Course Modulus for Practitioners
Fundamentals of seismic analysis and seismic design

Lecturer
Dr. Barbara Borzi

<u>Day 1:</u>
09:00 –10:30: Fundamentals of seismology
10:30 – 11:00: Coffee break
11:00 – 12:30: Seismic hazard in Palestine
12:30 – 14:30: Lunch break
14:30 – 16:00: Single Degree of Freedom System (SDOF)
16:00 – 16:30: Coffee break
16:30 – 19:00: Elastic Response Spectrum – Site effects EC8

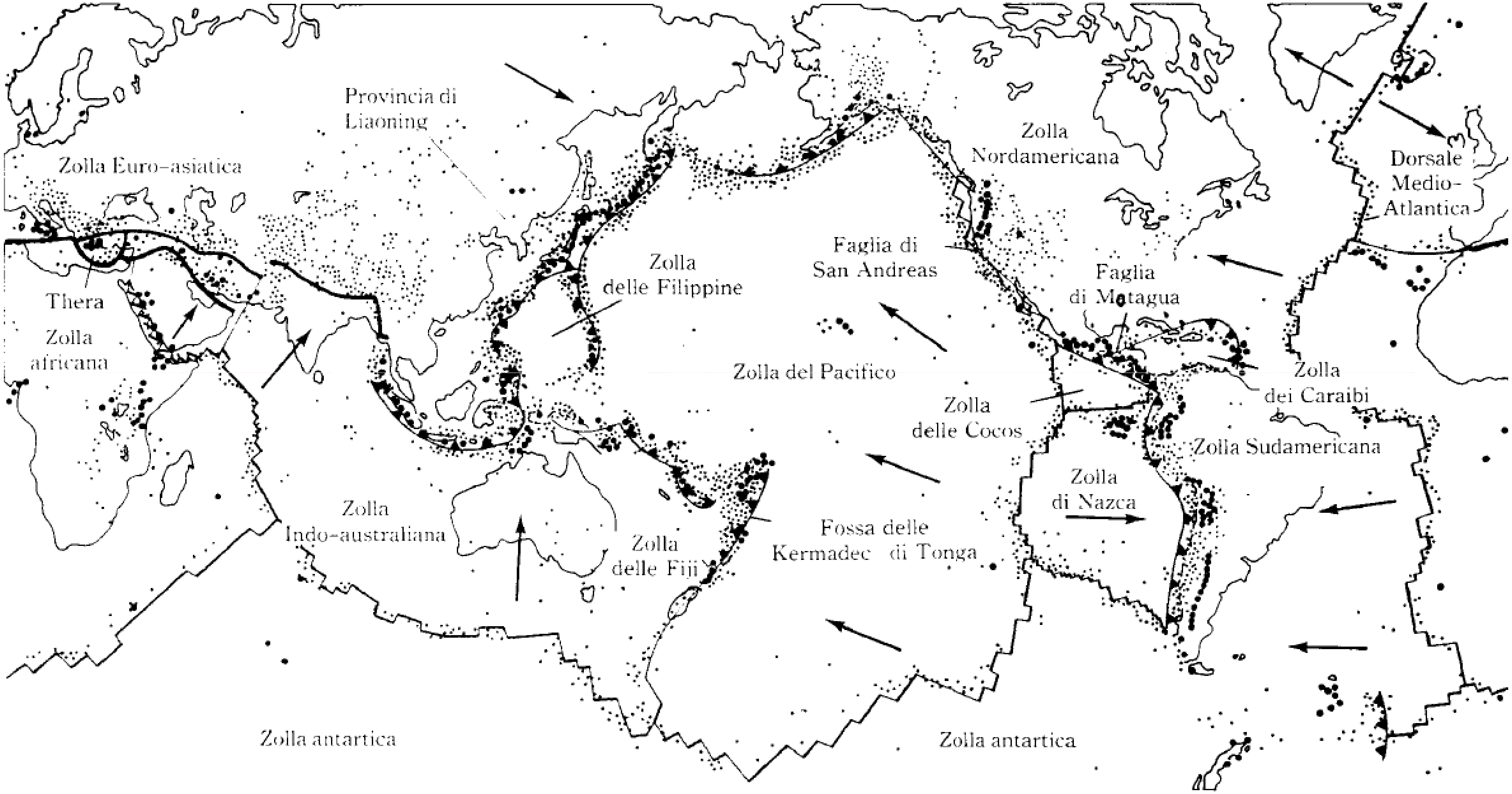
<u>Day 2:</u>
09:00 – 10:30: Fundamental of ductility and Inelastic Response Spectra
10:30 – 11:00: Coffee break
11:00 – 12:30: Conceptual seismic design
12:30 – 14:30: Lunch break
14:30 – 16:00: Seismic Analysis
16:00 – 16:30: Coffee break
16:30 – 19:00: Capacity Design of Buildings

<u>Day 3:</u>
09:00 – 10:30: Assignment 1
10:30 – 11:00: Coffee break
11:00 – 13:00: Assignment 2

Fundamentals of Seismology

Tectonics

Relations between tectonic plates, earthquakes and volcanoes



Volcanoes
Seismic zones
Subduction zones
Plate motion
Dorsal due to transform faults
Collision zone

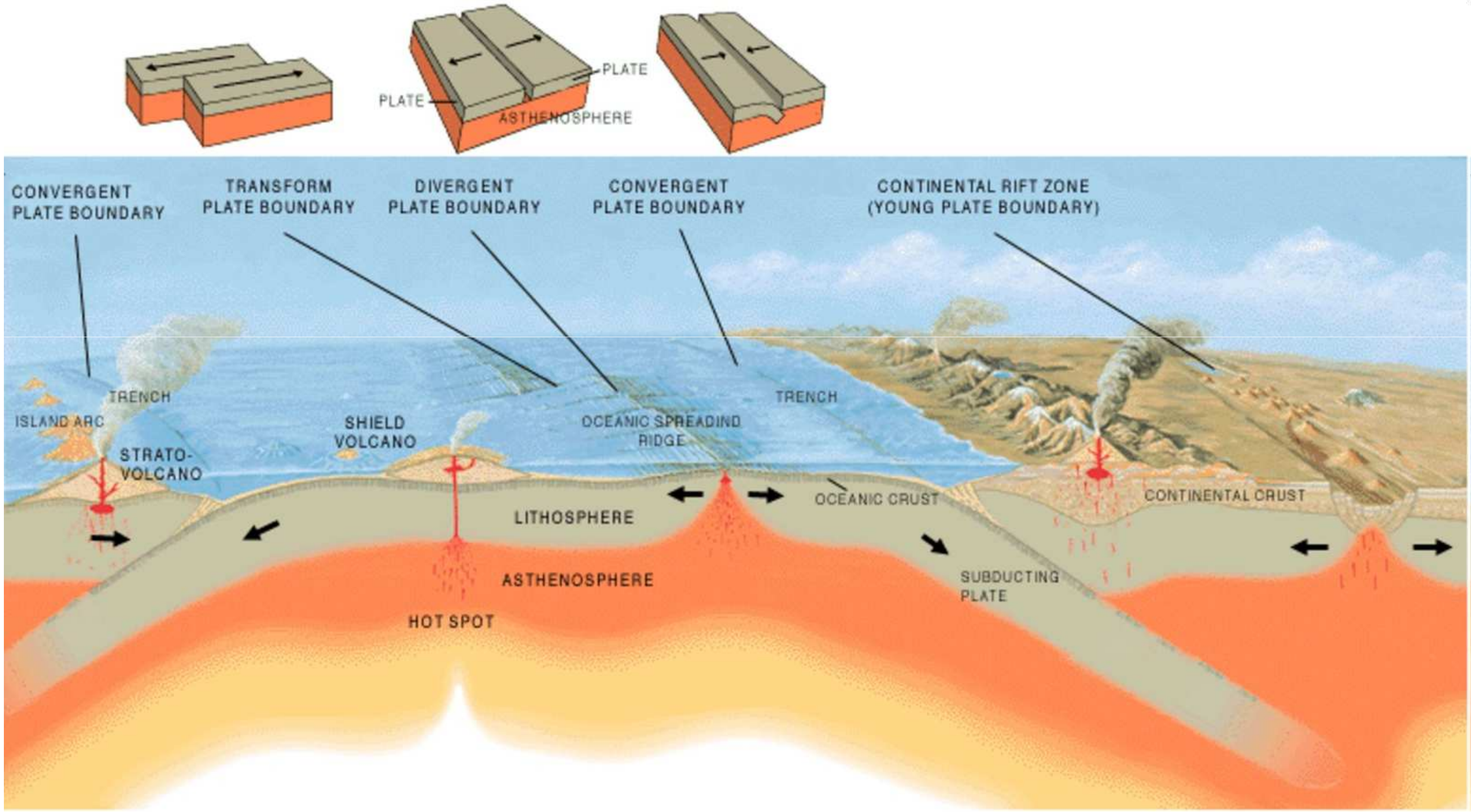
(From Faccioli, 2005)

