2 minutes over a specified interval of time.

The gust effect factor accounts only the effects in the along wind direction. It does not include allowances for across-wind loading effects, vortex shedding, instability due to galloping or flutter, or dynamic torsion effects.



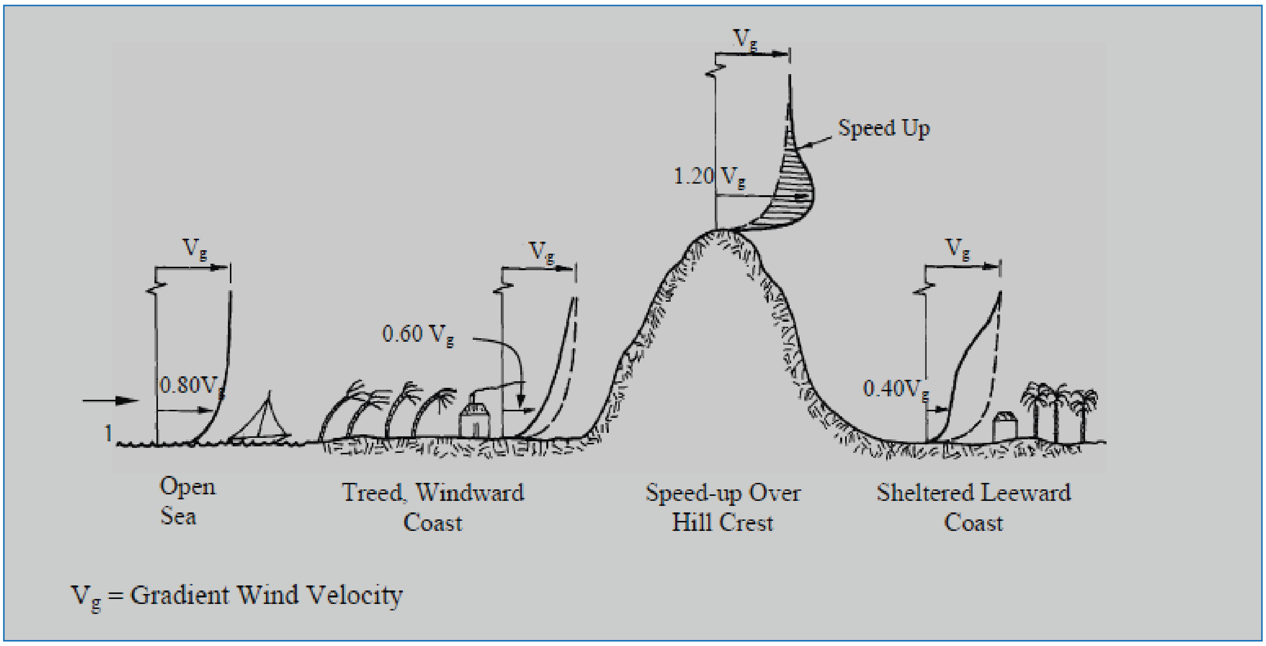
2) Vortex Shedding : When wind acts on a bluff body forces and moments in three mutually perpendicular direction are generated- out of which three are translation and three rotation. Mainly the flow of wind is considered two-dimensional consisting of along wind response and transverse wind response only.

Along wind response refer to drag forces, and transverse wind is the term used to describe crosswind. The crosswind response causing motion in a plane perpendicular to the direction of wind typically dominates over the along-wind response for tall buildings.



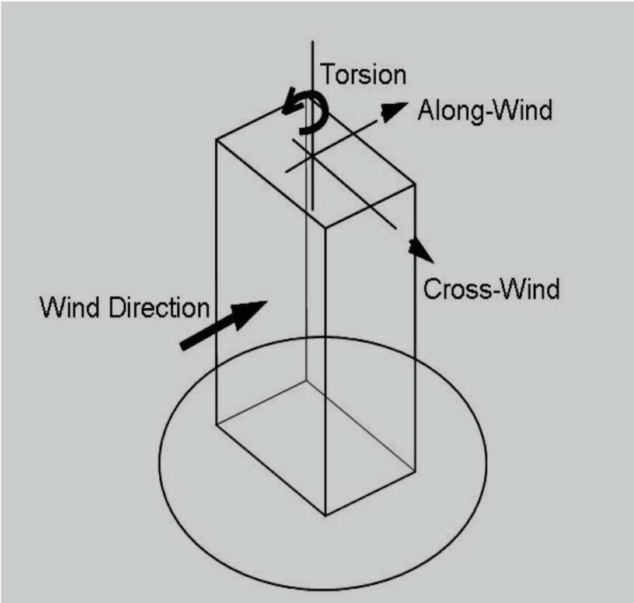
3) Buffeting : A downwind structure could oscillate due to vortex shedding of adjacent structure.
If one structure is located in the wake of another the vortices shed from the upstream may cause the oscillations of the downstream structure. This is called wake Buffeting

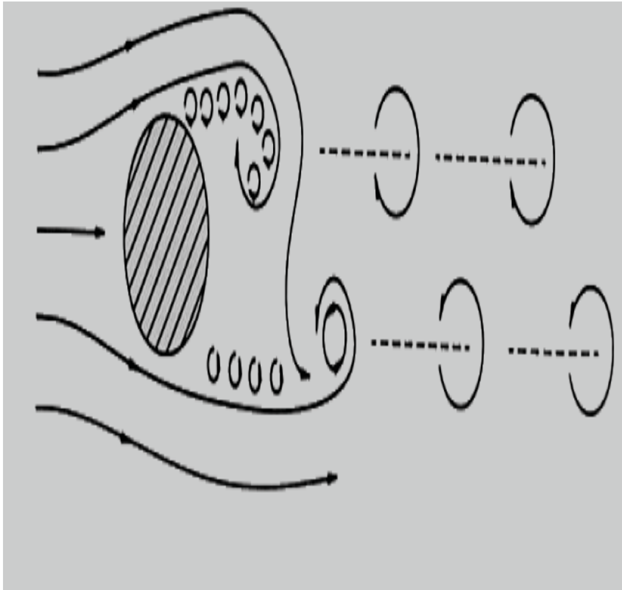
With in the earth's boundary layer, both components not only vary with height, but also depend upon the approach terrain and topography.



INFLUENCE OF TERRAIN AND TOPOGRAPHY

Most structures present bluff forms to the wind making it difficult to ascertain wind forces accurately.

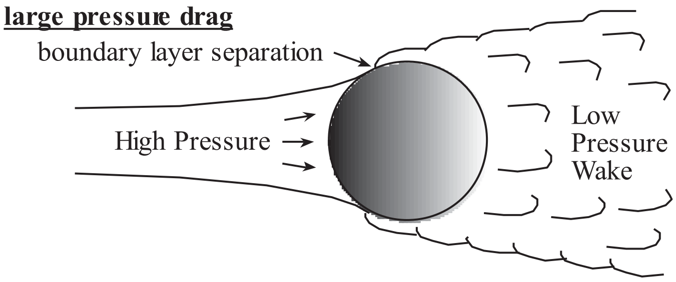




There are two main types of wind force on a tall structure – the first is its drag; that is the force measured along the wind direction, caused mainly by its resistance to the wind pressure on its face. The second is its cross-wind response; the static component is called lift (just like on the aeroplane wing) and the dynamic part is caused mainly by the vortex shedding. Special problems can occur if the regular frequency of the cross-wind vortex shedding coincides with the frequency of the structure's natural motion in that direction. Such response is relatively common and can occur at low (5 – 20 m/s) wind speeds, compared with "design" wind speeds. For example the famous failure at the Tecoma Narrows Bridge was at only 18 m/s.

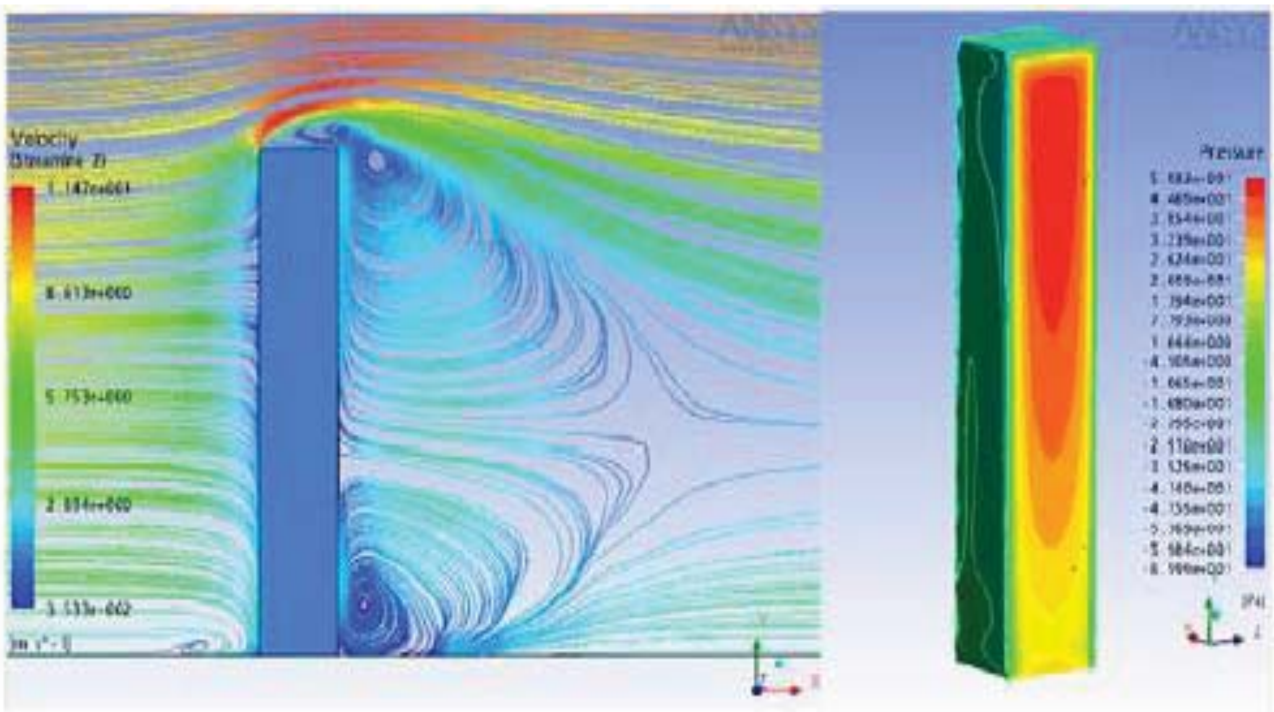
In non-cyclonic areas design wind speeds are around 40m/s and in cyclonic or typhoon areas up at about 60m/s.

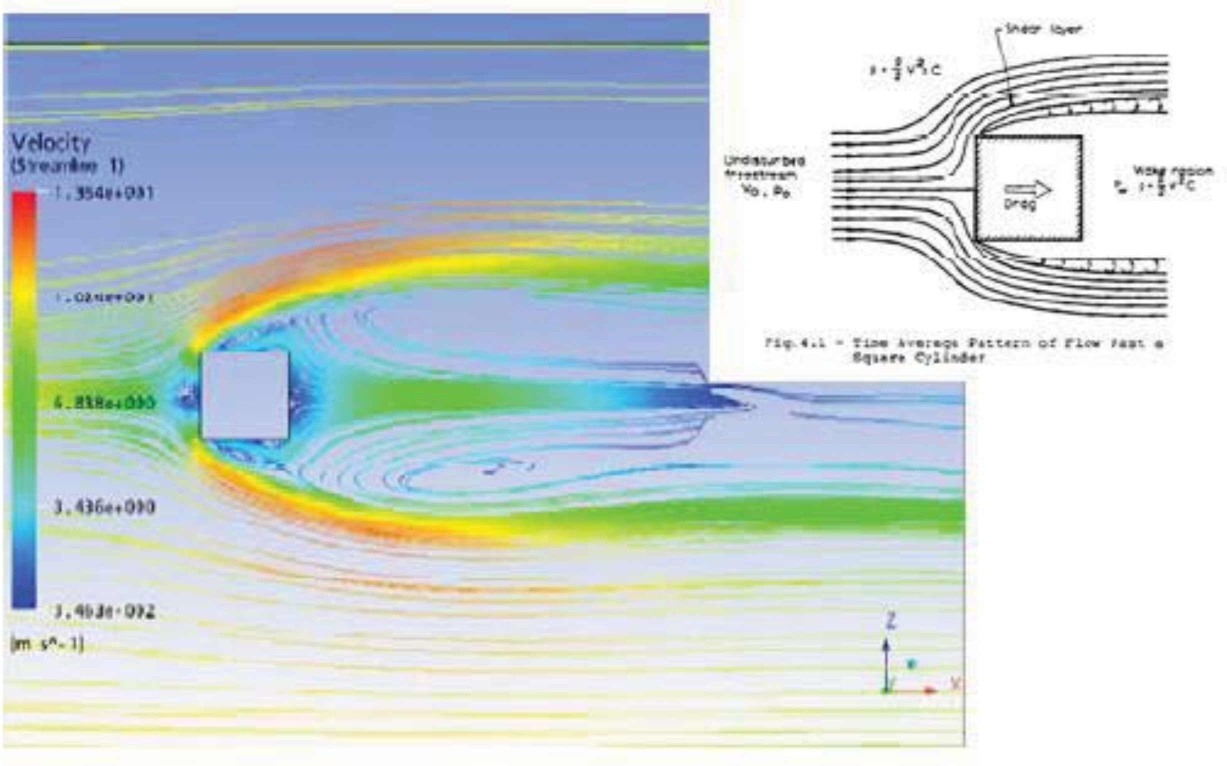
Non-aerodynamic shape

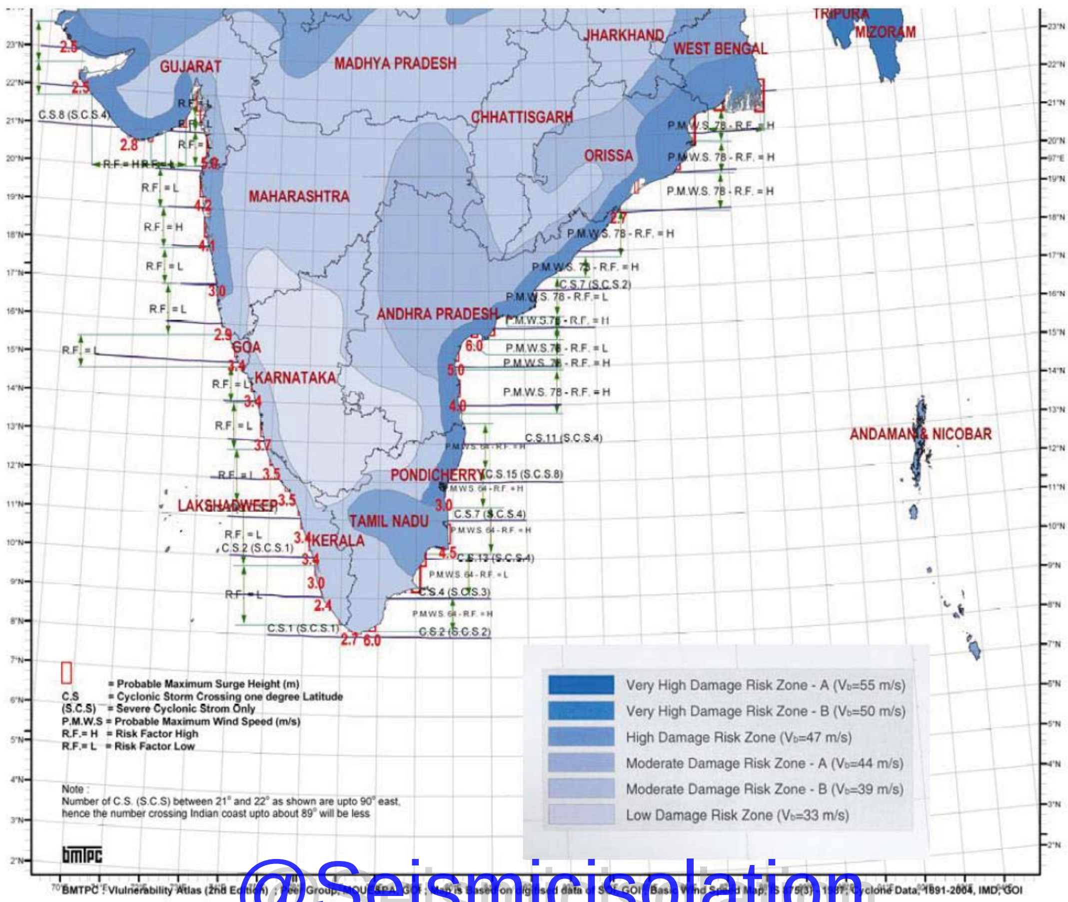


Aerodynamic shape

low pressure drag no separated flow region




Velocity
Shear layer
Undisturbed freestream
Drag
Wake region
Fig.4.1 - Time Average Pattern of Flow Past a Square Cylinder


GUJARAT
MADHYA PRADESH
JHARKHAND
WEST BENGAL
TRIPURA
MIZORAM
CHHATTISGARH
ORISSA
MAHARASHTRA
ANDHRA PRADESH
GOA
KARNATAKA
PONDICHERRY
TAMIL NADU
KERALA
LAKSHADWEEP
ANDAMAN & NICOBAR
= Probable Maximum Surge Height (m)
C.S = Cyclonic Storm Crossing one degree Latitude
(S.C.S) = Severe Cyclonic Strom Only
P.M.W.S = Probable Maximum Wind Speed (m/s)
R.F.= H = Risk Factor High
R.F.= L = Risk Factor Low
Note : Number of C.S. (S.C.S) between 21° and 22° as shown are upto 90° east, hence the number crossing Indian coast upto about 89° will be less
Very High Damage Risk Zone - A (Vb=55 m/s)
Very High Damage Risk Zone - B (Vb=50 m/s)
High Damage Risk Zone (Vb=47 m/s)
Moderate Damage Risk Zone - A (Vb=44 m/s)
Moderate Damage Risk Zone - B (Vb=39 m/s)
Low Damage Risk Zone (Vb=33 m/s)

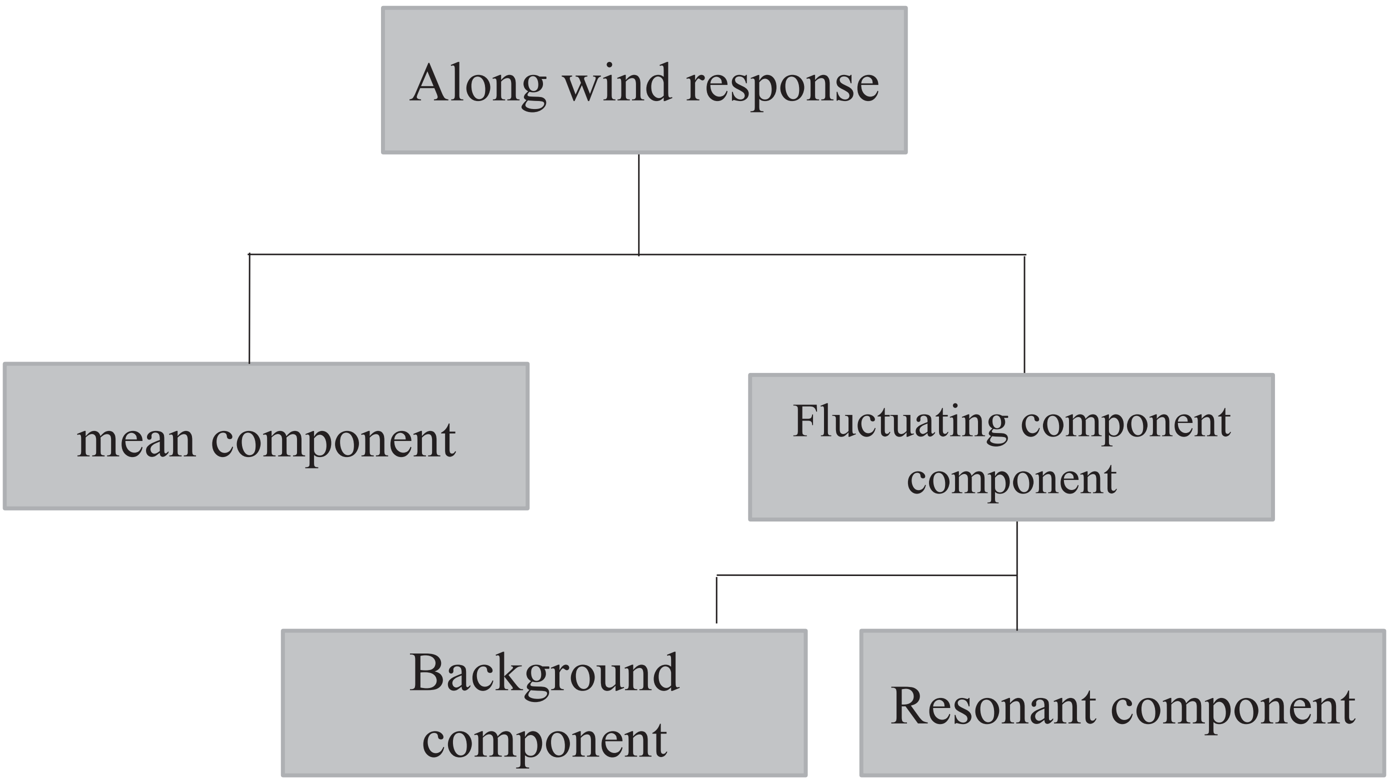
Following are some of the criteria that are important in designing for wind:

1. Strength and stability.
2. Fatigue in structural members and connections caused by fluctuating wind loads.
3. Excessive lateral deflection that may cause cracking of internal partitions and external cladding, misalignment of mechanical systems, and possible permanent deformations of nonstructural elements.
4. Frequency and amplitude of sway that can cause discomfort to occupants of tall, flexible buildings.
5. Possible buffeting that may increase the magnitude of wind velocities on neighboring buildings.
6. Wind-induced discomfort in pedestrian areas caused by intense surface winds.
7. Annoying acoustical disturbances.
8. Resonance of building oscillations with vibrations of elevator hoist ropes.

Table 3. Human perception levels

LEVEL	ACCELERATION (m/sec^2)	EFFECT
1	< 0.05	Humans cannot perceive motion
2	0.05 - 0.1	a) Sensitive people can perceive motion; b) hanging objects may move slightly
3	0.1 - 0.25	a) Majority of people will perceive motion; b) level of motion may affect desk work: c) long - term exposure may produce motion sickness
4	0.25 - 0.4	a) Desk work becomes difficult or almost impossible; b) ambulation still possible
5	0.4 - 0.5	a) People strongly perceive motion; b) difficult to walk naturally; c) standing people may lose balance.
6	0.5 - 0.6	Most people cannot tolerate motion and are unable to walk naturally
7	0.6 - 0.7	People cannot walk or tolerate motion.
8	> 0.85	Objects begin to fall and people may be injured

	Wind	Earthquake Effects
(1) Source of loading	External force due to wind pressure	Applied movements from ground vibration
(2) Type and duration of loading	Wind storm of several hours duration; loads fluctuate but predominantly in one direction	Transient cyclic loads of at most a few minutes duration; loads change direction repeatedly
(3) Predictability of loads	Usually good, by extrapolation of records or by analysis of site and wind patterns	Poor; little statistical certainty of magnitude of vibrations or their effects
(4) Influence of local soil conditions on response	Unimportant	Can be important
(5) Main factors affecting building response	External shape and size of building; dynamic properties unimportant except for very slender structures	Response governed by building dynamic properties: fundamental period and mass
(6) Normal design basis for maximum credible event	Elastic response required	Inelastic response permitted, but ductility must be provided; design is for a small fraction of the loads corresponding to elastic response
(7) Design of non-structural elements	Loading confined to external cladding	Entire building contents shaken and must be designed appropriately



Indian wind code calculates wind load from three different points of view;

(i) The building and structure taken as a whole;

(ii) Individual structural elements such as roofs and walls; and

(iii)Individual cladding units such as sheeting and glazing including their fixtures.

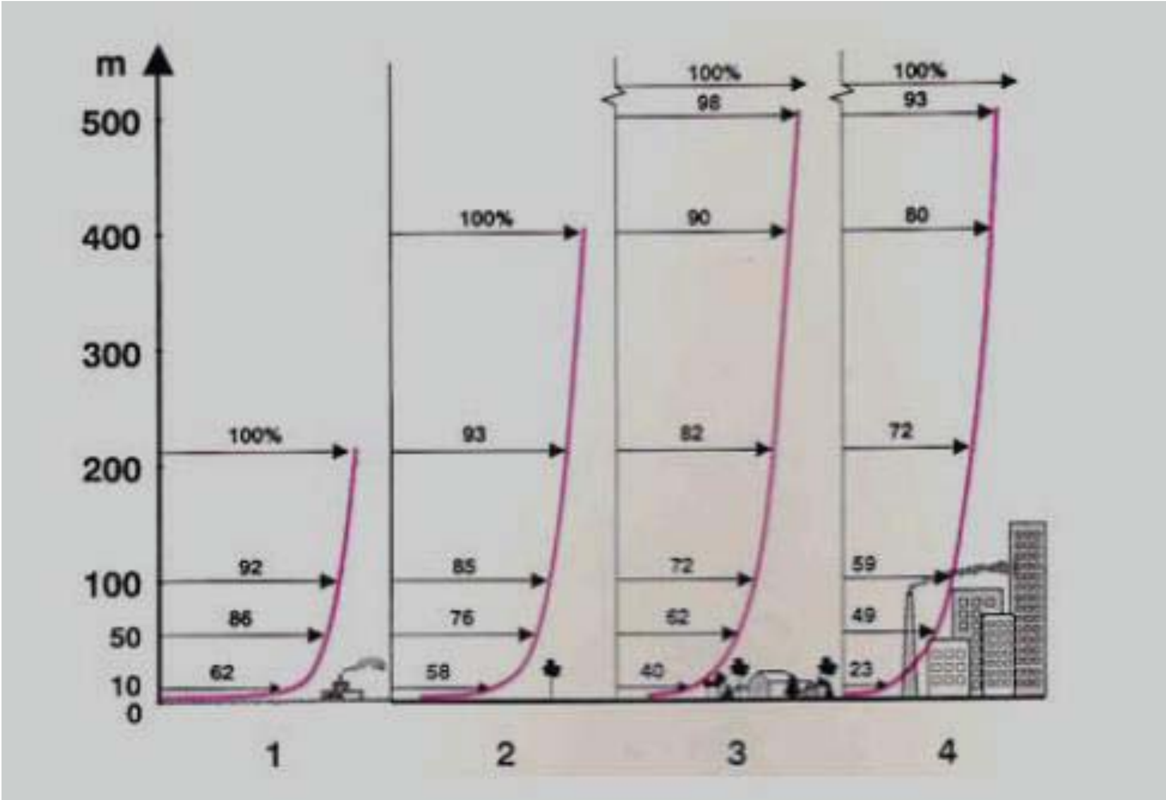
Considering the building and structure as a whole wind load can be calculated by using Force coefficient method
or Gust factor method depending on type of building or structure.

Three equations are used to calculate wind load according to the Force coefficient method and these are:

$$V_z = V_b\, k_1 k_2 k_3$$

$$p_z = 0.6 Vz^2$$

$$F = C_f A_e\, p_d$$



Boundary layer profile for different approach terrains

The dynamic response induced by the wind can be attributed to the following actions of wind:

(a) Non-correlation of the fluctuating along-wind pressures over the height and width of a structure.

(b) Resonant vibrations of a structure.

(c) Vortex shedding forces acting mainly in a direction normal to the direction of wind causing across-wind as well as torsional response.

Wind load on structures under the buffeting action of wind gusts have traditionally been treated by the “gust loading factor” (GLF)
Codes and Standards utilize the “gust loading factor” (GLF) approach for estimating dynamic effect on high-rise structures.

The concept of GLF was first introduced by Davenport in 1967

Indian wind code stipulates that buildings and structures with a height to minimum lateral dimension ratio of more than about 5.0, and buildings and structures whose natural frequency in the first mode is less than 1.0 Hz shall be examined for the dynamic effects of Wind.

The Gust factor method must be considered for the flexible buildings and the more severe of the two estimates, namely
1) by Gust factor method of load estimation and
2) by Static wind method of load estimation, is taken for design.

In this method hourly mean wind speed at any height at particular location is calculated similarly as prescribed by Eqn. (1), with only exception that the terrain category factor k2 has to be read from a separate table containing a relatively lower value.

At the same time, the lower frequency components of the wind speed and pressures have the greatest energy, so that the higher frequency modes of a structure would be subjected to lower excitation forces. Thus, generally the major dynamic response of a flexible structure due to wind is confined only to the fundamental mode of vibration of the structure.

TABLE 33 HOURLY MEAN WIND SPEED FACTOR k_2 IN DIFFERENT TERRAINS FOR DIFFERENT HEIGHTS

(Clauses 8.2 and 8.2.1)

HEIGHT m	TERRAIN			
	Category 1	Category 2	Category 3	Category 4
(1)	(2)	(3)	(4)	(5)
Up to 10	0·78	0·67	0·50	0·24
15	0·82	0·72	0·55	0·24
20	0·85	0·75	0·59	0·24
30	0·88	0·79	0·64	0·34
50	0·93	0·85	0·70	0·45
100	0·99	0·92	0·79	0·57
150	1·03	0·96	0·84	0·64
200	1·06	1·00	0·88	0·68
250	1·08	1·02	0·91	0·72
300	1·09	1·04	0·93	0·74
350	1·11	1·06	0·95	0·77
400	1·12	1·07	0·97	0·79
450	1·13	1·08	0·98	0·81
500	1·14	1·09	0·99	0·82

Further, the along wind load on a strip area (Ae) at any height (z) is given by Fz as follows.

Fz = Cf Ae z p G (4)

G = gust factor = (peak load/ mean load), and is given by

$$G = 1 + g_f r \sqrt{B(1+\phi)^2 + \frac{SE}{\beta}}$$

The effect of non-correlation of the peak pressures is considered by defining a **size reduction factor, S.**

It also accounts for the resonant and the non-resonant effects of the random wind forces.

The equation for G contains terms, one for the low frequency wind speed variations called the non-resonant or **'background'** effects, and the other for resonance effects. The first term accounts for the quasi-static dynamic response below the natural frequency of vibration of the structure.

