3rd Conference of IWGoRS, Christchurch, NZ, September 23rd-25th, 2013

Eurocode 8 estimation of rotational ground motion effects on the Bell Tower of Parma Cathedral

by **Rossi Andrea[1], Zembaty Zbigniew[2], Spagnoli Andrea[1],**

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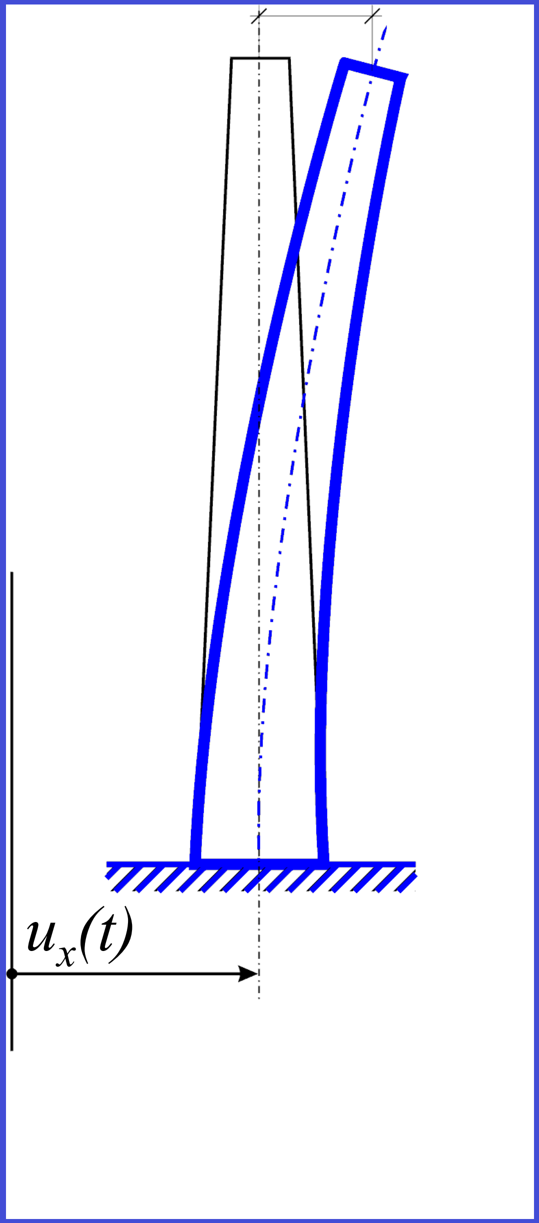
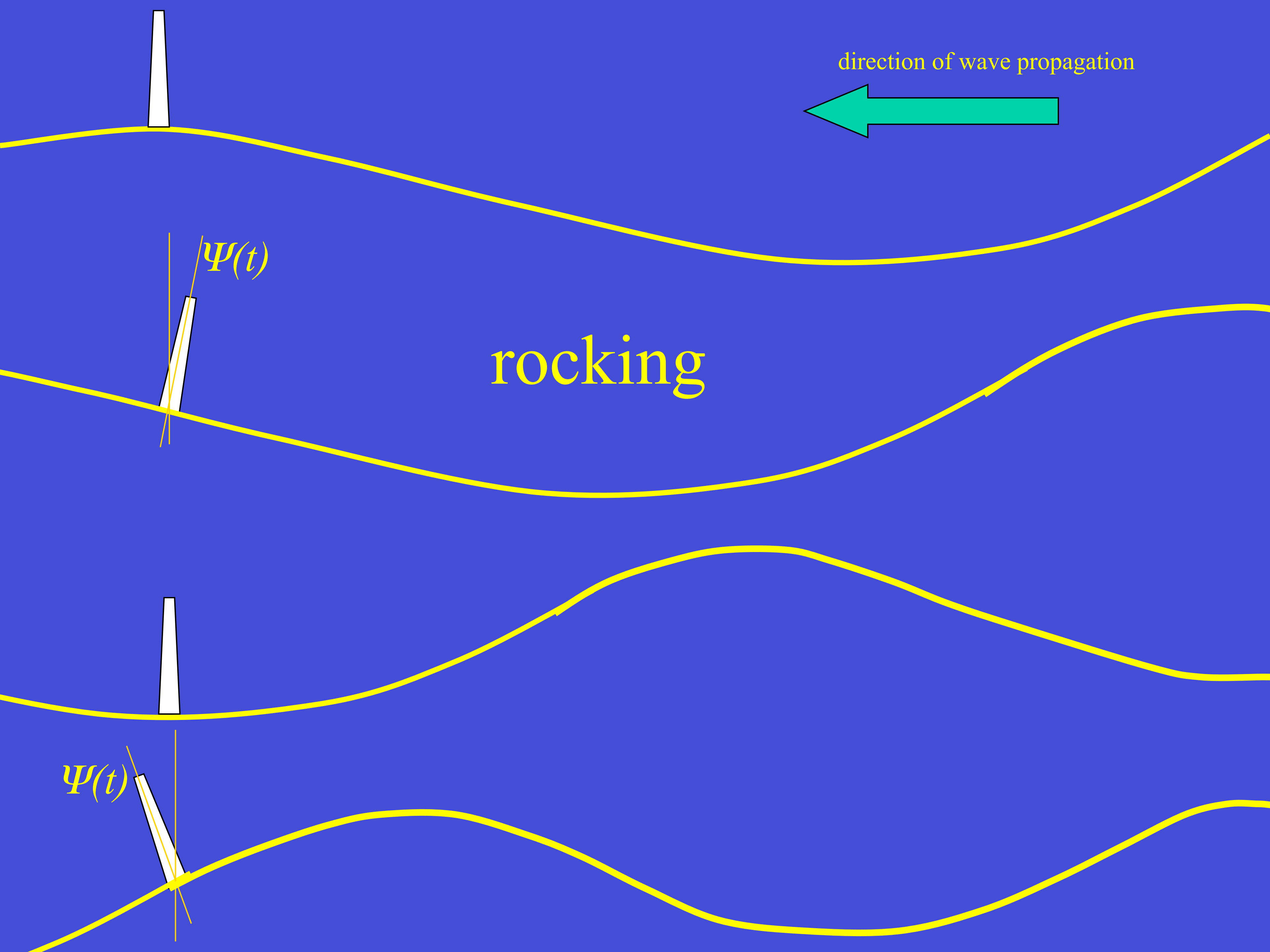
$q(t)=q_{hor}$

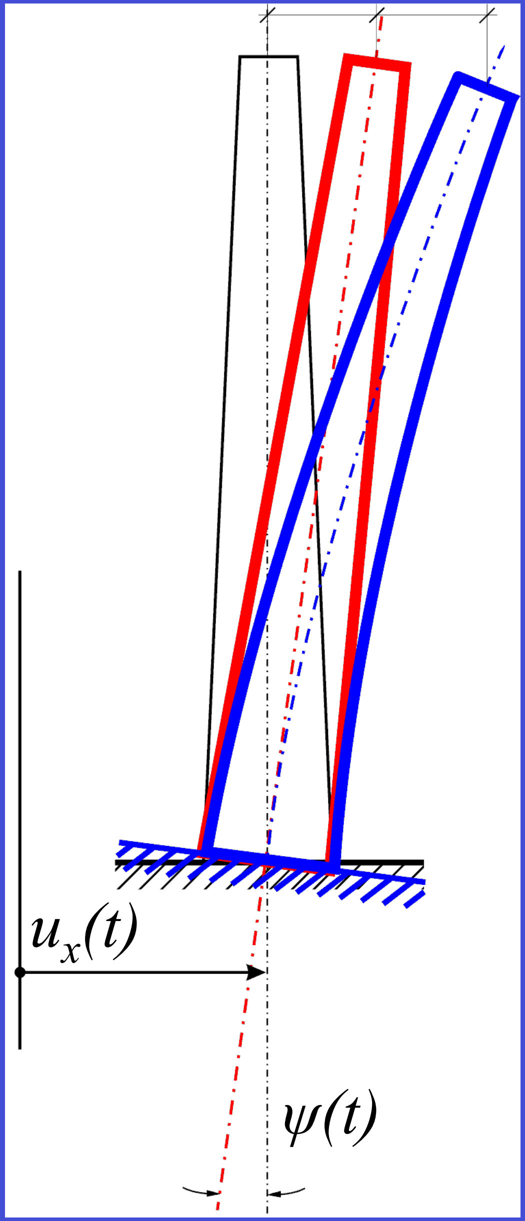


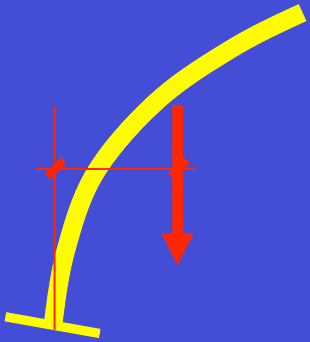
Typical assumption: only HORIZONTAL seismic excitations