Notes:

- The largest earthquakes typically occur at plate boundaries
- Earthquakes almost always occur on faults, which represent the boundary between two rigid media that are capable of relative motion
- Earthquakes that occur on land and close enough to the surface often leave visible evidence in the form of ground dislocation e.g. the San Andreas fault.
- Earthquakes are generally classified depending on their focal depth:
    - Crustal (shallow) up to60 km
    - Intermediate from 60 km to 150-200 km
    - Deep up to 600 km

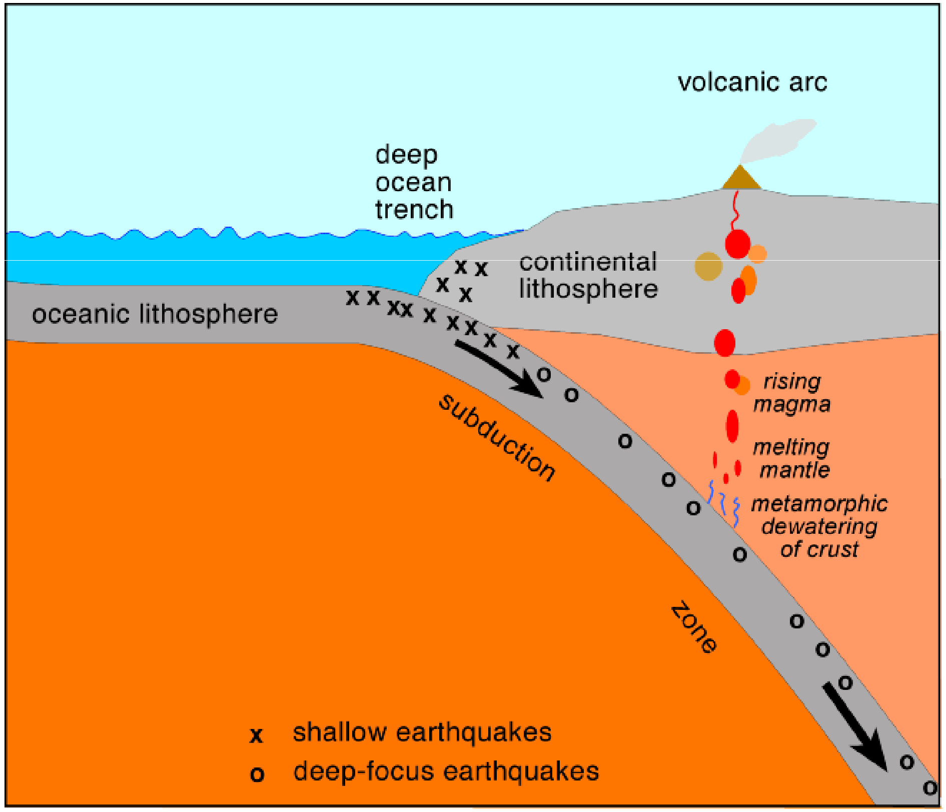
From Faccioli, 2005

Plate tectonics is the reference frame to understand earthquakes



Subduction boundary:

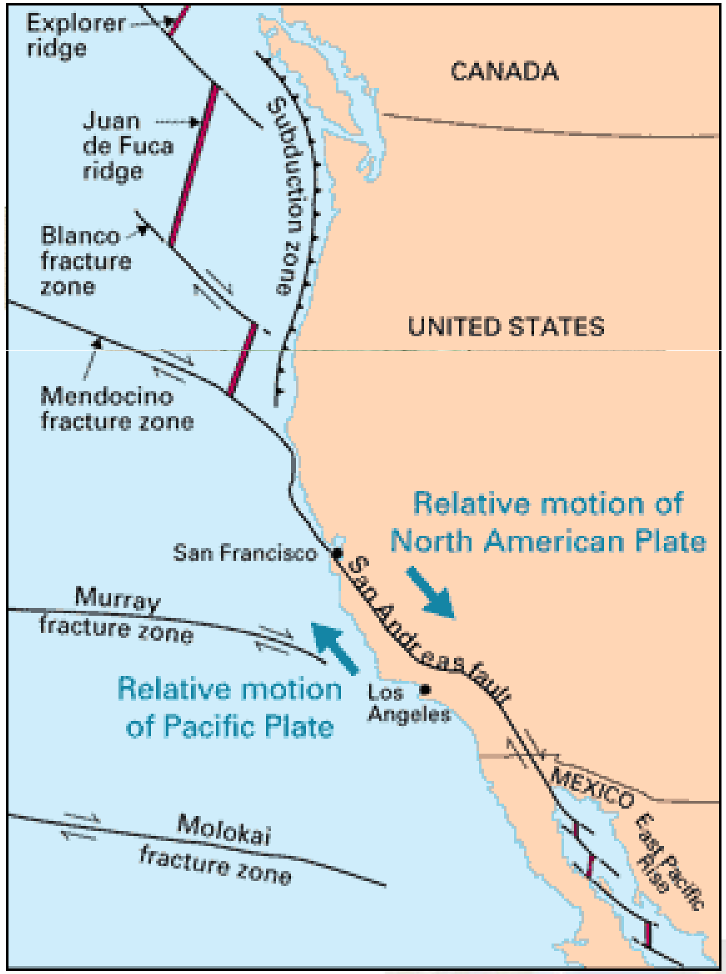
- Where two tectonic plates converge, if one or both of the plates is oceanic lithosphere, a subduction zone will form. An oceanic plate will sink back into the mantle.



- Planes of earthquakes are associated to subduction zone, shallow near the trench and descending beneath and beyond the volcanic arc They the result of seafloors subducting beneath continents. These planes are called Wadati-Benioff zones
- Earthquakes of Wadati-Benioff zones are believe to delineate the upper surface of the descending plate (or slab)
- Subduction margins are zones where the most quantity of energy is released

Sliding boundary:

Where two plates are sliding horizontally past one another a transform-fault boundary will form.



- Earthquakes originate in the transform fault, or in parallel strike-slip faults, when a frictional resistance in the fault system is overcome and the plates suddenly move.
- Most transform faults are found in the ocean where they offset spreading ridges creating a zigzag pattern between the plates. Some transform faults occur on land.
- E.g.: The San Andreas is one of the few transform faults exposed on land. The San Andreas fault zone, is about 1,300 km long and in places tens of kilometers wide. It slices through two thirds of the length of California. Along it, the Pacific Plate has been grinding horizontally past the North American Plate for 10 million years, at an average rate of about 5 cm/yr.

Magnitude scales

Definition:

Magnitude is a measure of earthquake size

Various scales of magnitudes:

First Magnitude scale at the base of the modern quantitative seismology:

- M_L: Local or Richter magnitude

Modern seismic Magnitudes:

- M_S: Surface-wave magnitude (Rayleigh Wave)
- M_b: Body-wave magnitude (P-wave)
- M_w: Moment magnitude

M_L: Local or Richter magnitude

It was defined in 1930's by Richter in California on the basis of two main observations:

- Peak amplitudes on seismograms from 2 events of different intensities, recorded at the same focal depth, by the same seismograph, at similar distances are different: the strongest event generates higher amplitudes.
  Only if epicentral distances differ and the smaller event is very near to the station, it can generate a seismogram with higher amplitude than the stronger event.

- If earthquakes are recorded at various stations at various distances, the recorded peaks of amplitude as function of distances give a curve for each earthquake. The higher curve is associated to the strongest event and the peak amplitudes decrease with distance in a similar manner for different earthquakes.