The other term depends on the **gust energy** and aerodynamic admittance at the natural frequency of vibration as well as on the damping in the system.

$$B_s = \frac{1}{1 + \frac{\left[36(h-s)^2 + 64 b_{sh}{}^2\right]^{0.5}}{2L_h}}$$

$$S = \frac{1}{\left[1 + \frac{4 f_0 h (1 + g_v I_h)}{V_h}\right]\left[1 + \frac{4 f_0 b_{0h} (1 + g_v I_h)}{V_h}\right]}$$

E = (π/4) times the spectrum of turbulence in the approaching wind stream, given as follows:

$$E = \frac{\pi N}{\left(1 + 70 N^2\right)^{5/6}}$$

g_R = peak factor for resonant response (1 hour period) given by:

$$g_R = \sqrt{\left[2 \log_e (3600 f_0)\right]}$$

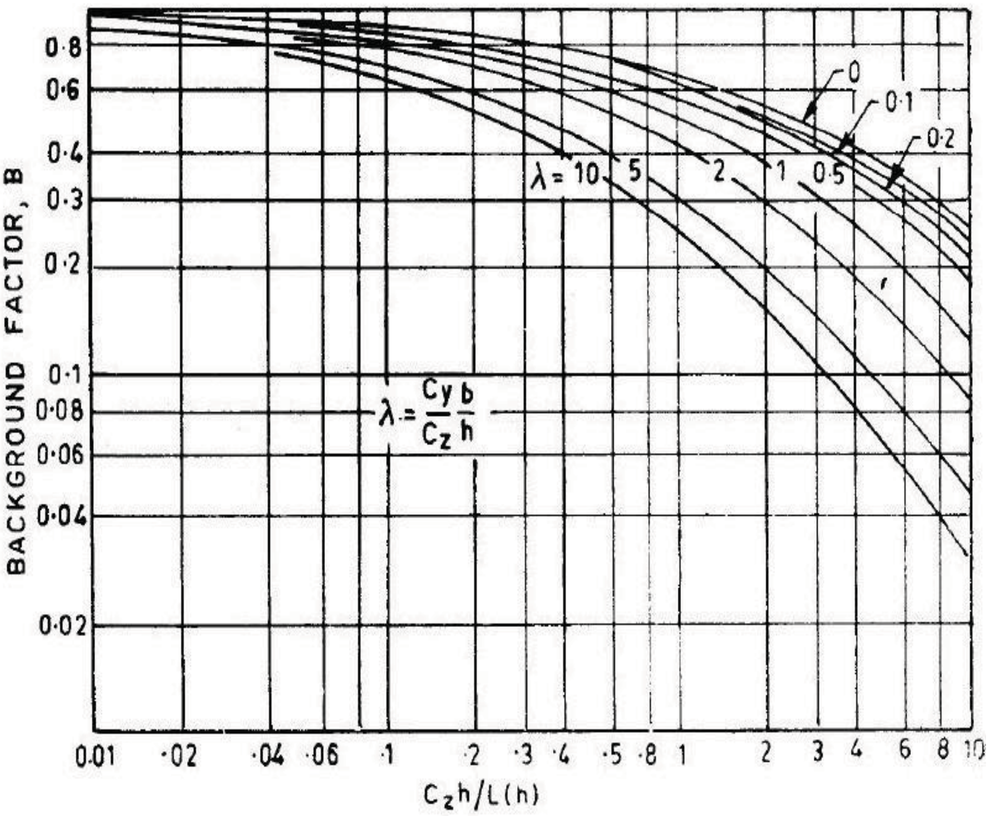


FIG. 9 BACKGROUND FACTOR *B*

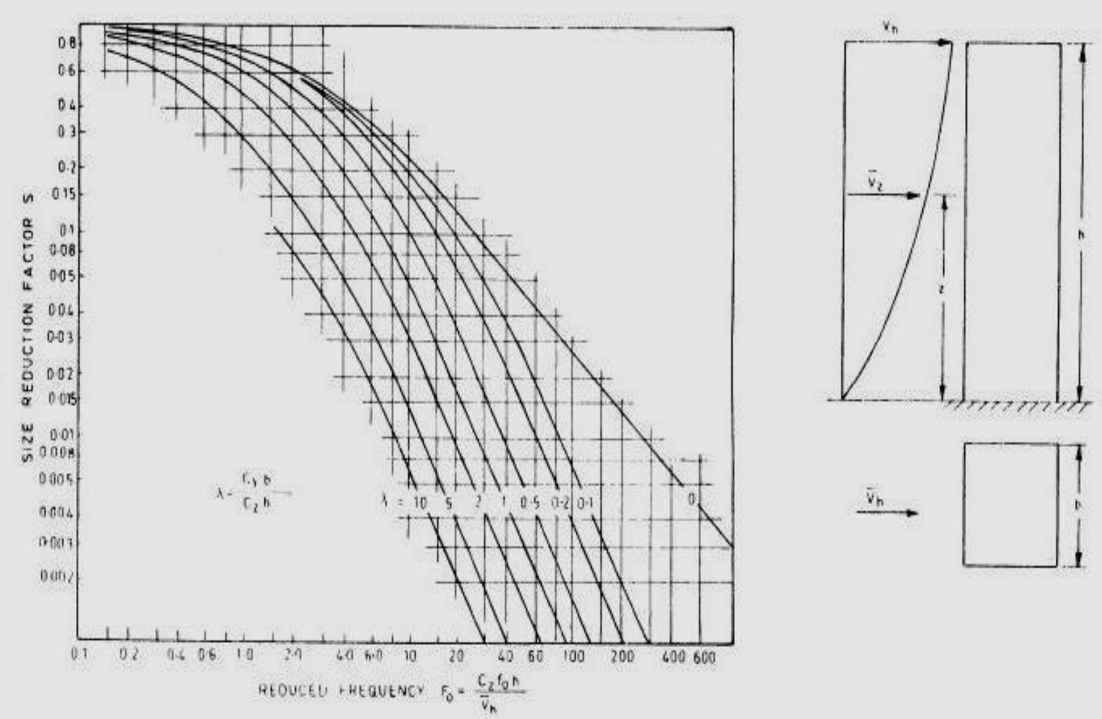


FIG. 10 SIZE REDUCTION FACTOR, *S*

IS : 875 (Part 3) - 19

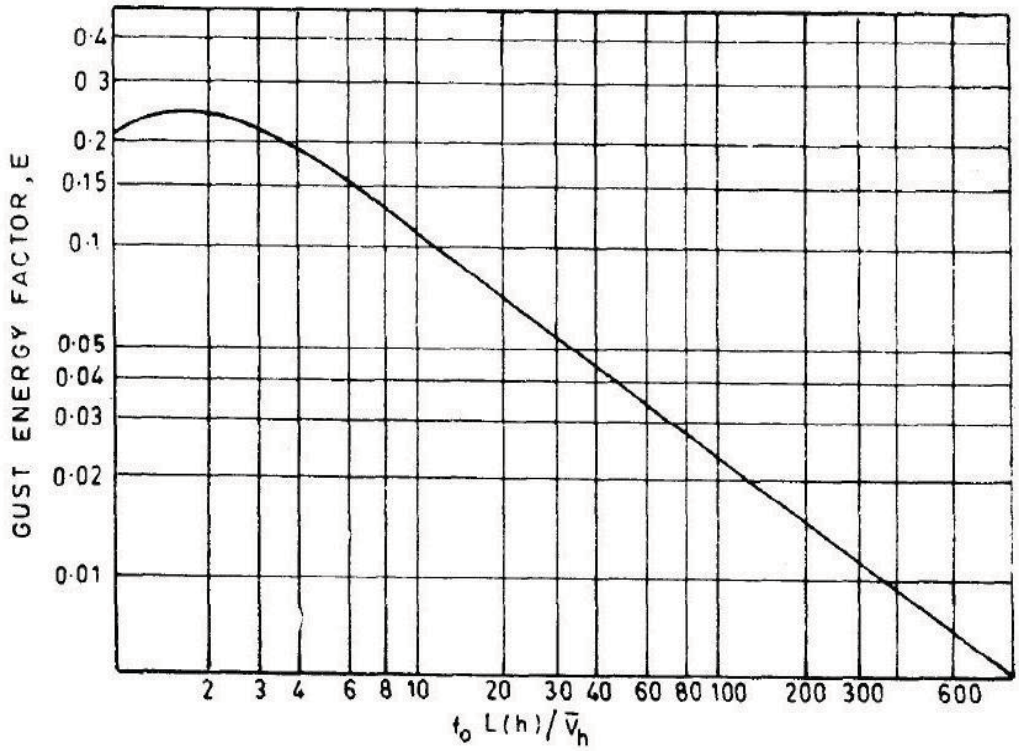


FIG. 11 GUST ENERGY FACTOR, E

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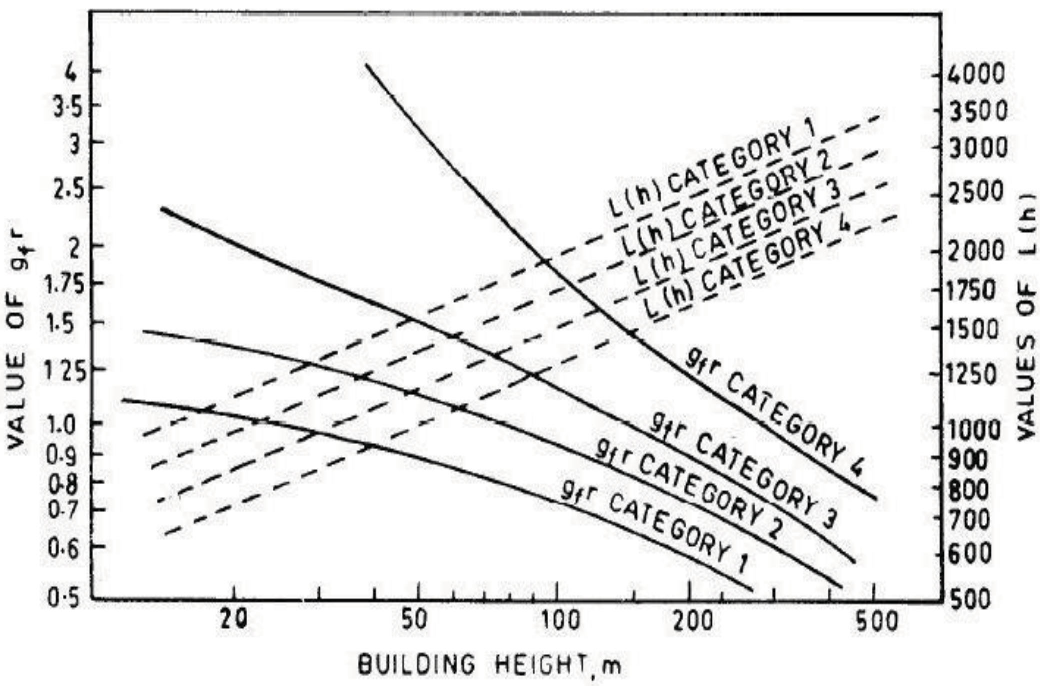

FIG. 8 VALUES OF $g_f r$ AND L (h)

L_h = measure of the integral turbulence length scale at height h = 100 (h/10)$^{0.25}$

Nature of Structure	Damping Coefficient, β
Welded steel structures	0.010
Bolted steel structures	0.020
Reinforced or prestressed concrete	0.020

The peak acceleration along the wind direction at the top of the structure
is given as:

$$a = (2\pi f_o)^2 \bar{x} g_f r \sqrt{\frac{SE}{\beta}}$$

Acceleration is assumed to be produced only by the resonant component of the response hence second term under the square root has been considered.

Only the first mode response is assumed to dominate . The mode shape is assumed linear. Hence acceleration also varies linearly with height.

Slender Structures

the eddy shedding frequency η shall be determined by the following formula:

where $\eta = \frac{S_r V_z}{b}$

V_z = design wind speed, and b = breadth of a structure or structural member normal to the wind direction as well as the axis of the structure/member.

a) Circular Structures – For structures circular in cross-section:
S_r = 0.20 for bV_z not greater than 7, and
= 0.25 for bV_z greater than 7.
b) For rectangular cross section:
S_r = 0.15 for all values of bV_z.

Wake Excitation : It is the most common type of across-wind excitation and is caused by shedding of the vortices by a structure at regular intervals alternately from its two opposite sides. The periodicity of eddy shedding is defined by Strouhal Number that depends on the shape of cross-section of the structure. Resonance would result when the frequency of eddy shedding matches the natural frequency of vibration of the structure. This would give rise to large amplitudes of vibration which are limited only by the damping present in the system. In case of tall structures the wind speed as well as turbulence vary with the height of structure. The latter is spread over a band of frequencies. For this reason wake excitation includes also the response due to non-resonant frequencies. The draft Code describes the methods of computing the cross-wind response at resonant wind speeds due to wake excitation

Cross-wind response of tall enclosed buildings and towers of rectangular cross-section

Equivalent static wind force
The equivalent cross–wind static force per unit height (W^e) *as a function of z in Newton per meter height, shall be as follows:*

$W^e(z) = 0.6\,[V^h]^2\, d\, C^{dyn}$
where d = Lateral dimension of the structure parallel to the wind stream, and

$$C_{dyn} = 1.5 g_R \left(\frac{b}{d}\right) \frac{K_m}{(1+g_v I_h)^2} \left(\frac{z}{h}\right)^k \sqrt{\frac{\pi C_{fs}}{\beta}}$$

where
K^m *= mode shape correction factor for cross–wind acceleration*
= 0.76 + 0.24 k

where

k = mode shape power exponent for the fundamental mode of vibration
= 1.5 for a uniform cantilever
= 0.5 for a slender framed structure (moment resistant)
= 1.0 for building with central core and moment resisting façade
= 2.3 for a tower decreasing in stiffness with height, or with a large mass at the top
C^{fs} *= cross–wind force spectrum coefficient generalized for a linear mode shape*

G^R, g^v, I^h and β are the same as for along wind calculations.

Cross–wind base overturning moment and acceleration

The cross–wind base overturning moment (M^0) in Newton – meters is given by:

$$M_0 = 0.5 g_R b \left[\frac{0.6(V_z)^2}{(1+g_v I_h)^2}\right] h^2 \left(\frac{3}{k+2}\right) K_m \sqrt{\frac{\pi C_{fs}}{\beta}}$$

where the value $\left(\frac{3}{k+2}\right) K_m$ is the mode shape correction factor for cross–wind base overturning moment.

The peak acceleration at top of the building is given as :

$$\ddot{y}_h = \frac{0.90\, b g_R}{m_0}\left[\frac{V_h}{1+g_v I_h}\right]^2 K_m \sqrt{\frac{\pi C_{fs}}{\beta}}$$

where m_0 = mass per unit height.

Wind Load Evaluation

The wind load acting on a structure can be evaluated by

1. Analytical method (Code specific)

    a. quasi- static methods - (static method)

    b. With the dynamic effects – (Gust method)

2. Experimental methods (Wind Tunnel)

    a. For cladding study

    b. HFFB method

    c. HFPI method

    d. Aero-elastic model study

When the structure is stiff, the response due to the wind loads will be minuscule and quasi – static method can be adopted.

But if the structure is flexible, dynamic parameters like natural frequency and damping comes in to the picture .So dynamic effects needs to be considered in evaluating the wind forces. Code stipulates some methods for doing so. Also it lays down two checks for the need of it like frequency and slenderness ratio.

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Code has stipulated following provisions to check for the dynamic effects

1. The slenderness ratio is more than 5

2. The natural frequency in the first mode is less than 1Hz.

Wind Tunnel Method

Wind tunnels are widely used to reliably predict the wind loading on the cladding and glazing as well as on the structural frames of tall buildings.

It account for building geometry, local climate and surrounding details and this leads to cost effective and accurate wind loading on cladding and structural frames of tall buildings.

Wind tunnel model studies offer the best estimate of the wind loading acting on a building for cladding as well as the structural frame design.

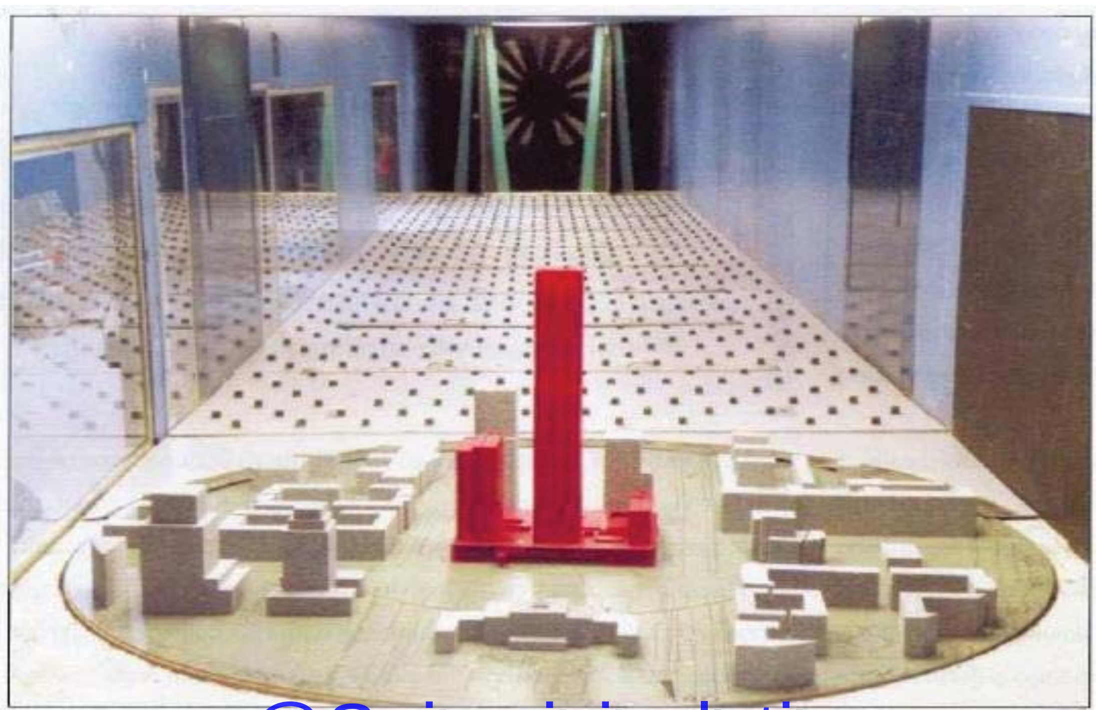
Types of wind tunnel studies

- Wind Engineering Studies
- Topographical Studies
- Cladding Wind Load Study
- Wind-Induced Response of Structures
- Structural Wind Load Study
- Wind Effects on Bridges
- Wind Effects on Special Structures

Methodology of wind Tunnel Testing

- A prototype Model with Scale of 1:300 to 1:500 is Prepared
- The Geometry / Shape of the model should reflect the original structure.
- Elevation features up to 1ft dimension are well taken care of.
- Buildings surrounding half a KM of the Structure are also to be modeled
- Proper sensors are mounted on the model at convenient and strategic location to monitor the dynamic effects
- The model is rotated and loads are applied for every 10 degree rotation. This accounts for the fact that the structure may get wind load from all possible directions.

Wind Tunnel Testing setup



The loads derived from Code analytical methods are often approximate, as they are based on box shaped buildings in isolated conditions.

Unlike wind tunnel tests, Codes have difficulty accounting for project specific factors such as:

The aerodynamic effect of the actual shape of the structure

The influence of adjacent buildings and topography

Detailed wind directionality effects.

- Aero elastic interaction between the structural motion and airflow.

- Torsional moments and also the load combinations with along and cross loads.

- Modal coupling effects between sway modes, torsion and combination of all.

For the reasons stated earlier, it is prudent to do the experimental tests for accurately finding out the wind load estimation.

The present study is done on two different structures with varying geometry, stiffness and height, designed for Mumbai conditions.

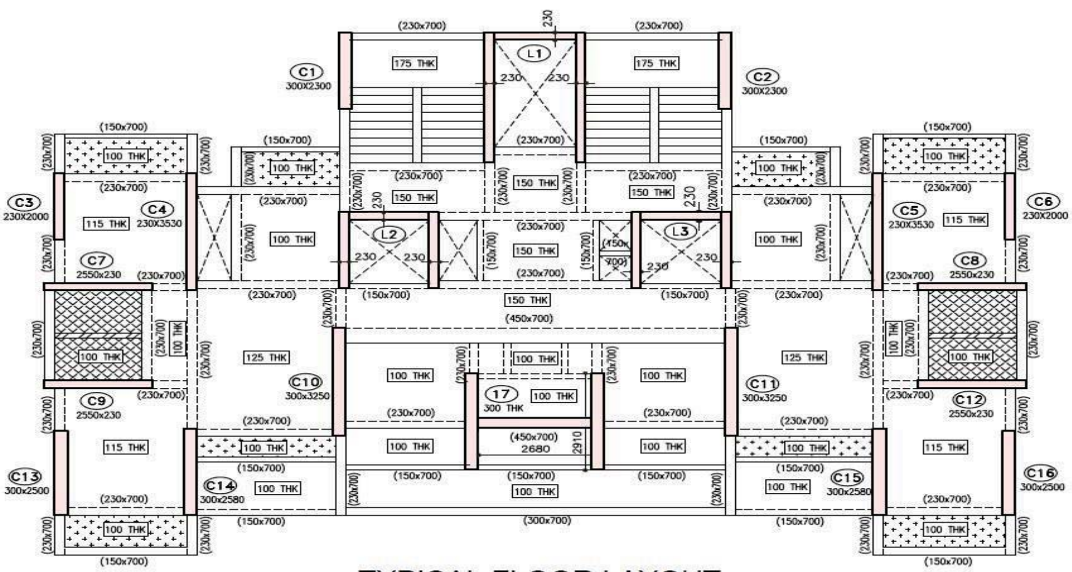
The code specific wind forces and their effect on the structure will be compared against that from the wind tunnel tests.
The need of the tunnel tests can be reviewed from the difference in the structural responses of both the buildings.

CASE STUDY 1
Structure height 38 floors 129m

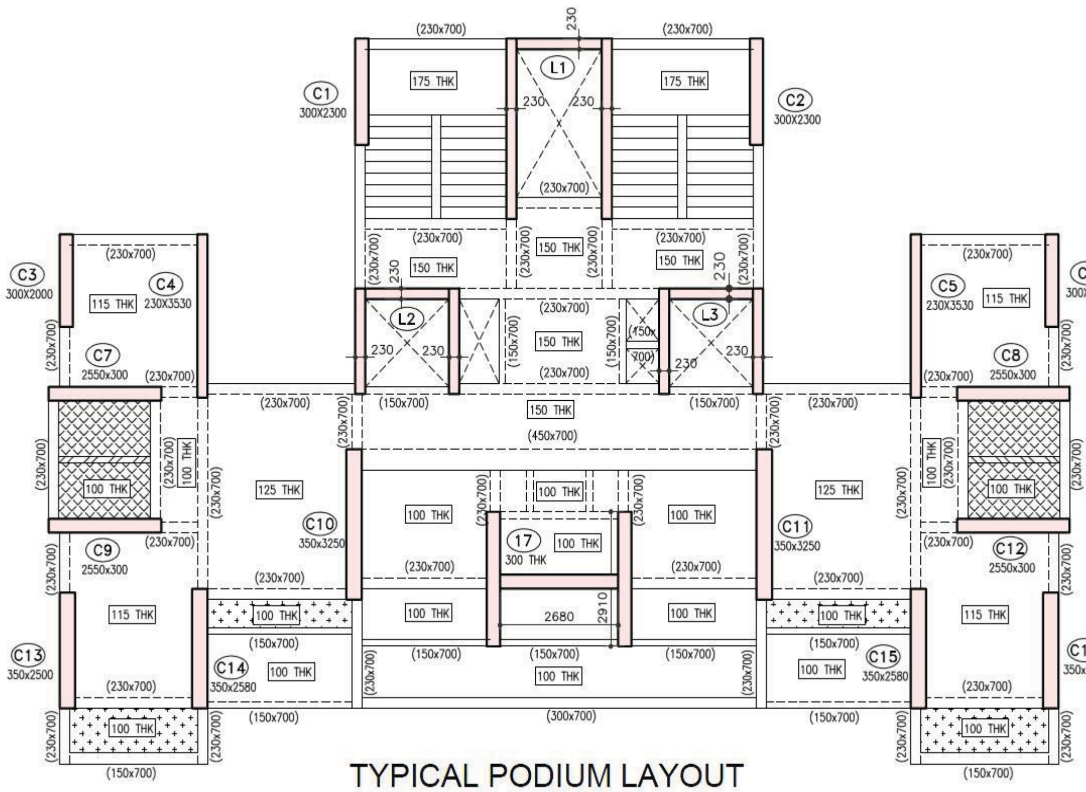
CASE STUDY 2
Structure height 87 floors 302m

CASE STUDY 1 – TOWER- A

The salient features of the structure are as follows.



TYPICAL FLOOR LAYOUT
COLUMN SIZES ARE SHOWN AS OF 10TH FLOOR.



TYPICAL PODIUM LAYOUT

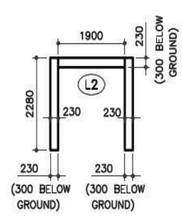
COLUMN SIZES ARE SHOWN AS OF 2ND PODIUM FLOOR.

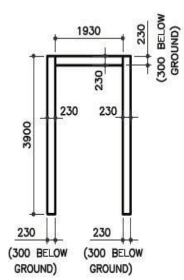
COLUMN SCHEDULE

GRADE	COLUMN NOS.	C1,C2	C3,C6	C4,C5	C7,C8 C9,C12	C10,C11	C13,C16	C14,C15	C17
M25	38TH FLOOR TO 23RD FLOOR	300x2300	230x2000	230x3530	230x2550	300x3250	300x2500	300x2580	300 THK
M40	22ND FLOOR TO 9TH FLOOR	300x2300	230x2000	230x3530	230x2550	300x3250	300x2500	300x2580	300 THK
M50	8TH FLOOR	300x2300	230x2000	230x3530	230x2550	300x3250	300x2500	300x2580	300 THK
	7TH FLOOR	300x2300	230x2000	230x3530	230x2550	300x3250	300x2500	300x2580	300 THK
	6TH FLOOR	300x2300	230x2000	230x3530	230x2550	300x3250	300x2500	300x2580	300 THK
	5TH PODIUM	300x2300	300x2000	230x3530	300x2550	350x3250	350x2500	350x2580	300 THK
	4TH PODIUM	300x2300	300x2000	230x3530	300x2550	350x3250	350x2500	350x2580	300 THK
	3RD PODIUM	300x2300	300x2000	230x3530	300x2550	350x3250	350x2500	350x2580	300 THK
	2ND PODIUM	300x2300	300x2000	230x3530	300x2550	350x3250	350x2500	350x2580	300 THK
	1ST PODIUM	300x2300	300x2000	230x3530	300x2550	350x3250	350x2500	350x2580	300 THK
	GROUND FLOOR	300x2300	300x2000	230x3530	300x2550	350x3250	350x2500	350x2580	300 THK
	BELOW GROUND LVL.	350x2350	350x2050	300x3580	350x2600	400x3300	400x2550	400x2630	350 THK

COLUMN SCHEDULE

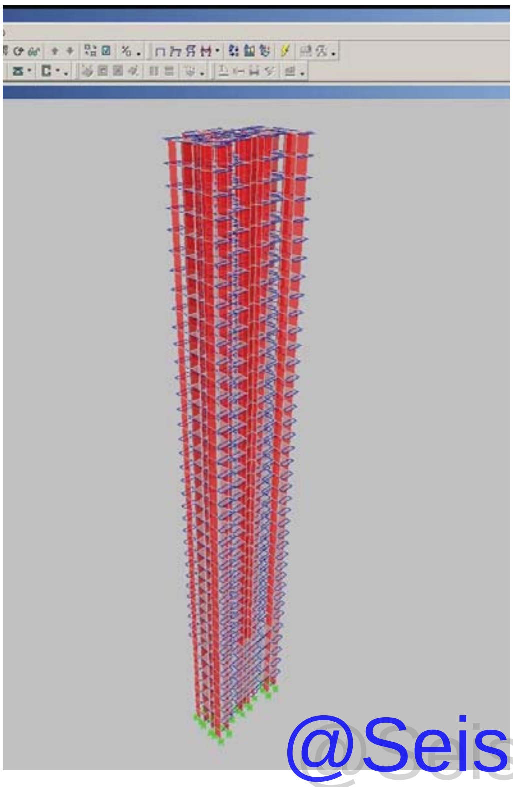
GRADE	COLUMN NOS.	L2,L3	L1
M25	38TH FLOOR TO 23RD FLOOR		
M30	22ND FLOOR TO 16TH FLOOR		
M40	15TH FLOOR TO 9TH FLOOR		
M50	8TH FLOOR		
	7TH FLOOR		
	6TH FLOOR		
	5TH PODIUM		
	4TH PODIUM		
	3RD PODIUM		
	2ND PODIUM		
	1ST PODIUM		
	GROUND FLOOR		
	BELOW GROUND LVL.		

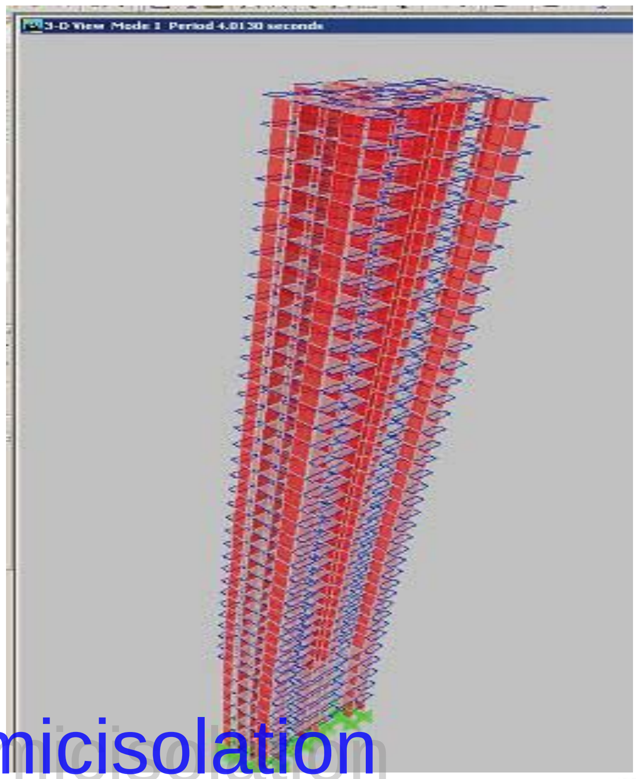




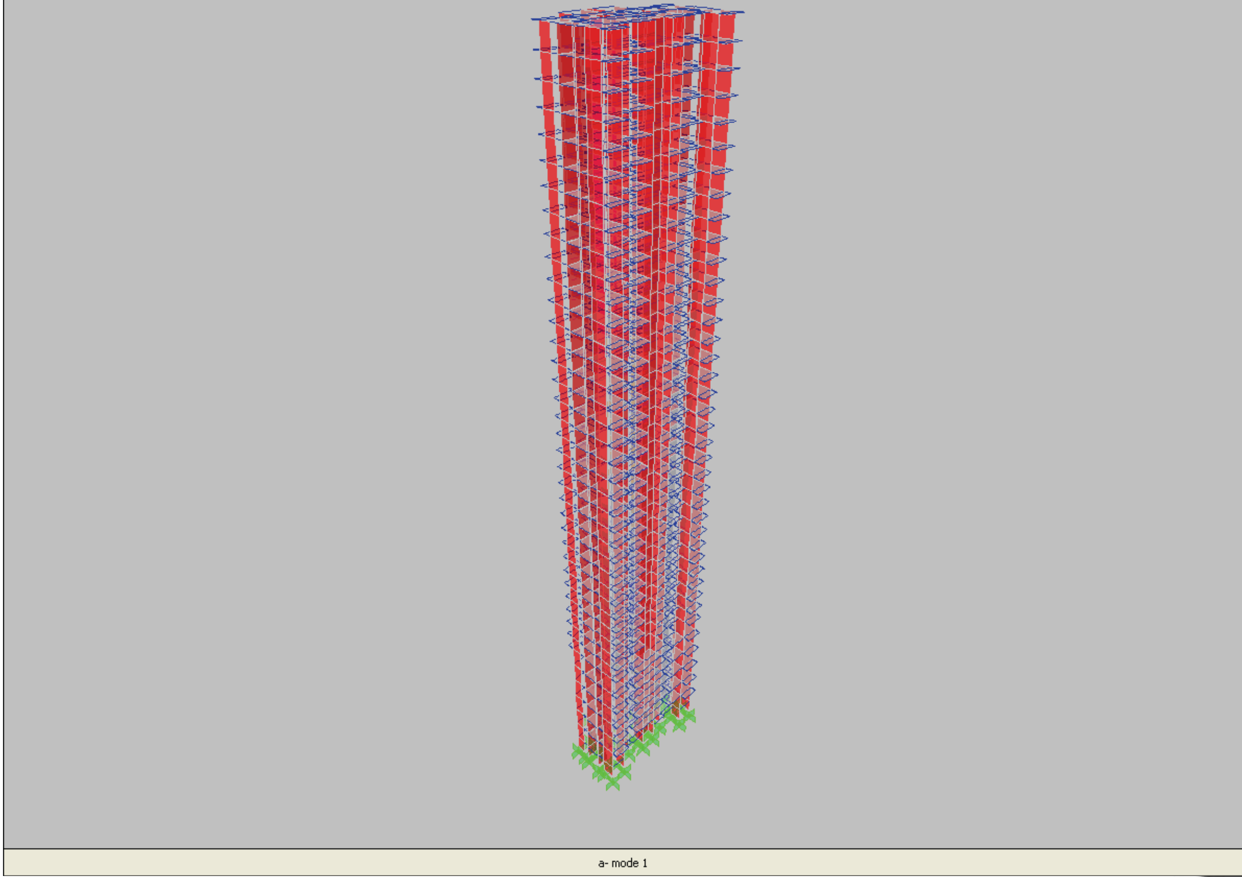
TOWER A DETAILS		
Max. Length X direction	23.15	m
Max. width Y direction	15.5	m
No. of Floors	G+38	
Floor height @ typical floors	3.05	m
Floor height @ typical Podium floors	4.2	m
Floor height @ Ground level	4.35	m
Total height of the structure	129	m
Foundation Type	Raft	
Soil type	Type 1	
General live load	0.2 T/m2	
Masonry type	Brick	

The analysis software used is ETABS Non Linear 1 Version 9.7.3





Tower A –First mode 4.01s UX =0, UY =68.7, RZ =0



ALT1 . WIND LOAD EVALUATION BY STATIC METHOD

In this method, the force F acting in a specified direction of wind is given as

F = Cf. Ae.Pd , where

Ae = the effective frontal area of the structure
Cf = Force Coefficient for the building and
Pd = Design wind force on the structure.