Slender tower under seismic, horizontal excitations


direction of wave propagation
rocking
Ψ(t)
Ψ(t)

$$q(t) = q_{rot} + q_{hor}$$

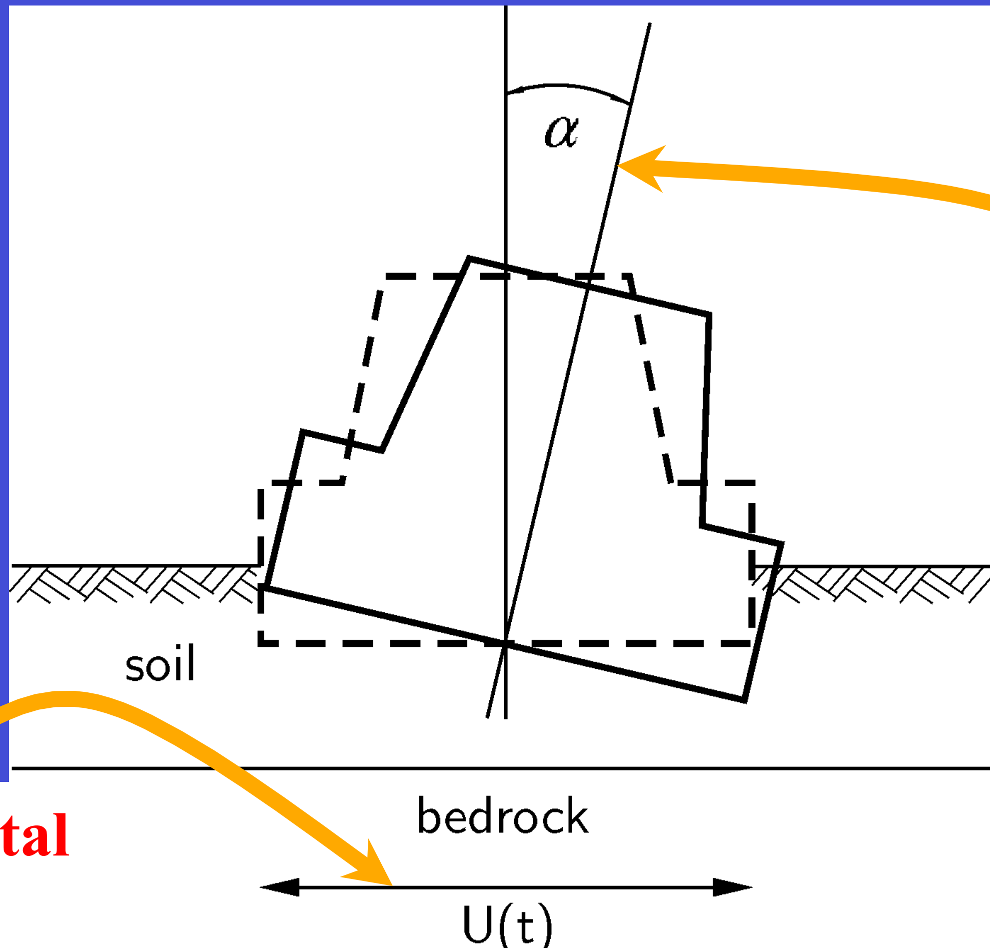




2nd order effects

Slender tower under seismic, <u>horizontal & rocking</u> excitations

A massive structure on a compliant soil



RESPONSE, not excitations

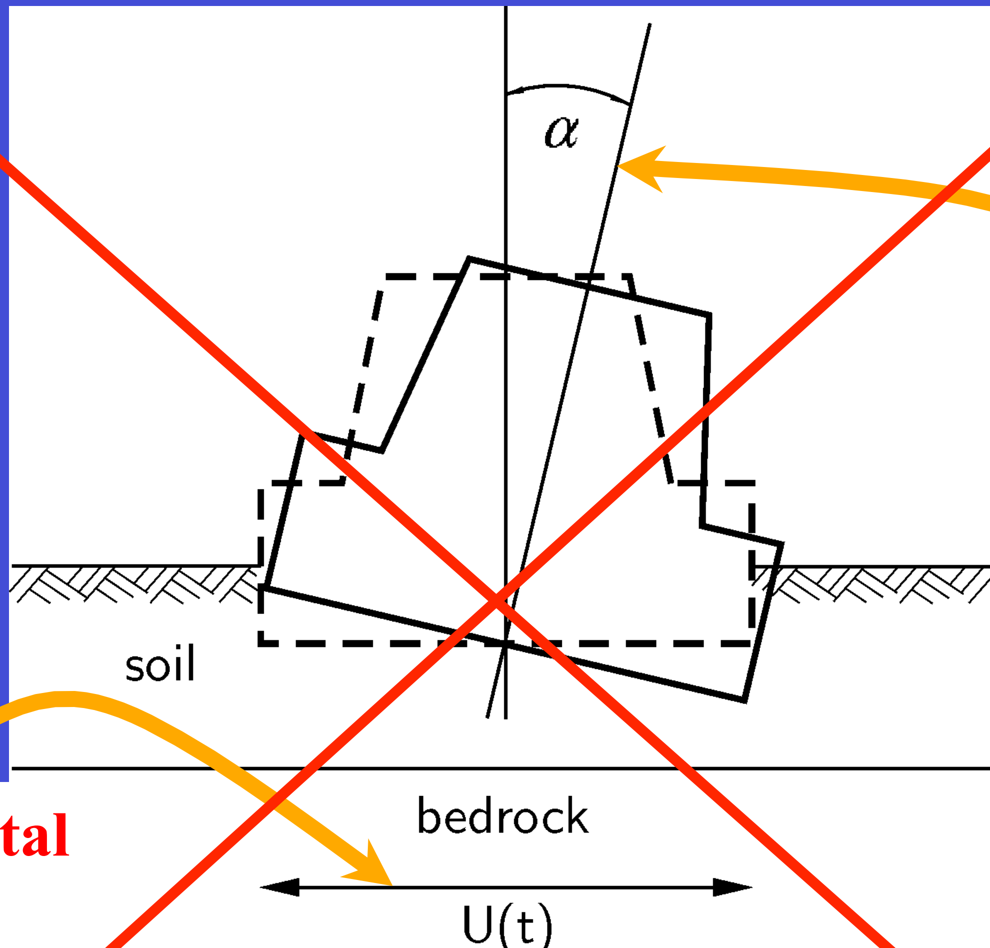
ONLY horizontal excitations

SSI effects

Some times such (response) rotations from very complaint (weak) soil and strong <u>horizontal</u> excitations can be very serious

Photograph taken after Kocaeli (1999) earthquake in Turkey

A massive structure on a compliant soil



RESPONSE, not excitations

SSI effects

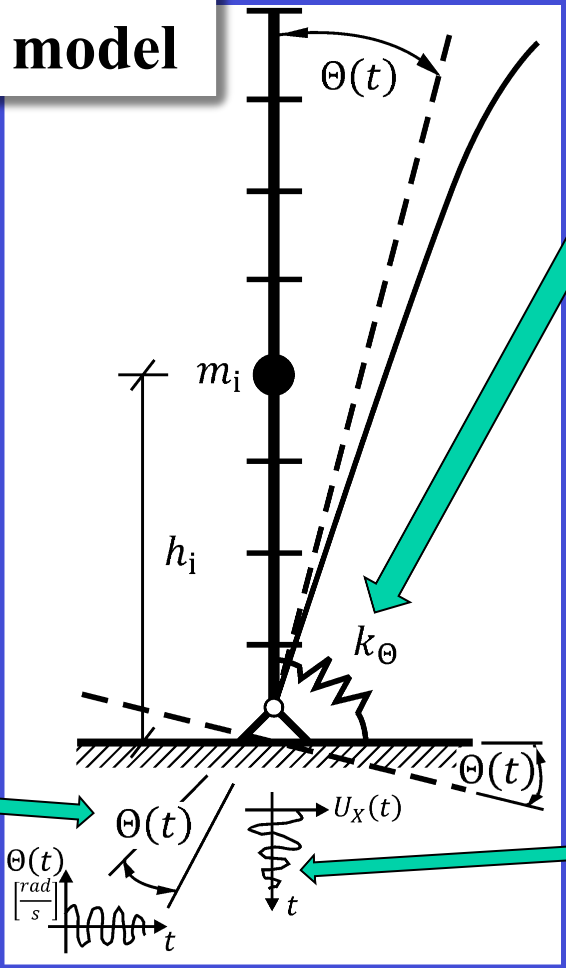
ONLY horizontal excitations

This is not what we consider by seismic rotational load

Conclusion:

Except for the structures founded directly on rock the **structural response due to rotational <u>excitations</u>** should be combined with rocking effects from soil compliance (or even SSI)

structural model



rotational excitations

horizontal excitations

Equation of motion of a plane, discrete system under horizontal kinematic excitations $u(t)$

$$\boldsymbol{M}\ddot{\boldsymbol{u}} + \boldsymbol{C}\dot{\boldsymbol{u}} + \boldsymbol{K}\boldsymbol{u} = -\boldsymbol{m}\ddot{u}$$

M - diagonal mass matrix

m - vector of masses $m_1, m_2, \ldots, m_n$

We may solve it using results of the eigenproblem (natural frequencies ω_i , modes $\mathbf{w}_i$) and mode superposition method

$$q_j(t) = \sum_{i=1}^{n} q_{ji}(t)$$

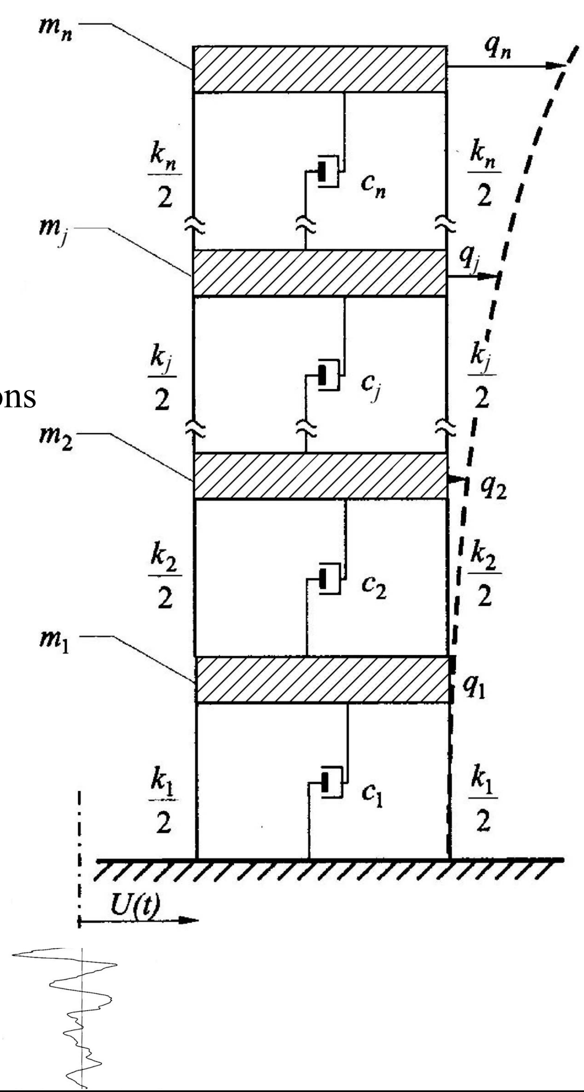
natural modes

modal impulse response functions

$$q_{ji} = w_{ji}\eta_i \int_0^t \ddot{u}(\tau)h_i(t-\tau)d\tau$$

modal participation factors

$$\eta_i = \frac{\boldsymbol{w}_i^{Tr}\boldsymbol{M}\boldsymbol{1}}{\boldsymbol{w}_i^{Tr}\boldsymbol{M}\boldsymbol{w}_i}$$