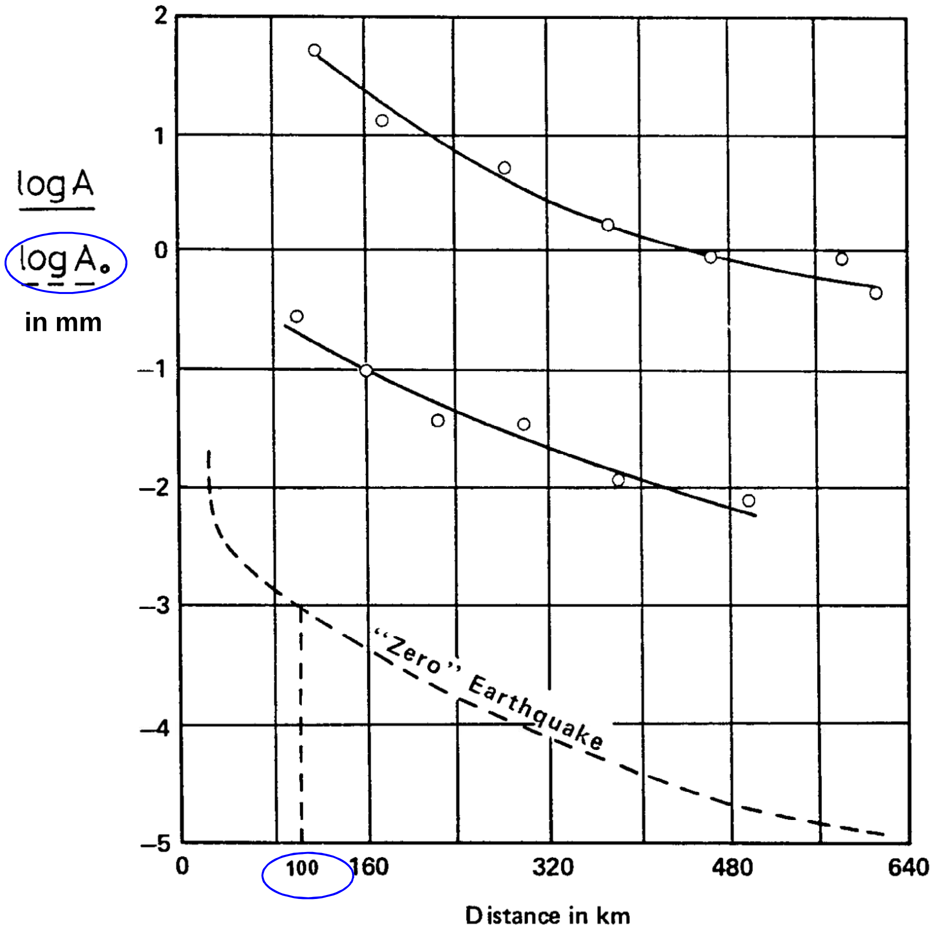
M_L: Local or Richter magnitude

In log scale the amplitude difference is distance independent

Quantitative relative measure between two earthquakes

Absolute measure needs a reference "zero" earthquake with M=0:
the event that generates an amplitude peak of 0.001 mm at 100 km, recorded by the Wood-Anderson (WA) seismograph

The event with M=0 is the smaller event recordable by WA instrument



M_L: Local or Richter magnitude

Def: $$M_L = \log A - \log A_0$$

A: peak amplitude (mm) of the recorded WA trace at a given distance

A_0 : corresponding amplitude of the “zero” earthquake at the same distance

- ✓ Magnitude is a measure of the earthquake intensity at the source
- ✓ Magnitude is distance independent
- ✓ Increase of a M unit induces the increase of 10 times in the amplitude of motion that defines the magnitude

Application:

- Amplitude record by WA in mm
- Definition of epicentral distance from earthquake localization
- For records from two sensors with perpendicular directions (NS, EW), M_L is obtained as an average of the two
- When records from various stations are available, M_L is calculated as the average of all the values (dispersion ± 0.3)

M_L: Local or Richter magnitude

Limits:

- ✓ Use of the specific, obsolete, short period WA seismometer: A_{max}=120 mm, A_{min}=0.1 mm
- ✓ A_0 values at various distances R depend on features of seismic waves damping at high frequencies in shallow crust of Southern California and logA_0 curve is calibrated for California

$$log\,A_0 = a\,log\frac{R}{100} + b\left(R - 100\right)$$

- ✓ Thus a proper curve should be calibrated for different regions
- ✓ M_L is defined for maximum distances of 600 km
- ✓ M_L saturation at 7-7.5 due to the limitation of the WA bandwidth

Advantages:

- ✓ M_L is useful for engineering applications since many structures have natural periods close to that of the WA (0.8s) or within the range of its pass-band (about 0.1 -1 s).

M_S: Surface wave magnitude (Rayleigh wave)

- It was introduced in 1945 by Gutenberg
- It is based on the use of surface waves of period T=20 sec., which generally carry the maximum values of seismograms recorded by long period seismometers and are slightly influenced by crust lateral heterogeneities.

$$M_S = log\, A_{H\,max}(\Delta) + \sigma_S(\Delta)$$

A_{Hmax}: maximum horizontal surface waves amplitude in μmm (vectorially combined) at periods 20±2 sec

Δ : distance in degrees

- Modified version officially adopted by IASPEI and U.S. Geological Survey in 1962

$$M_S = log\left(\frac{A}{T}\right)_{max} + \sigma_S(\Delta) = log\left(\frac{A}{T}\right)_{max} + 1.66\, log\,\Delta + 3.3$$

T: period

- ✓ Definition problems for M_S less than around 5.5
- ✓ M_S proper for shallow earthquakes with dept < 50 km and epicentral distances 2< Δ<160
- ✓ M_S maximum values about 8.5

M_b: Body wave magnitude (P wave)

- It was introduced in 1945 by Gutenberg
- It is based on the use of short period (0.5-12 s) body waves

$$M_b = log\left(\frac{A}{T}\right) + Q(\Delta, h)$$

Q: empirically defined function

Δ : distance in degrees

h: focal depth

- In the practice short period vertical seismograms of the world network WWSSN are used; they are generally dominated by P waves with period about 1 s.
- A/T values have to be determined in the first time window of 5 s

✓ It is suitable for both shallow and deep earthquakes

✓ M_b saturation at values less than 6.5

M_W: Moment magnitude

- It was introduced in 1979 by Hanks and Kanamori
- It is based on the use of the seismic moment M_0

$$M_W = \frac{2}{3} \log M_0 - cost$$

M_0: seismic moment

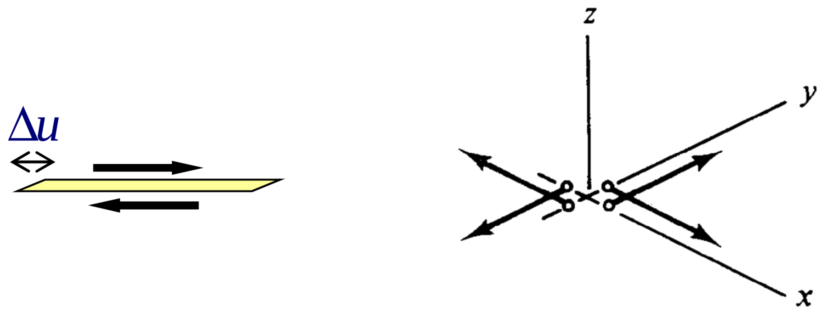
$cost$: 10.7 if M_0 [dyne·cm]; 6 if M_0 [N·m];

- ✓ M_W is the best measure of earthquake size
- ✓ M_W does not saturate, since M_0 can increase without limits as fault and dislocation dimensions increase
- ✓ It can be estimated from geological observations and paleoseismologic studies
- ✓ It can be tied to plate motions and recurrence relations

<u>***Seismic moment***</u> M_0:

- Moment of a double couple force system which generates the same strain induced by a shear dislocation on an infinitesimal element of surface.

Equivalence shear dislocation-double couple force system



- Measure of the relevant features of an earthquake source.
- It is proportional to the energy released during the earthquake.