Design wind pressure Pd is given in clause 5.4 of the code as

Pd = 0.6 Vz^2 where
Vz = Vb.k1.k2.k3

Vz = Design wind pressure at any height z in m/s
K1 = Probability factor (clause 5.3.1)
K2 = Terrain, height and structure size factor (clause 5.3.2)
K3 = topography factor (clause 5.3.3)

WIND FORCES STATIC ANALYSIS - TOWER A

LEVEL	X DIX.	Y DIX.	LEVEL	X DIX.	Y DIX.
TRFL	8.85	14.06	19FL	8.52	13.54
38FL	9.85	15.66	18FL	8.44	13.41
37FL	9.8	15.58	17FL	8.36	13.29
36FL	9.75	15.49	16FL	8.28	13.16
35FL	9.69	15.41	15FL	8.2	13.04
34FL	9.64	15.32	14FL	8.11	12.89
33FL	9.59	15.24	13FL	7.97	12.67
32FL	9.54	15.16	12FL	7.83	12.44
31FL	9.48	15.07	11FL	7.69	12.22
30FL	9.41	14.96	10FL	7.55	11.99
29FL	9.33	14.83	9FL	7.41	11.77
28FL	9.25	14.7	8FL	7.27	11.55
27FL	9.17	14.57	7FL	7.08	11.25
26FL	9.08	14.44	6FL	8.12	12.9
25FL	9	14.31	5FL	9.01	14.32
24FL	8.92	14.18	4FL	8.44	13.41
23FL	8.84	14.05	3FL	7.73	12.29
22FL	8.76	13.92	2FL	7.27	11.56
21FL	8.68	13.79	1FL	7.36	11.71
20FL	8.6	13.66	GRFL	3.46	5.96

TOWER A DIAPHRAGM CM DISPLACEMENTS – STATIC – TOWER A

Story	Load	UX	UY
TRFL	WINDX	0.107	0.000
TRFL	WINDY	0.000	0.198
38FL	WINDX	0.106	0.000
38FL	WINDY	0.000	0.193
37FL	WINDX	0.104	0.000
37FL	WINDY	0.000	0.188
36FL	WINDX	0.102	0.000
36FL	WINDY	0.000	0.183
35FL	WINDX	0.100	0.000
35FL	WINDY	0.000	0.178
34FL	WINDX	0.099	0.000
34FL	WINDY	0.000	0.173
33FL	WINDX	0.097	0.000
33FL	WINDY	0.000	0.168
32FL	WINDX	0.095	0.000
32FL	WINDY	0.000	0.162
31FL	WINDX	0.093	0.000

ALT2 . WIND LOAD EVALUATION BY GUST FACTOR METHOD

Along wind load on a structure of strip area Ae and height z is given as

Fz = Cf. Ae. Pz.G, where

Cf = Force coefficient of the building
Ae =Effective frontal area considered for the structure at height z,
Pz = Design pressure at height z due to hourly mean wind obtained as 0.6Vz2,

and
Gust factor G =1+gfr. sq.rt (B (1+φ) ^2+SE/β), where

gf = peak factor defined as the ratio of the expected peak value to the root mean value of a fluctuating load, and

TOWER A - WIND FORCES: GUST FACTOR, FORCE COEFFICIENT,ACCELERATION AND WIND FORCES AT ALL LEVELS - PART 1/2						
Size X Direction (m)	23.15	Category	3		*X Dix*	*Y Dix*
Size Y Direction (m)	15.5	Class	C	a/b	1.49	0.67
Basic Wind speed (m/s)	44	K1	1	h/b	8.13	5.44
gfr	1.1	K3	1	Force Coeff. (Cf)	1.4	1.5
L(h)	1625	Cy	10	Time period (Modal)	3.92	4.01
β	0.016	Cz	12	f0 (Natural freq.)	0.26	0.25

TOWER A - WIND FORCES: GUST FACTOR, FORCE COEFFICIENT,ACCELERATION AND WIND FORCES AT ALL LEVELS - PART 2/2

	X Dix	Y Dix		X Dix	Y Dix		
λ = Cyb/Czh	0.10	0.15	Deflection at the Roof level (mm)	133.4	244.4		
Cz.h/L(h)	0.9	0.9	Acceleration (m/s2)	0.377	0.617		
Back gr.Factor (B)	0.65	0.65					

GUST CALCULATION TOWER A

LEVEL	X- Dim	Y-Dim	HEIGHT	TOTAL HEIGHT	K2	Vz* (m/s)	Pz* (N/m2)	F0 = Cz.f0.h / Vz*		Size redn.Factor (S)		f0.L(h) / Vz*		Gust Ene.Fact. (E)		φ = (gf.r*√B)/4		Gust Factor G.		Wind Force (T)	
								X dix	Y dix	X dix	Y dix	X dix	Y dix	X dix	Y dix	X dix	Y dix	X dix	Y dix	X dix	Y dix
TRFL	23.15	15.5	3.05	126	0.80	35.22	744	11.0	10.7	0.16	0.14	11.8	11.5	0.10	0.10	0.22	0.22	2.54	2.49	12.53	19.66
FL38	23.15	15.5	3.05	123	0.80	35.16	742	11.0	10.7	0.16	0.14	11.8	11.5	0.10	0.10	0.22	0.22	2.54	2.49	12.49	19.60
FL37	23.15	15.5	3.05	120	0.80	35.11	740	11.0	10.7	0.16	0.14	11.8	11.5	0.10	0.10	0.22	0.22	2.54	2.49	12.45	19.54
FL36	23.15	15.5	3.05	117	0.80	35.06	737	11.0	10.8	0.16	0.14	11.8	11.6	0.10	0.10	0.22	0.22	2.54	2.49	12.42	19.48
FL35	23.15	15.5	3.05	114	0.80	35.00	735	11.0	10.8	0.16	0.14	11.8	11.6	0.10	0.10	0.22	0.22	2.54	2.49	12.38	19.42
FL34	23.15	15.5	3.05	111	0.79	34.95	733	11.0	10.8	0.16	0.14	11.9	11.6	0.10	0.10	0.22	0.22	2.54	2.49	12.34	19.36
FL33	23.15	15.5	3.05	108	0.79	34.90	731	11.1	10.8	0.16	0.14	11.9	11.6	0.10	0.10	0.22	0.22	2.54	2.49	12.30	19.30
FL32	23.15	15.5	3.05	105	0.79	34.84	728	11.1	10.8	0.16	0.14	11.9	11.6	0.10	0.10	0.22	0.22	2.54	2.49	12.26	19.24

WIND FORCES GUST FACTOR ANALYSIS						
LEVEL	X DIX.	Y DIX.		LEVEL	X DIX.	Y DIX.
TRFL	12.53	19.66		19FL	10.08	15.83
38FL	12.49	19.60		18FL	9.93	15.59
37FL	12.45	19.54		17FL	9.78	15.36
36FL	12.42	19.48		16FL	9.63	15.12
35FL	12.38	19.42		15FL	9.48	14.89
34FL	12.34	19.36		14FL	9.33	14.65
33FL	12.30	19.30		13FL	9.09	14.27
32FL	12.26	19.24		12FL	8.85	13.89
31FL	12.23	19.18		11FL	8.61	13.52
30FL	11.94	18.75		10FL	8.30	13.02
29FL	11.77	18.49		9FL	8.07	12.67
28FL	11.61	18.23		8FL	7.85	12.31
27FL	11.44	17.97		7FL	7.55	11.85
26FL	11.28	17.72		6FL	9.75	15.33
25FL	11.12	17.47		5FL	9.10	14.30
24FL	10.96	17.21		4FL	8.20	12.89
23FL	10.80	16.97		3FL	7.13	11.21
22FL	10.64	16.72		2FL	6.41	10.07
21FL	10.39	16.31		1FL	6.64	10.43
20FL	10.24	16.07		GRFL	0.00	0.00

TOWER A -GUST DISPLACEMENTS		
Story	UX	UY
TRFL	0.1334	0.2444
38FL	0.1313	0.2381
37FL	0.1292	0.2319
36FL	0.127	0.2256
35FL	0.1248	0.2192
34FL	0.1224	0.2128
33FL	0.12	0.2063
32FL	0.1174	0.1997
31FL	0.1148	0.1931
30FL	0.112	0.1864
29FL	0.1092	0.1797
28FL	0.1062	0.1729
27FL	0.1032	0.1661
26FL	0.1	0.1592
25FL	0.0968	0.1523
24FL	0.0934	0.1454
23FL	0.09	0.1386
22FL	0.0866	0.1317
21FL	0.0831	0.1249
20FL	0.0795	0.1181
19FL	0.0759	0.1113
18FL	0.0722	0.1046
17FL	0.0685	0.0979
16FL	0.0647	0.0913
15FL	0.0609	0.0847

Rowan Williams Davies & Irwin Inc. (RWDI) has conducted the wind tunnel study of this project.

The model was tested in the presence of all surroundings within a full-scale radius of 1600 ft, in RWDI's 8 ft × 6.5 ft boundary layer wind tunnel facility in Guelph, Ontario, Canada.

The influence of the upwind terrain on the planetary boundary layer was simulated by appropriate roughness on the wind tunnel floor and flow conditioning spires at the upwind end of the working section for each wind direction.

Methodology of wind Tunnel Testing

- The model was tested in the presence of all surroundings within a full-scale radius of 1600 ft, in RWDI’s 8 ft × 6.5 ft boundary lay er wind tunnel facility in Guelph, Ontario
- A prototype Model with Scale of 1:400 is Prepared.
- Elevation features up to 1ft dimension are well taken care of.
- Buildings surrounding half a KM radius of the Structure are also modeled
- Proper sensors are mounted on the model at convenient and stra tegic location to monitor the dynamic effects

- The model is rotated and loads are applied for every 10 degree rotati on. This accounts for the fact that the structure may get wind load fr om all possible directions.
- The Damping ratio is taken as 2% of critical.
- The fundamental building vibration frequencies were used for finding out the loads on the structures.
- The measurements of forces and moments are made at the base for each 10 deg wind attack angle.
- Thereafter, the forces and torsional moment will be distributed over t he building height based on width, height, mass distribution, mode s hapes and computed acceleration.

TOWER –A – STRUCTURAL DYNAMIC PROPERTIES

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ETABS v9.7.3  File:STATIC-3  Units:Ton-m  April 9, 2013 20:22  PAGE 2

M O D A L   P E R I O D S   A N D   F R E Q U E N C I E S

MODE                   PERIOD          FREQUENCY       CIRCULAR FREQ
NUMBER                 (TIME)        (CYCLES/TIME)    (RADIANS/TIME)

Mode 1                4.01297          0.24919           1.56572
Mode 2                3.91768          0.25525           1.60380
Mode 3                2.90717          0.34398           2.16127
Mode 4                1.18665          0.84271           5.29489
Mode 5                1.01351          0.98667           6.19946
Mode 6                0.90231          1.10827           6.96346
Mode 7                0.59592          1.67809          10.54372
Mode 8                0.47055          2.12516          13.35278
Mode 9                0.46774          2.13794          13.43307
Mode 10               0.37618          2.65829          16.70256
Mode 11               0.30021          3.33105          20.92958
Mode 12               0.28840          3.46735          21.78600

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ETABS v9.7.3  File:STATIC-3  Units:Ton-m  April 9, 2013 20:22  PAGE 4

M O D A L   P A R T I C I P A T I N G   M A S S   R A T I O S

MODE            X-TRANS       Y-TRANS       Z-TRANS       RX-ROTN       RY-ROTN       RZ-ROTN
NUMBER        %MASS <SUM>   %MASS <SUM>   %MASS <SUM>   %MASS <SUM>   %MASS <SUM>   %MASS <SUM>