Equation of motion of a plane, discrete system under horizontal kinematic excitations *u*(*t*) – solution by response spectrum method

$$\boldsymbol{M\ddot{u}} + \boldsymbol{C\dot{u}} + \boldsymbol{Ku} = -\boldsymbol{m}\ddot{u}$$

M - diagonal mass matrix

m - vector of masses $m_1, m_2, \ldots, m_n$

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modal participation factors

$$\eta_i = \frac{\boldsymbol{w}_i^{Tr}\boldsymbol{M1}}{\boldsymbol{w}_i^{Tr}\boldsymbol{M}\boldsymbol{w}_i}$$

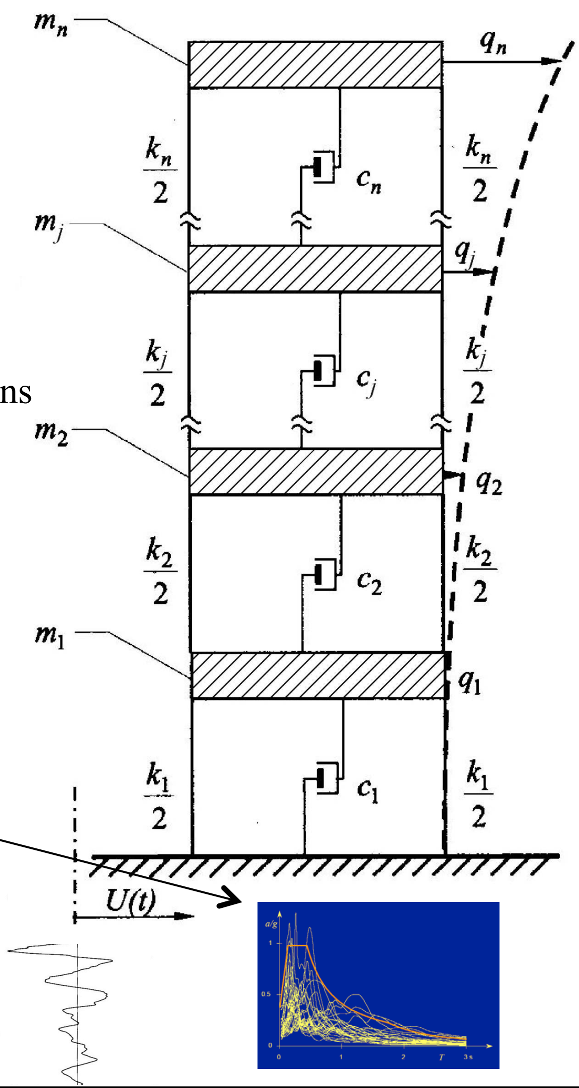
using response spectrum method
one can asses maxima the modal displacements

smooth, design
acceleration response spectrum

$$\max q_{ji} = \left| w_{ji}\eta_i \frac{1}{\omega_i^2} S_a(\xi_i, \omega_i) \right|$$

Total displacement response by the SRSS rule

$$q_j \cong \sqrt{q_{j1}^2 + q_{j2}^2 + q_{j3}^2 + \cdots}$$



Equation of motion of a plane, discrete system under combined horizontal excitations $u_x(t)$ and rocking excitations $\theta(t)$

$$\boldsymbol{M\ddot{u}} + \boldsymbol{C\dot{u}} + \boldsymbol{Ku} = -\left(\boldsymbol{m}\ddot{u} + \{\boldsymbol{mh}\}\ddot{\theta}\right)$$

{mh} - vector containing multiplications

$m_1{\cdot}h_1$, $m_2{\cdot}h_2$, … $m_i{\cdot}h_i$, …, $m_n{\cdot}h_n$,

modal displacement response from horizontal excitations

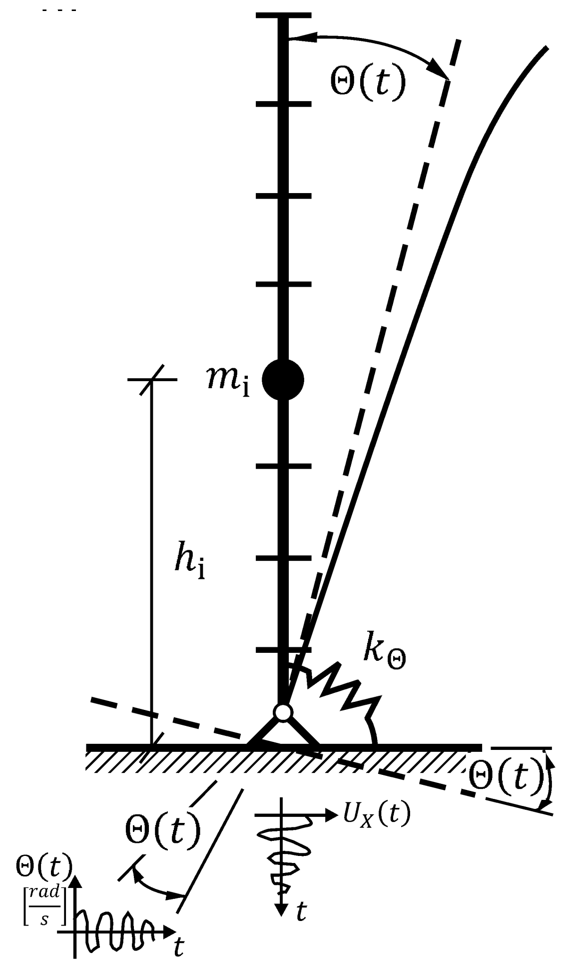
$$\max q_{ji} \cong \left| w_{ji}\eta_i \frac{1}{\omega_i^2} S_a(\xi_i, \omega_i) \right|$$

modal displacement response from rotational excitations

$$\max q_{ji}^{rot} \cong \left| w_{ji}\eta_i \frac{1}{\omega_i^2} S_a^{\ddot{\theta}}(\xi_i, \omega_i) \right|$$

total displacements again by SRSS rule

$$\max q_j = \sqrt{\sum_{i=1}^{n} (\max q_{ji})^2 + \sum_{i=1}^{n} (\max q_{ji}^{rot})^2}$$



Base shear & bending moments:

For the high buildings and slender towers, important response measures can be base shears & overturning moments

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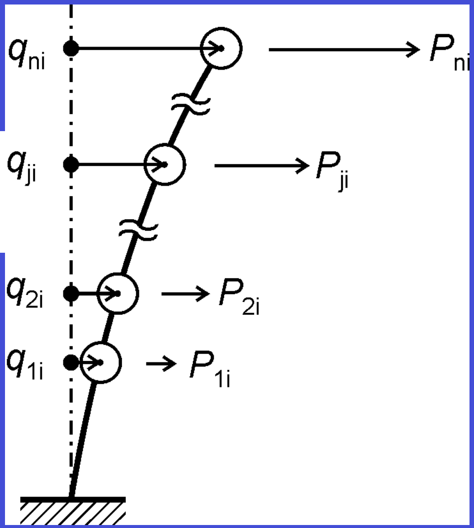
First we define equivalent pseudo-static seismic forces (i.e. leading to the same response displacements)

$$P_{ji} = m_j w_{ji} \eta_i S_a(\omega_i, \xi_i)$$



Base shear & bending moments:

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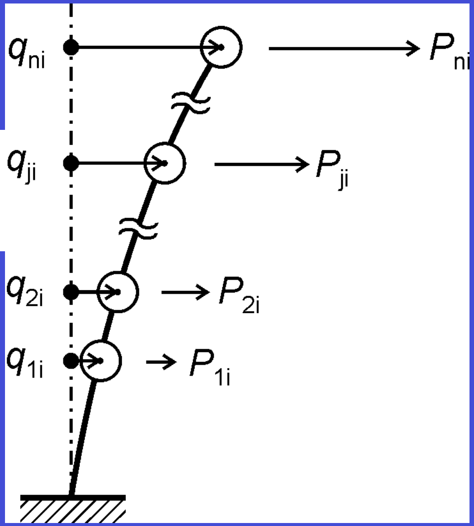