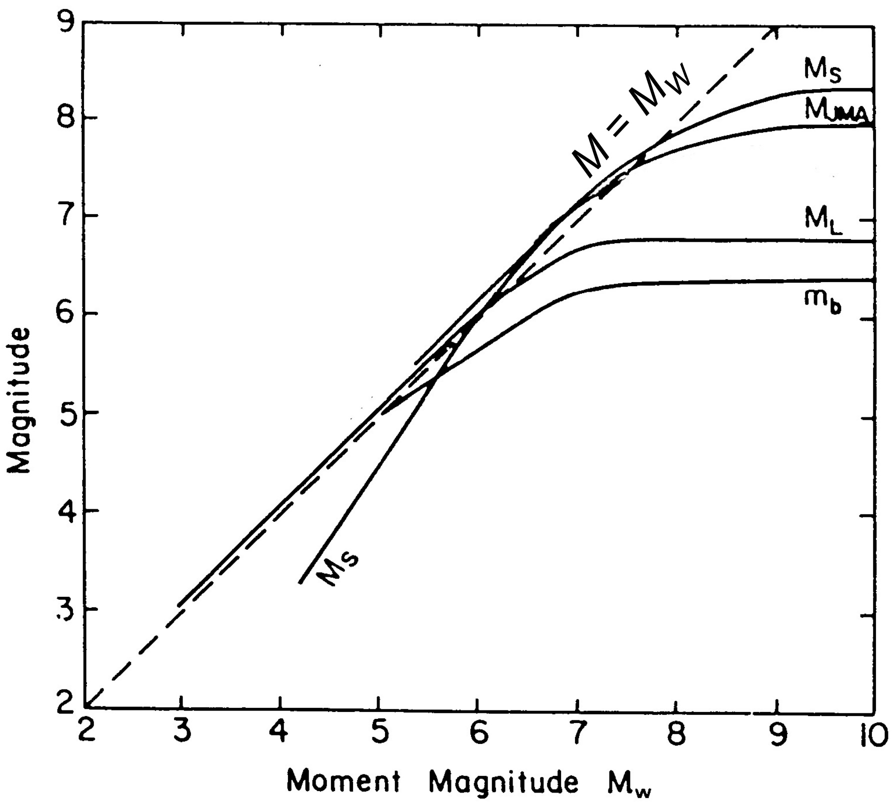
$$M_0 = \mu \overline{\Delta u} A$$

μ: shear modulus

$\overline{\Delta u}$: average relative displacement on the dislocation surface

A: area of the dislocation surface

Magnitude scales relations:



M_{JMI} scale defined by the Japanese Meteorological Agency

$$M_W = M_b \text{ for } M_W \le 5$$

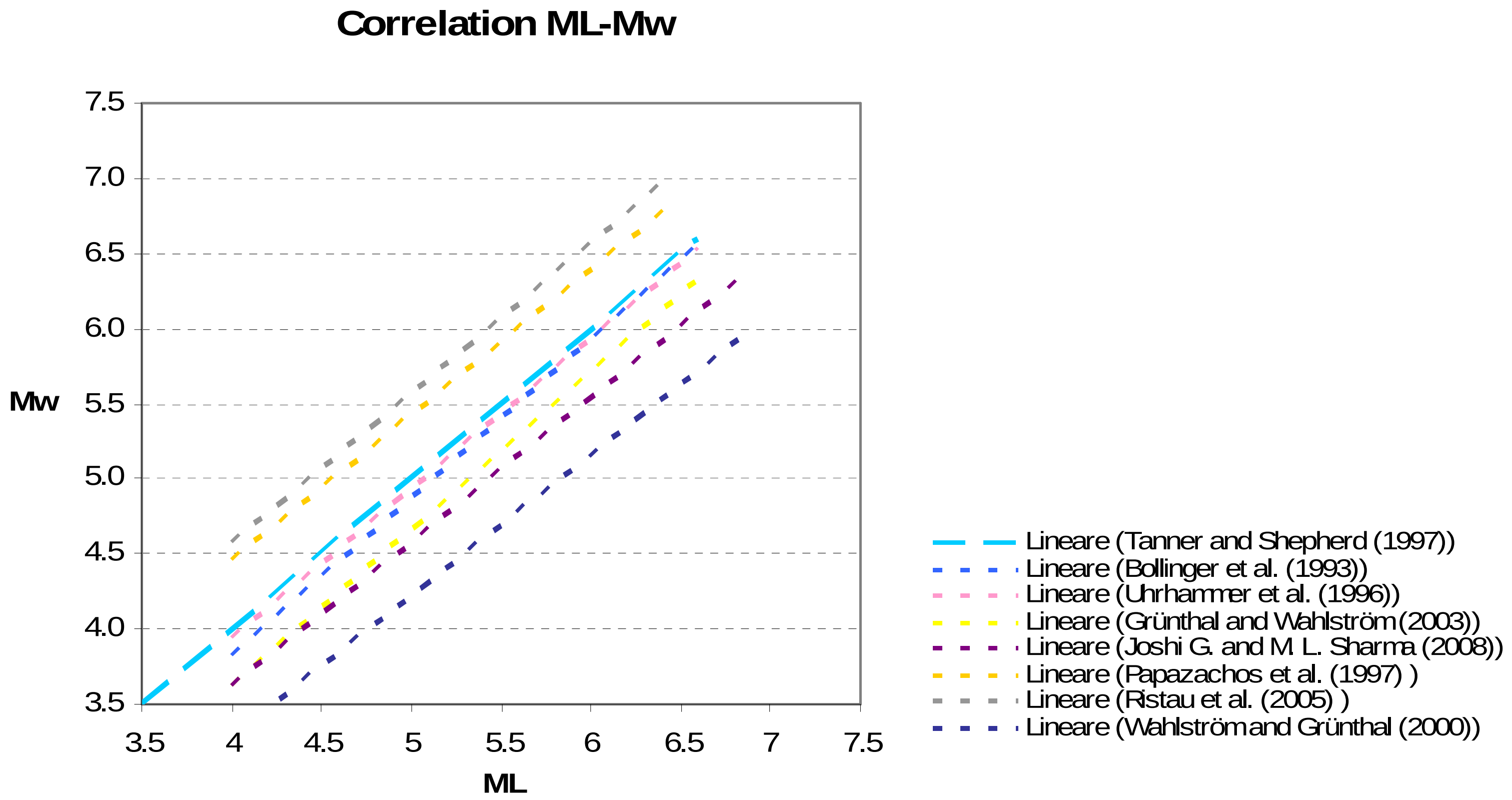
$$M_W = M_L \text{ for } M_W \le 6.2$$

$$M_W = M_S \text{ for } 6.2 \le M_W \le 8$$

(Modified from Kanamori, 1983)

Magnitude scales conversions:

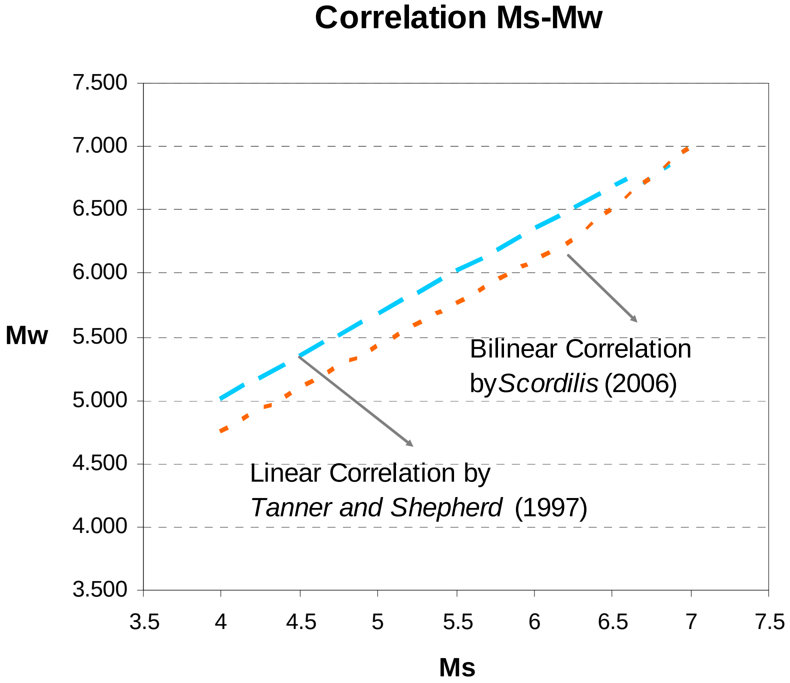
Empirical relations can be used to convert between magnitude scales. This is important in deriving magnitude recurrence statistics for a region or source zone, as all magnitudes should be first reported on the same scale before



Magnitude scales conversions:

M_S - M_W

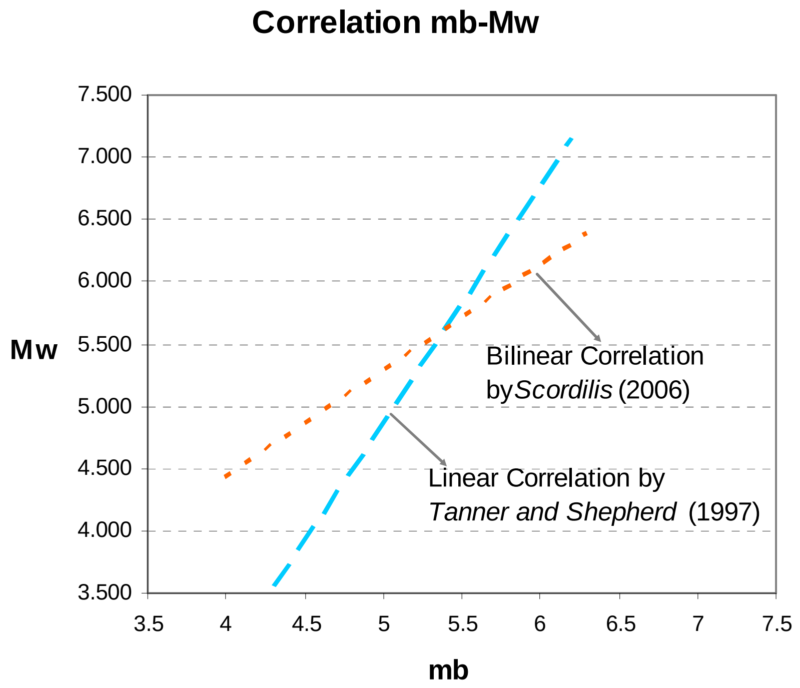
CORRELATION	SOURCE	STUDIED ZONE
$M_W=0.67M_S + 2.07$ ($M_S \le 6.2$) $M_W=0.99M_S + 0.08$ ($M_S > 6.2$)	Scordilis, 2006	Worldwide
$M_W=0.667M_S + 2.34$ ($M_S \le 6.6$) $M_W=M_S$ ($M_S > 6.6$)	Tunner & Shepherd, 1997	Latin America and Caribbean



Magnitude scales conversions:

M_b - M_W

CORRELATION	SOURCE	STUDIED ZONE
$M_W = 0.85 M_b + 1.03$	Scordilis, 2006	Worldwide
$M_W = 1.9048 M_b - 4.6619$	Tunner & Shepherd, 1997	Latin America and Caribbean



EUCENTRE
European Centre for Training and Research in Earthquake Engineering

IUSS

SEVENTH FRAMEWORK PROGRAMME

Macroseismic intensity scales

PAVIA RISK CENTRE

Definition:

Macroseismic intensity is a classification of the strength of shaking at any place during an earthquake, in terms of its observed effects on humans, objects, nature, and damage to buildings.

Main developed Scales:

- RF, end ‘800 : Rossi and Forel, (X degrees)
- Mercalli, 1883, 1902 (RF revised)
- Cancani,1904 (Mercalli revised and amplified to XII degrees)
- MCS, 1930: Mercalli-Cancani-Sieberg (Cancani improved)
- MM, 1931, 1956: Modified Mercalli
- MSK, 1964: Medvedev, Sponheuer and Kárník (MCS, previous Medvedev scale, Wood and Neumann and Richter’s work rearranged)
- EMS98, 1998: European Macroseismic scale (MSK revised, XII degrees)
- JMA, 1996: Japanese Meteorological Agency scale (9 degrees)

MM scale

I. Not felt except by a very few under especially favorable conditions.

II. Felt only by a few persons at rest, especially on upper floors of buildings.

III. Felt quite noticeably by persons indoors, especially on upper floors of buildings. Many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibrations similar to the passing of a truck. Duration estimated.

IV. Felt indoors by many, outdoors by few during the day. At night, some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.

V. Felt by nearly everyone; many awakened. Some dishes, windows broken. Unstable objects overturned. Pendulum clocks may stop.

VI. Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight.

VII. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken.

VIII. Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned.

IX. Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations.

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations. Rails bent.

XI. Few, if any (masonry) structures remain standing. Bridges destroyed. Rails bent greatly.

XII. Damage total. Lines of sight and level are distorted. Objects thrown into the air.

EMS scale: main aspects

- basis: MSK scale

- robustness, i.e. minor differences in diagnostics should not make large differences in the assessed intensity; further to this, the scale should be understood and used as a compromise solution, since no intensity scale can hope to encompass all the possible disagreements between diagnostics that may occur in practice
- such disagreements may also reflect differences in cultural conditions in the regions where the scale is used
- simplicity of use
- rejection of any intensity corrections for soil conditions or geomorphological effects, because detailed macroseismic observations should just be a tool for finding and elaborating such amplification effects
- understanding of intensity values as being representative for any village, small town or part of a larger town instead of being assigned to a point (for one house etc)

EMS scale: problems to be solved

- need to include new types of buildings, especially those including earthquake-resistant design features, which are not covered by previous versions of the scale
- need to address a perceived problem of non-linearity in the scale arrangement at the junction of the degrees VI and VII (which, after thorough discussion for preparing the EMS-92, as well as for the EMS-98, proved to be illusory)
- need to generally improve the clarity of the wording in the scale
- need to decide what allowance should be made for including high-rise buildings for intensity evaluations
- whether guidelines for equating intensities to physical parameters of strong ground motions, including their spectral representations, should be included
- to design a scale that not only meets the needs of seismologists alone, but which also meets the needs of civil engineers and other possible users
- to design a scale which should be suitable also for the evaluation of historical earthquakes
- need for a critical revision of the usage of macroseismic effects visible in the ground (rock falls, fissures etc.) and the exposure of underground structures to shakings

EMS scale (short form)

I. **Not felt** Not felt.

II. **Scarcely felt** Felt only by very few individual people at rest in houses.

III. **Weak** Felt indoors by a few people. People at rest feel a swaying or light trembling.

IV. **Largely Observed** Felt indoors by many people, outdoors by very few. A few people are awakened. Windows, doors and dishes rattle.

V. **Strong** Felt indoors by most, outdoors by few. Many sleeping people awake. A few are frightened. Buildings tremble throughout. Hanging objects swing considerably. Small objects are shifted. Doors and windows swing open or shut.

VI. **Slightly Damaging** Many people are frightened and run outdoors. Some objects fall. Many houses suffer slight non-structural damage like hair-line cracks and fall of small pieces of plaster.

VII. **Damaging** Most people are frightened and run outdoors. Furniture is shifted and objects fall from shelves in large numbers. Many well built ordinary buildings suffer moderate damage: small cracks in walls, fall of plaster, parts of chimneys fall down; older buildings may show large cracks in walls and failure of fill-in walls.

VIII. **Heavily Damaging** Many people find it difficult to stand. Many houses have large cracks in walls. A few well built ordinary buildings show serious failure of walls, while weak older structures may collapse.

IX. **Destructive** General panic. Many weak constructions collapse. Even wellc built ordinary buildings show very heavy damage: serious failure of walls and partial structural failure.

X. **Very Destructive** Many ordinary well built buildings collapse.

XI. **Devastating** Most ordinary well built buildings collapse, even some with good earthquake resistant design are destroyed.

XII. **Completely Devastating** Almost all buildings are destroyed.

EMS scale (short form)

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III. **Weak** Felt indoors by a few people. People at rest feel a swaying or light trembling.

VIII. **Heavily Damaging**

EMS (complete form)

a) Many people find it difficult to stand, even outdoors.

b) Furniture may be overturned. Objects like TV sets, typewriters etc. fall to the ground. Tombstones may occasionally be displaced, twisted or overturned. Waves may be seen on very soft ground.

c) Many buildings of vulnerability class A suffer damage of grade 4; a few of grade 5.
Many buildings of vulnerability class B suffer damage of grade 3; a few of grade 4.
Many buildings of vulnerability class C suffer damage of grade 2; a few of grade 3.
A few buildings of vulnerability class D sustain damage of grade 2.

plaster, parts of chimneys fall down, older buildings may show large cracks in walls and failure of fill-in walls.