Mode 1         0.03 <  0>   68.69 < 69>    0.00 <  0>   98.71 < 99>    0.04 <  0>    0.00 <  0>
Mode 2        70.35 < 70>    0.03 < 69>    0.00 <  0>    0.04 < 99>   94.15 < 94>    3.75 <  4>
Mode 3         4.07 < 74>    0.00 < 69>    0.00 <  0>    0.00 < 99>    5.58 <100>   72.67 < 76>
Mode 4        12.38 < 87>    0.00 < 69>    0.00 <  0>    0.00 < 99>    0.04 <100>    0.85 < 77>
Mode 5         0.00 < 87>   18.29 < 87>    0.00 <  0>    1.07 <100>    0.00 <100>    0.00 < 77>
Mode 6         0.85 < 88>    0.00 < 87>    0.00 <  0>    0.00 <100>    0.00 <100>   10.53 < 88>
Mode 7         3.50 < 91>    0.00 < 87>    0.00 <  0>    0.00 <100>    0.13 <100>    0.51 < 88>
Mode 8         0.33 < 92>    0.01 < 87>    0.00 <  0>    0.00 <100>    0.02 <100>    3.30 < 92>
Mode 9         0.00 < 92>    4.38 < 91>    0.00 <  0>    0.12 <100>    0.00 <100>    0.00 < 92>
Mode 10        1.88 < 93>    0.00 < 91>    0.00 <  0>    0.00 <100>    0.01 <100>    0.30 < 92>
Mode 11        0.17 < 94>    0.00 < 91>    0.00 <  0>    0.00 <100>    0.00 <100>    1.73 < 94>
Mode 12        0.00 < 94>    2.14 < 94>    0.00 <  0>    0.02 <100>    0.00 <100>    0.00 < 94>
```


EQUIVALENT-STATIC - FORCES

Floor	Fx (T)	Fy'(T)	Mz (T- m)
TRFL	18.7	33.1	129.8
38 FL	15.2	26.6	101.7
37 FL	14.8	25.7	94.7
36 FL	14.6	25.1	97.1
35 FL	14.3	24.2	90.2
34 FL	15.0	24.8	96.7
33 FL	14.6	23.9	90.0
32 FL	14.4	23.2	91.1
31 FL	14.0	22.2	85.0
30 FL	13.2	20.9	82.6
12 FL	5.8	5.9	28.1
11 FL	5.7	5.5	25.6
10 FL	5.2	4.7	23.4
9 FL	5.1	4.3	21.2
8 FL	4.7	3.6	19.2
7 FL	4.5	3.1	16.2
6 FL	5.2	2.9	17.1
5 FL	5.2	2.3	15.3
4 FL	4.7	2.1	12.7
3 FL	4.2	2.1	10.1
2 FL	3.8	2.1	8.0
1 FL	3.6	2.2	6.3
GRFL	1.7	1.0	2.6

PROPOSED LOAD COMBINATIONS

Case	X Forces (Fx)	Y Forces (Fy)	Torsion (Mz)
1	100%	45%	45%
2	100%	45%	-45%
3	100%	-30%	45%
4	100%	-30%	-45%
5	-100%	45%	35%
6	-100%	45%	-45%
7	-100%	-30%	35%
8	-100%	-30%	-45%
9	30%	100%	30%
10	30%	100%	-45%
11	30%	-100%	30%
12	30%	-100%	-45%
13	-50%	100%	30%
14	-50%	100%	-45%
15	-50%	-100%	30%
16	-50%	-100%	-45%
17	45%	40%	85%
18	30%	40%	-100%
19	45%	-40%	85%
20	30%	-40%	-100%
21	-55%	40%	85%
22	-60%	40%	-100%
23	-55%	-40%	85%
24	-60%	-40%	-100%

Load combination factors have been produced through consideration of the structure's response to various wind directions, modal coupling, correlation of wind gusts and the directionality of strong winds in the local wind climate.

THERE WILL BE 24 LOAD COMBINATIONS !!

Level	Fx (T)	Fy (T)	Mz (T- m)	Fx (T)	Fy (T)	Mz (T- m)
	WIND 1			WIND 2		
40	18.7	14.9	58.4	18.7	14.9	-58.4
39	15.2	12.0	45.8	15.2	12.0	-45.8
38	14.8	11.6	42.6	14.8	11.6	-42.6
37	14.6	11.3	43.7	14.6	11.3	-43.7
36	14.3	10.9	40.6	14.3	10.9	-40.6
35	15.0	11.2	43.5	15.0	11.2	-43.5
34	14.6	10.7	40.5	14.6	10.7	-40.5
33	14.4	10.4	41.0	14.4	10.4	-41.0

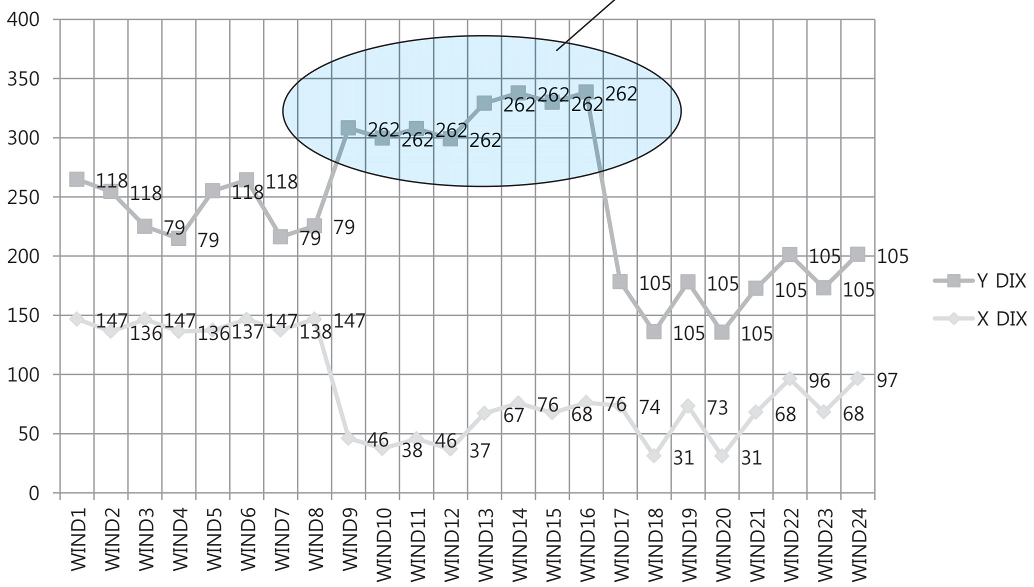
Level	Fx (T)	Fy (T)	Mz (T- m)	Fx (T)	Fy (T)	Mz (T- m)
	WIND 5			WIND 6		
40	-18.7	14.9	45.4	-18.7	14.9	-58.4
39	-15.2	12.0	35.6	-15.2	12.0	-45.8
38	-14.8	11.6	33.1	-14.8	11.6	-42.6
37	-14.6	11.3	34.0	-14.6	11.3	-43.7
36	-14.3	10.9	31.6	-14.3	10.9	-40.6
35	-15.0	11.2	33.8	-15.0	11.2	-43.5
34	-14.6	10.7	31.5	-14.6	10.7	-40.5
33	-14.4	10.4	31.9	-14.4	10.4	-41.0
32	-14.0	10.0	29.8	-14.0	10.0	-38.3

Level	Fx (T)	Fy (T)	Mz T- m)	Fx (T)	Fy (T)	Mz(T- m)
	WIND 9			WIND 10		
40	5.6	33.1	38.9	5.6	33.1	-58.4
39	4.5	26.6	30.5	4.5	26.6	-45.8
38	4.4	25.7	28.4	4.4	25.7	-42.6
37	4.4	25.1	29.1	4.4	25.1	-43.7
36	4.3	24.2	27.1	4.3	24.2	-40.6
35	4.5	24.8	29.0	4.5	24.8	-43.5
34	4.4	23.9	27.0	4.4	23.9	-40.5
33	4.3	23.2	27.3	4.3	23.2	-41.0
32	4.2	22.2	25.5	4.2	22.2	-38.3
31	4.0	20.9	24.8	4.0	20.9	-37.2
30	4.0	20.5	23.9	4.0	20.5	-35.9

Level	Fx (T)	Fy (T)	Mz (T- m)	Fx (T)	Fy (T)	Mz (T- m)
	WIND 23			WIND 24		
40	-10.3	-13.2	110.3	-11.2	-13.2	-129.8
39	-8.3	-10.7	86.4	-9.1	-10.7	-101.7
38	-8.1	-10.3	80.5	-8.9	-10.3	-94.7
37	-8.1	-10.0	82.5	-8.8	-10.0	-97.1
36	-7.9	-9.7	76.7	-8.6	-9.7	-90.2
35	-8.3	-9.9	82.2	-9.0	-9.9	-96.7
34	-8.0	-9.5	76.5	-8.8	-9.5	-90.0
33	-7.9	-9.3	77.4	-8.7	-9.3	-91.1
32	-7.7	-8.9	72.3	-8.4	-8.9	-85.0
31	-7.3	-8.3	70.2	-7.9	-8.3	-82.6

TOWER A

LOADS 9 TO 16 ARE GOVERNING



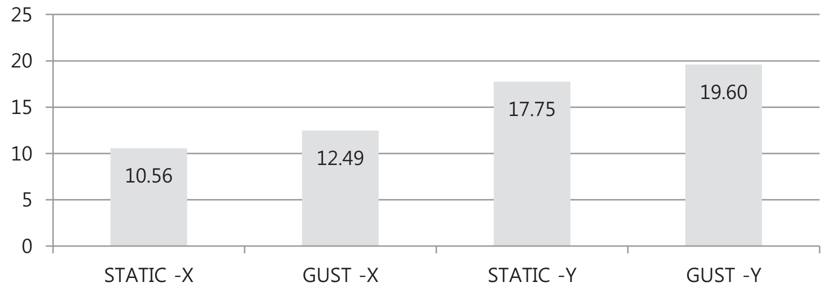
TOWER A - ROOF DISPLACEMENTS FOR TUNNEL LOADING in mm.

TOWER A - TUNNEL - DISPLACEMENTS

LOAD	X DIX	Y DIX
WIND1	147	118
WIND2	136	118
WIND3	147	79
WIND4	136	79
WIND5	137	118
WIND6	147	118
WIND7	138	79
WIND8	147	79
WIND9	46	262
WIND10	38	262
WIND11	46	262
WIND12	37	262
WIND13	67	262
WIND14	76	262
WIND15	68	262
WIND16	76	262
WIND17	74	105
WIND18	31	105
WIND19	73	105
WIND20	31	105
WIND21	68	105
WIND22	96	105
WIND23	68	105
WIND24	97	105

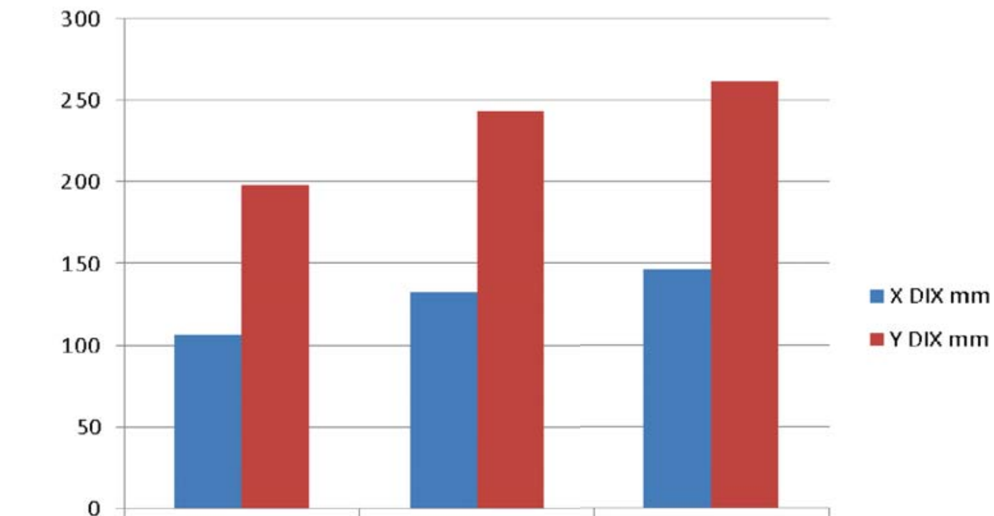
COMPARISON OF RESULTS

TOWER A -38FL LOADS IN Ton.

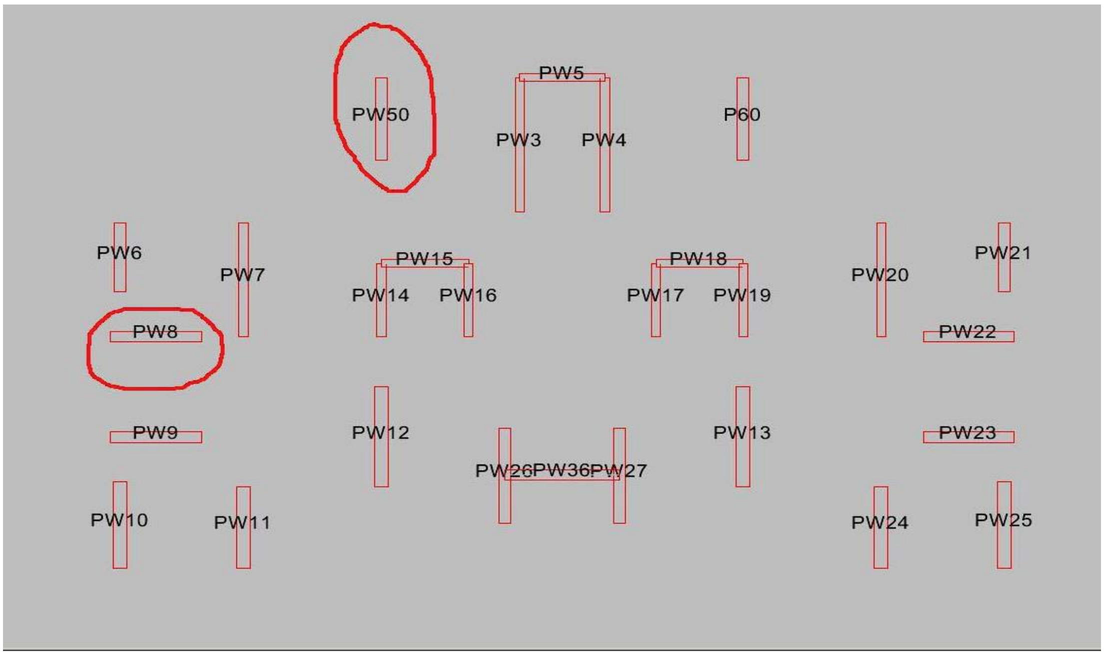


- There is 10% to 20% increment in wind forces in GUST method over STATIC. But 25% more structural response in Gust method.

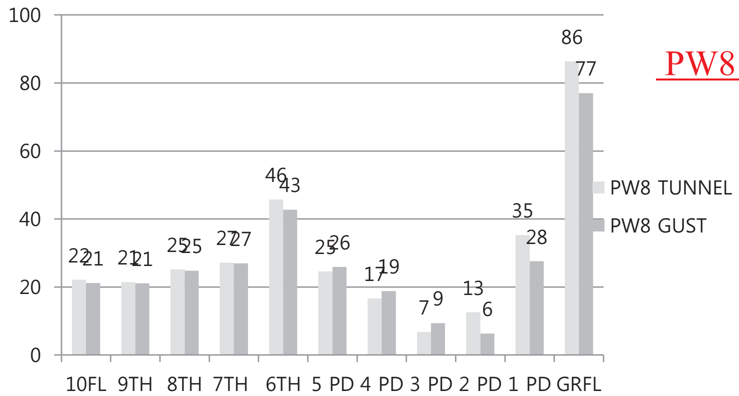
MAX.DISPLACEMENT- TOWER A



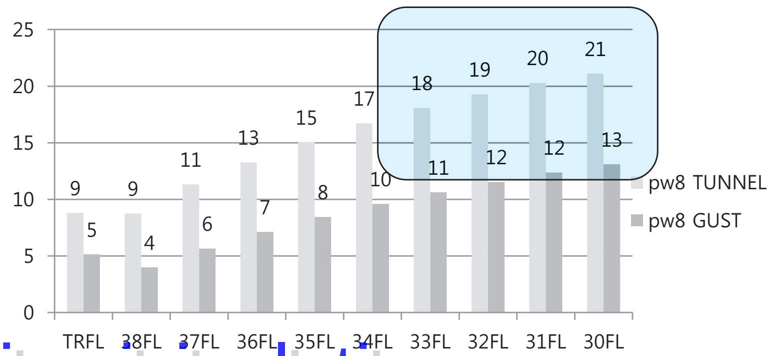
	Static	Gust	Tunnel
X DIX mm	107	133	147
Y DIX mm	198	244	262

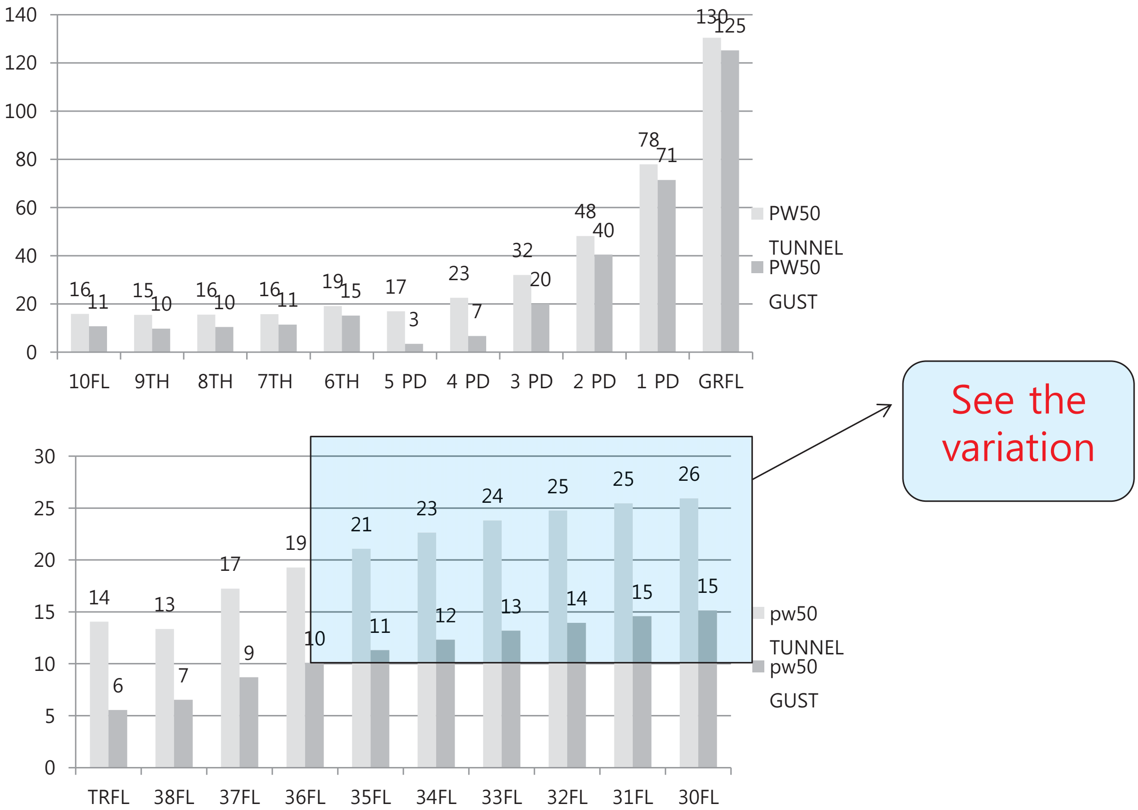


TOWER A – Location of columns for case studies- PW8 and PW50.



PW8 Comparison of M3 moment in mm.





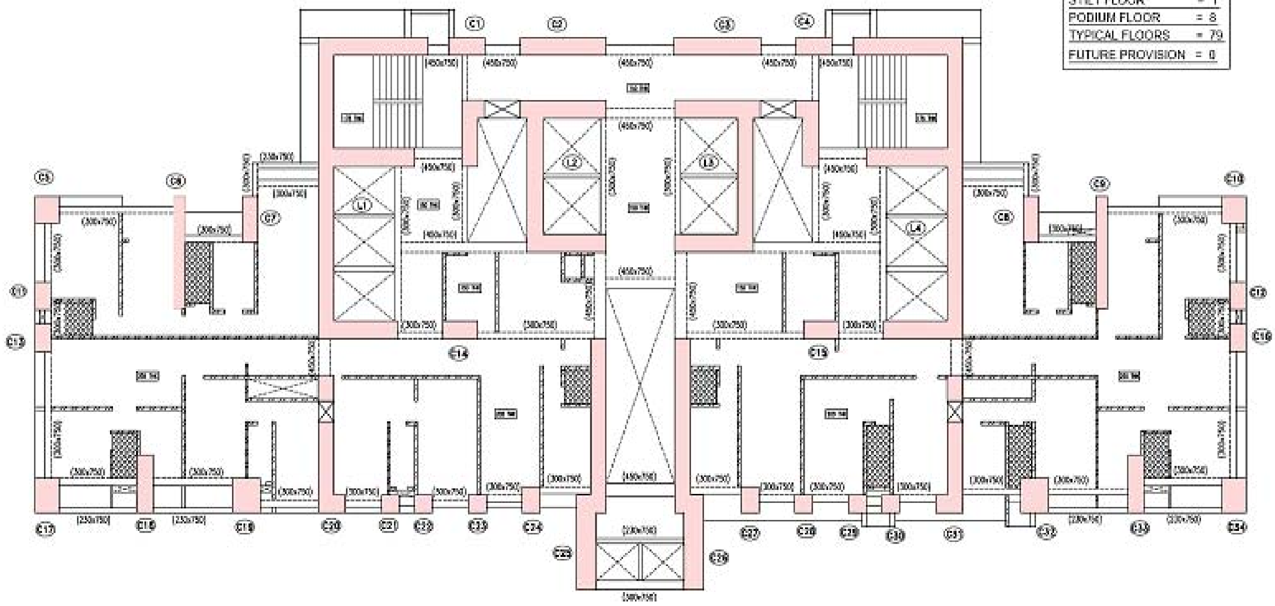
PW50 Comparison of M3 moment in mm.

CONCLUSION:

- The Tunnel loads are a combination of Fx, Fy and Mz forces resulting due to the dynamic effects of wind.
- The torsional moments as well as the directionality effect is also getting captured in this analysis.
- The overall displacement at the roof level is about 10% more than the GUST factor analysis.
- The wind moments on columns are getting increased by more than 60 % at the upper most levels, where as these are around 15% only at the lower levels.
- The cross wind effect of the dynamic effect is causing the larger structural response in this building.
- The inference of the above will be confirmed by doing a second case study.

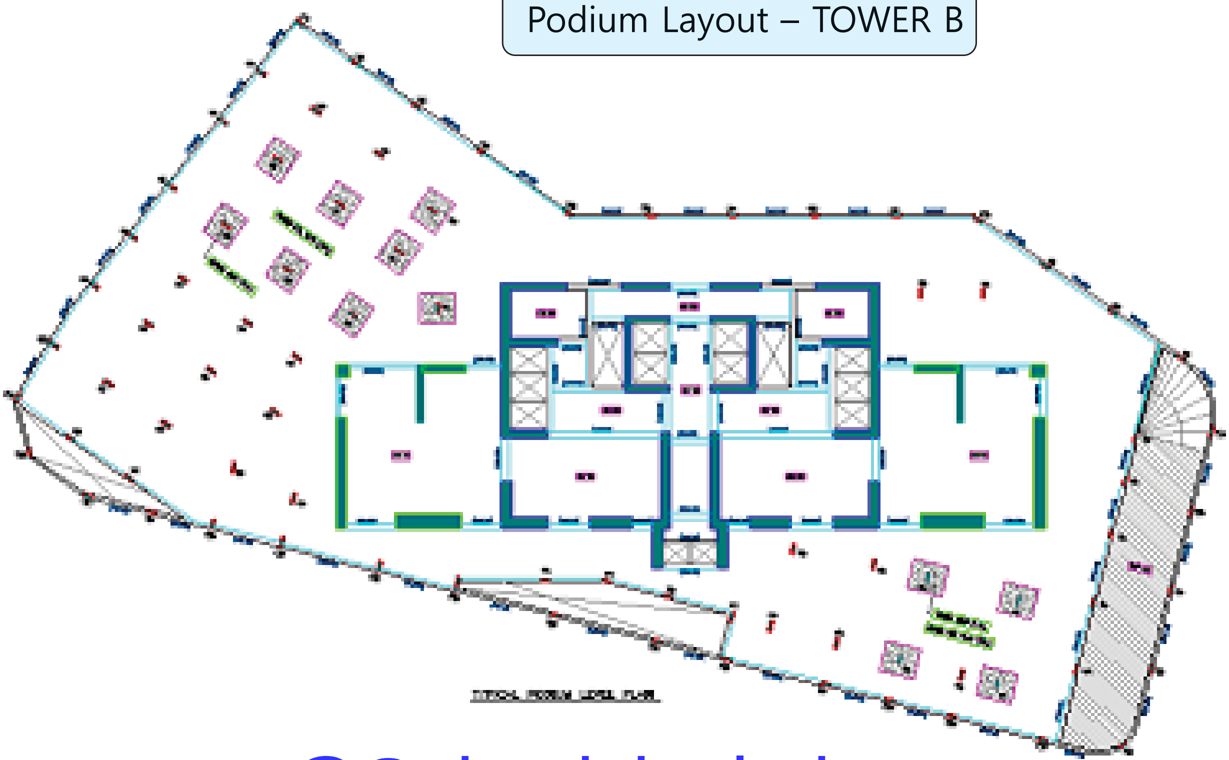
CASE STUDY 2 - TOWER B :

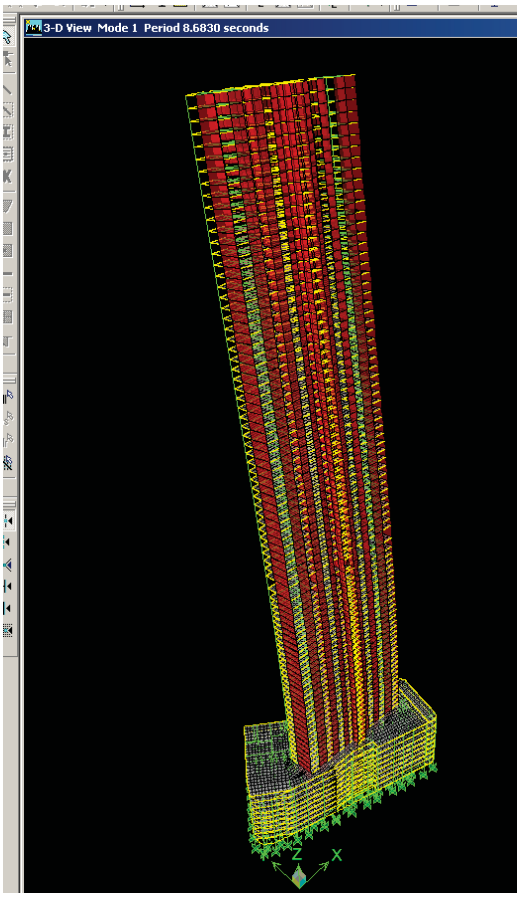
Typical Floor –Tower B



NO. OF FLOORS	
BASEMENT LVL.	= 1
STILT FLOOR	= 1
PODIUM FLOOR	= 8
TYPICAL FLOORS	= 79
FUTURE PROVISION	= 0

Podium Layout – TOWER B





Tower B First Mode

8.68 seconds Ux=0, UY=56.0, RZ=0.06

TOWER B DETAILS		