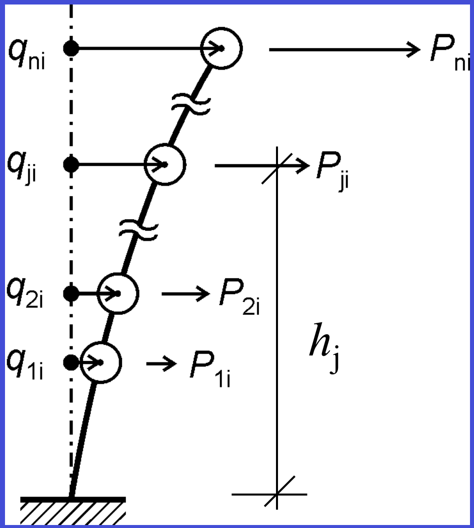
ignoring damping effects in force response and summing down all the P_{ji} forces one arrives at so called BASE SHEAR force which is a good, approximate measure of seismic load for any structure
After some algebra, for the *i*-th mode, this force equals:

$$F_{base\,\boxed{i}} = \frac{\left(\sum_k m_k w_{ki}\right)^2}{\sum_k m_k w_{ki}^2} S_a(\omega_i, \xi_i)$$

overturning moments:

bending moment at the base **contributed by i-th** mode can be obtained by summing down multiplications of seismic force by respective heights of their applications

$$M_{base\,\boxed{i}} = \sum_j M_{ji} = \sum_j h_j P_{ji}$$



Total base shear & overturning moments:

Total response (base shear or overturning moments) from all modal responses is again approximated by the SRSS rule

$$F_{base} = \sqrt{\sum_{i} F_{base\,\boxed{i}}^{2}}$$

$$M_{base} = \sqrt{\sum_{i} M_{base\,\boxed{i}}^{2}}$$

Eurocode 8 part 6

Design of structures for earthquake resistance -
Towers, masts and chimneys

Eurocode 8 part 6

Design of structures for earthquake resistance - Towers, masts and chimneys

Rotational response spectrum:

$$S_a^{\ddot{\theta}}(\omega) = \frac{0.85\omega}{v_s} S_a(\omega)$$

a multiplier and familiar, horizontal, acceleration, design response spectrum $S_a(T)$

or in terms of the natural period T=2π/ω

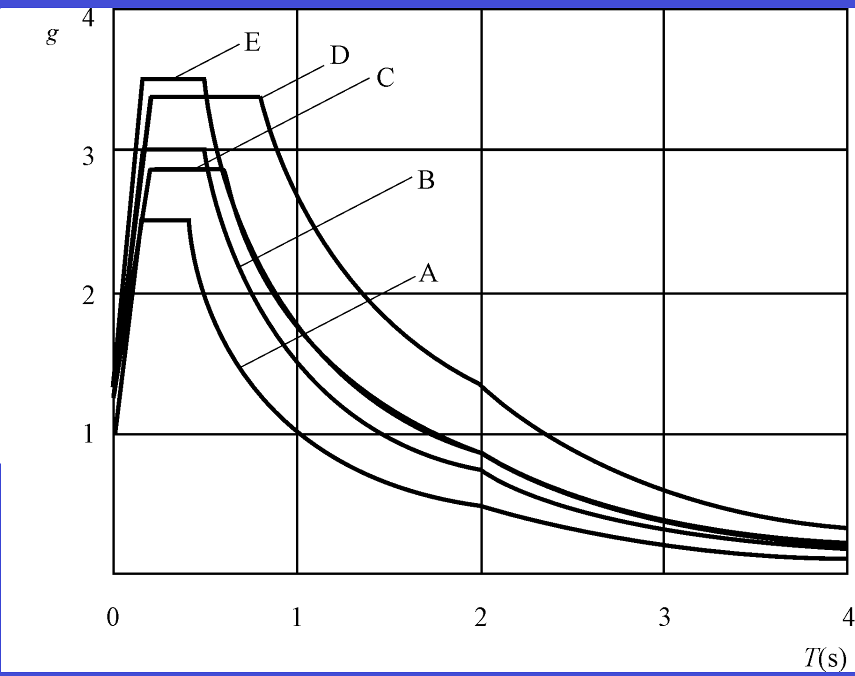
$$S_a^{\ddot{\theta}}(T) = \frac{1.7}{v_sT} S_a(T)$$

Eurocode 8 part 6

Design of structures for earthquake resistance - Towers, masts and chimneys

S_a(T) – familiar, horizontal, acceleration, design response spectrum depending on local soil profile

$$S_a(T) = \begin{cases} S\left[1 + \dfrac{T}{T_B}(2.5\eta - 1)\right] & 0 < T \leq T_C \\ S \cdot 2.5\eta & T_B < T \leq T_C \\ S \cdot 2.5\eta \dfrac{T_C}{T_B} & T_C < T \leq T_D \\ S \cdot 2.5\eta \dfrac{T_C T_B}{T^2} & T_D < T \leq 4s \end{cases}$$



Eurocode 8 part 6

Design of structures for earthquake resistance - Towers, masts and chimneys

$$S_a^{\ddot{\theta}}(T) = \frac{1.7}{v_s T} S_a(T)$$

Conclusion –
The rocking “multiplier” decreases with increasing natural period and increases for softer soils (with increasing v_S)

Important question:
What is the origin of such calibration of the rocking component?
Answer:
More angineering intuition than the actual data
– so far there is no reliable rotational record of STRONG earthquake like the famous “El Centro” signal from 1941

Reasons:

- It is difficult to measure very STRONG seismic rotation
- We have to wait until a truly strong earthquake occur close to a reliable, **strong motion**, rotational instrument

What can be done?

1. Promote networks of **strong motion** rotational sensors in active seismic regions
2. Try to record very strong rockbursts (from deep mining) M_L about 4 to 5+
3. Try to trace very strong aftershocks (M_L=6+) with well designed portable strong motion rotational instruments
4. While waiting for the benchmark strong rocking record, in the mean time, we can investigate appropriate methods of calculating rocking effects on structures

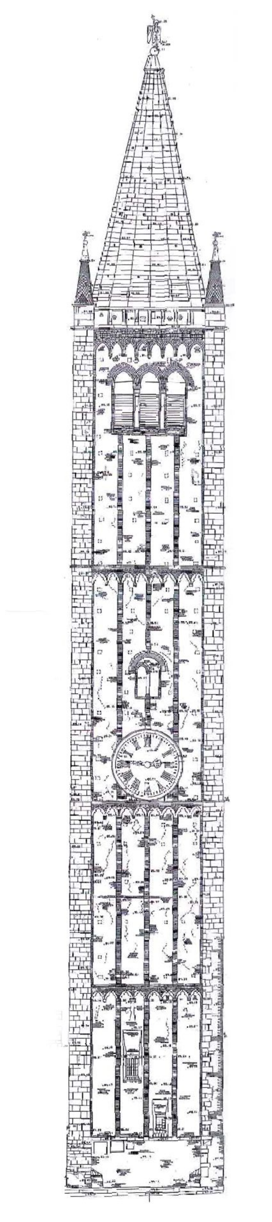
Purpose of this lecture:

To check how to practically model the Eurocode 8 part 6 load for typical, old tower

Parma Bell Tower

Built in gothic style
between 1284 and 1294
(more than 700 years old)
Masonry structure
with marble corners

double walls made of clay
bricks with an infill of a
mixture of stone and
masonry rubble