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EMS scale: main distinguishing features

- Key aspects of the classification defined with a High level of detail, i.e.: building type, the associated seismic vulnerability, damage degree, involved quantities
- First scale with associated figures and photos to illustrate the meaning of damage degree
- Introduction of differentiation of different types of buildings (masonry, reinforced concrete, wood, …) into vulnerability classes (A, B,…, F). The vulnerability associated to a building has to be identified accounting for degradation state, construction quality, plant and vertical regularity, level of seismic norms applied at the design stage.
- Introduction of different damage classification for different types of structures, i.e. masonry and reinforced concrete
- Introduction of quantity definition: few (up to 15-20%), many (from 15 to 55-60%), most from 55 to 100%),

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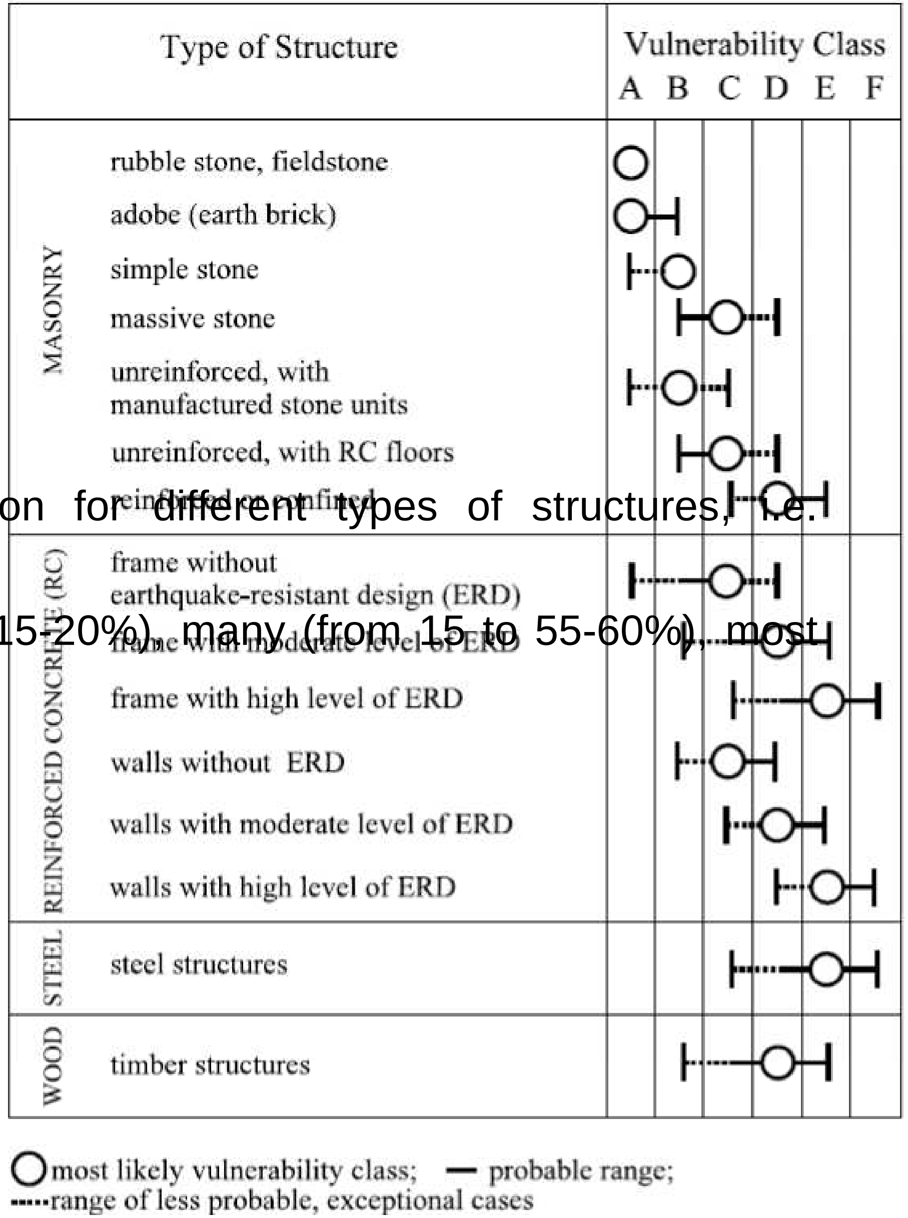

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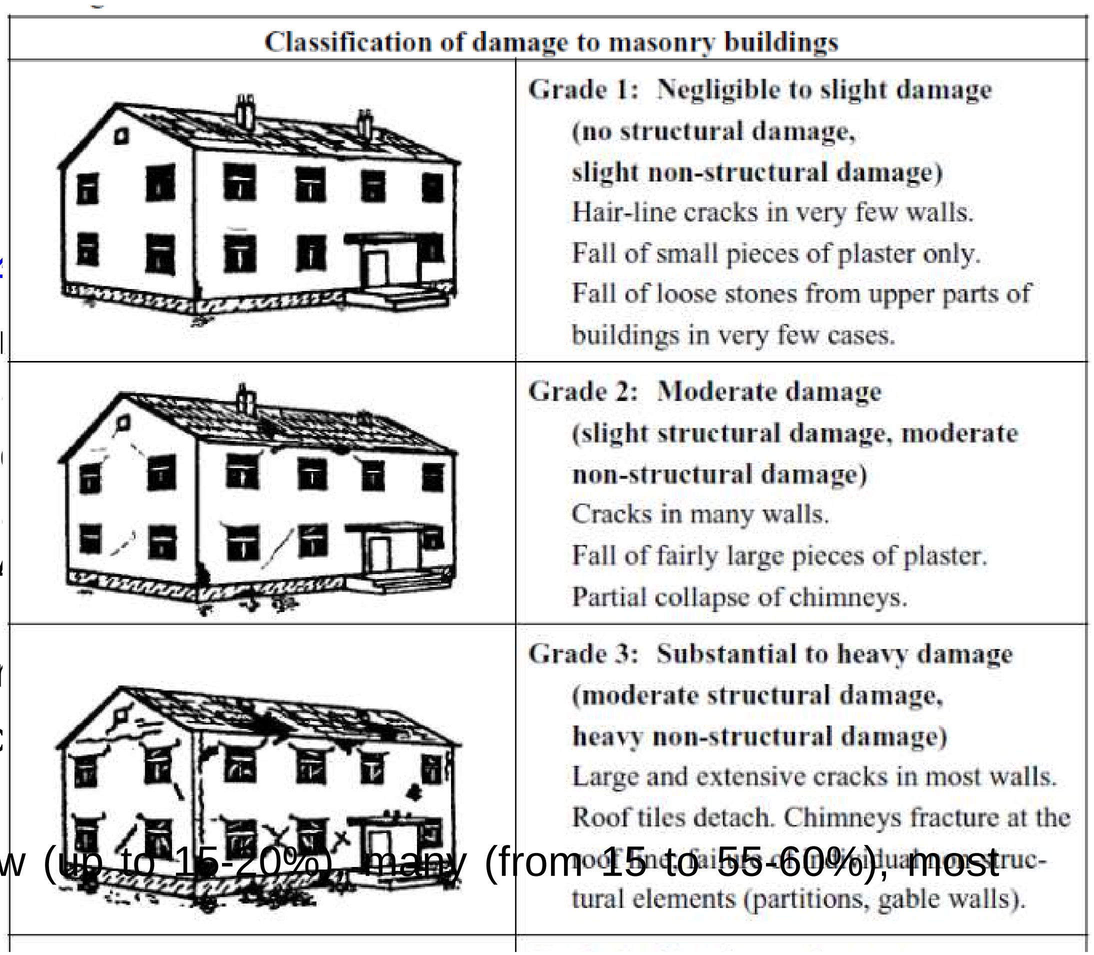
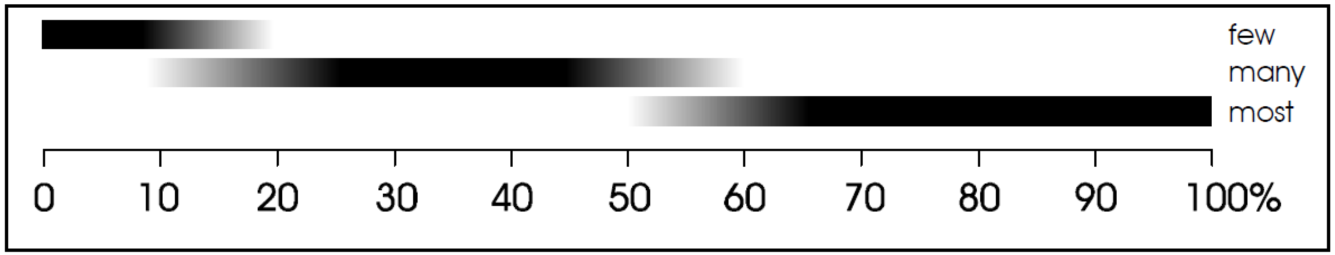


Definitions of quantity



JMA scale

- It is made by converting instrumental ground motion parameters into pseudo-intensity values (Yamazaki et al 1998); so one has no idea whether a quoted intensity value for a modern Japanese earthquake actually corresponded to the description of effects for that degree of the scale being observed or not (Musson et al., 2010)
- Sites where equal seismic intensities were observed did not necessarily suffer the same degree of damage, since damage depends on the type of construction used and on the nature of the seismic ground motion
- Seismic intensity is a value observed at a site where a seismic intensity meter is installed. Seismic intensity is usually measured on the ground surface, so in general, the shaking on upper stories of buildings may be amplified greatly
- A large earthquake generates long-period seismic waves. Even at locations far from the epicenter, where the seismic intensity is rather small, the long-period waves may occasionally cause unusual types of damage, such as the sloshing of oil in a tank or troubles with elevators
- The scale is prepared based mainly on the examples collected from recent destructive earthquakes. It is subject to revision when new examples are collected or the present descriptions become inconsistent with actual situations, due to the improvement of earthquake resistant buildings, etc.