Max. Length X direction (Typical Floor)	60	m
Max. width Y direction (Typical Floor)	25	m
No. of Floors	G+87	
Floor height @ typical floors	3.5	m
Floor height @ typ.Podium floors	4.2	m
Floor height @ Ground level	4.35	m
Total height of the structure	302	m
Foundation Type	Piled Raft	
Soil type	Type 1	
General live load	0.2 T/m2	
Masonry type	Brick	

TOWER B -MODAL PROPERTIES											
Mode	Period	UX	UY	SumUX	Sum UY	RX	RY	RZ	Sum RX	SumRY	SumRZ
1	8.68	0.0	56.0	0.0	56.0	97.8	0.0	0.06	98	0	0
2	6.60	62.5	0.0	62.5	56.0	0.0	96.8	1.13	98	97	1
3	5.35	1.9	0.0	64.4	56.0	0.0	2.8	47.47	98	100	49
4	2.08	13.6	0.1	78.0	56.2	0.0	0.0	0.09	98	100	49
5	1.98	0.1	17.7	78.1	73.8	1.7	0.0	0.02	100	100	49
6	1.62	0.1	0.0	78.3	73.8	0.0	0.0	12.43	100	100	61
7	1.15	5.3	0.0	83.6	73.8	0.0	0.3	0.05	100	100	61
8	0.87	0.0	7.2	83.6	81.0	0.3	0.0	0.06	100	100	61
9	0.84	0.0	0.0	83.6	81.0	0.0	0.0	9.63	100	100	71
10	0.76	3.8	0.0	87.4	81.0	0.0	0.0	0.12	100	100	71
11	0.56	2.8	0.0	90.3	81.0	0.0	0.0	0.23	100	100	71
12	0.53	0.1	0.2	90.4	81.2	0.0	0.0	9.80	100	100	81
13	0.50	0.0	4.6	90.4	85.8	0.1	0.0	0.10	100	100	81
14	0.42	1.7	0.0	92.1	85.8	0.0	0.0	0.02	100	100	81
15	0.38	0.0	0.1	92.1	85.9	0.0	0.0	5.84	100	100	87

TOWER B STATIC WIND FORCES

Story	FX	FY	Story	FX	FY
TRFL	14.39	40.87	FL-48	16.43	46.68
FL-87	18.27	51.91	FL-47	16.37	46.51
FL-86	18.23	51.79	FL-46	16.31	46.34
FL-85	18.19	51.67	FL-45	16.23	46.1
FL-84	18.15	51.55	FL-44	16.13	45.82
FL-83	18.11	51.43	FL-43	16.03	45.54
FL-82	18.06	51.32	FL-42	15.93	45.26
FL-81	18.02	51.2	FL-41	15.83	44.98
FL-80	17.98	51.08	FL-40	21.33	60.58
FL-79	17.94	50.96	FL-39	22.71	64.52
FL-78	17.9	50.84	FL-38	15.91	45.19
FL-77	17.86	50.72	FL-37	13.18	37.43
FL-76	17.81	50.61	FL-36	13.11	37.23
FL-75	17.77	50.49	FL-35	13.03	37.03
FL-74	17.73	50.37	FL-34	12.96	36.83
FL-73	17.69	50.25	FL-33	12.89	36.63
FL-72	17.65	50.14	FL-32	12.82	36.43
FL-71	17.61	50.02	FL-31	12.74	36.19
FL-70	17.57	49.9	FL-30	12.63	35.88

TOWER B STATIC WIND - DISPLACEMENTS		
Story	UX	UY
TRFL	0.0941	0.5233
79FL	0.0936	0.5165
78FL	0.093	0.5096
77FL	0.0925	0.5027
76FL	0.0919	0.4959
75FL	0.0914	0.489
74FL	0.0908	0.482
73FL	0.0902	0.4751
72FL	0.0895	0.4681
71FL	0.0889	0.4611
70FL	0.0882	0.4541
69FL	0.0876	0.4471
68FL	0.0869	0.44
67FL	0.0861	0.433
66FL	0.0854	0.4259
65FL	0.0846	0.4187
64FL	0.0838	0.4115
63FL	0.083	0.4044
62FL	0.0822	0.3971
61FL	0.0814	0.3899
60FL	0.0805	0.3826
59FL	0.0796	0.3753
58FL	0.0787	0.368

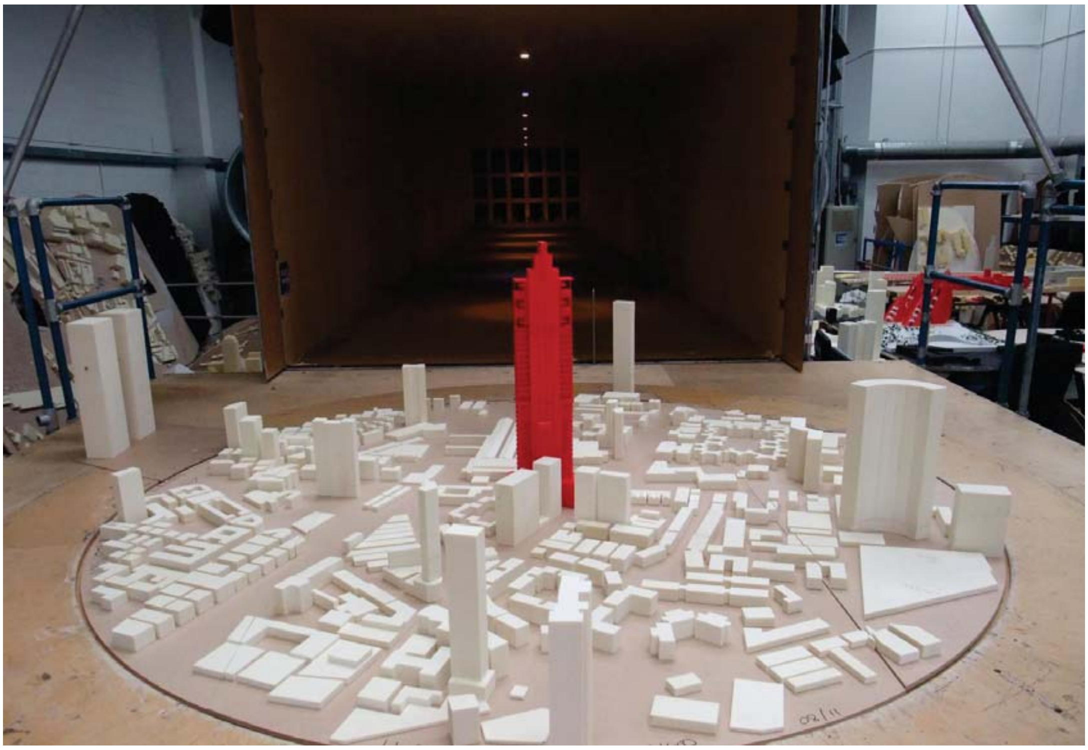
TOWER B - WIND FORCES: GUST FACTOR, FORCE COEFFICIENT, ACCELERATION AND WIND FORCES AT ALL LEVELS												
Size X Direction (m)	60	Category	3		*X Dix*	*Y Dix*		*X Dix*	*Y Dix*		*X Dix*	*Y Dix*
Size Y Direction (m)	25	Class	C	a/b	2.40	0.42	λ = Cyb/Czh	0.07	0.17	Deflection at Roof level (mm)	117	614
Basic Wind speed (m/s)	44	K1	1	h/b	11.90	4.96	Cz.h/L(h)	4.8	4.8	Acceleration (m/s2)	0.120	0.358
gfr	0.72	K3	1	Force Coeff. (Cf)	1.25	1.4	Back gr.Factor (B)	0.38	0.33			
L(h)	740	Cy	10	Time period (Modal)	6.6	8.68						
β	0.016	Cz	12	f0 (Natural freq.)	0.15	0.12						

TOWER B- GUST WIND FORCES AT ALL LEVELS

LEVEL	X- Dim	Y-Dim	HEIGHT	TOTAL HEIGHT	K2	Vz* (m/s)	Pz* (N/m2)	F0 = Cz.f0.h / Vz*		Size redn.Factor (S)		f0.L(h) / Vz*		Gust Ene.Fact. (E)		φ = (gf.r*√B)/4		Gust Factor G.		Wind Force (T)	
								X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
TRFL	60	25	3.5	298	0.93	40.88	1003	13.2	10.1	0.18	0.16	2.7	2.1	0.22	0.24	0.11	0.10	2.24	2.21	24.51	65.00
79FL	60	25	3.5	294	0.93	40.82	1000	13.3	10.1	0.18	0.16	2.7	2.1	0.22	0.24	0.11	0.10	2.24	2.21	24.44	64.80
78FL	60	25	3.5	291	0.93	40.75	997	13.3	10.1	0.18	0.16	2.8	2.1	0.22	0.24	0.11	0.10	2.24	2.21	24.37	64.61
77FL	60	25	3.5	287	0.92	40.69	994	13.3	10.1	0.18	0.16	2.8	2.1	0.22	0.24	0.11	0.10	2.24	2.21	24.29	64.41

TOWER - B - DIAPHRAGM CM DISPLACEMENTS		
Story	UX	UY
TRFL	0.117	0.6135
79FL	0.1164	0.6054
78FL	0.1157	0.5972
77FL	0.115	0.5891
76FL	0.1142	0.5809
75FL	0.1135	0.5727
74FL	0.1127	0.5644
73FL	0.1119	0.5562
72FL	0.1111	0.5479
71FL	0.1103	0.5396
70FL	0.1094	0.5313
69FL	0.1085	0.5229
68FL	0.1076	0.5145
67FL	0.1067	0.5061
66FL	0.1057	0.4976
65FL	0.1047	0.4892
64FL	0.1037	0.4806
63FL	0.1027	0.4721
62FL	0.1016	0.4635
61FL	0.1005	0.4549
60FL	0.0994	0.4463

TOWER B WIND TUNNEL MODEL



TOWER B WIND TUNNEL MODEL - SURROUNDINGS

TOWER B- WIND TUNNEL FORCES – EQ. STATIC

Floor	Fx (N)	Fy'(N)	Mz (N- m)
TRFL	548866	1319641	5417023
STORY87	543432	1306575	5363389
STORY86	538051	1293639	5310286
STORY85	532724	1280830	5257709
STORY84	527449	1268149	5205653
STORY83	522227	1255593	5154112
STORY82	517057	1243161	5103081
STORY81	511937	1230853	5052555
STORY80	506869	1218666	5002530
STORY79	501850	1206600	4953000
STORY78	380800	861900	4963000
STORY77	377500	840600	4750000
STORY76	511800	1231400	9965000
STORY75	500700	1206600	9781000
STORY74	481200	1126400	8839000
STORY73	348200	769500	4869000
STORY72	342600	771000	5115000
STORY71	339400	745800	4773000
STORY70	476900	1122100	9523000

TOWER B Tunnel Load Combinations

Case	X Forces (Fx)	Y Forces (Fy)	Torsion (Mz)
1	+75%	+30%	+40%
2	+75%	+30%	-30%
3	+75%	-50%	+40%
4	+75%	-50%	-30%
5	-100%	+55%	+45%
6	-100%	+55%	-80%
7	-100%	-45%	+45%
8	-100%	-45%	-80%
9	+30%	+100%	+40%
10	+30%	+100%	-30%
11	+30%	-90%	+30%
12	+30%	-90%	-50%
13	-60%	+100%	+40%
14	-60%	+100%	-30%
15	-30%	-90%	+30%
16	-40%	-90%	-60%
17	+30%	+55%	+75%
18	+30%	+30%	-100%
19	+30%	-30%	+75%
20	+30%	-45%	-100%
21	-65%	+55%	+75%
22	-70%	+30%	-100%
23	-65%	-30%	+75%
24	-70%	-45%	-100%

Tower B - Wind Tunnel Forces

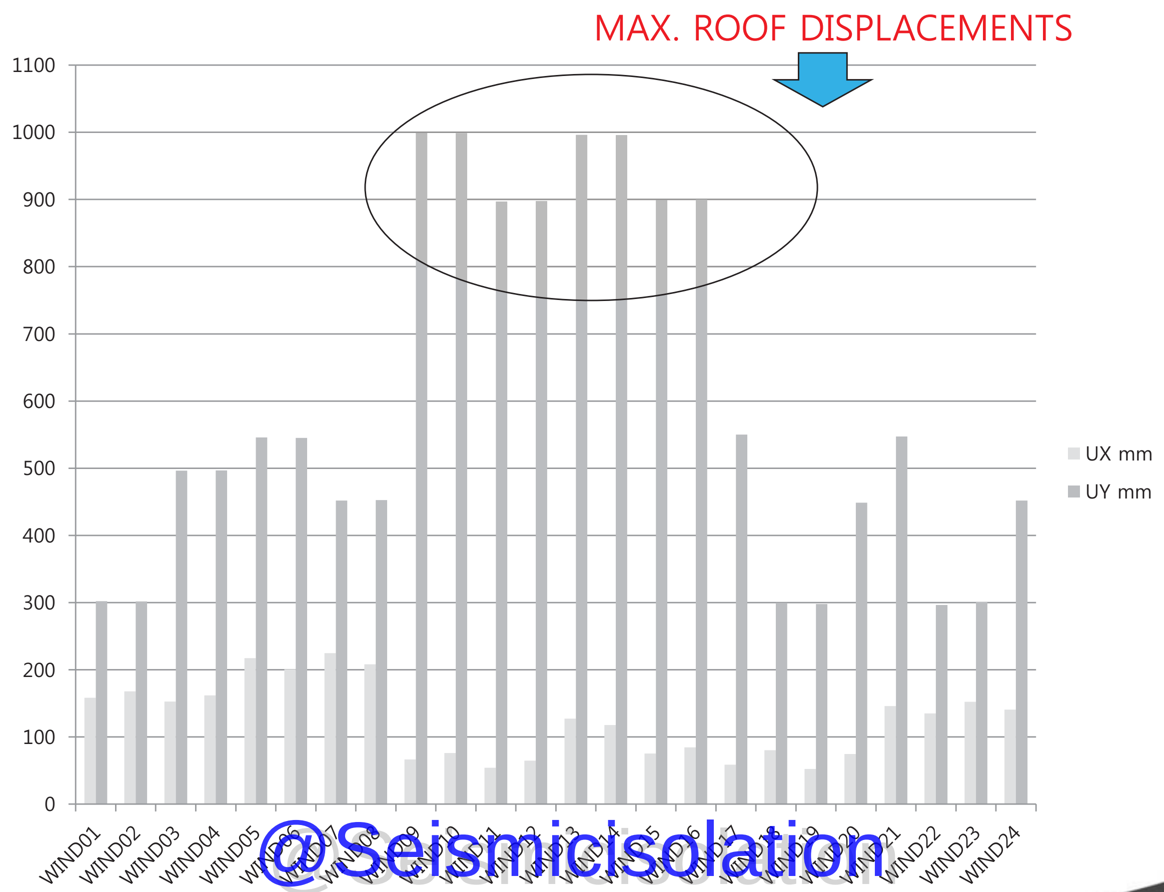
Fx (T)	Fy (T)	Mz (T- m)	Fx (T)	Fy (T)	Mz (T- m)	Fx (T)	Fy (T)	Mz (T- m)
WIND 1			WIND 2			WIND 3		
41.2	39.6	216.7	41.2	39.6	-162.5	41.2	-66.0	216.7
40.8	39.2	214.5	40.8	39.2	-160.9	40.8	-65.3	214.5
40.4	38.8	212.4	40.4	38.8	-159.3	40.4	-64.7	212.4
40.0	38.4	210.3	40.0	38.4	-157.7	40.0	-64.0	210.3
39.6	38.0	208.2	39.6	38.0	-156.2	39.6	-63.4	208.2
39.2	37.7	206.2	39.2	37.7	-154.6	39.2	-62.8	206.2
38.8	37.3	204.1	38.8	37.3	-153.1	38.8	-62.2	204.1
38.4	36.9	202.1	38.4	36.9	-151.6	38.4	-61.5	202.1
38.0	36.6	200.1	38.0	36.6	-150.1	38.0	-60.9	200.1
37.6	36.2	198.1	37.6	36.2	-148.6	37.6	-60.3	198.1
28.6	25.9	198.5	28.6	25.9	-148.9	28.6	-43.1	198.5
28.3	25.2	190.0	28.3	25.2	-142.5	28.3	-42.0	190.0
38.4	36.9	398.6	38.4	36.9	-299.0	38.4	-61.6	398.6