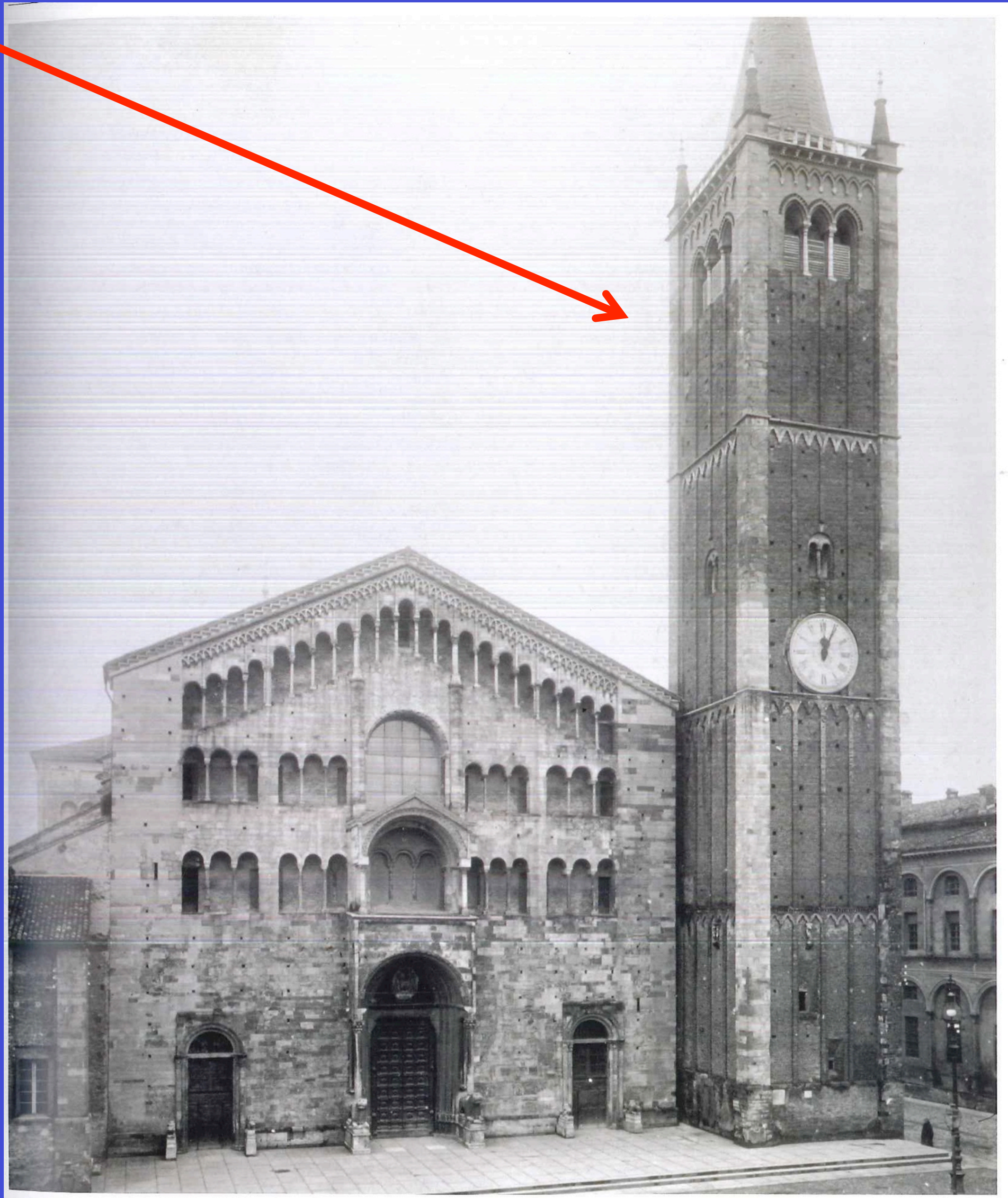
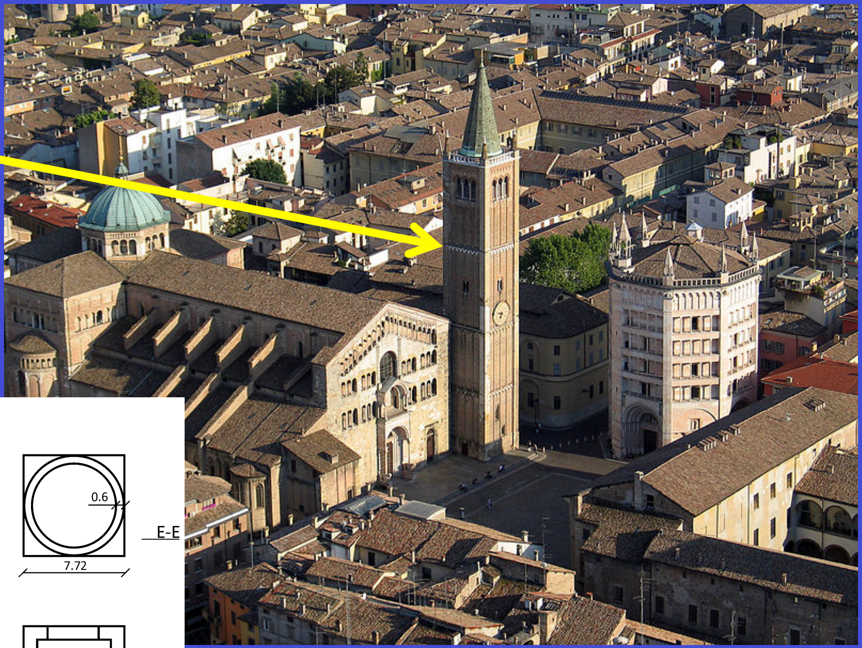
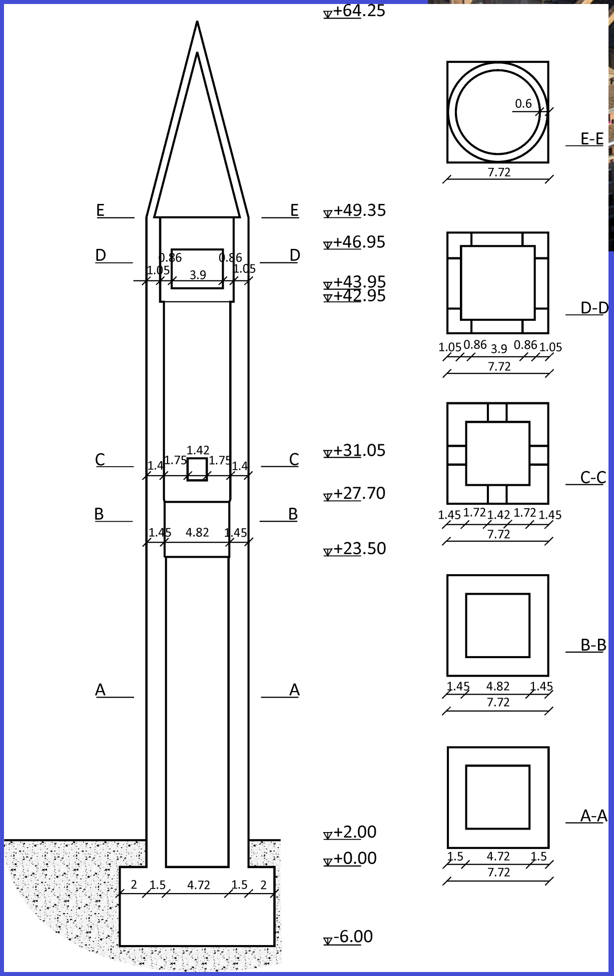


Figura 23. Archivio Fratelli Alinari, Firenze. Facciata del Duomo di Parma 1900- 1915 c.

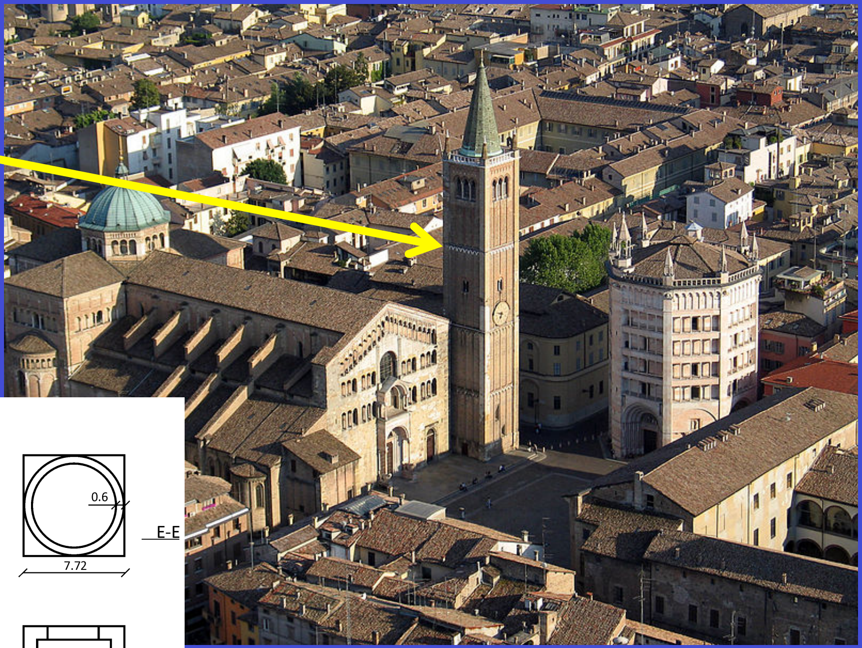
Parma Bell Tower





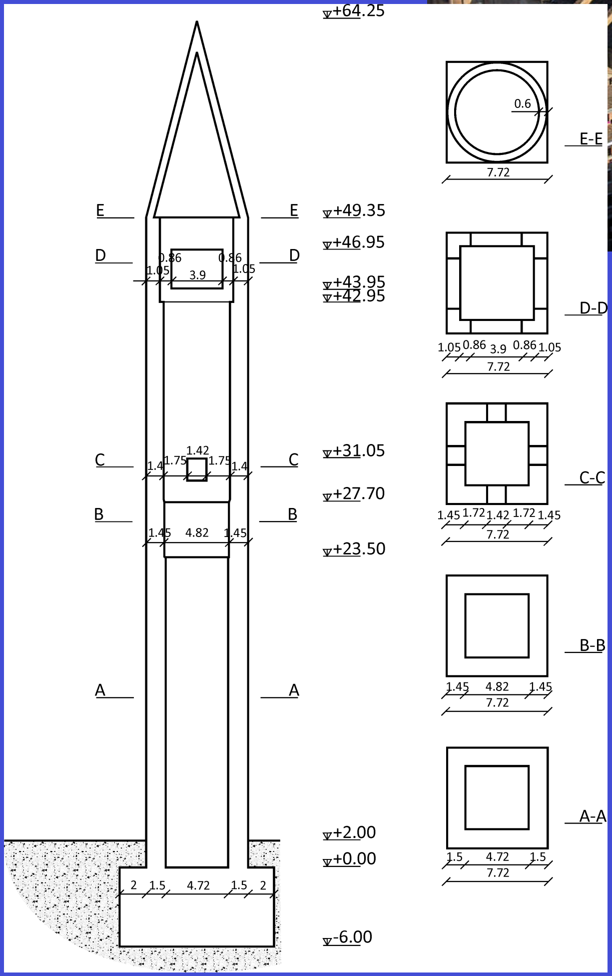
Parma Bell Tower

(dimensions & material data)



square cross section
7.72x7.72m

64.25m

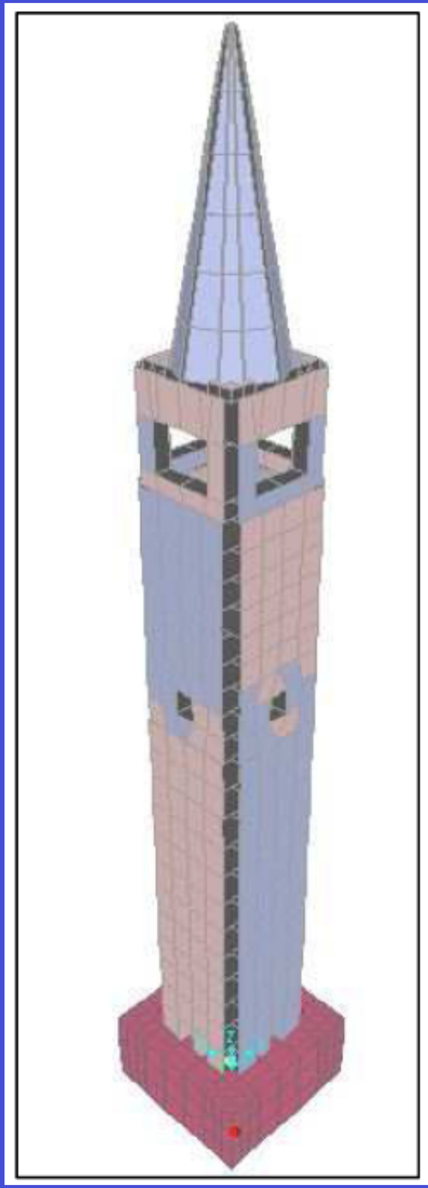


wall thickness
decreasing with height
from 1.40 to 1.05m

The soil underneath the tower consist of alluvial deposits with prevailing slimy clays, clayey sandy slimes and slimy sands.
Average shear wave velocity with v_S=200m/s
Soil C according to EC-8

The **average** mechanical masonry characteristics were measured by means of in-situ tests and through analogy with similar constructions in the region.
Young modulus E=3000MPa

Parma Bell Tower – 3D Finite Element Method (FEM) model in SAP 2000



Thick, shell element combining membrane and plate, bending behaviour to model two-way, out-of-plane, plate rotational stiffness components and a translational stiffness component in the direction normal to the plane of the element. The thick-plate (Mindlin/Reissner) formulation is applied which includes the effects of transverse shearing deformation.