37.6	36.2	391.2	37.6	36.2	-293.4	37.6	-60.3	391.2
36.1	33.8	353.6	36.1	33.8	-265.2	36.1	-56.3	353.6
26.1	23.1	194.8	26.1	23.1	-146.1	26.1	-38.5	194.8

Tower B - Wind Tunnel Forces

Fx (T)	Fy (T)	Mz (T- m)	Fx (T)	Fy (T)	Mz (T- m)	Fx (T)	Fy (T)	Mz (T- m)
WIND 14			WIND 15			WIND 16		
-32.9	132.0	-162.5	-16.5	-118.8	162.5	-22.0	-118.8	-325.0
-32.6	130.7	-160.9	-16.3	-117.6	160.9	-21.7	-117.6	-321.8
-32.3	129.4	-159.3	-16.1	-116.4	159.3	-21.5	-116.4	-318.6
-32.0	128.1	-157.7	-16.0	-115.3	157.7	-21.3	-115.3	-315.5
-31.6	126.8	-156.2	-15.8	-114.1	156.2	-21.1	-114.1	-312.3
-31.3	125.6	-154.6	-15.7	-113.0	154.6	-20.9	-113.0	-309.2
-31.0	124.3	-153.1	-15.5	-111.9	153.1	-20.7	-111.9	-306.2
-30.7	123.1	-151.6	-15.4	-110.8	151.6	-20.5	-110.8	-303.2
-30.4	121.9	-150.1	-15.2	-109.7	150.1	-20.3	-109.7	-300.2
-30.1	120.7	-148.6	-15.1	-108.6	148.6	-20.1	-108.6	-297.2
-22.8	86.2	-148.9	-11.4	-77.6	148.9	-15.2	-77.6	-297.8
-22.7	84.1	-142.5	-11.3	-75.7	142.5	-15.1	-75.7	-285.0
-30.7	123.1	-299.0	-15.4	-110.8	299.0	-20.5	-110.8	-597.9
-30.0	120.7	-293.4	-15.0	-108.6	293.4	-20.0	-108.6	-586.9
-28.9	112.6	-265.2	-14.4	-101.4	265.2	-19.2	-101.4	-530.3
-20.9	77.0	-146.1	-10.4	-69.3	146.1	-13.9	-69.3	-292.1
-20.6	77.1	-153.5	-10.3	-69.4	153.5	-13.7	-69.4	-306.9

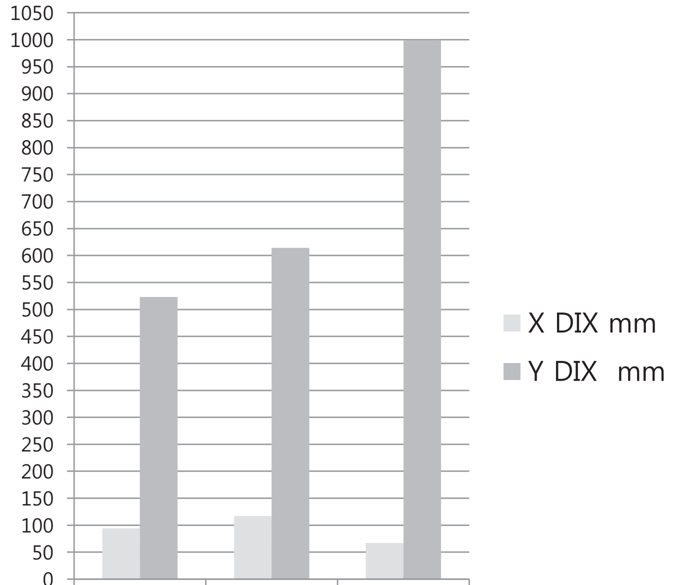
Tower B – Roof Displacements - Tunnel

Load	UX mm	UY mm
WIND01	158	302
WIND02	168	302
WIND03	153	496
WIND04	162	497
WIND05	218	546
WIND06	201	545
WIND07	225	452
WIND08	208	453
WIND09	67	**999**
WIND10	76	**999**
WIND11	54	**897**
WIND12	65	**898**
WIND13	127	**996**
WIND14	118	**996**
WIND15	75	899
WIND16	85	900
WIND17	59	550
WIND18	80	300
WIND19	52	298
WIND20	75	449
WIND21	146	547
WIND22	135	297
WIND23	152	301
WIND24	141	452



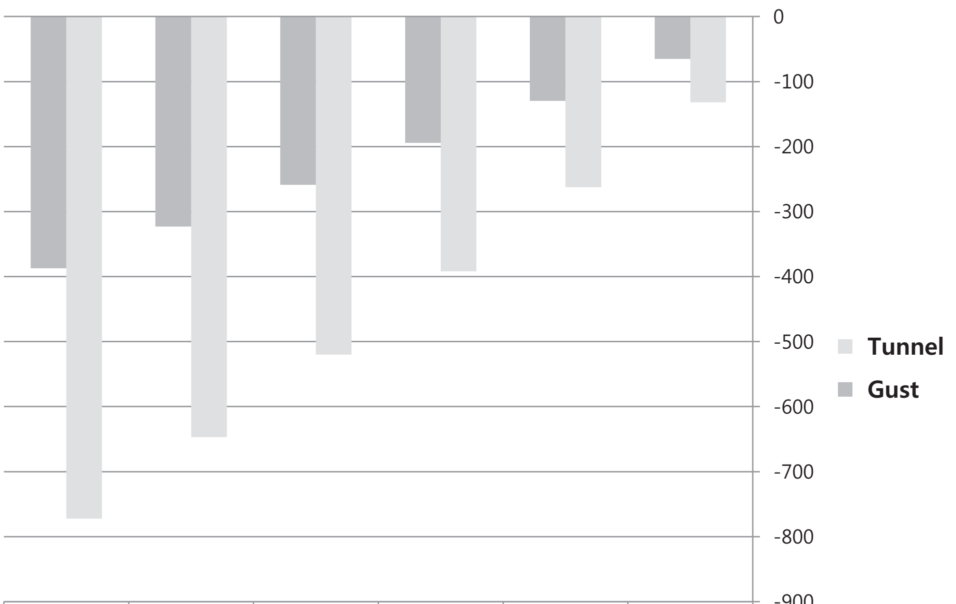
COMPARISON OF RESULTS -TOWER B

TOWER B MAX.DISPLACEMENT



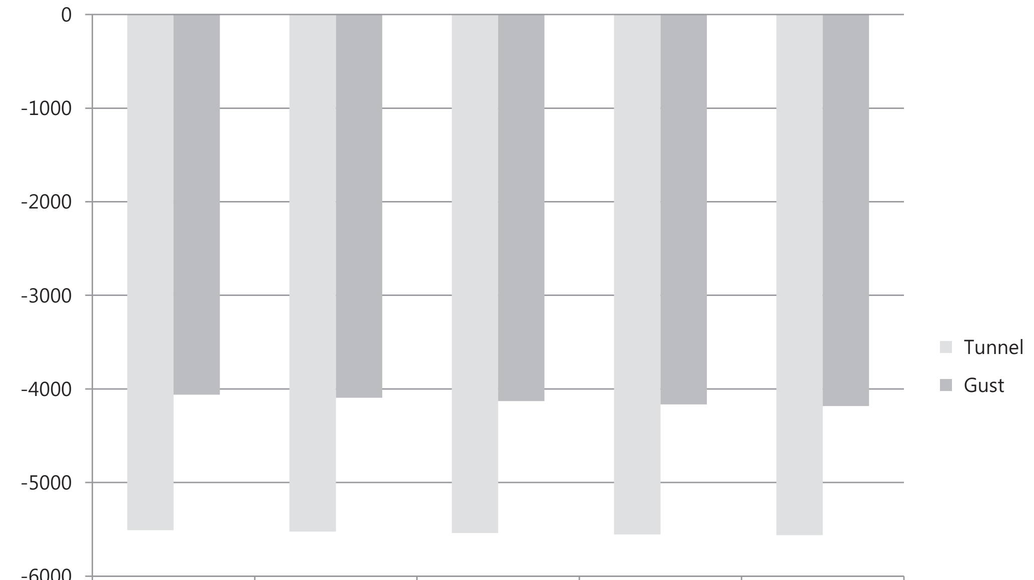
	Static	Gust	Tunnel
X DIX mm	94	117	67
Y DIX mm	523	614	999

Story Shear in Y dix. Tons -Tower B



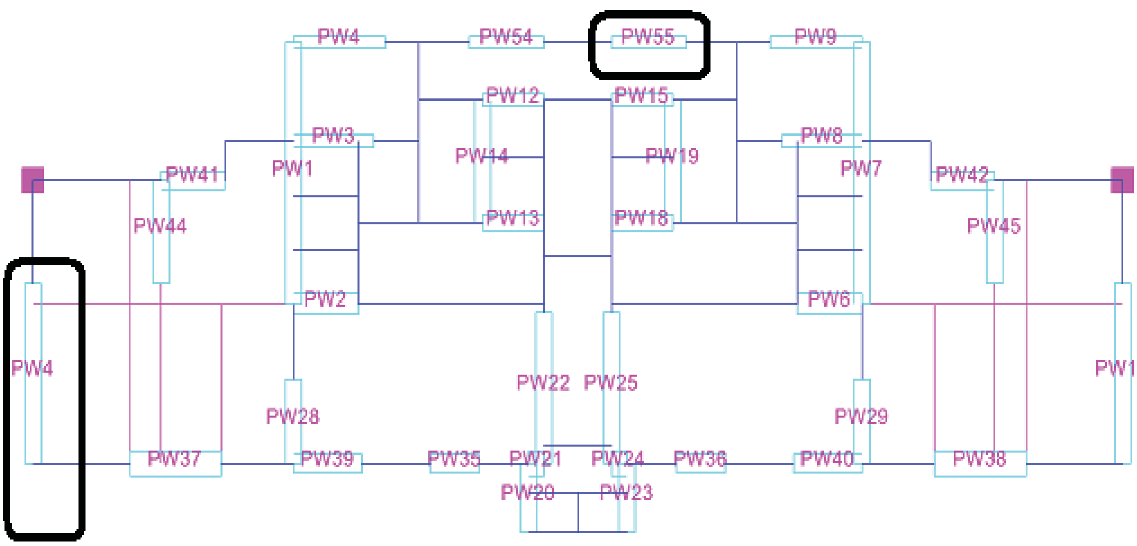
	75FL	76FL	77FL	78FL	79FL	TRFL
Tunnel	-772	-647	-520	-392	-263	-132
Gust	-387	-323	-259	-194	-130	-65

STORY SHEAR Y DIX . AT BASE - COMPARISON



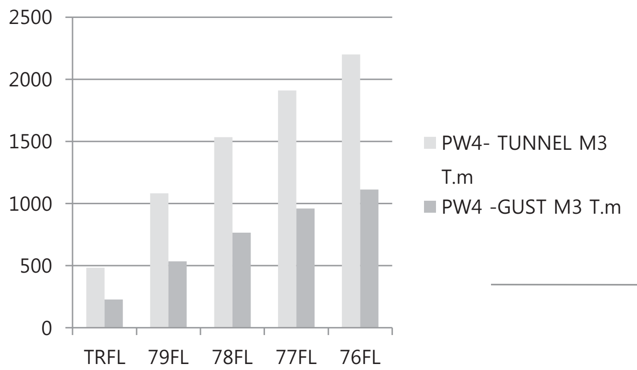
	POD-4	POD-3	POD-2	POD-1	GRFL
Tunnel	-5508	-5523	-5538	-5554	-5563
Gust	-4062	-4095	-4130	-4166	-4183

TOWER B PIER LABEL

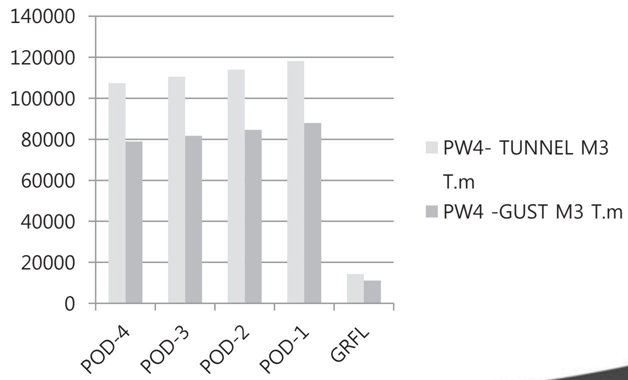


PW4 COMPARISON OF MOMENTS

VERTICAL PIER

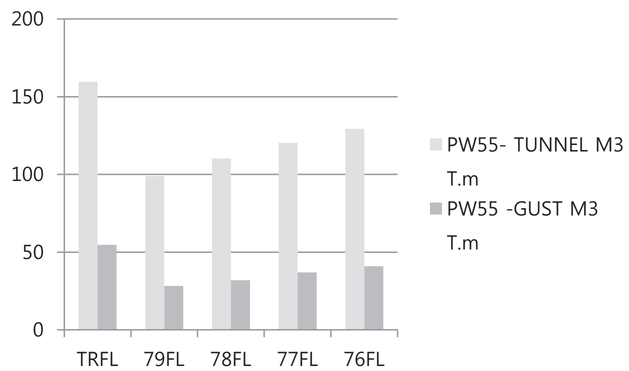


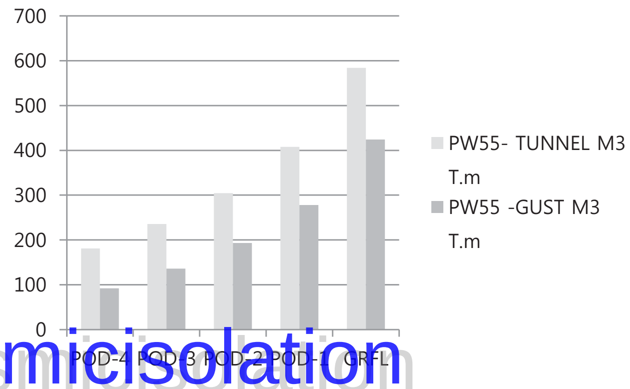
SEE THE VARIATION IN VALUES



PW55 MOMENT COMPARISON

HORZ. PIER





Summary and Conclusion

1. In Tower A, there is only 10% increment in displacement. But at the topmost columns, moments are increased substantially.

2. In Tower B, 60% increment in displacement in Y direction, but it reduces in x direction.

3. Pier moments and storey shears are increased considerably.

4. Displacement has to be based on 20 year return period only.

Summary and Conclusion

5. Gust method clearly underestimates response of the 88 storey building.

6. Even the 40 storied building cannot be convincingly designed for the Gust loads.

7. Gust method is equal or conservative in the stiff direction, but it is reverse in the flexible direction.

8. Gust method lacks the across wind forces, geometry effects, surrounding effects etc.

Summary and Conclusion

9. Gust method can be used for initial planning and also for concept design of tall buildings.

10. Experimental results are to be relied upon for design of tall buildings.

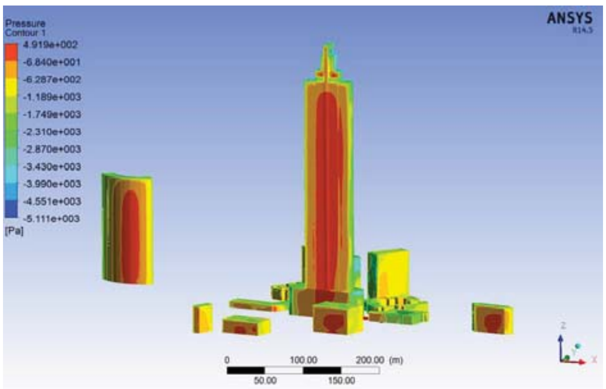
11. Accelerations are 16.8mg and 10.6mg based on Tunnel results.

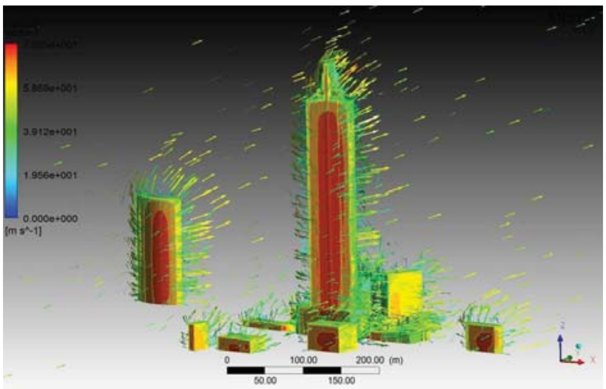
12. Direct comparison with the code method is not correct, as the codal method only follows peak along wind acceleration.

13. New draft code of I.S.875 discusses about the across wind effects, but only for a rectangular building.

Computational Fluid dynamics (CFD)

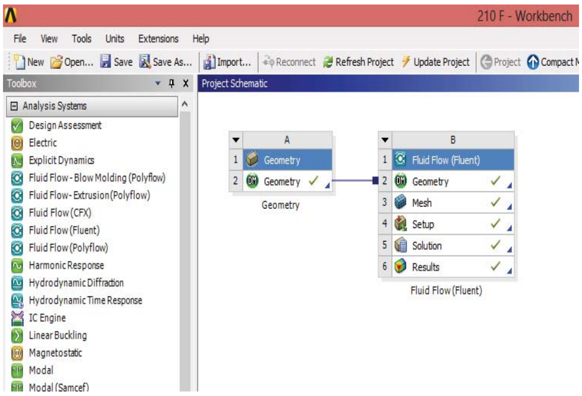
Branch of fluid mechanics that uses numerical methods and algorithms to solve and analyze problems that involve fluid flows. Computers are used to perform the calculations required to simulate the interaction of liquids and gases with surfaces defined by boundary conditions.





CFD PROCESS

- During preprocessing
- The geometry & domain of the problem is defined.
- The volume occupied by the fluid is divided into discrete cells (the mesh).
- The physical modeling is defined – for example, the equations of motions.
- Boundary conditions are defined. This involves specifying the fluid behavior and properties at the boundaries of the problem.
- The simulation is started and the equations are solved iteratively as a steady-state or transient.
- Finally a postprocessor is used for the analysis and visualization of the resulting solution



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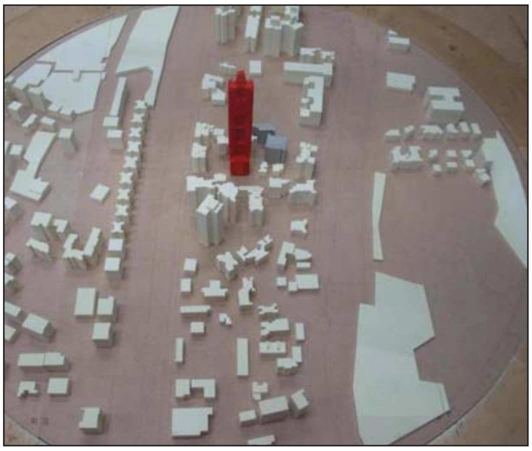


Fig. 3:-Wind tunnel model of tower 'A'

Fig.4:-Wind tunnel model of tower 'B'

Summary of Predicted Peak Overall Structural Wind Loads

DESCRIPTION	Fx (N)	Fy (N)	Mx (N-m)	My (N-m)	Mz (N-m)
TOWER 'A'	3.69E+06	6.41E+06	5.78E+08	3.03E+08	2.1E+07
TOWER'B'	2.27E+07	4.68E+07	8.45E+09	4.05E+09	4.97E+08

CFD MODELLING OF TOWER ‘A’ and TOWER ‘B’

The creation of the domain and the geometry:-

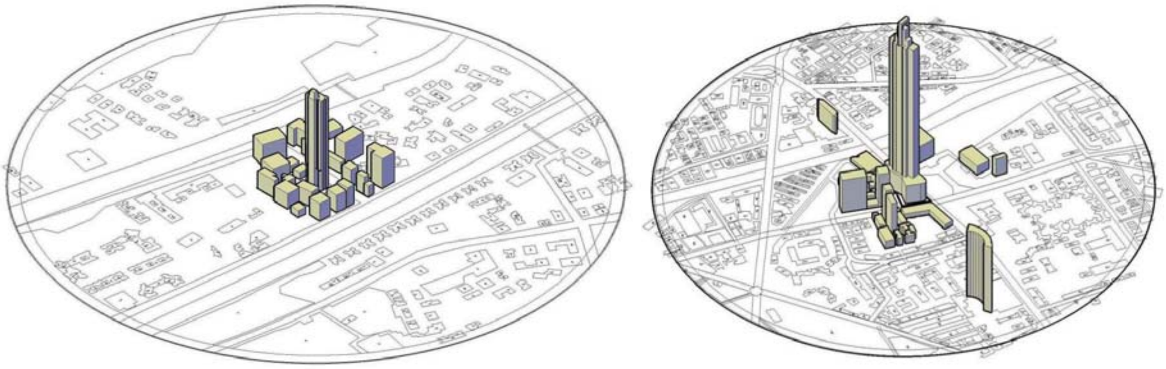


Fig.5 & 6:-Layout of wind tunnel model of tower ‘A’ & tower ‘B’ in AUTOCAD CIVIL 3D

- Wind tunnel models are simulated in AUTOCAD CIVIL 3D & then imported to ANSYS workbench.
- A building with height ‘H’ is extruded if its distance from the studied building is less than 6H [COST-CFD Guidelines].

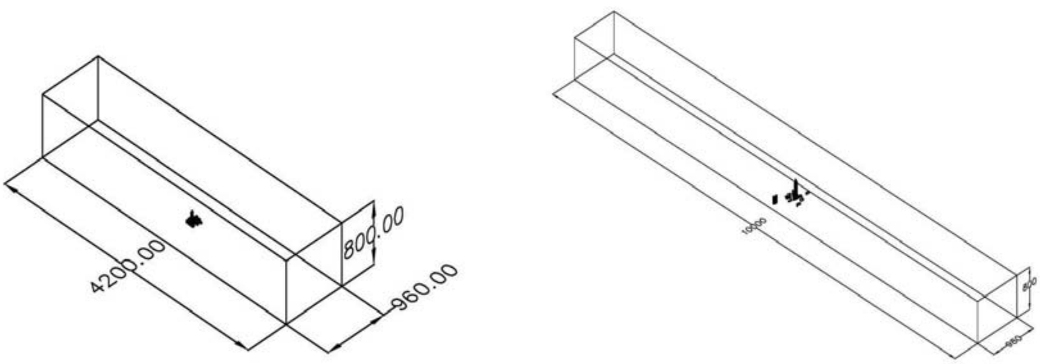


Fig.7 & 8 The built area and surrounding computational domain of tower ‘A’ and tower ‘B’

According to [COST] practice guidelines for the CFD simulation of flows in the urban environment, if the simulations are to be compared with boundary layer wind tunnel measurements, then it is recommended to use the cross section of the wind tunnel’s test section for the computational domain, i.e. the computational domain should have the same height and lateral extent as the boundary layer wind tunnel.

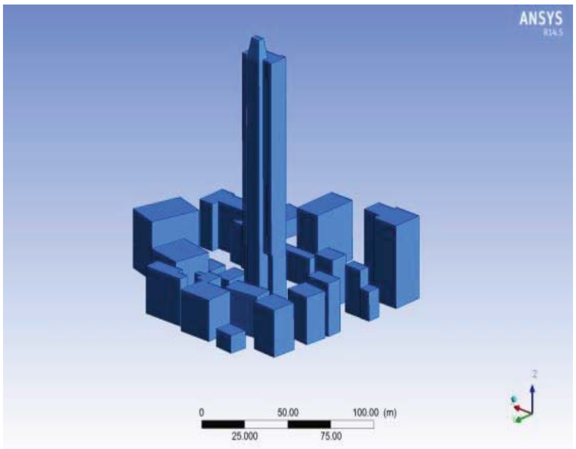


Fig. 9 Enlarged view of the built area of tower 'A'

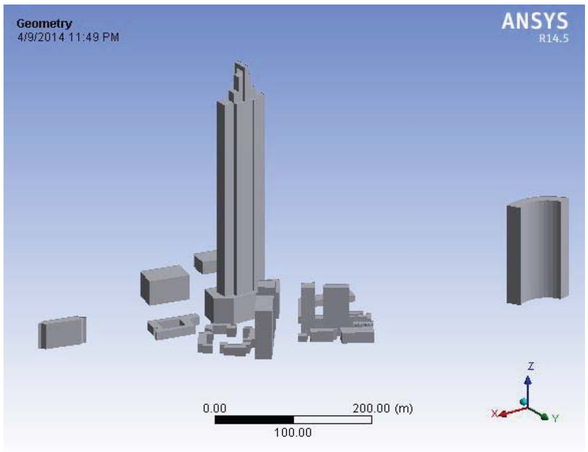


Fig. 10 Enlarged view of the built area of tower 'B'

Setting the meshing scheme:-

An unstructured tetrahedral grid was used to mesh all the faces and volumes in the domain.

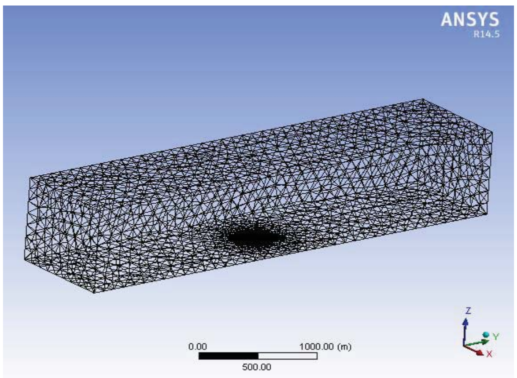


Fig.11 a)

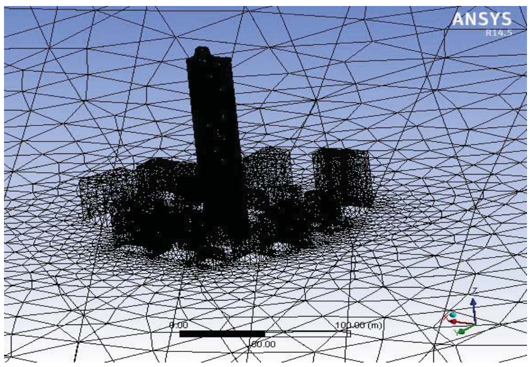


Fig.11 b)

Fig.11 Unstructured tetrahedral grid: a) Domain, b) the built area

Defining the boundary types:-

Boundary type consist of the selection of a surface and specifying it as inlet, outlet, wall and/or symmetry boundary conditions.

The boundaries types of the domain for this study are as follows:
Building Wall: - No slip wall condition
Top plane ,Lateral Sides & Ground: : - No slip wall condition
Outlet: - Outflow
Inlet: - Velocitv Inlet

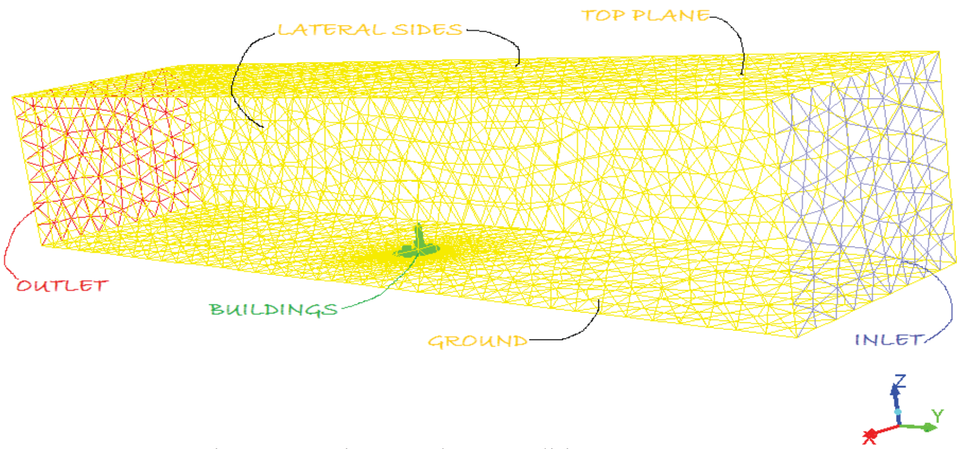


Fig.12 Domain Boundary Conditions

Cell zone condition: Fluid

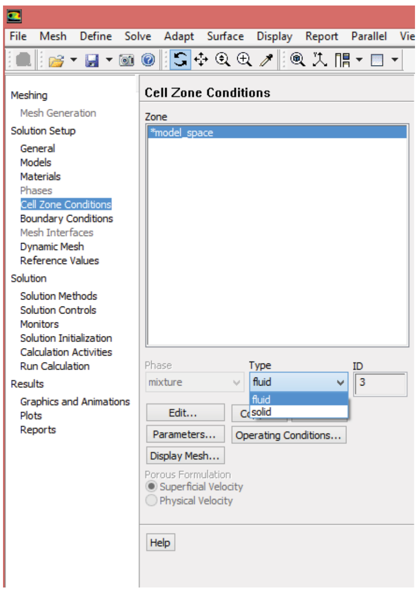


Fig. 15

Boundary Conditions:

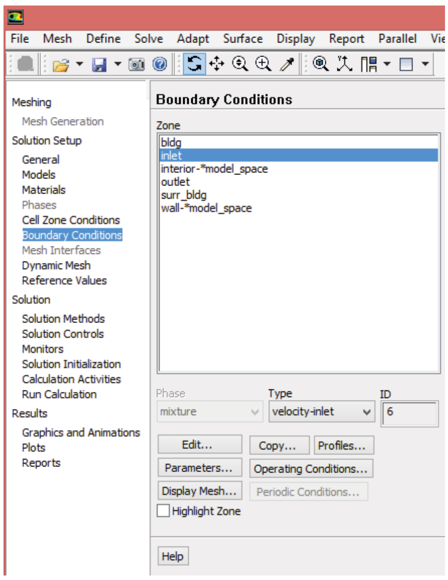


Fig. 16

- Inlet: Velocity Inlet = 44 m/s
- Outlet: Outflow
- Lateral sides, Top Side and Ground: Wall
- Building : Wall

Monitors: Residuals and drag forces, moments.

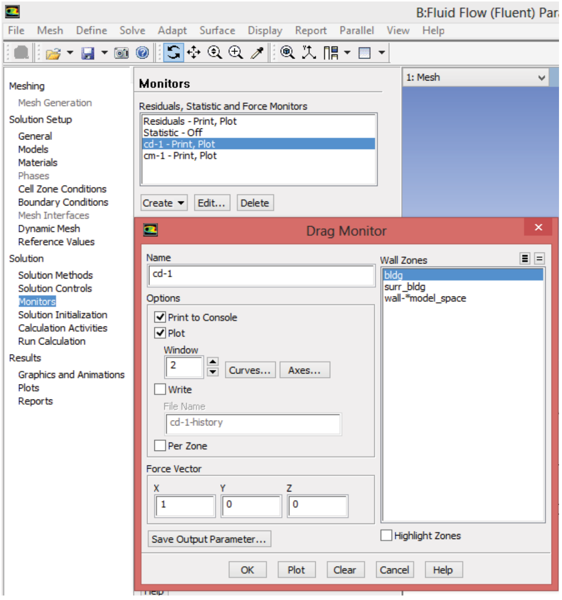


Fig. 17

Solution Initialization: Solution is initialized from inlet region with set initial values.

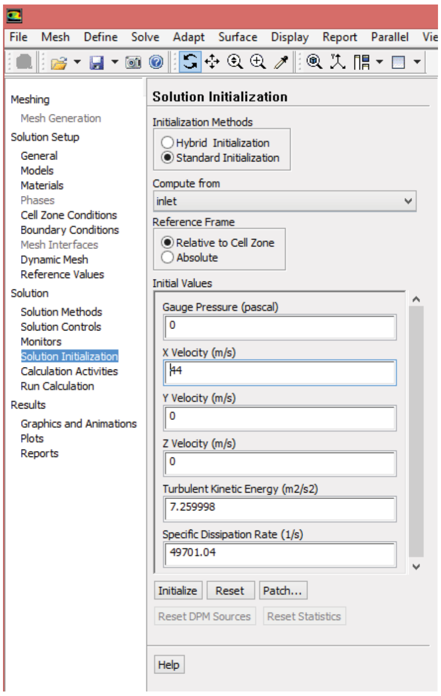


Fig. 18

- Run Calculation: Run the Calculation till convergence

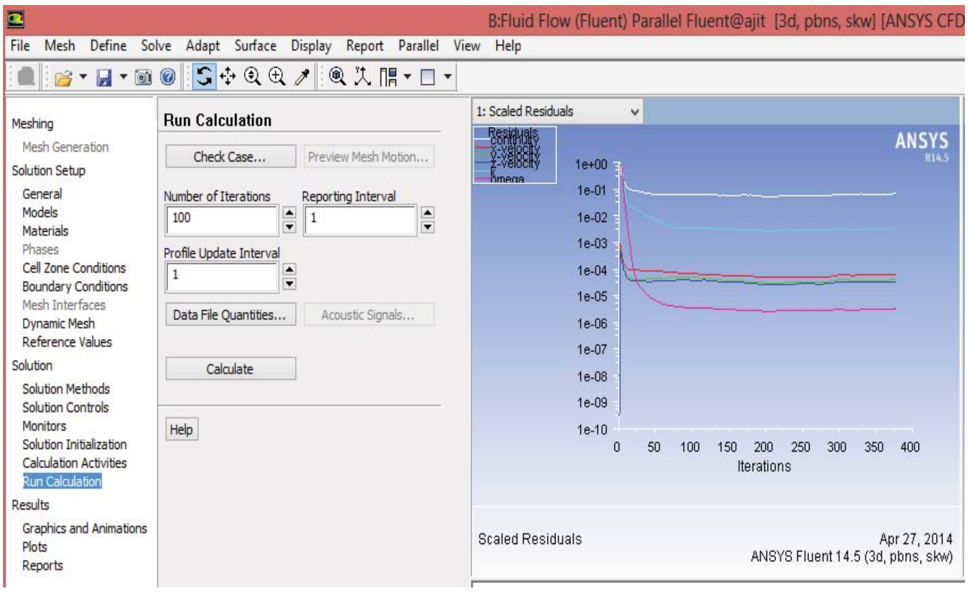


Fig. 19

CFD Test Results and Comparison

In the wind tunnel testing for structural response, the measurements of forces and moments are made at the base for each 10 deg wind attack angle. The wind tunnel results are based on a single wind speed. Similarly in CFD test, results are calculated for each 20 deg wind attack angle by rotating the domain. For each direction wind speed of 44 m/s is applied at inlet.

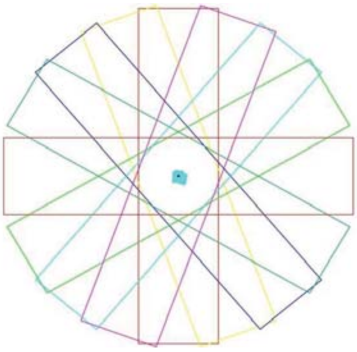


Fig. 20 Rotation of domain for different wind attack angles for tower 'A'

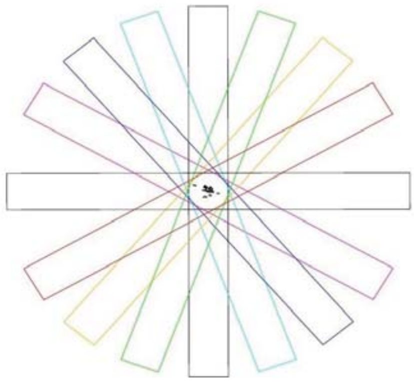


Fig. 21 Rotation of domain for different wind attack angles for tower 'B'

The drag forces in x and y direction and moments Mx, My & Mz at base of tower 'A' & 'B' are calculated around the same point which is used as reference point for wind tunnel test.

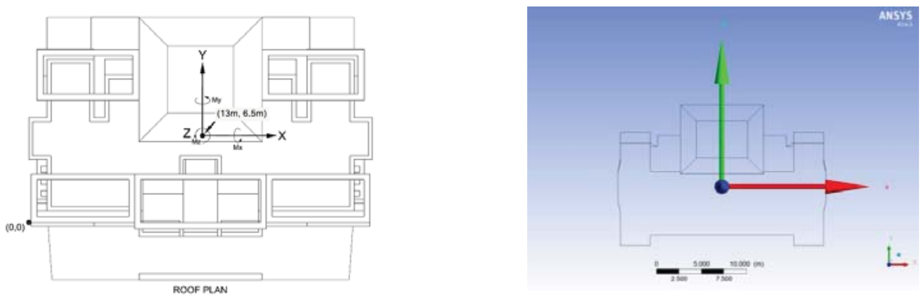


Fig. 22 Wind tunnel and CFD reference points for computation of drag forces and moments for tower 'A'

Fig. 23 Wind tunnel and CFD reference points for computation of drag forces and moments for tower 'B'

7.1 CFD results for Tower 'A'

The maximum values of Fx and My are found at 210 deg. wind attack angle whereas maximum values of Fy and Mx are found at 310 deg. wind attack angle.

The peak values obtained from wind tunnel and CFD measurements for tower 'A' are as follows:-

DESCRIPTION	Fx (N)	Fy (N)	Mx (N-m)	My (N-m)
WIND TUNNEL	3.69E+06	6.41E+06	5.78E+08	3.03E+08
CFD	3.78E+06	7.02E+06	4.85E+08	2.62E+08

The variation of wind tunnel and CFD results of tower 'A' are as follows:-

Fx (N)	Fy (N)	Mx (N-m)	My (N-m)
2.35%	9.56%	16.15%	13.4%

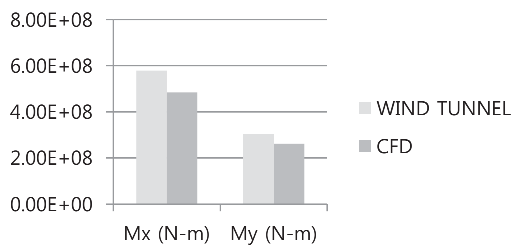


Fig. 25 Peak values of Mx & My for tower 'A'

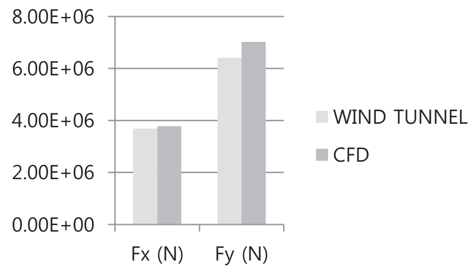


Fig. 24 Peak values of Fx & Fy for tower 'A'

Wind attack angle 210 degree

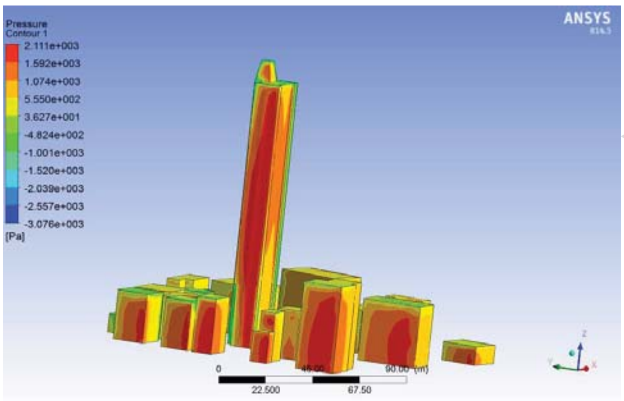


Fig. 26 Contours of pressure on built area of tower 'A' for max. Fx and My

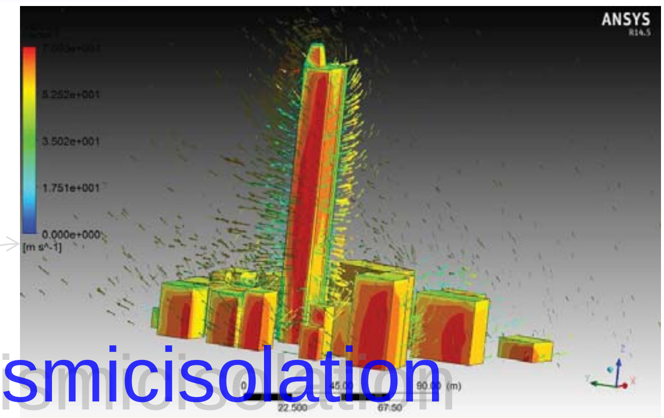


Fig. 27 Velocity Vector for max. Fx and My for tower 'A'

Wind attack angle 310 degree

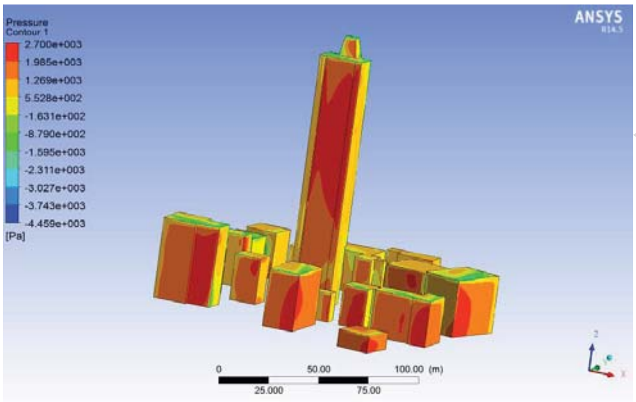


Fig. 28 Contours of pressure on built area of tower 'A' for max. Fy and Mx

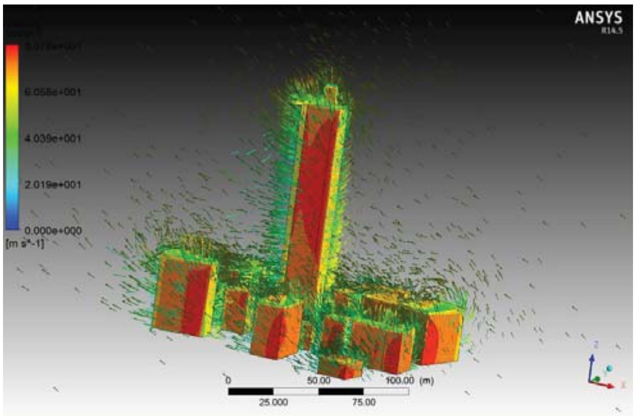


Fig. 29 Velocity Vector for max. Fy and Mx for tower 'A'

CFD results for Tower 'B'

The maximum values of Fx and My are found at 30 deg. wind attack angle whereas maximum values of Fy and Mx are found at 270 deg. wind attack angle. Maximum value of Mz is found at 130 deg. wind attack angle.

The peak values obtained from wind tunnel and CFD measurements for tower 'B' are as follows:-

DESCRIPTION	Fx (N)	Fy (N)	Mx (Nm)	My (Nm)	Mz (N m)
WIND TUNNEL	2.27E+07	4.68E+07	8.45E+09	4.05E+09	4.97E+08
CFD	2.41E+07	4.95E+07	7.86E+09	3.93E+09	4.12E+08

The variation of wind tunnel and CFD results of tower 'B' are as follows:-

Fx (N)	Fy (N)	Mx (N-m)	My (N-m)	Mz (N-m)
6.200%	5.77%	6.98%	2.9%	17.1%

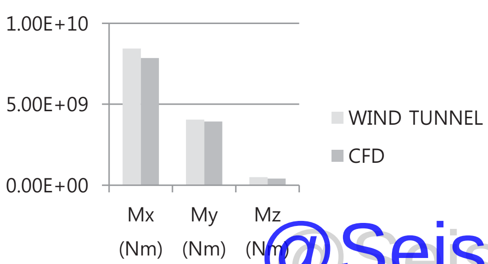


Fig. 31 Peak values of Mx, My & Mz for tower'B'

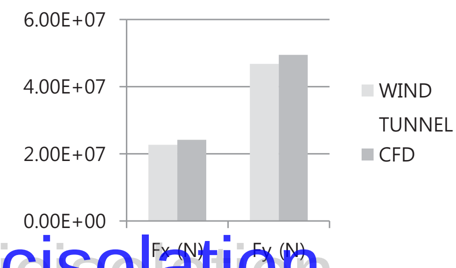


Fig. 30 Peak values of Fx & Fy for tower 'B'

Wind attack angle 30 degree

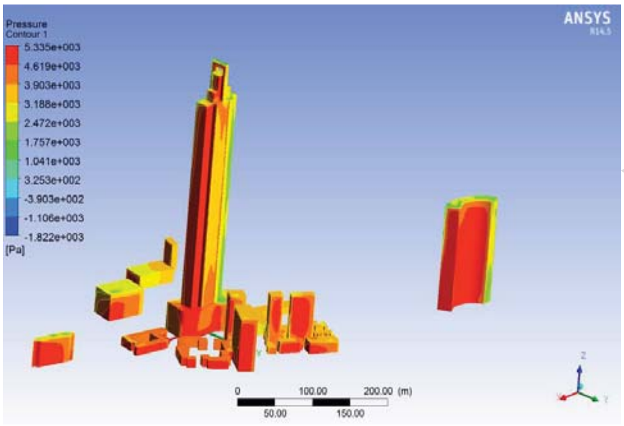


Fig. 32 Contours of pressure on built area of tower 'B' for max. Fx and My

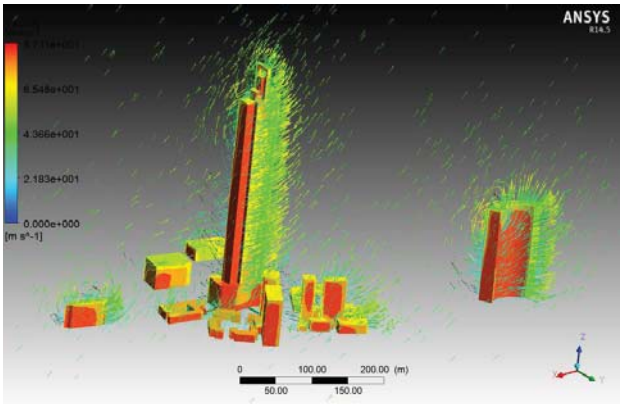


Fig. 33 Velocity Vector for max. Fx and My for tower 'B'

Wind attack angle 270 degree

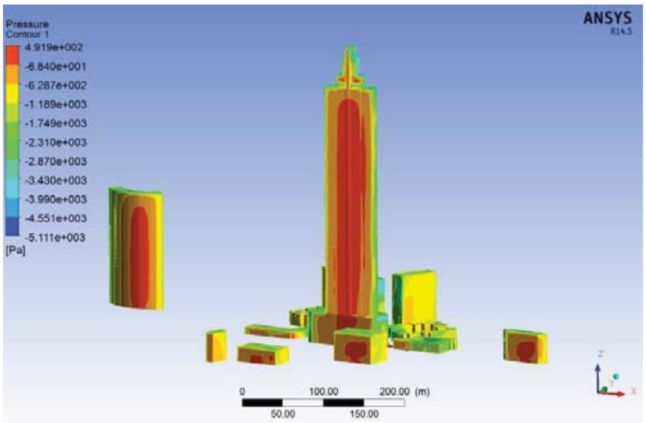


Fig 34 Contours of pressure on built area of tower 'B' for max. Fy and Mx

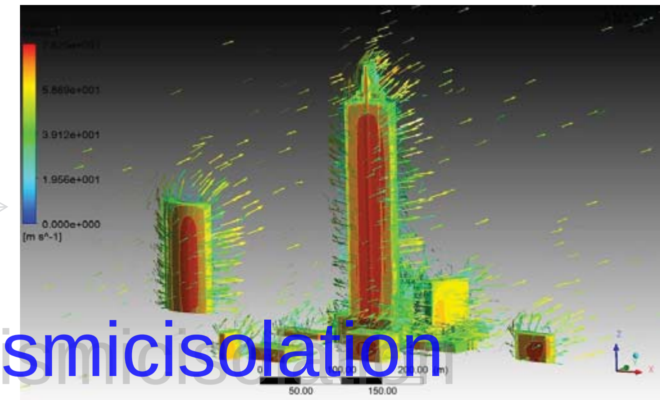


Fig.35 Velocity Vector for max. Fy and Mx for tower 'B'

Wind attack angle 130 degree

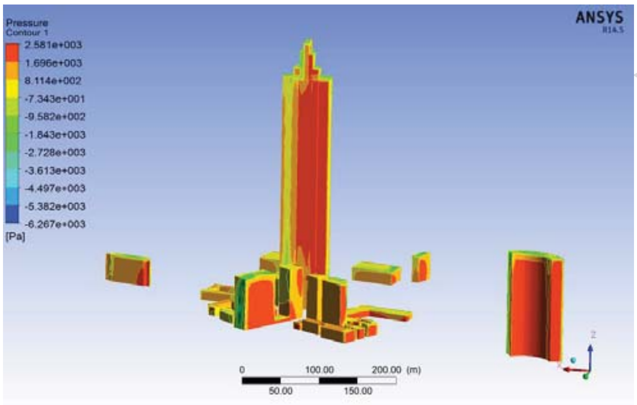


Fig. 36 Contours of pressure on built area of tower 'B' for max. MZ

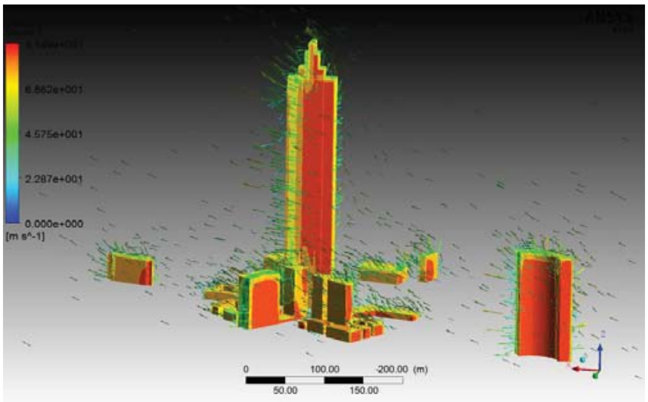


Fig. 37 Velocity Vector for max. Mz for tower 'B'

Conclusions

1. wind tunnel model of G+38 and G+87 storey building is simulated using CFD. The available wind tunnel results are compared with the results obtained from CFD simulation. From the preceding discussions, the following conclusions can be made:
2. CFD is a powerful tool for the determination of drag forces due to wind on buildings.
3. CFD techniques can provide detailed predictions of air velocities around buildings.
4. CFD can be effectively used for determination of moments due to wind at base of building.
5. CFD and Wind Tunnel Results showed reasonable agreement. The variation in CFD and wind tunnel results is observed due to simplified geometry used in ANSYS Fluent.

Thank you

Challenges & Solutions for Tall Building Design

Effect of wind loading on tall buildings

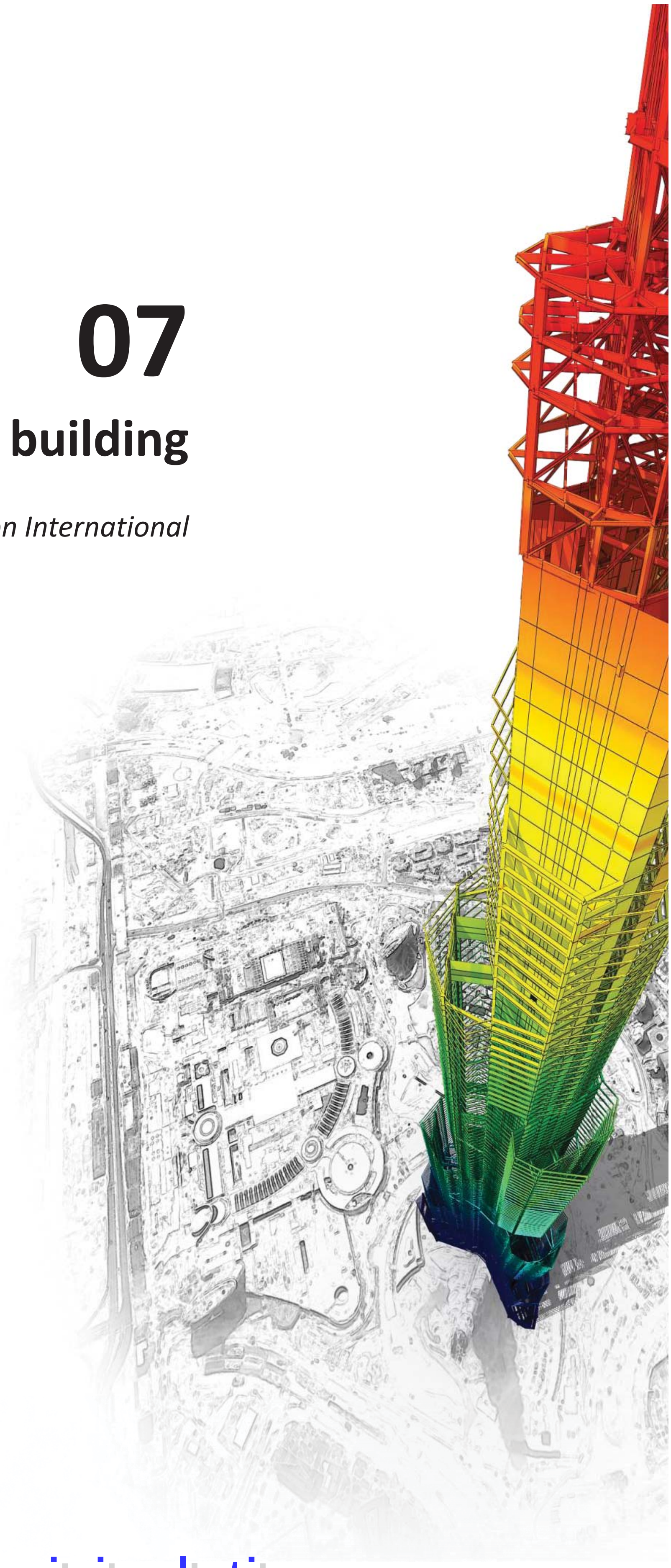
2014 MIDAS Technical Seminar