Parma Bell Tower

(results of eigenproblem)

Table 11.4 – Natural periods, frequencies and eigenvalues

(Periodi naturali, frequenze ed auto valori)

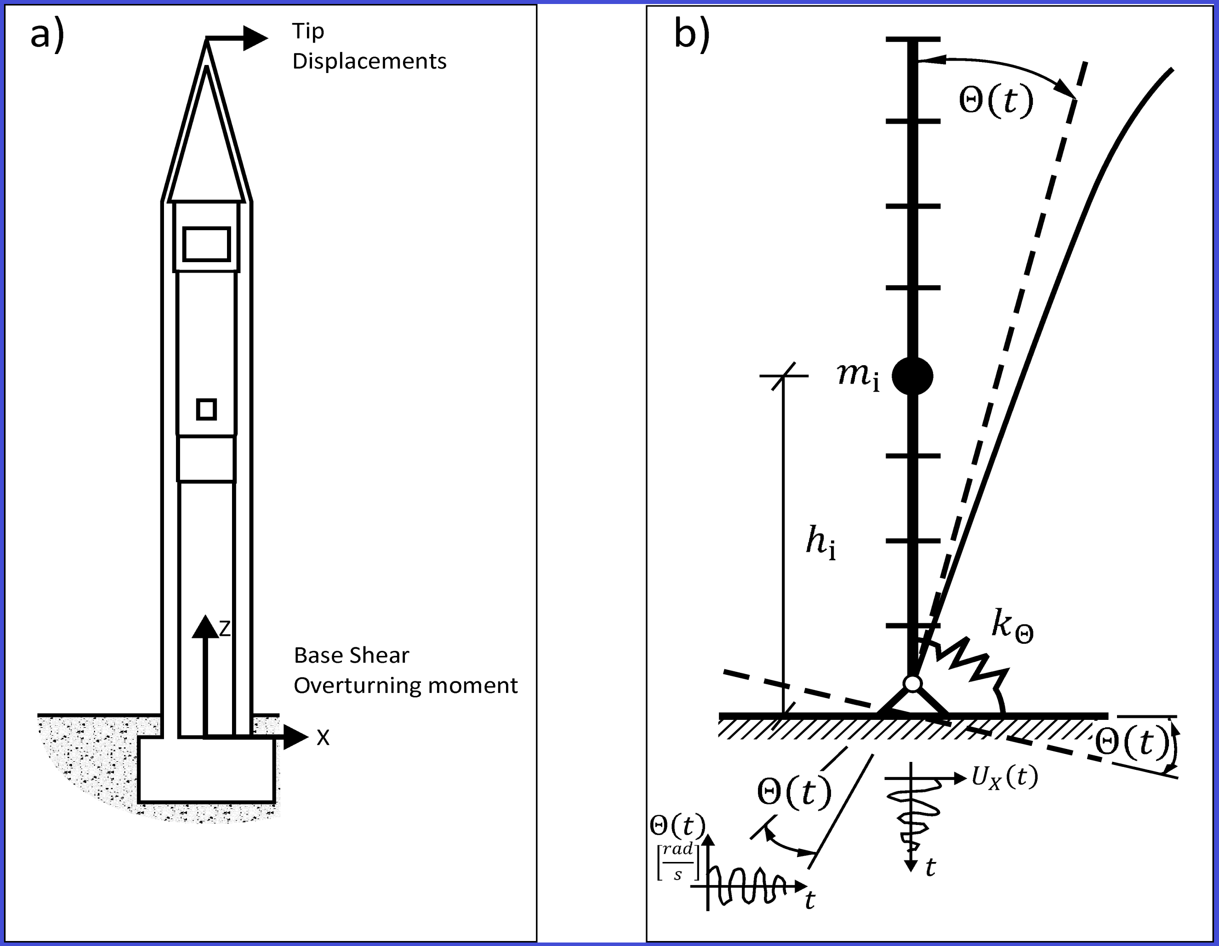
Mode	T	f	ω
	s	Hz	Rad/s
1	1.3660	0.7321	4.5998
2	0.3038	3.2914	20.6810
3	0.1511	6.6195	41.5910
4	0.1499	6.6703	41.9100
5	0.1456	6.8696	43.1630
6	0.0997	10.0320	63.0300
7	0.0817	12.2430	76.9240
8	0.0759	13.1710	82.7580
9	0.0600	16.6550	104.6500
10	0.0568	17.6000	110.5900
11	0.0537	18.6110	116.9300
12	0.0487	20.5380	129.0400

Parma Bell Tower – Timoshenko beam model

So far **none** civil engineering FEM software (including SAP-2000) and other FEM programs allow direct rotational kinematic excitations, particularly in response spectrum format

Question: How to obtain dynamic response of slender tower to rotational excitations?
Answer: to build your own full scale FEM software
or
to build a simplified FEM model

Parma Bell Tower – Timoshenko beam model prepared in Matlab

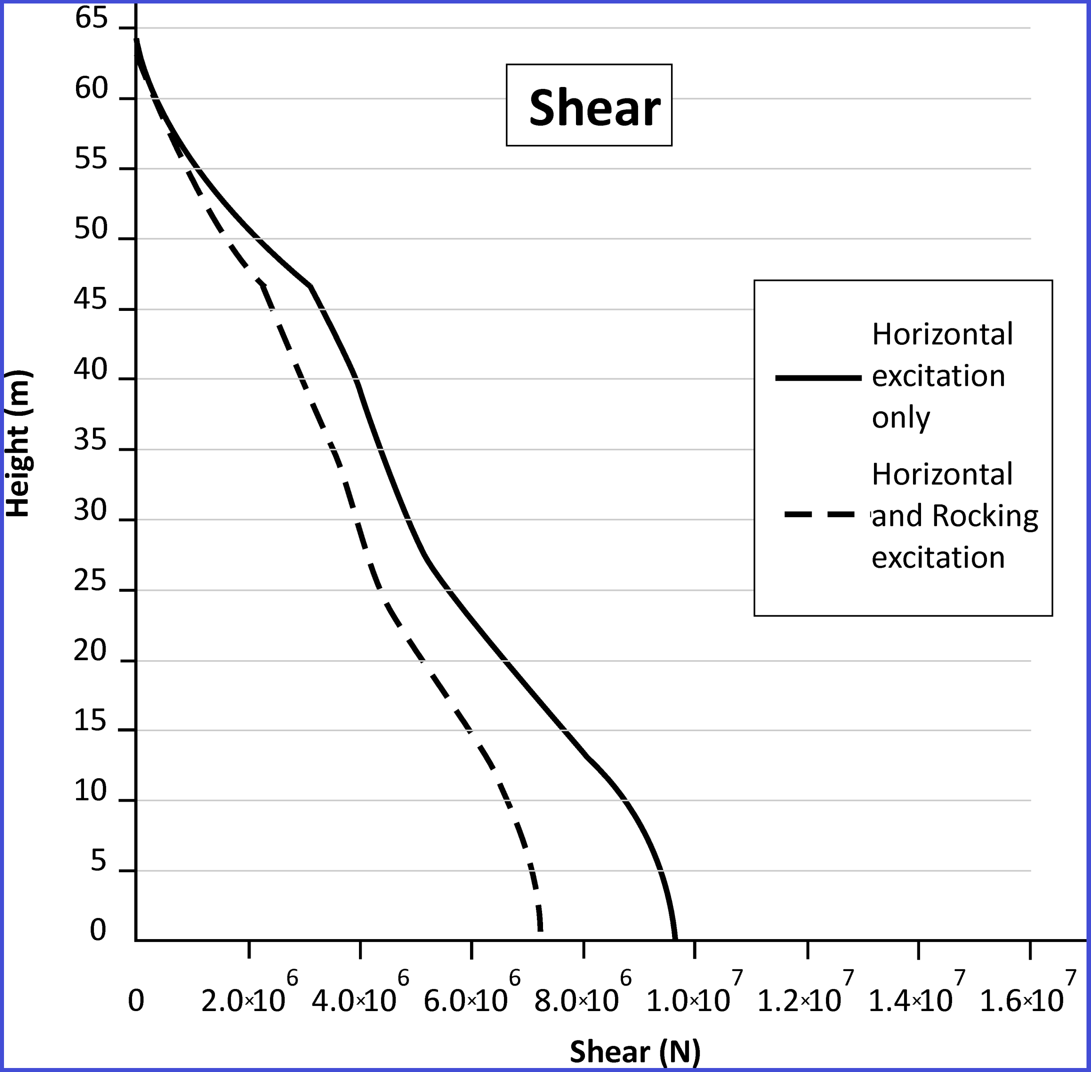


equivalence of the Matlab-Timoshenko model vs. SAP 2000 thick shell element model
minimization of difference between natural frequencies + and tip displacements among both models