Challenges & Solution for Tall Building Design

07

Foundation for tall building

Jaydeep Wagh, Geocon International



FOUNDATION STRUCTURAL DESIGN BY FEM SOFTWARES

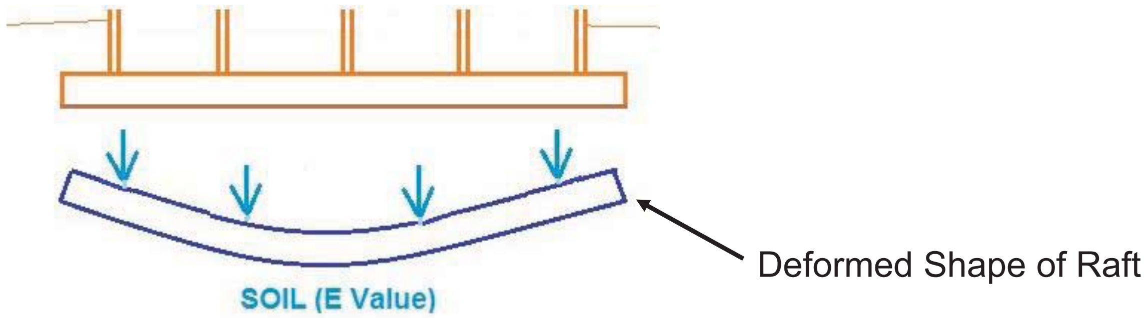
midas Gen + Spring Constant by Geotechnical engineers

ETABS → SAFE + Spring Constant by Geotechnical Engineers

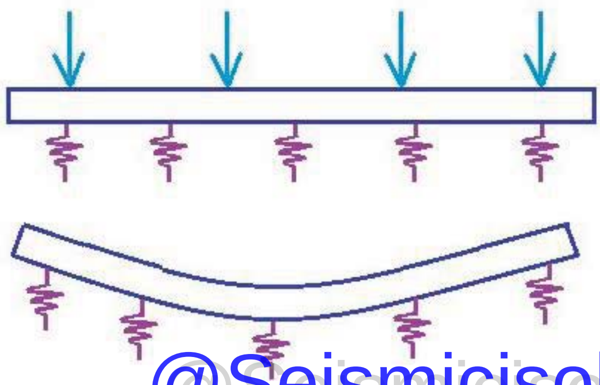
STAAD → STAAD FOUNDATION + Spring Constant by Geotechnical engineers

WHAT IS THIS SPRING CONSTANT ???

DEFORMATION OF RAFT ON SOIL AS DETERMINED BY A GEOTECHNICAL ENGINEER



RAFT ON SPRING CONCEPT.



K = Spring Constant = Modulus of Subgrade Reaction= $\frac{Raft\ Pressure}{Raft\ Settlement} = \frac{Q}{S}$

- MODULUS OF SUBGRADE REACTION OR SPRING CONSTANT OR "K VALUE" IS ONE OF THE MOST MISUNDERSTOOD AND MISUSED TERM IN DESIGN OF FOUNDATIONS.

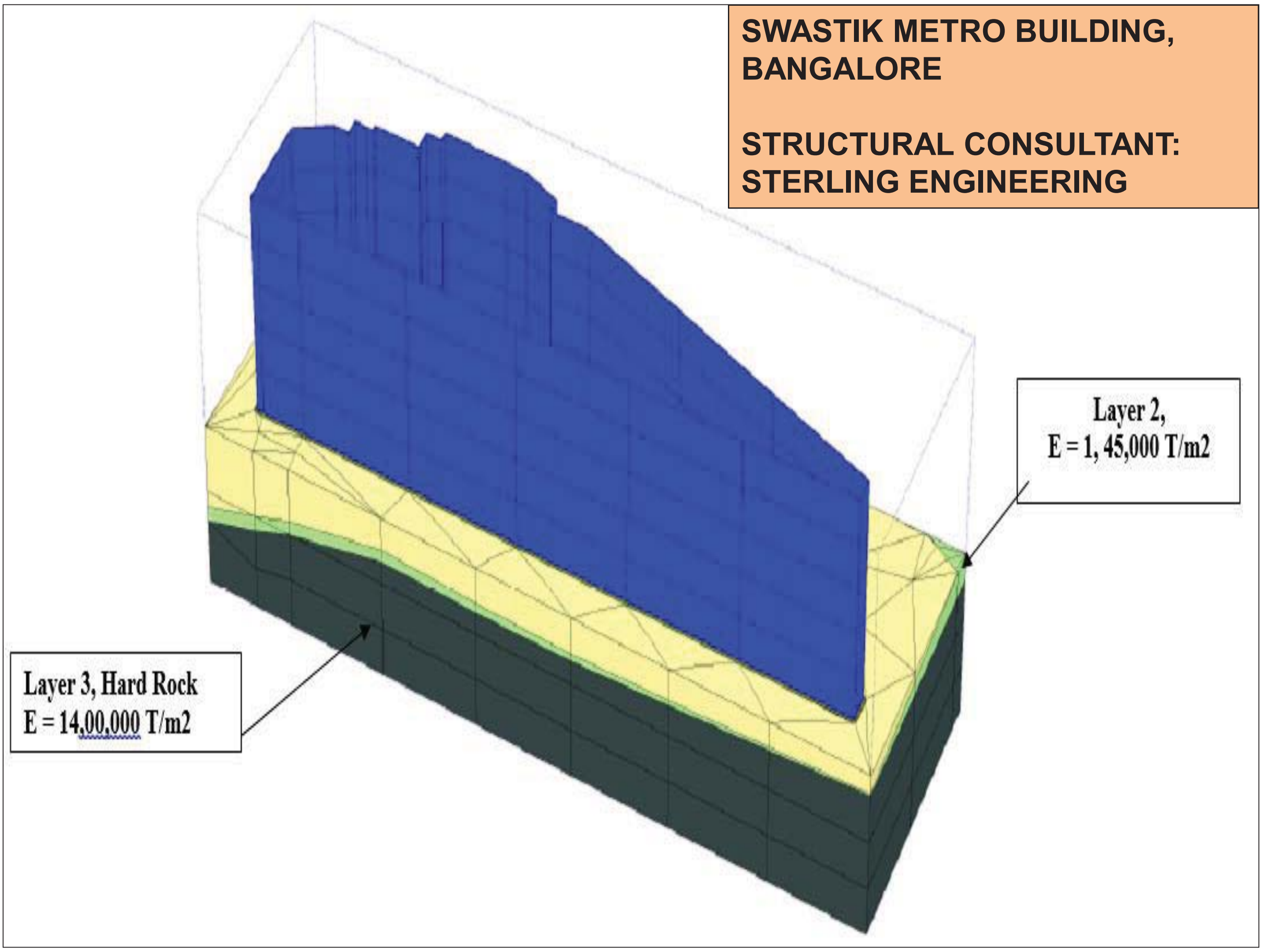
$$K = \frac{\textit{Raft Pressure}}{\textit{Raft Settlement}}$$

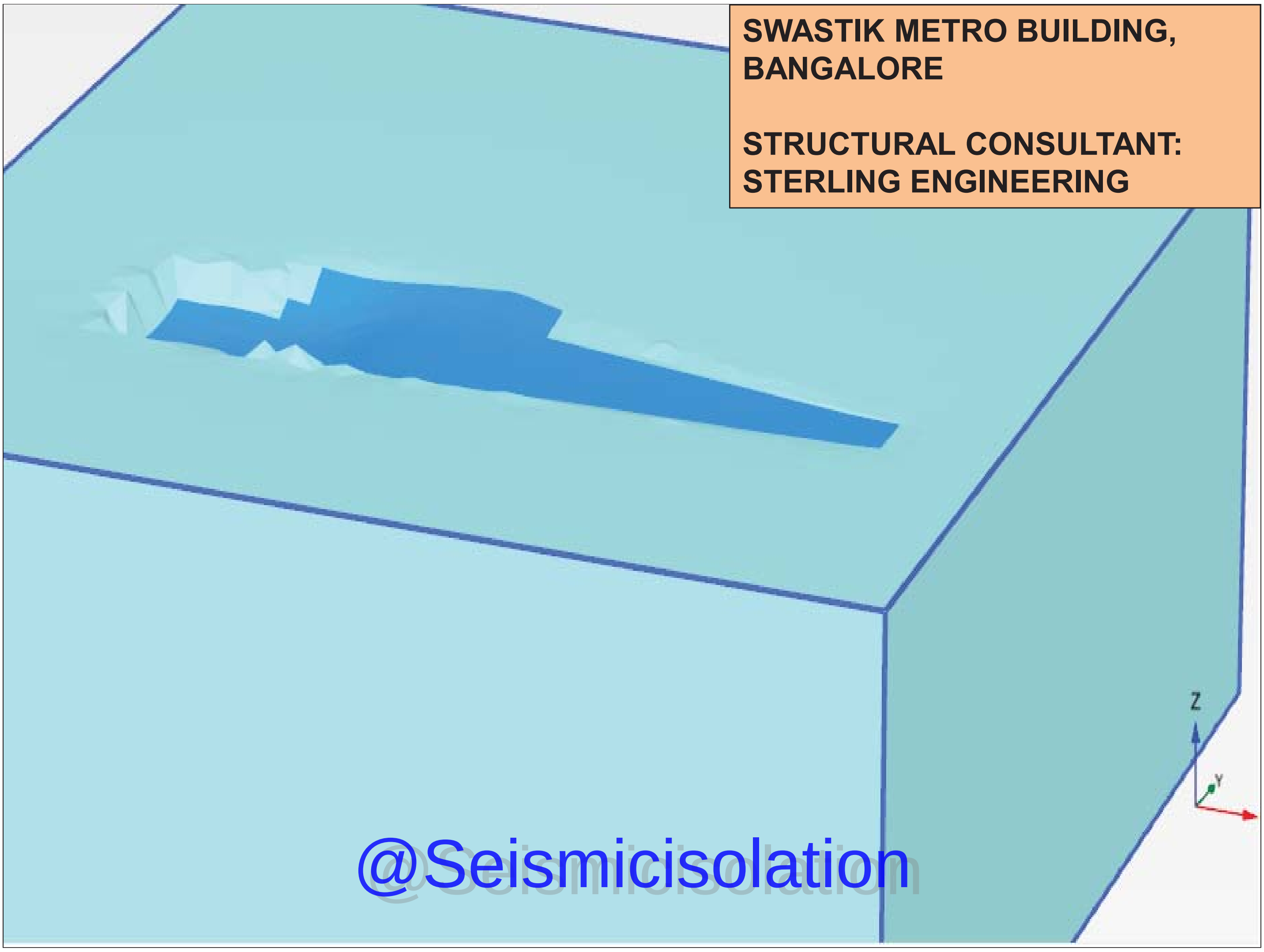
Thus K value depends on

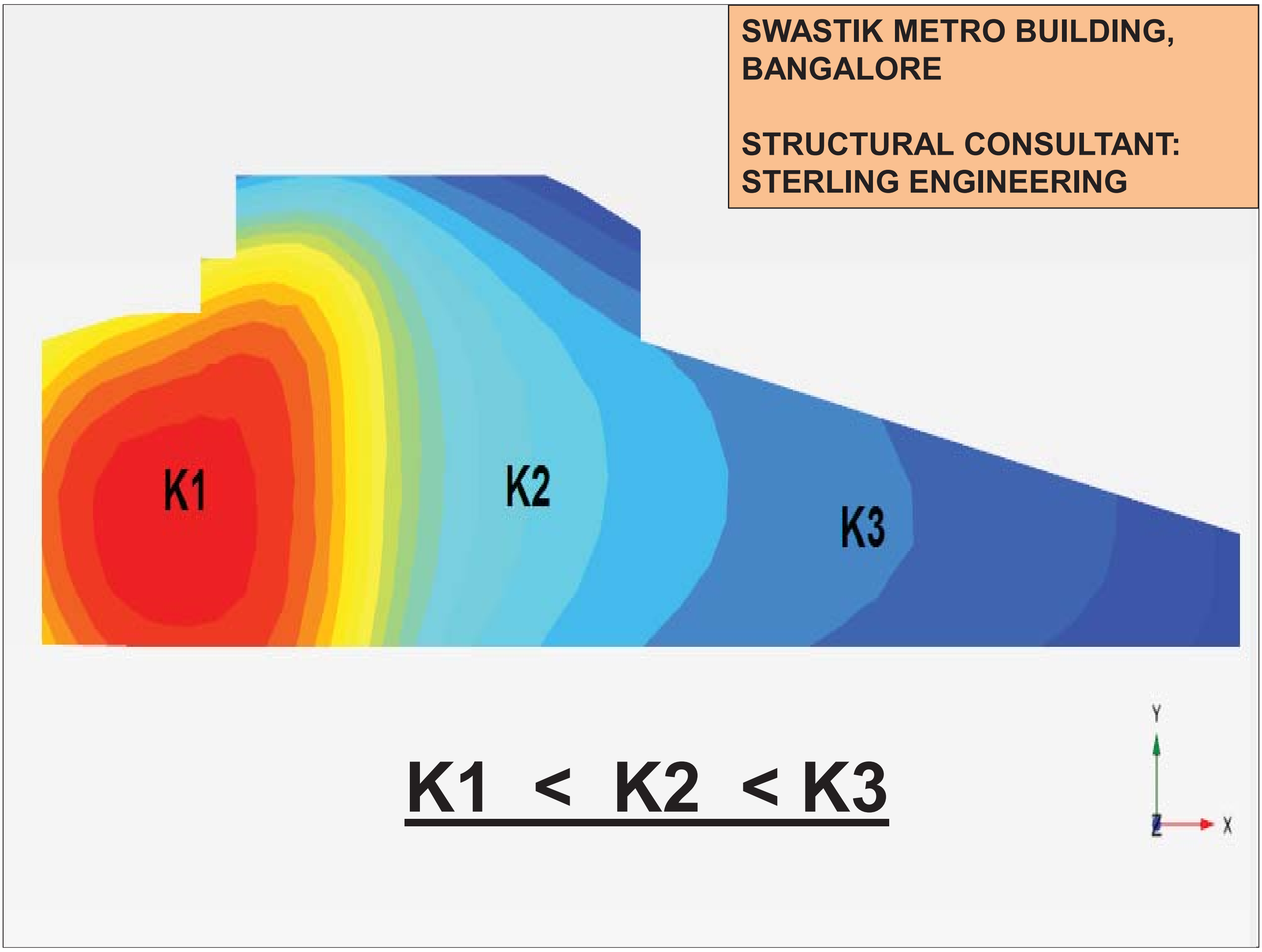
1)Width of footing.

2)Variations in founding strata.

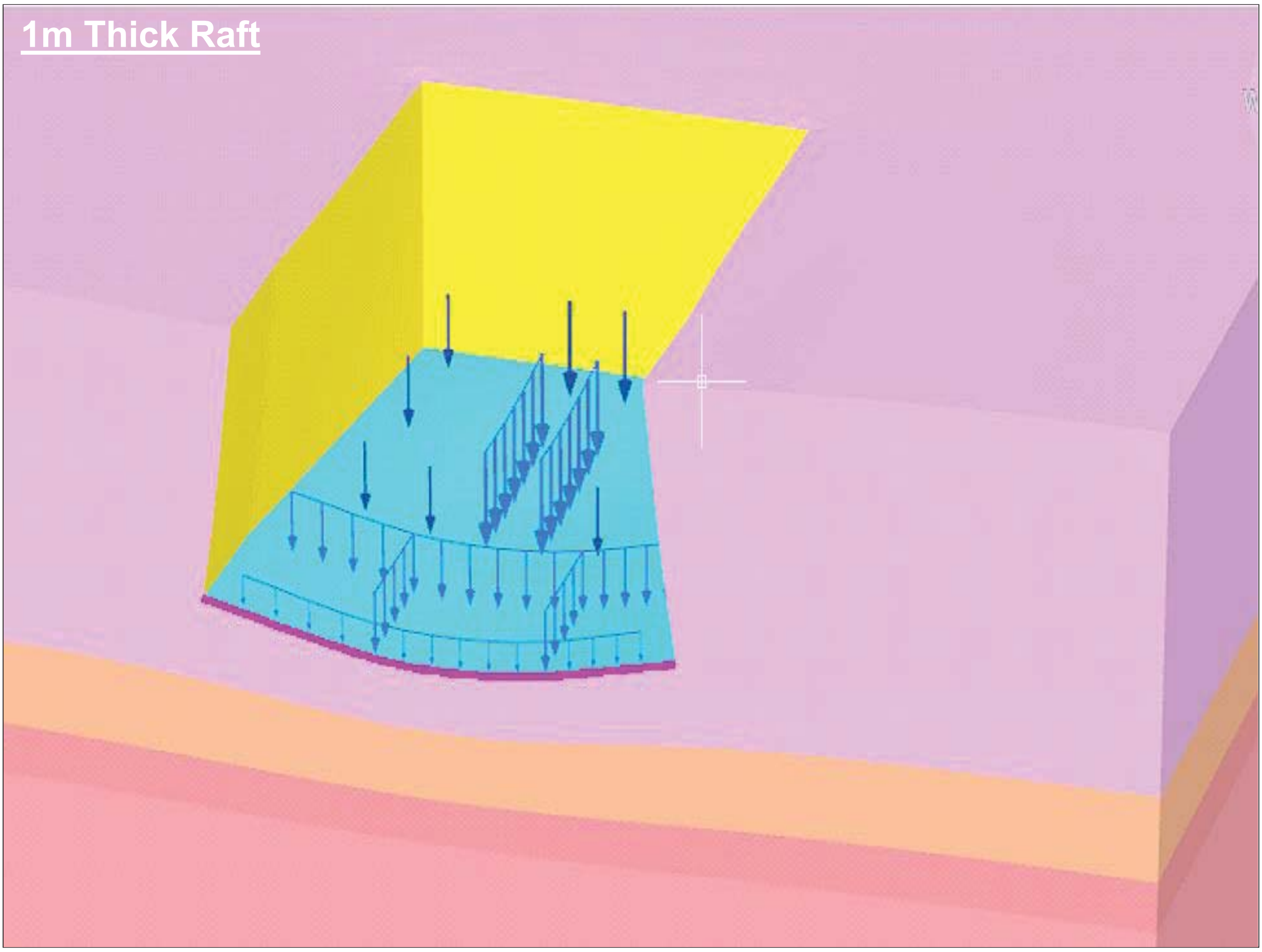
3)Rigidity of Raft & superstructure

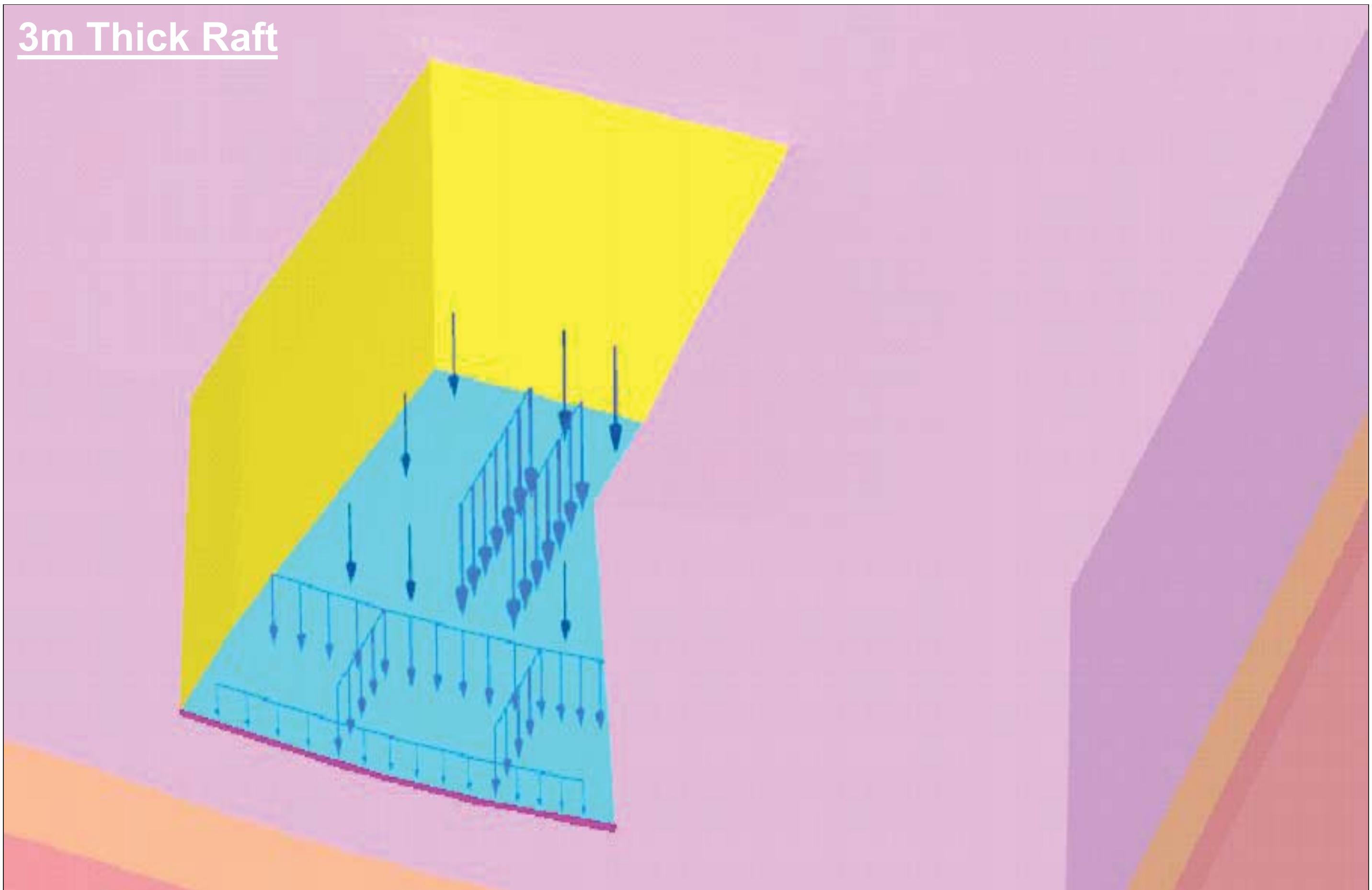

SWASTIK METRO BUILDING, BANGALORE
STRUCTURAL CONSULTANT: STERLING ENGINEERING
Layer 2,
E = 1, 45,000 T/m2
Layer 3, Hard Rock
E = 14,00,000 T/m2


SWASTIK METRO BUILDING, BANGALORE
STRUCTURAL CONSULTANT: STERLING ENGINEERING
Z
Y


SWASTIK METRO BUILDING, BANGALORE
STRUCTURAL CONSULTANT: STERLING ENGINEERING
K1
K2
K3
K1 < K2 < K3
Y
X
Z

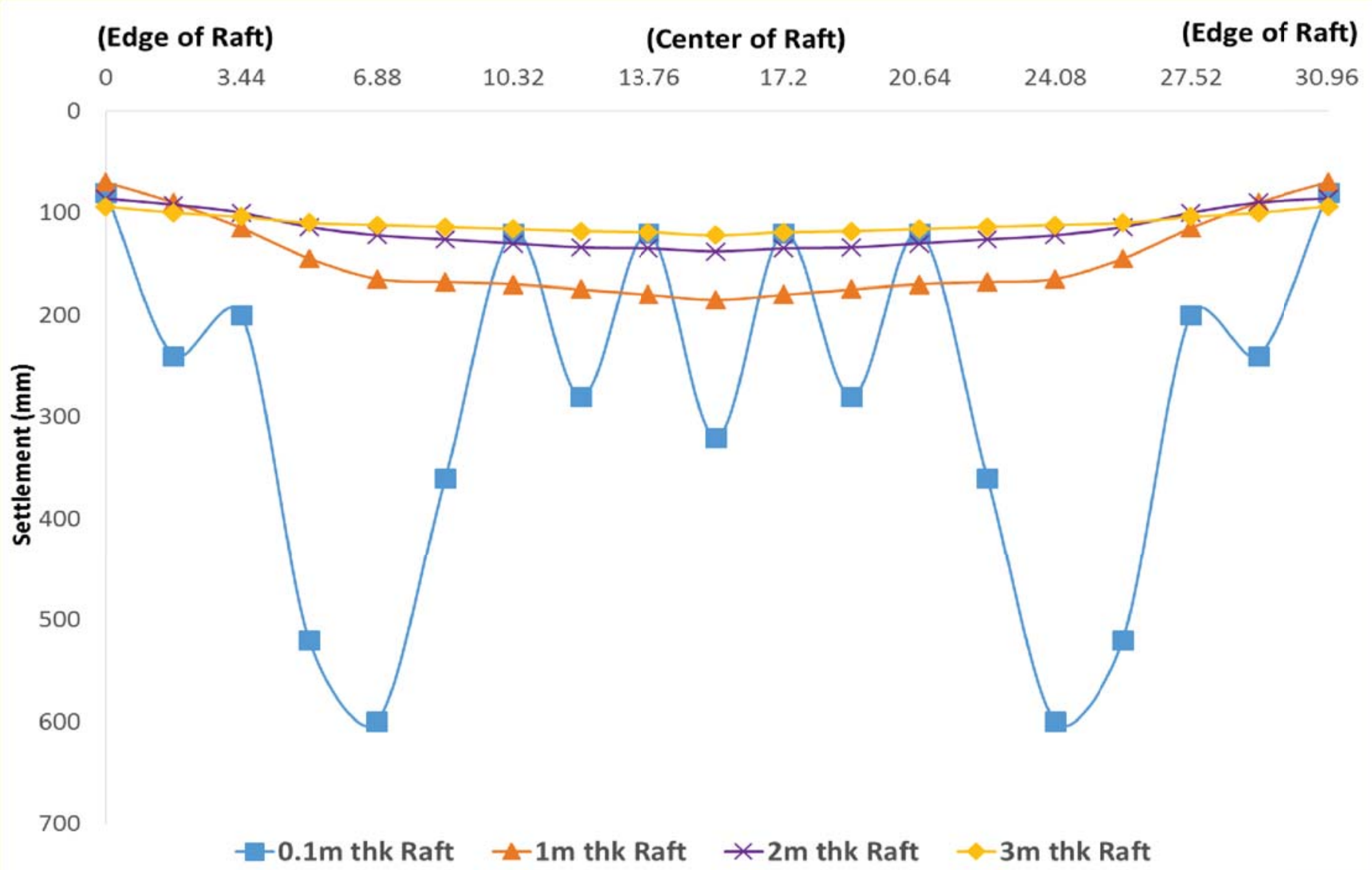

0.1m Thick Raft



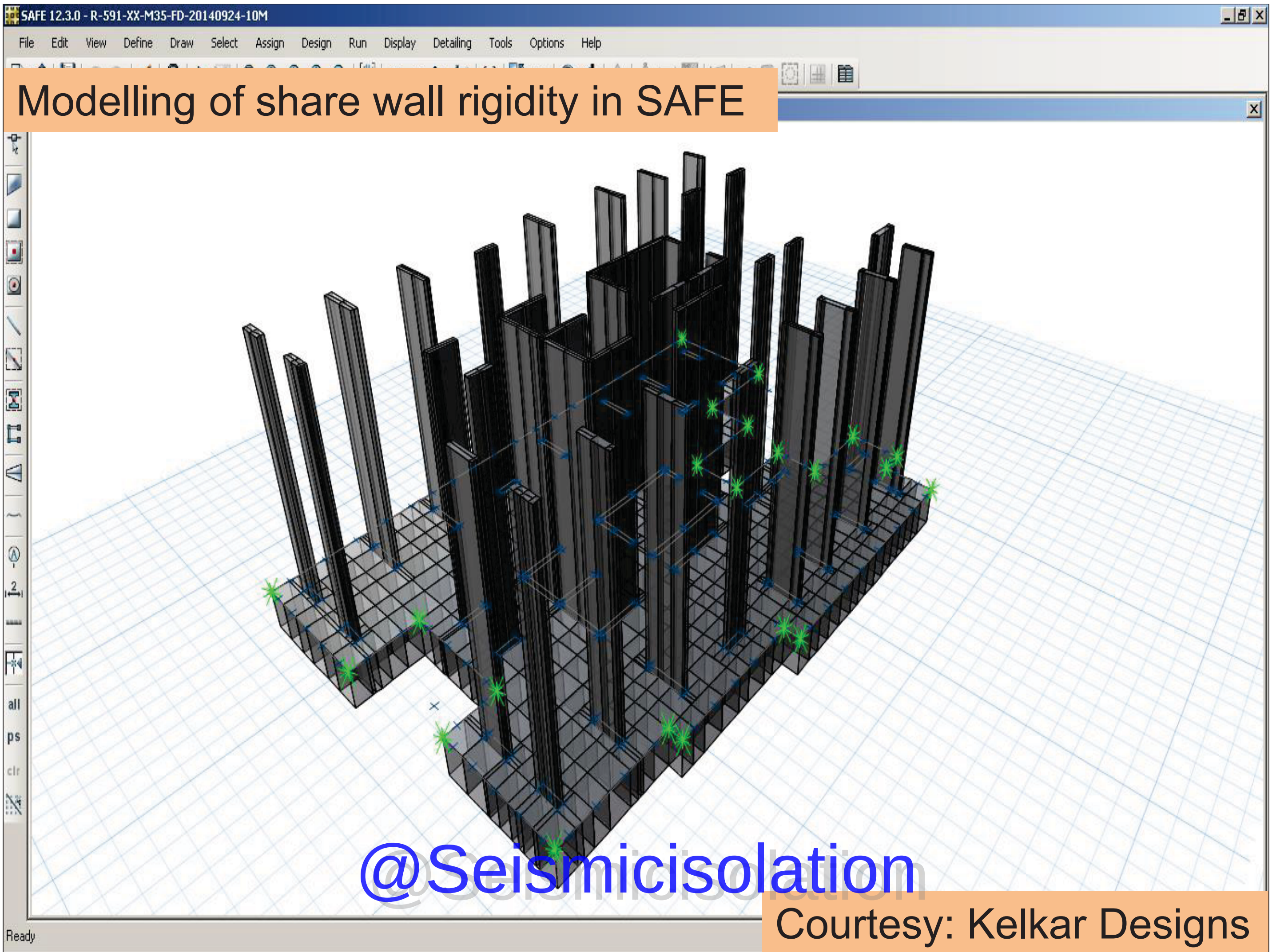


SETTLEMENTS ARE MUCH LOWER IN A 3M THICK RAFT FOUNDATION. SUGGESTING THAT AN ITERATIVE ANALYSIS BETWEEN GEOTECHNICAL AND STRUCTURAL ENGINEERS CAN HELP ECONOMIZ

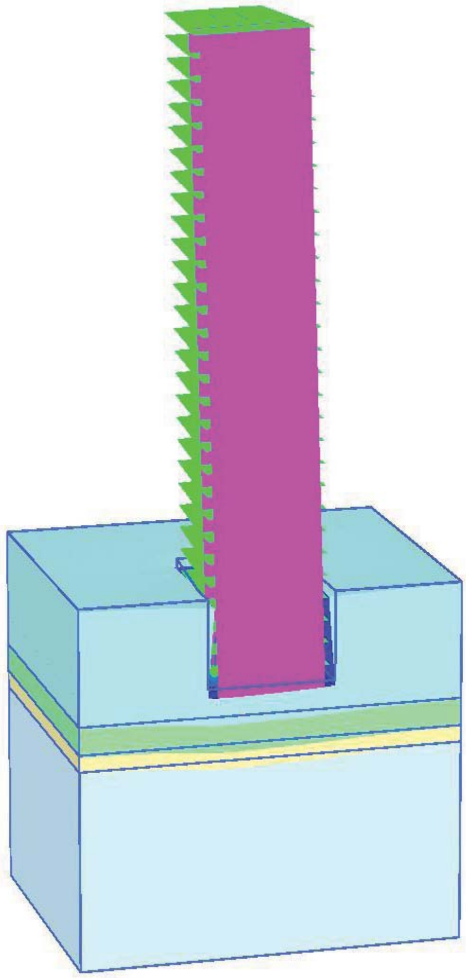
SETTLEMENT WITH VARYING THICKNESS



Modelling of share wall rigidity in SAFE



Courtesy: Kelkar Designs

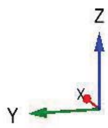


NOVA TOWER

MODELING OF WIND +Y CONDITION

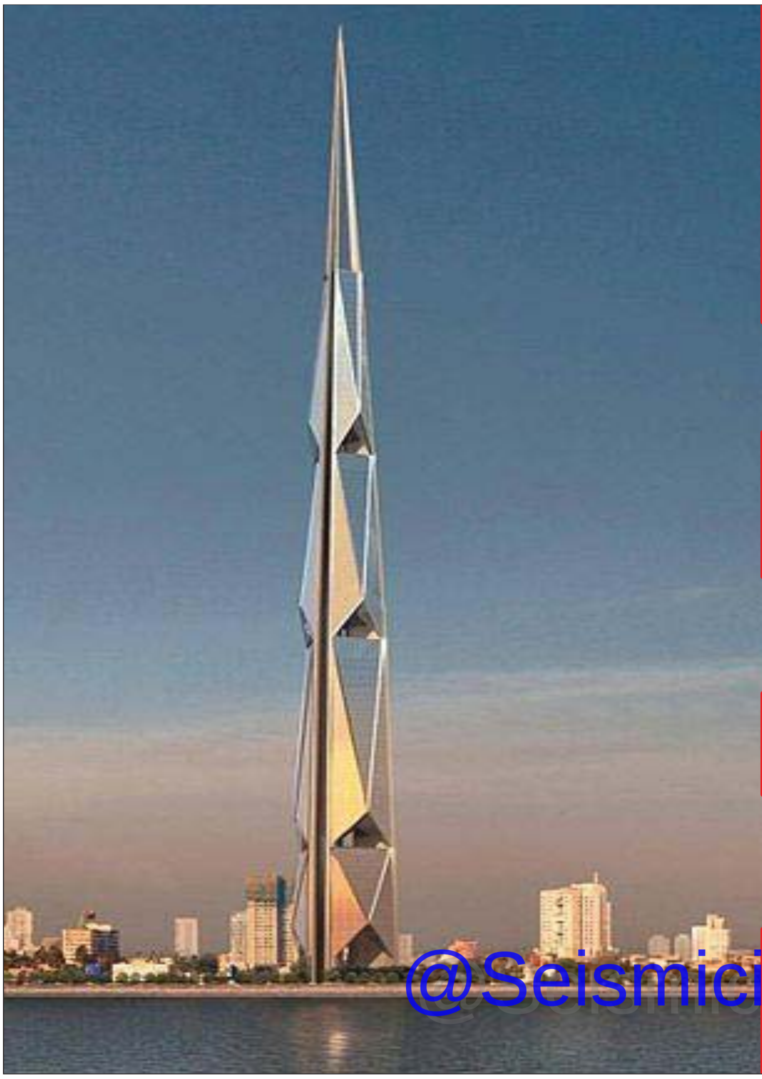
INCORPORATING STRUCTURAL RIGIDITY OF SUPERSTRUCTURE.

Structural Consultant: Buro Happold



MIDASGEN SOFTWARE HAS CAPABILITY OF INCORPORATING COLUMNS AND BEAM RIGIDITY IN RAFT SETTLEMENT ANALYSIS.

GEOTECHNICAL CASE HISTORIES OF FEW TALL BUILDINGS IN INDIA



INDIA TOWER , MUMBAI

(6B+G+130 FLOORS)

CLIENT: DB REALTY

RCC CONSULTANT: MAHIMTURA / WSP

The highest allowable bearing capacity utilized in India for shallow foundations (500 t/m2)

Extensive geotechnical investigation completed.

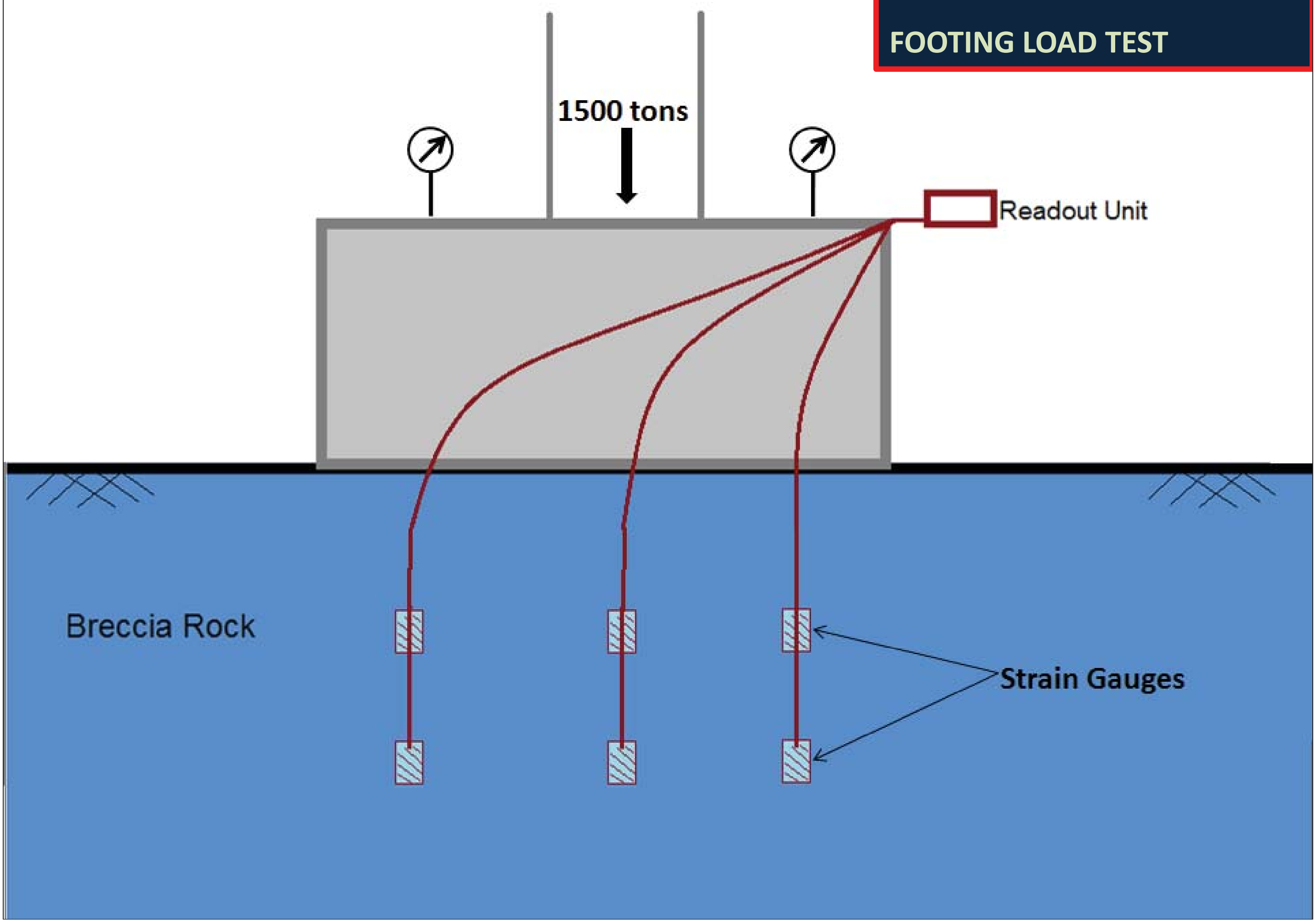
Soil structure interaction with finite element analysis helped reduce raft thickness from 6.5m to 5m.


INDIA TOWER , MUMBAI
6 BASEMENTS 27m DEEP
CLIENT: DB REALTY


INDIA TOWER , MUMBAI
FOOTING LOAD TEST

INDIA TOWER , MUMBAI

FOOTING LOAD TEST



“PALAIS ROYALE”
CLIENT: SHREE RAM URBAN

3B + G + 75 FLOORS

CURRENTLY ONE OF THE TOP THREE TALLEST BUILDINGS IN INDIA

STRUCTURAL CONSULTANT: STERLING ENGINEERING

SOIL STRUCTURE INTERACTION

"PALAIS ROYALE"
CLIENT: SHREE RAM URBAN

3B + G + 75 FLOORS

STRUCTURAL CONSULTANT: STERLING ENGINEERING

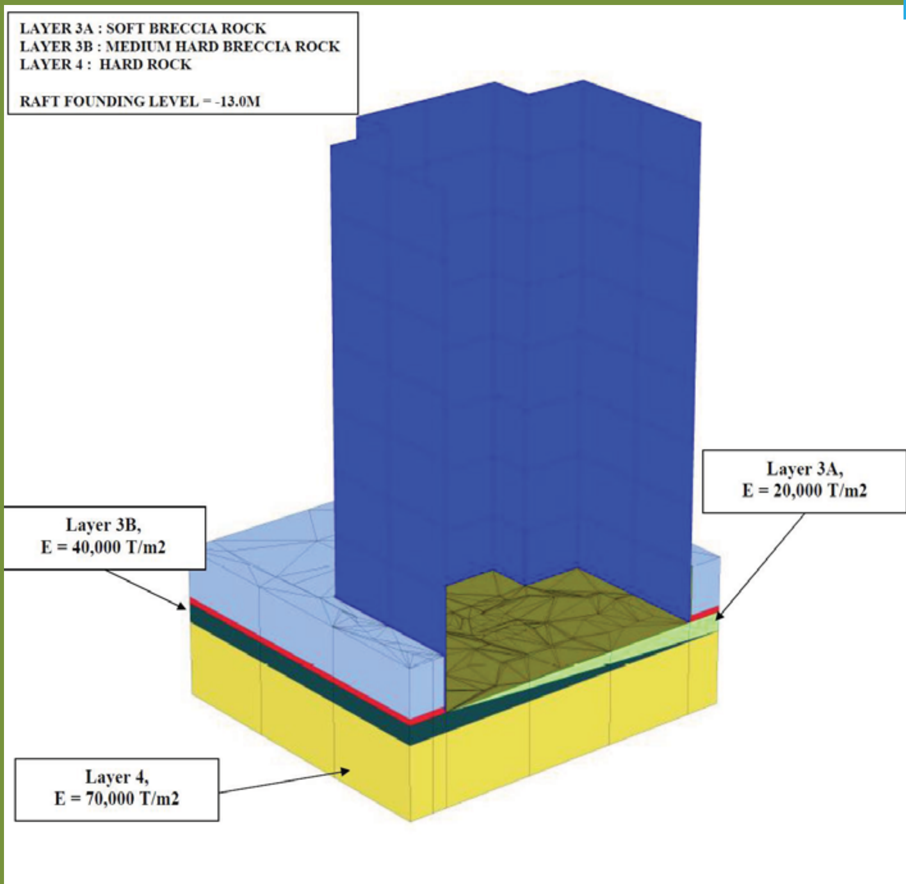
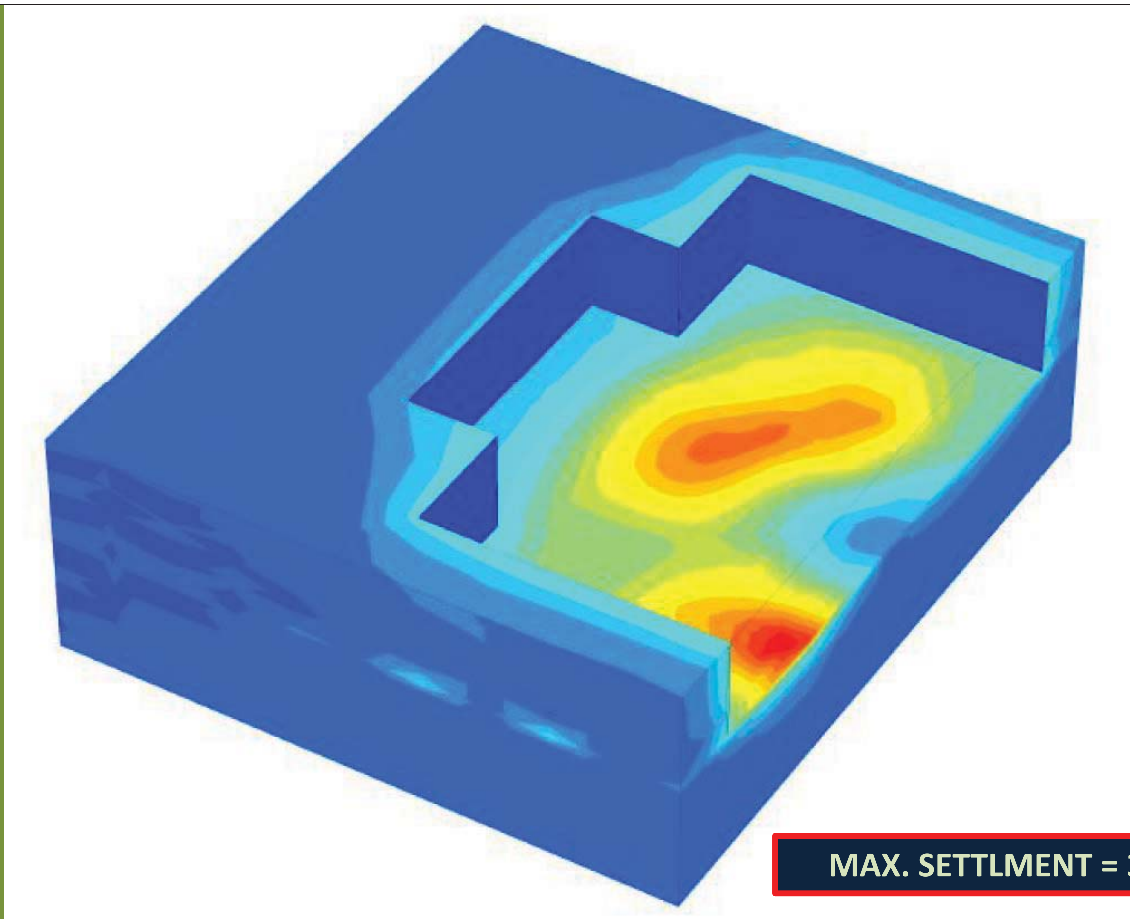


FIG 2: PLAXIS MODEL

ONLY LEFT HALF PORTION OF THE BUILDING IS ANALYZED DUE TO SYMMETRY



MAX. SETTLMENT = 30mm

FIG 3: SETTLEMENT CONTOURS (3D VIEW)

ONLY LEFT HALF PORTION OF THE BUILDING IS ANALYZED DUE TO SYMMETRY

NATHANI HEIGHTS, MUMBAI

2 B + G + 70 FLOORS

STRUCTURAL CONSULTANT: THORNTUN TOMASETTI

ONE OF THE MOST SLENDER BUILDINGS IN THE WORLD WITH SLENDERNESS RATIO OF 1:11.

PROBABLY THE HIGHEST PILE CAPACITY UITILIZED IN INDIA (1700 TONS ON A 1.2m DIA PILE)

PILE LOAD TEST CONDUCTED UP TO A TEST LOAD OF 2500TONS.

PILE LOAD TEST IN PROGRESS

PILE DIAMETER = 1200mm

MAX. PILE TEST LOAD = 2500 TONS

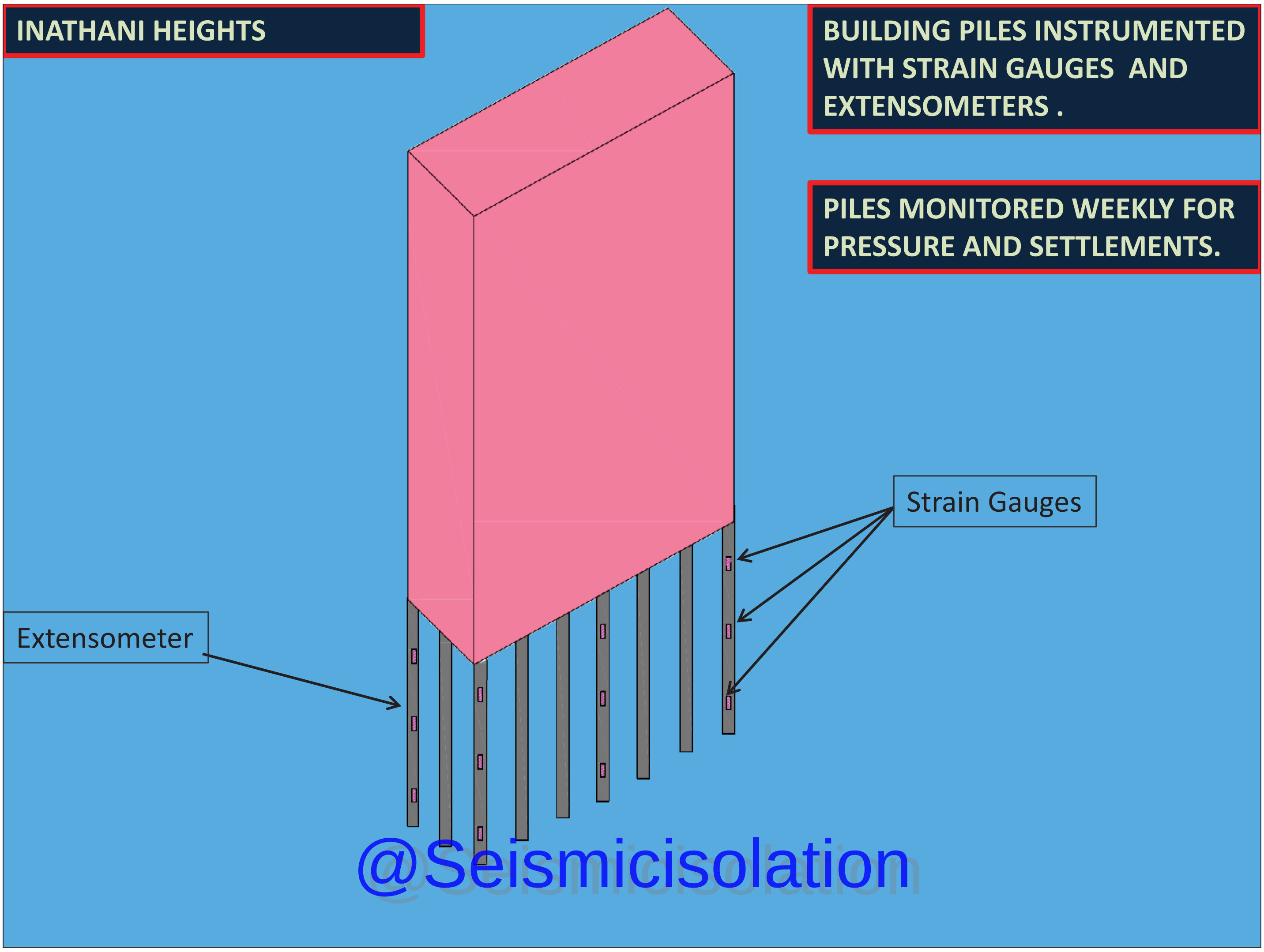
MAX. SETTLEMENT = 5mm

NATHANI HEIGHTS, MUMBAI

STRUCTURAL CONSULTANT: THORNTUN TOMASETTI

TESTING AGENCY: STEFFON


NATHANI HEIGHTS, MUMBAI
STRUCTURAL CONSULTANT:
THORNTUN TOMASETTI
PILE DIAMETER = 1200mm
PILE CAPACITY = 1700 TONS

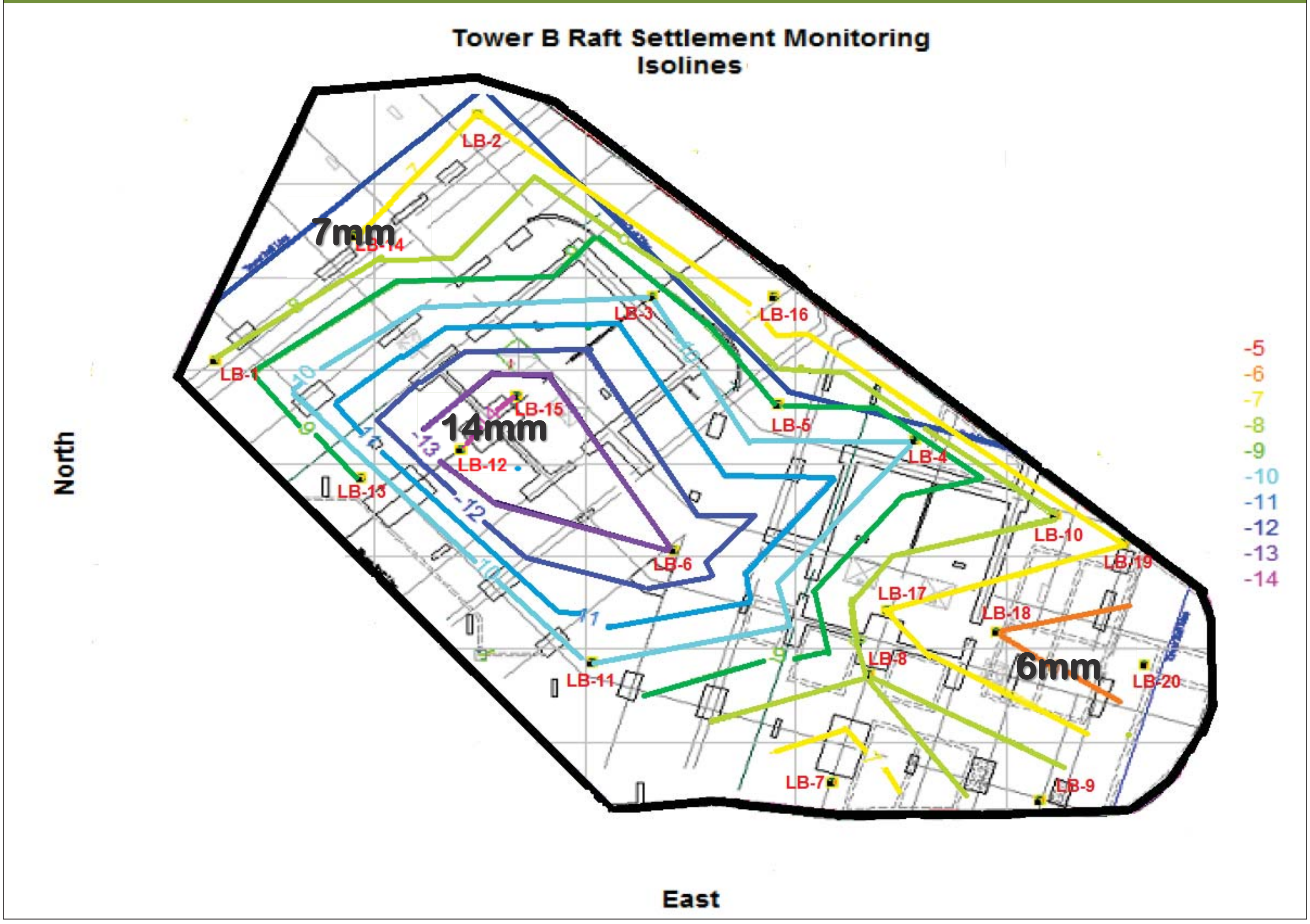

INATHANI HEIGHTS
BUILDING PILES INSTRUMENTED WITH STRAIN GAUGES AND EXTENSOMETERS .
PILES MONITORED WEEKLY FOR PRESSURE AND SETTLEMENTS.
Strain Gauges
Extensometer

OASIS TOWER

CLIENT: OBEROI REALTY

STRUCTURAL CONSULTANT: LERA

3B + G + 85 FLOORS



WAVE CITY CENTER, NOIDA

RCC CONSULTANT: AECOM

"THE 42", KOLKATA

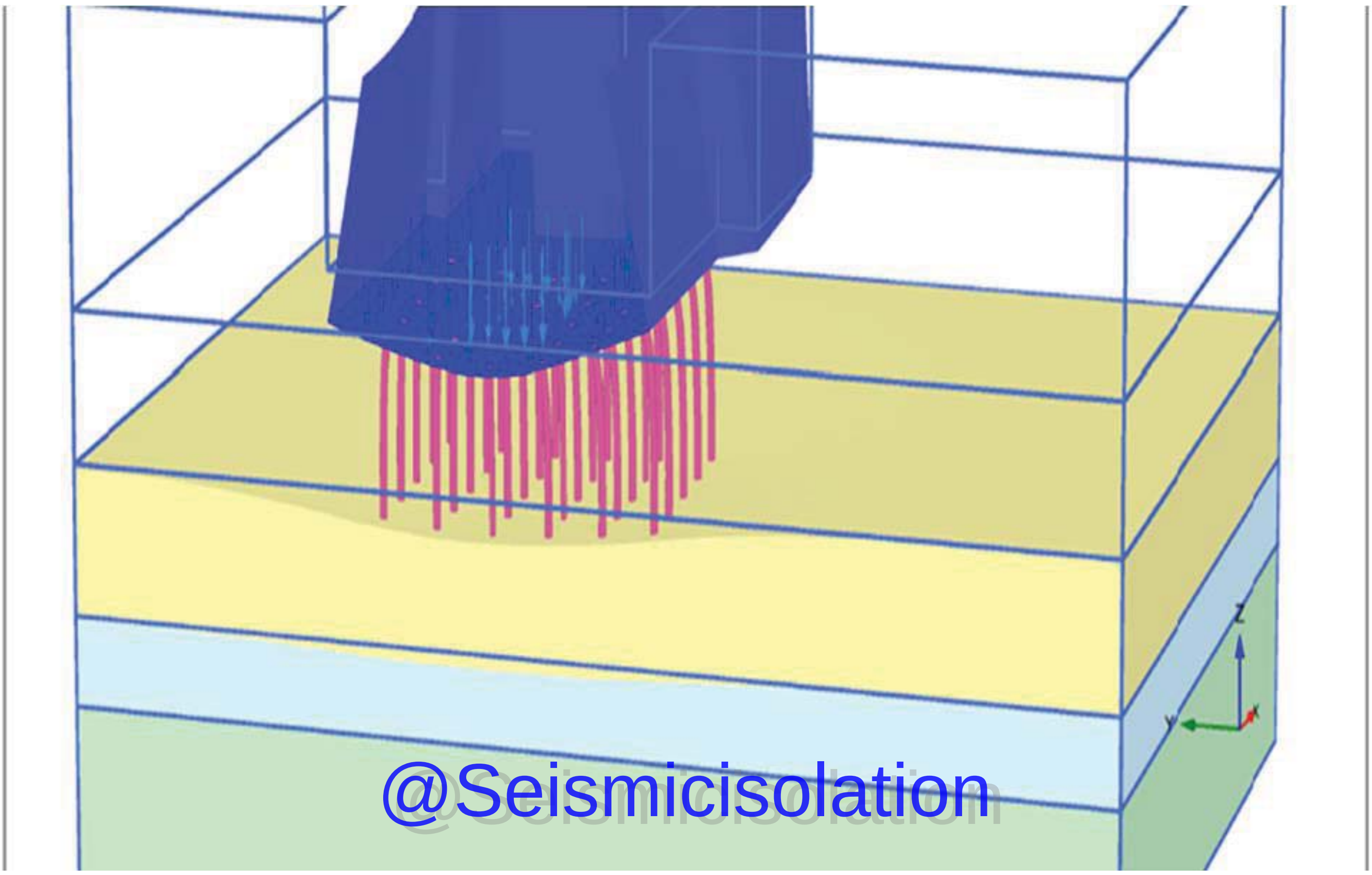
G + 60 FLOORS

STRUCTURAL CONSULTANT: JW CONSULTANTS

GROUND AT THIS SITE NOT CAPABLE OF SUPPORTING BUILDING ON CONVENTIONALLY DESIGNED FOUNDATIONS.

SOIL STRUCTURE INTERACTION CONDUCTED TO VERIFY THAT ALL DIFFERENTIAL SETTLEMENTS ARE WITHIN PERMISSIBLE LIMIT.

"THE 42", KOLKATA



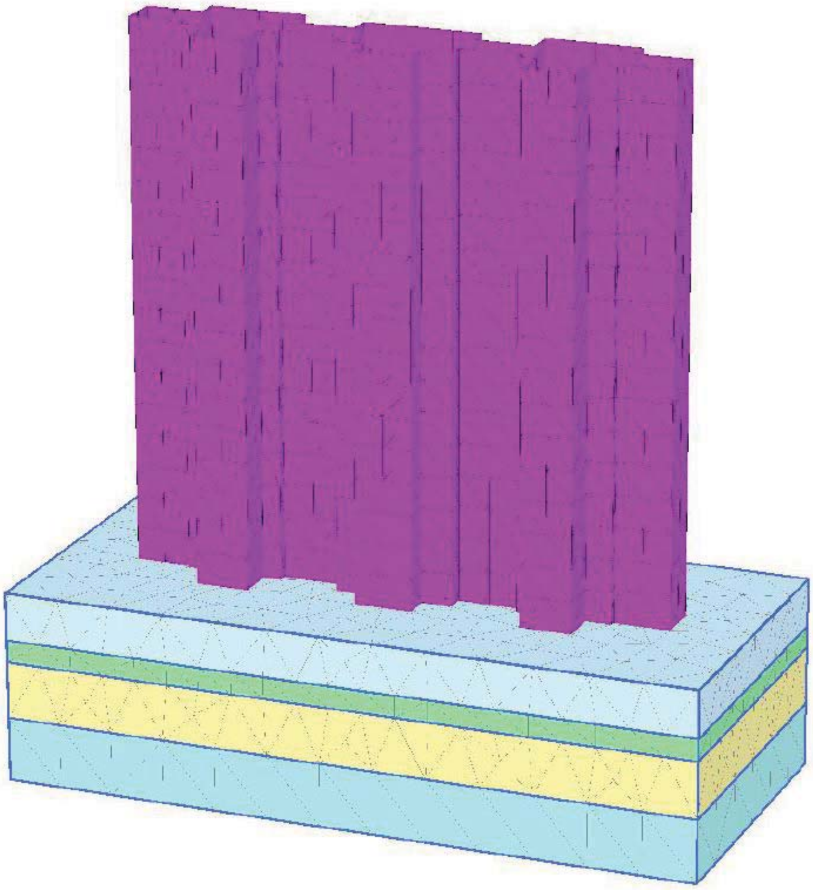
SETTLEMENT REDUCING PILES

RAHEJA REVANTA, GURGAON | **RCC CONSULTANT: THORNTUN TOMASETTI**

BEFORE OUR INVOLVEMENT BUILDING PROPOSED ON FULL FLEDGED PILE FOUNDATION.

OUR INVOLVEMENT RECOMMENDED BY STRUCTURAL CONSULTANTS

NUMBER OF PILES REDUCED TO 25%.



RAHEJA REVANTA,

Settlement Reducing Piles

RAHEJA REVANTA

1) Piles placed at strategic locations only to reduce settlements.

2) Piles designed for ultimate pile capacities. i.e. piles are designed to fail.

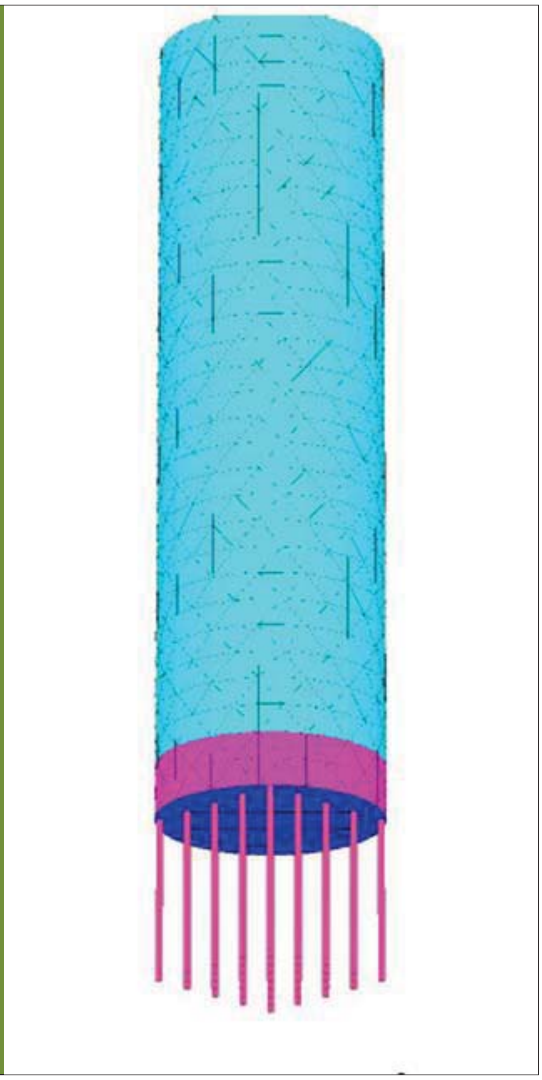
ORB TOWERS, NOIDA

2B + G + 50

ONLY 41 PILES

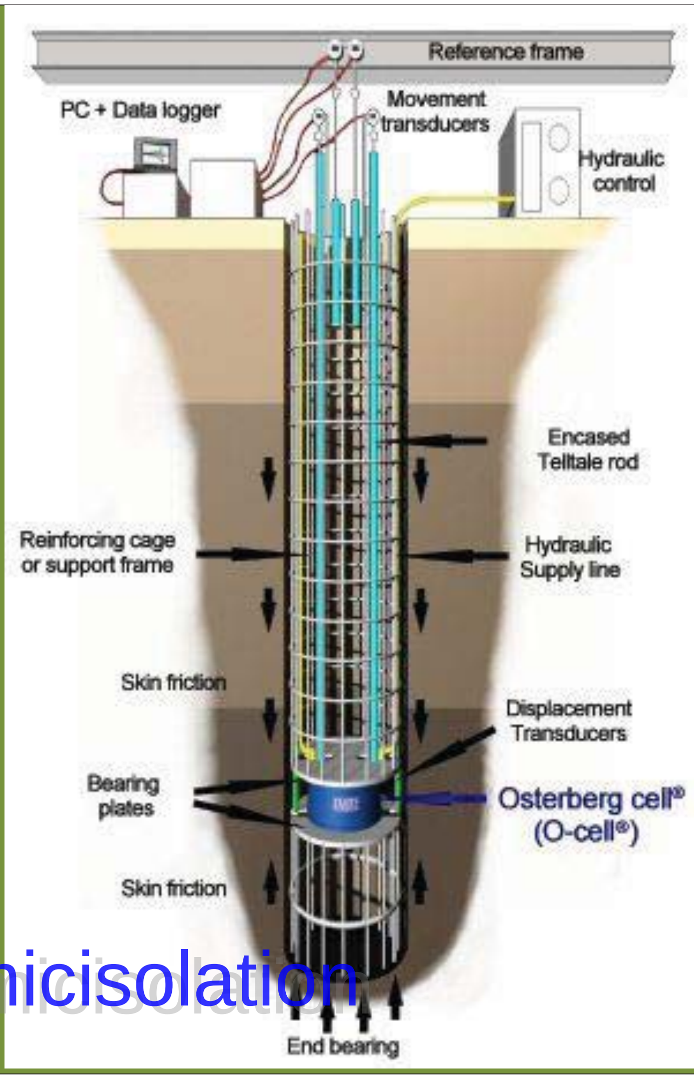
85% of load taken by Raft
15% of load taken by Piles

Structural Consultant:
VMS Consultants



GEOTECHNICAL INVESTIGATION FOR SPIRA TOWER

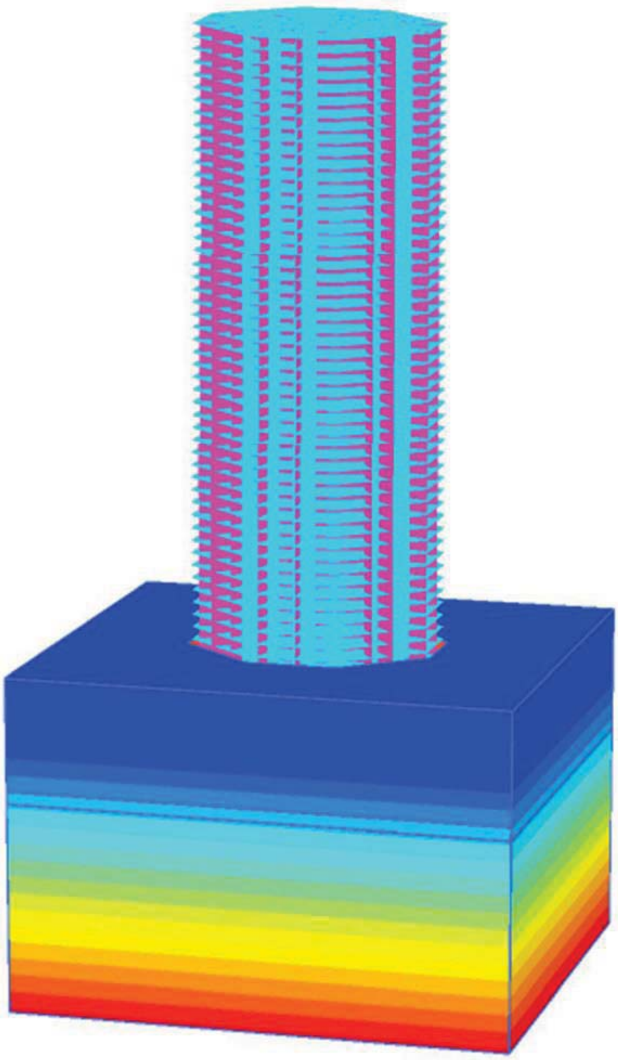
- **24 pressuremeter tests.**
- **4 static cone penetration tests.**
- **2 static pile load tests.**
- **1 instrumented Osterberg Load Cell test.**

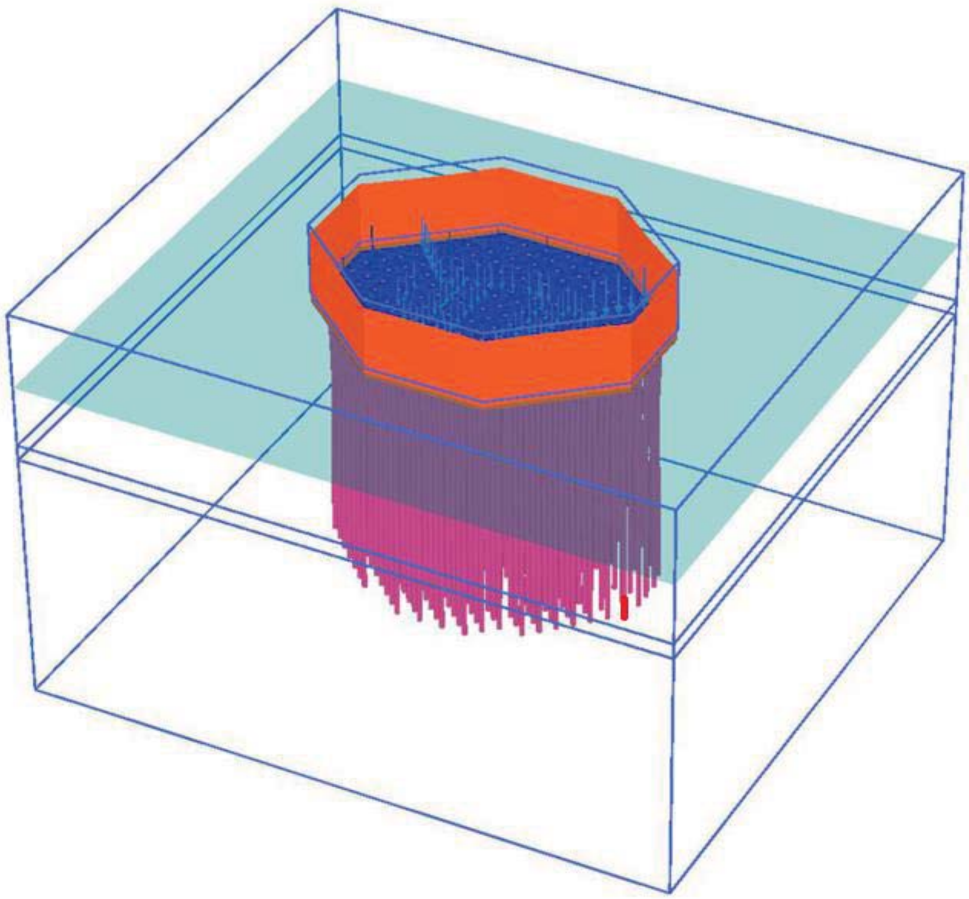


O CELL TESTING BY FUGRO LOAD CELL

ICONIC TOWER (SUPERNOVA)

SOIL STRUCTURE INTERACTION USING FINITE ELEMENT ANALYSIS





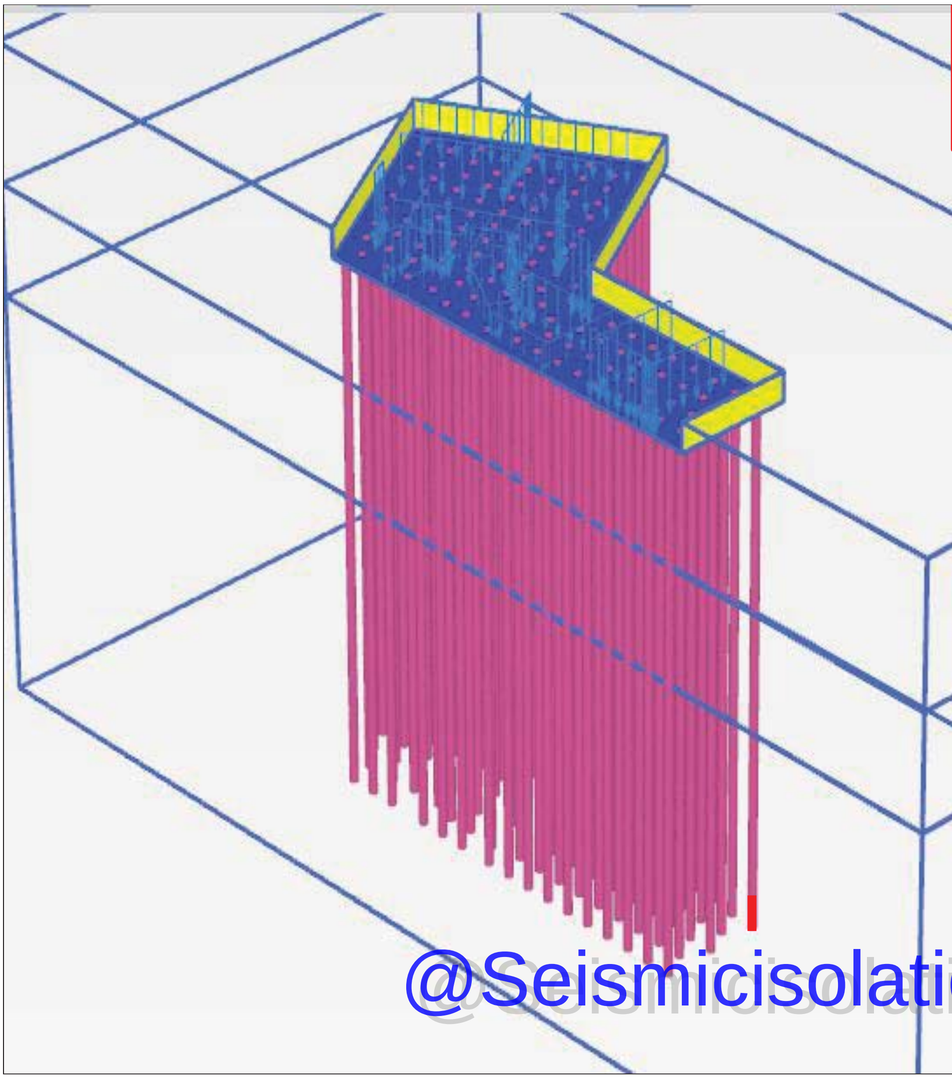
ICONIC TOWER (SUPERNOVA)

1.2m DIA PILES.
LOAD ON PILES 1500 TONS UNDER STATIC LOADS
AND 1900 TONS UNDER WIND LOADS.

BRYS BUZZ, NOIDA

G + 81 FLOORS

STRUCTURAL CONSULTANT:
BEST CONSULTING ENGINEERS, DUBAI



BRYS BUZZ, NOIDA

PCC PILES

THANK YOU VERY MUCH.