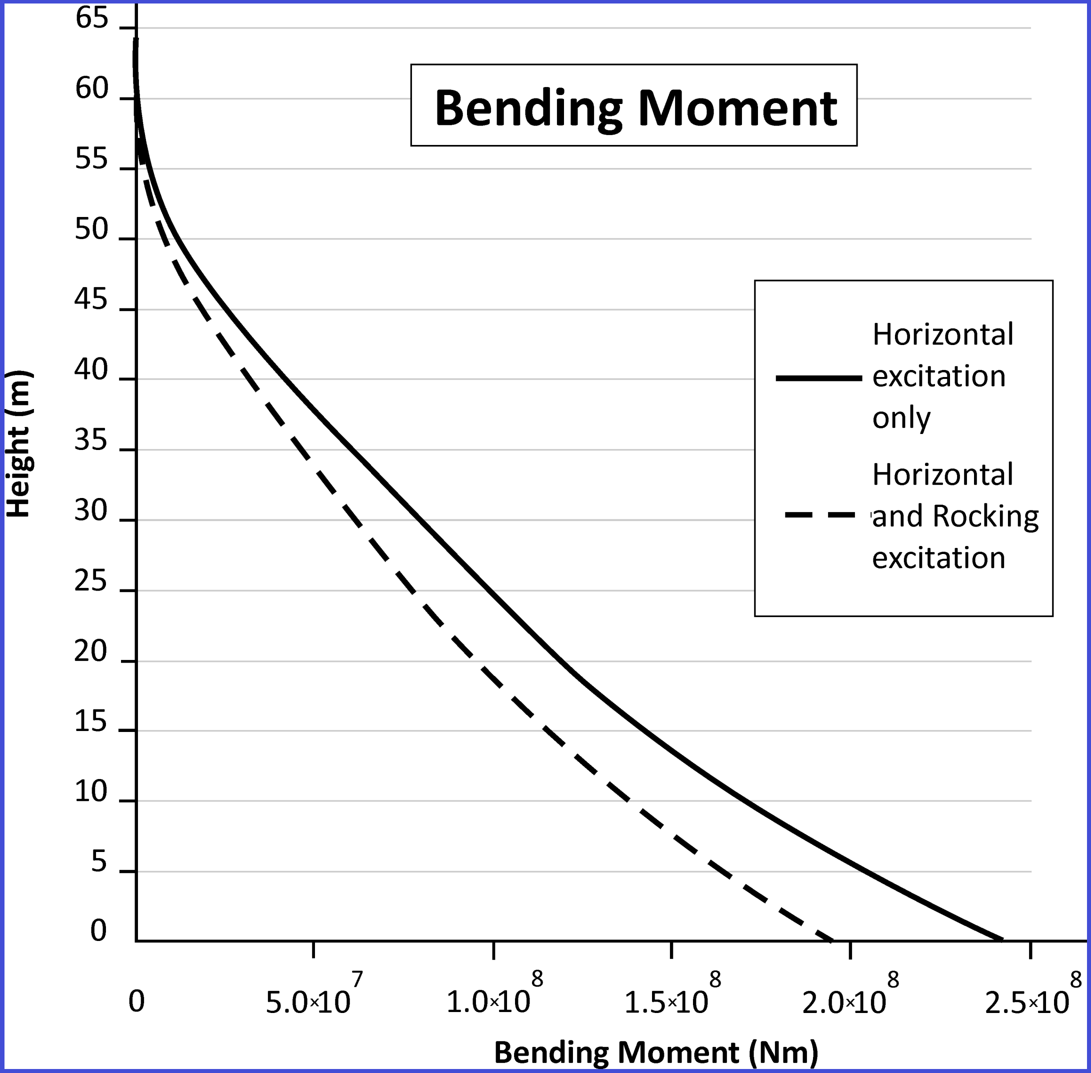
Parma Bell Tower
– results of seismic response analyses with Eurocode 8 response spectrum

design ground accelerations a_g=2.146m/s^2
&
soil C = average compliant stiffness

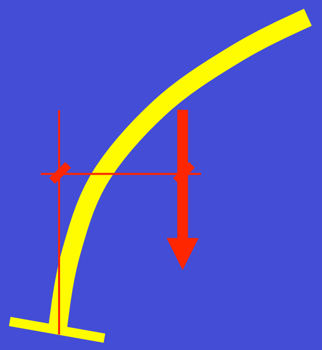
Parma Bell Tower – results of seismic response analyses with Eurocode 8 response spectrum



Parma Bell Tower
– results of seismic response analyses with Eurocode 8 response spectrum



Parma Bell Tower assesment of P-Δ effects



2nd order effects

		Horizontal Excitation	Horizontal and Rotational Excitation	
M	Base Overturning moment (Nm)	194673195.33	242836852.37	Nm
$M_{p\Delta}$	Additional Overturning Moment due to P-delta (Nm)	2363489.87	2923752.13	Nm
M_{TOT}	Total Overturning Moment(Nm)	197036685.20	245760604.50	Nm
$100*M_{p\Delta}/M_{TOT}$	Increasing of Moment due to P-delta	1.20%	1.19%	

Parma Bell Tower
– rotational ground motion excitation effects

Conclusions

1. Seismic response spectrum computations for the old tower were carried out with and without rocking excitation effects
2. A 3D Finite Element Method was applied to solve the eigenproblem of the tower
3. An equivalent Timoshenko beam model using 1D FEM was prepared in order to include also rocking excitation effects. This was done because so far <u>none of the available commercial civil engineering software allows to include rocking excitations,</u> particularly in format pf response spectrum
4. The applied Eurocode 8 part 6 rotation, rocking seismic load definition still requires calibrations which may happen when some true and reliable <u>strong</u> motion rocking records are acquired
5. The actual EC-8.6 response spectrum model showed quite substantial 20% contribution of rocking excitation effects in the analysed case
6. The 2nd order p-Δ delta effects appeared to be small for the response case analyzed (about 1.2%)

Thank you for your attention

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Short Note

Rotational Seismic Load Definition in Eurocode 8, Part 6, for Slender Tower-Shaped Structures

by Zbigniew Zembaty

Abstract This note describes the rotational seismic load definition as included in Part 6 of Eurocode 8 (EC8.6, 2005). The Eurocode 8, Part 6 (EC8.6, 2005), definition of the rotational ground-motion component depends upon the structural subsoil compliance, which is controlled by the shear-wave velocity in the top 30 m of ground. A comparison of the effects of the rocking ground motion and the horizontal ground motion on the response of a 160 m reinforced concrete chimney shows that for the Eurocode 8, Part 6 (EC8.6, 2005), definition of the rotational seismic ground motion, the rocking excitations contribute significantly to the overall response of the structure. The engineering code formulas for the rocking component of ground motion, however, should be calibrated and reconciled with the results of the latest empirical research.

Bulletin of the Seismological Society of America, Vol. 99, No. 2B, pp. 1483–1485, May 2009, doi: 10.1785/0120080252

Introduction

For some structures such as slender towers, the rocking excitations can contribute substantial additional seismic response. In spite of the lack of recorded data on the rotational strong ground motion, the problem has been studied, and it has been shown that the classic response spectrum method can be formulated to also include the rotational excitations (Castellani and Boffi, 1986, 1989).

On 22 February 2005, Eurocode 8, Part 1 (EC8.1, 2005), was formally approved for use in 28 European countries. Part 6 of Eurocode 8 (EC8.6, 2005), which was approved on 25 September 2005 proposed to include (in addition to traditional horizontal seismic actions) three rotational excitations. This is probably one of the first codified rotational seismic loads ever proposed. The purpose of this note is to briefly describe the load definitions of Eurocode 8, Part 6 (EC8.6, 2005), that apply to rotational excitation.

Formal Seismic Load Definition for Slender Towers

Eurocode 8, Part 6, deals with the design rules for tower-shaped structures, including bell towers, intake towers, radio and TV towers, masts, chimneys (including free-standing industrial chimneys), and lighthouses, with additional special provisions for reinforced concrete and steel chimneys. Point 3.6 of Eurocode 8, Part 6 (EC8.6, 2005), proposes to take into account one vertical and two horizontal components of seismic ground motion acting simultaneously. It also proposes to take into account the corresponding simultaneous action of rotational components of seismic load for the tall structures designed in regions of high seismicity. The formal decision to eventually include rotational components of seismic ground motion is left to the national authorities of the countries implementing the codes. Eurocode 8, Part 6, recommends including the rotational seismic excitations for structures higher than 80 m and for cases in which seismic design acceleration defined by product $a_g S$ is not less than $0.25g$; a_g is design acceleration for type A ground and S represents the soil factor (g is the acceleration of gravity). Eurocode 8, Part 6 (EC8.6, 2005), recommends either the time history or the response spectrum method for analysis. In the first case one should apply simultaneous action of six records of seismic ground motion (three translations and three rotations). In the second case for the translational loads, the response spectrum defined by Eurocode 8, Part 1 (EC8.1, 2005), should be applied. To account for the three rotations Eurocode 8, Part 6 (EC8.6, 2005), recommends the response spectrum method, in which the rotational response spectra about two horizontal axes (x and y) and the vertical z are defined by

$$\mathrm{R}_{\mathrm{x}}^{\theta}(T) = \frac{1.7\pi S_e(T)}{V_S T}, \tag{1}$$

$$\mathrm{R}_{\mathrm{y}}^{\theta}(T) = \frac{1.7\pi S_e(T)}{V_S T}, \tag{2}$$

$$\mathrm{R}_{\mathrm{z}}^{\theta}(T) = \frac{2.0\pi S_e(T)}{V_S T}, \tag{3}$$

where $S_e(T)$ (m/sec^2) is the elastic horizontal response spectrum defined for the site, T is the natural period (sec), and V_S is the average S-wave velocity (m/sec) in the top 30 m of the ground profile (Eurocode 8, Part 6 [EC8.6, 2005] recommends applying the value corresponding to low-amplitude soil vibrations, i.e., to shear deformations on the order of 10^{-6}). When the top 30 m shear-wave velocity is not known from experiments, the values corresponding to ground types A, B, C, and D as proposed by Eurocode 8, Part 1 (EC8.1, 2005), may be used ($V_S = 800$, 580, 270, and 150 m/sec, respectively).

The extension of the response spectrum method to include rotational excitations is formulated in terms of the discrete mathematical representation of the structural model. For the simultaneous action of horizontal translation excitations along axis x and rocking excitations about horizontal, orthogonal axis y (vibrations in plane x-z), the equation of motion of the discrete system is given by

$$[\mathrm{M}]\{\ddot{\mathrm{u}}\} + [C]\{\dot{\mathrm{u}}\} + [K]\{\mathrm{u}\} = -(\{\mathrm{m}\}\ddot{x} + \{mh\}\ddot{\theta}), \qquad (4)$$

in which

$\{\ddot{\mathrm{u}}\}$ is the vector representing the accelerations of the degrees of freedom of the structure relative to the base;

$\{\dot{\mathrm{u}}\}$ is the vector representing the velocities of the degrees of freedom of the structure;

$\{\mathrm{u}\}$ is the vector representing the displacements of the degrees of freedom of the structure;

$\{\mathrm{m}\}$ is the vector comprising the translational masses in the horizontal direction of the translational excitation; it coincides with the main diagonal of the mass matrix [M], if the vector $\{\mathrm{u}\}$ includes only the translational displacements in the horizontal direction of the excitation;

$\ddot{x}(t)$ is the translational ground acceleration, represented by S_e;

$\ddot{\theta}(t)$ is the rotational acceleration of the base, represented by R^{Θ};

and the participation factor in the modal analysis of mode k is defined as

$$a_{kx} = \frac{\{\Phi^{\mathrm{Tr}}\}\{\mathrm{m}\}}{\{\Phi^{\mathrm{Tr}}\}[\mathrm{M}]\{\Phi\}}. \qquad (5)$$

For the term $\{mh\}\ddot{\theta}$ the participation factor is

$$a_{k\Theta} = \frac{\{(\Phi h)^{\mathrm{Tr}}\}\{\mathrm{m}\}}{\{\Phi^{\mathrm{Tr}}\}[\mathrm{M}]\{\Phi\}}, \qquad (6)$$

where

$\{\Phi\}$ is the kth modal vector;

$\{\Phi h\}$ is the vector of the products of the modal amplitude Φ_i at the ith degree of freedom and its elevation h_i;

and Tr stands for transposition.

The effects of the rotational ground excitations may be combined with those of the translational excitation via the square root of the sum of the squares rule.

Eurocode 8, Part 6 (EC8.6, 2005), emphasizes the need to properly account for soil structure interaction effects as well as for the second-order effects, which for slender towers on compliant soil under rotational (rocking) excitations can play an important role. Eurocode 8, Part 6 (EC8.6, 2005), recommends neglecting the second-order effects if the overturning moment due to inclusion of second-order effects does not exceed the basic overturning moment by 10%. It should also be noted that the definition of torsional seismic response spectrum in Eurocode 8, Part 6 (EC8.6, 2005, equation 3, rotation around the vertical axis) is of less importance for slender towers, but it has been included in Eurocode 8, Part 6 (EC8.6, 2005), for the completeness of the formal definition of the rotational components of seismic ground motion.

Numerical Example

In the article by Zembaty and Boffi (1994) the seismic response of a 160 m reinforced concrete chimney, based on the horizontal response spectrum defined by Eurocode 8, Part 1, and the rotational (rocking) response spectrum defined by Eurocode 8, Part 6 (EC8.6, 2005, equation 1), were calculated for the damping ratio $\xi = 0.05$. The analyzed chimney and its basic dimensions are shown in Figure 1, and the Young modulus of the concrete used in the dynamic calculations is 1.776×10^{10} N/m^2. Neither the shaft of the

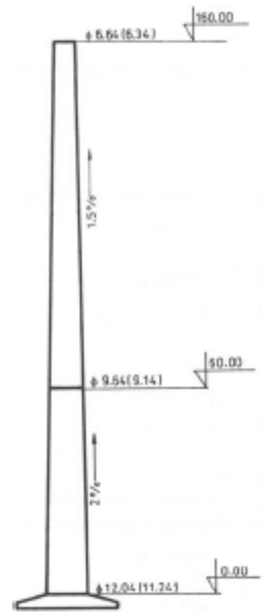

Figure 1. Sketch showing the 160 m reinforced concrete chimney analyzed in the numerical example. All dimensions are shown in meters (the internal diameters are shown in parentheses). The diameter of the foundation equals 20 m.

chimney nor its foundation were designed to withstand seismic effects. In the computations, in which equations (1) and (2) and (4)–(6) have been applied, the soil compliance effects were included for the shear-wave velocity of the soil $V_S = 200$ m/sec, density of the soil $\rho = 1800$ kg/m^3, and Poisson ratio $\nu = 0.25$. Only the rocking and horizontal flexibility of the subsoil have been taken into account (vibrations in the vertical plane).

In Figure 2 the plot of bending moments in the chimney shaft due to joint action of horizontal and rocking excitations is shown and denoted as total. The contribution of only rocking effects to the bending moments is also plotted and denoted as rotation. The response spectrum calculations were carried out for the design acceleration $a_g = 0.1g$.

Discussion and Conclusion

The definition of the rotational seismic load as proposed by Eurocode 8, Part 6 (EC8.6, 2005), for slender, tower-shaped structures has been presented. Example calculations of the seismic response of a tall industrial chimney show a substantial contribution of the rocking effects in the overall seismic response. The rotational seismic load around the horizontal and vertical axes is defined in Eurocode 8, Part 6 (EC8.6, 2005) as the multipliers of horizontal response spectra dependent upon the shear-wave velocity of the top 30 m of the ground. Such a definition means that the rotational seismic load as defined for slender towers is quite arbitrary and depends only upon the soil compliance and not on the seismological parameters of the expected earthquake and its detailed wave propagation characteristics.

In addition, as has been shown in the articles by Lee and Trifunac (1985, 1987, 2009), the ratio of rocking to horizontal spectra depends not only upon the shear-wave velocity in the top soil layer but also on the waves with higher phase velocities associated with the deeper ground layers. In figure 1 of the article by Lee and Trifunac (2009) an example of dispersion curves is shown for a ground profile at El Centro, California. It can be seen from this figure that the higher the period of the waves the more the wave velocities contribute to rocking ground motion. For the period of 1–10 sec, respective phase velocities reach maxima at about 3–4 km/sec, which means that the rocking excitations will depend also upon the high-velocity wave components. The actual Eurocode 8, Part 6 (EC8.6, 2005), proposal given in equations (1)–(3) suggests just the opposite relation and may lead to erroneous results, which means that further development and empirical scaling of these formulas should follow. The problem is not easy to resolve because typical code formulas have to cover various load and structural scenarios and usually represent a conservative compromise between the actual state of the art in the research and engineering simplicity.

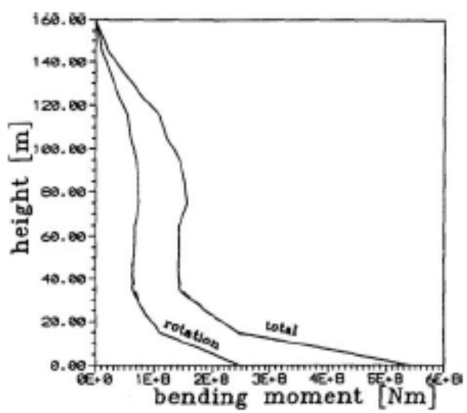

Figure 2. Plot of bending moment along the shaft of 160 m high reinforced concrete chimney and contribution of the rotational (rocking) effects according to Eurocode 8, Part 6.

Data and Resources

All data used in this article came from published sources listed in the references.

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References

Castellani, A., and G. Boffi (1986). Rotational components of seismic motion, *Earthq. Eng. Struct. Dyn.* **14,** 751–767.

Castellani, A., and G. Boffi (1989). On the rotational components of seismic motion, *Earthq. Eng. Struct. Dyn.* **18,** 785–797.

EC8.1 (2005). EN 1998-1 Eurocode 8: Design of structures for earthquake resistance; Part 1: General rules, seismic actions and rules for buildings, February 2005, European Committee for Standardization, Management Centre: rue de Stassart, 36 B-1050 Brussels (see also http://eurocodes.jrc.ec.europa.eu/, last accessed February 2009).

EC8.6 (2005). EN 1998-6 Eurocode 8: Design of structures for earthquake resistance; Part 6: Towers, masts, and chimneys, September 2005, European Committee for Standardization, Management Centre: rue de Stassart, 36 B-1050 Brussels (see also: http://eurocodes.jrc.ec.europa.eu/) last accessed February 2009).

Lee, V. W., and M. D. Trifunac (1985). Torsional accelerograms, *Soil Dyn. Earthq. Eng.* **4,** 132–139.

Lee, V. W., and M. D. Trifunac (1987). Rocking strong earthquake accelerations, *Soil Dyn. Earthq. Eng.* **6,** 75–89.

Lee, V. W., and M. D. Trifunac (2009). Empirical scaling of rotational spectra of strong earthquake ground motion, *Bull. Seismol. Soc. Am.* **99,** no. 2B, 1378–1390.

Zembaty, Z., and G. Boffi (1994). Effect of rotational seismic ground motion on dynamic response of slender towers, *Eur. Earthq. Eng.* **8,** 3–11.

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