JMA scale

JMA Seismic Intensity Scale	People	Indoor situations	Outdoor situations
0	Imperceptible to people.		
1	Felt by only some people in the building.		
2	Felt by many people in the building. Some sleeping people awakened.	Hanging objects such as lamps swing slightly.	
3	Felt by most people in the building. Some people are frightened.	Dishes in a cupboard rattle occasionally.	Electric wires swing slightly.
4	Many people are frightened. Some people try to escape from danger. Most sleeping people are awakened.	Hanging objects swing considerably and dishes in a cupboard rattle. Unstableornaments fall occasionally.	Electric wires swing considerably. People walking on a street and some people driving automobiles feel the tremor.
5Lower	Most people try to escape from danger. Some people find it difficult to move.	Hanging objects swing violently. Most unstable ornaments fall. Occasionally, dishes in a cupboard and books on a bookshelf fall and furniture moves.	People notice electric-light poles swaying. Occasionally, window panes are broken and fall, unreinforced concrete-block walls collapse, and roads suffer damage.
5Upper	Many people are very frightened and find it difficult to move.	Most dishes in a cupboard and most books on a bookshelf fall. Occasionally, a TV set on a rack falls, heavy furniture such as a chest of drawers falls, sliding doors slip out of their groove, and the deformation of a door frame makes it impossible to open the door. Some doors get unhinged.	In many cases, unreinforced concrete-block walls collapse and tombstones overturn. Many automobiles come to a stop because it becomes difficult to drive. Occasionally, poorly installed vending machines fall over.
6Lower	Difficult to keep standing.	A lot of heavy and unbolted furniture moves and falls, it is impossible to open the door in many cases.	In many buildings, wall tiles and windowpanes are damaged and fall.
6Upper	Impossible to keep standing or to move without crawling.	Most heavy and unbolted furniture moves and falls. Occasionally, sliding doors are thrown out of their grooves.	In many buildings, wall tiles and windowpanes are damaged and fall. Most unreinforced concrete-block walls collapse.
7	Thrown by the shaking and impossible to move at will.	Most furniture moves to a large extent and some jumps up.	In most buildings, wall tiles and windowpanes are damaged and fall. In some cases, reinforced concrete-block walls collapse.

JMA scale

JMA Seismic Intensity Scale	Wooden houses	Reinforced-concrete buildings	Basic infrastructure	Ground and slopes
0				
1				
2				
3				
4				
5Lower	Occasionally, less earthquake-resistant houses suffer damage to walls and pillars.	Occasionally, cracks are formed in walls of less earthquake-resistant buildings.	A safety device cuts off the gas service at some houses. On rare occasions water pipes are damaged and water service is interrupted. [Electrical service is interrupted at some houses.]	Occasionally, cracks appear in soft ground. Rock falls and small slope failures take place in mountainous districts.
5Upper	Occasionally, less earthquake-resistant houses suffer heavy damage to walls and pillars and lean to one side.	Occasionally, large cracks are formed in walls, crossbeams and pillars of less earthquake-resistant buildings, and even highly earthquake-resistant buildings develop cracks in walls.	Occasionally, gas pipes and/or water mains are damaged. [Occasionally, gas service and/or water service are interrupted in some regions.]	Occasionally, cracks appear in soft ground. Rock falls and small slope failures take place in mountainous districts.
6Lower	Occasionally, less earthquake-resistant houses collapse and even walls and pillars of highly earthquake-resistant houses are damaged.	Occasionally, walls and pillars of less earthquake-resistant buildings are destroyed, and even highly earthquake-resistant buildings develop large cracks in walls, crossbeams and pillars.	Gas pipes and/or water mains are damaged. [In some regions, gas service and water service are interrupted and electrical service is interrupted occasionally.]	Occasionally, cracks and slope failures take place in mountainous districts.
6Upper	Many less earthquake-resistant houses collapse. In some cases, even walls and pillars of highly earthquake-resistant houses are heavily damaged.	Occasionally, less earthquake-resistant buildings collapse. In some cases, even highly earthquake-resistant buildings suffer damage to walls and pillars.	Occasionally gas mains and/or water mains are damaged. [Electrical service is interrupted in some regions. Occasionally, gas service and/or water service are interrupted over a large area.]	Occasionally, cracks and slope failures take place in mountainous districts.
7	Occasionally, even highly earthquake-resistant houses are severely damaged and lean to one side.	Occasionally, even highly earthquake-resistant buildings are severely damaged and lean to one side.	[Electrical service, gas service and water service are interrupted over a large area.]	The ground is considerably distorted by large cracks and fissures, and slope failures and landslides take place, which occasionally change local topographic features.

Intensity Scales conversions:

Mercalli (1883)	EMS-98
1	2 or 3
2	4
3	5
4	6 or 7
5	8 or 9
6	10 or 11

RF	EMS-98	MCS	EMS-98	MMI 56	EMS-98	MSK	EMS-98	JMA-96	EMS-98
								0	1
1	1	1	1	1	1	1	1	1	2 or 3
2	2	2	2	2	2	2	2	2	4
3	3	3	3	3	3	3	3	3	4 or 5
4	4	4	4	4	4	4	4	4	5
5	5	5	5	5	5	5	5	5L	6
6	5	6	6	6	6	6	6	5U	7
7	6	7	7	7	7	7	7	6L	8
8	7 or 8	8	8	8	8	8	8	6U	9 or 10
9	9	9	9	9	9	9	9	7	11
10	-[a]	10	10	10	10	10	10		
11	11	11	-[a]	11	11				
12	-[a]	12	-[a]	12	-[a]				

[a] This intensity is defined in such a way that it relates to phenomena that do not represent strength of shaking, e.g. those due to surface faulting, or reaches a saturation point in the scale where total damage refers to total damage to buildings without antiseismic design.

(From Musson et al., 2010)

- ✓ Conversions should be used with care, and checks made if at all possible.
- ✓ Conversion in this manner should only be used where absolutely necessary.

Notes:

- ✓ Intensity is often used in seismic hazard, since it relates directly to damage and yields hazard values, which are relevant to planners and insurers
- ✓ Site classification by intensity is associated to quality and typology of constructions and depends on housing concentration. Therefore, a strong earthquake that strikes a desert site can be classified with a low intensity degree, while a moderate earthquake which strikes a site with vulnerable buildings and causes victims is classified with a relative high intensity degree
- ✓ A magnitude scale expresses the seismic energy released by an earthquake, while an intensity scale denotes how strongly an earthquake affects a specific place.

M-I relations:

- General forms of empirical relations:

$$M = aI_0 + c'$$

$$M = aI_0 + b\log h + c$$

h: earthquake depth

a, b, c, c': coefficients

- Empirical relationship derived by Gutemberg and Richter (1956), based exclusively on data from Californian earthquakes:

$$M = 1 + \frac{2}{3} I_0$$

M: Modified Mercalli-1931

I_0: maximum earthquake intensity

M-I relations:

E. Peterschmitt [1]	$M = 0{\cdot}8 I_0 - 0{\cdot}9$	Schwäbische Alb (Germany)
Savarensky-Dzhibladze [2]	$M = 0{\cdot}69 I_0 + 0{\cdot}9$	Caucasus
J. V. Aivazov [3]	$M = 0{\cdot}93 I_0 + 1{\cdot}14 \log h - 3\mathrm{i}0$	Caucasus
Csomor-Kiss [4]	$M = 0{\cdot}6 I_0 + 0{\cdot}3$	Hungary
	$M = 0{\cdot}6 I_0 + 1{\cdot}8 \log h - 1{\cdot}3; \quad h = n$	
Grigorov-Grigorova [5]	$M = 0{\cdot}51 I_0 + 1{\cdot}1$	Bulgaria (M based on S and Sg!)
Petrescu-Radu [6]	$M = 0{\cdot}51 I_0 + 2{\cdot}55$	Vrancea, Rumania, $h = 100$ km $\pm$
W. Sponheuer [7]	$M = 0{\cdot}52 I_0 + 1{\cdot}56 \log h + 0{\cdot}7 \alpha h$	Germany
V. Kárník [8]	$M = 0{\cdot}5 I_0 + 1{\cdot}8$	Europe
	$M = 0{\cdot}55 I_0 + 0{\cdot}93 \log h + 0{\cdot}14$	Czechoslovakia
N. V. Shebalin [9]	$M = 0{\cdot}67 I_0 + 2{\cdot}3 \log h - 2{\cdot}0; \; h = n,$	
	$M = 0{\cdot}67 I_0 + 2{\cdot}3 \log h - 3{\cdot}6; \; h = i, d$	
G. Galanopoulos [10]	$M = 0{\cdot}83 \log r^2 + 0{\cdot}28 I_0 - 0{\cdot}13$	Greece
A. Adlung [11]	$M = 0{\cdot}5 I_0 + \log h + 1{\cdot}32$	Schwäbische Alb, Germany
Marcelli-Montecchi [12]	$M = 0{\cdot}481 I_0 + 1{\cdot}407$	Italy
The relations for non-European countries are:		
Gutenberg-Richter [13]	$M = 0{\cdot}6 I_0 + 1{\cdot}8 \log h - 1.0$	
	$M = 2/3 I_0 + 1{\cdot}0$	California, $h = 18$ km $\pm$
Mei Shi Yun [14]	$M = 2/3 I_0 + 4/5 \log h - 1/2$	China
S. P. Lee [15]	$M = 0{\cdot}58 I_0 + 1{\cdot}5$	China
V. I. Bune [16]	$M = 0{\cdot}61 I_0 + 2/3 \log h + 0{\cdot}39$	Tadzhik SSR

From Karnik, 1965

Ground motion prediction equations (GMPEs)

Definition:

Models used in engineering seismology applications to estimate the distribution of future ground-motions, to quantify the severity of an earthquake at a site with the aim of structures design or risk assessment

Main properties:

- Global (based on worldwide data) or regional (developed for selected region)
- Simple models that only require knowledge of a few representative parameters of ground motion
- Use of one measure of earthquake intensity at source: magnitude
- Use of one measure accounting of effects of waves propagation from source to site: distance
- Significant variability associated with the estimates of these equations
  - Partly reflects the simplicity of the models
  - Partly reflects the inherent variability of earthquake ground-motions

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Functional form:

- The basic form is motivated by waves propagating from a point source

DISTANCE DEPENDENCE:

Point source in a whole space

$$Y \propto \frac{e^{-cR}}{R}$$

Functional form

$$\log Y = a_4 R + a_5 \log R$$

with $R = \sqrt{h^2 + D^2}$

Y: measure of the ground motion

D : measure of the source-site distance

h : “pseudo-depth” to be determined from the regression, correction term for distance

a_i: regression coefficient which can be function of the period

- Magnitude scaling is accounted for with a first or second order polinomial

MAGNITUDE DEPENDENCE:

$$log\,Y = a_1 + a_2\left(M - M_{r1}\right) + a_3\left(M - M_{r2}\right)^2 + a_4 R + a_5\, log\, R$$

M: earthquake magnitude

$M_{r1,\, r2}$: reference magnitudes to center the equation and to attemp prevention of decreasing motion for large M

- More complicated forms are developed, e.g.:

$$R = D + a_6\, exp\left(a_7 M\right), \quad R = \sqrt{D^2 + \left(a_6 + exp\left(a_7 M\right)\right)^2}, \quad \ldots$$

To accomodate saturation at close distances

NOTES:

- Describe the INCREASE of amplitude with magnitude at a given distance
- Describe the CHANGE of amplitude with distance for a given magnitude (usually, but not necessarily, a DECREASE of amplitude with increasing distance).

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- Additional terms to capture further effects

FURTHER DEPENDENCES:

- Site
- Stile of faulting
- Non linear site response
- Directivity
- Footwall, hanging wall
- Basin response

$$\log Y = a_1 + a_2\left(M - M_{r1}\right) + a_3\left(M - M_{r2}\right)^2 + a_4 R + a_5 \log R +$$

$$+ f_{site} + f_{fault} + f_{nl} + f_{dir} + f_{hng} + f_{basin}$$

- Measure of data dispersion

$$\log Y = \ldots + \varepsilon$$

ε: runcertainty of the predictive relation. It is a random variable assumed with normal distribution with null average value. The standard deviation is used to quantify the error associated to the average value estimated by the correlation. Intra-event standard deviation and inter-event aleatory uncertainty can be accounted for.

Derivation:

- When adequate data are available:
  - Based on empirical approach
  - Derived by regression analysis of a large suite of recorded earthquake motions

- When adequate data are lacking:
  - Based on regression analysis of data simulation (making use of motions from smaller events if available to constrain distance dependence of motions)
  - Based on hybrid methods, capturing complex source effects from observed data and modifying for regional differences

- Basic procedure:
  - Dependent variable Y is calculated from available data
  - Starting from the adopted functional form a statistical multivariate regression (linear or not linear) is computed as function of the independent variables M and D
  - Various iterations are performed to determine the assumed coefficients (i.e. h) for which the residual is minimum

Measure of the ground motion:

- Macroseismic intensity
- Peak Ground Acceleration PGA, Velocity PGV or Displacement PGD
- Ordinates of acceleration S_a, velocity S_v or displacement S_d response spectra at a fixed period

Data origin:

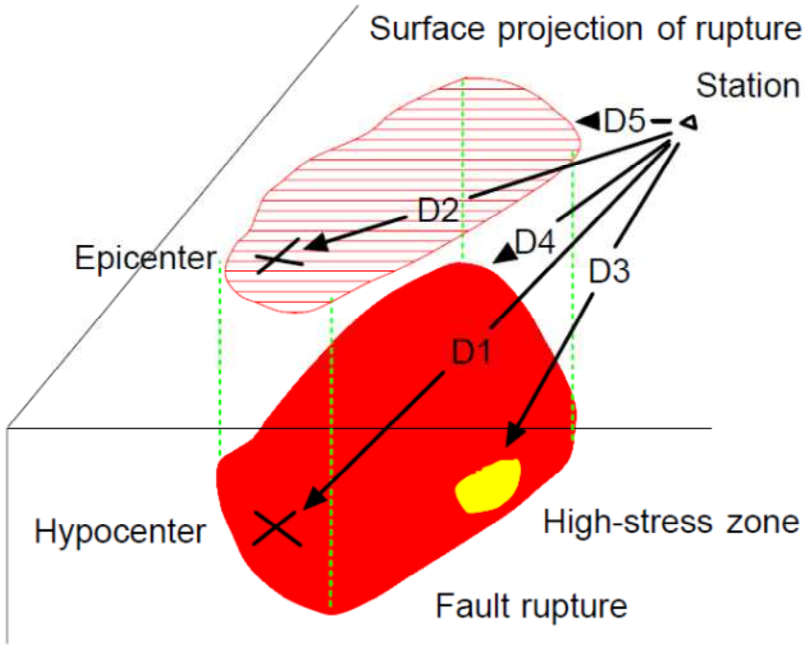
- World-wide
- American
- European
- Japanese
- Regional

Seismotectonic environment:

- Shallow earthquakes in active tectonic regions
- Stable continental regions
- Subduction zones

Prediction variables:

- Preferred moment **magnitude** M_w
- Different **distance** measured are used (), although the closest distance to the rupture surface is probably the distance most commonly used



D1: Hypocentral

D2: Epicentral

D3: from the most energetic zone

D4: from the source

D5: from the source projection to the surface

Used site classifications:

- Rock/Soil
  - Rock: less than 5 m soil over rock (granite, limestone, etc…)
  - Soil: everything else

- Continuous variable V_{S30}

- NEHRP Site classes

TABLE 4.
Definition of NEHRP site classes (BSSC, 1994)

Site Class	Range of Shear Velocities*
A	greater than 1500 m/sec
B	760 m/sec to 1500 m/sec
C	360 m/sec to 760 m/sec
D	180 m/sec to 360 m/sec
E	less than 180 m/sec

* Shear velocity is averaged over the upper 30 m.

- EC8 Site classes

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

Main criteria of reliability:

- Homogeneity of recording instruments and processing method
- Influence of features of recording sites (interactions, burial depth with respect to the ground level, as recordings at the base of buildings; in fact amplification of acceleration peaks can be highlighted due to uppermost layers of a few meters)
- M and D are teoretically assumed independent variables, while there is a degree of correlation due to the fact that earthquakes with high M are rare with respect to those with low M and the probability to record them at small D is lower

Some of most widely applied GMPEs:

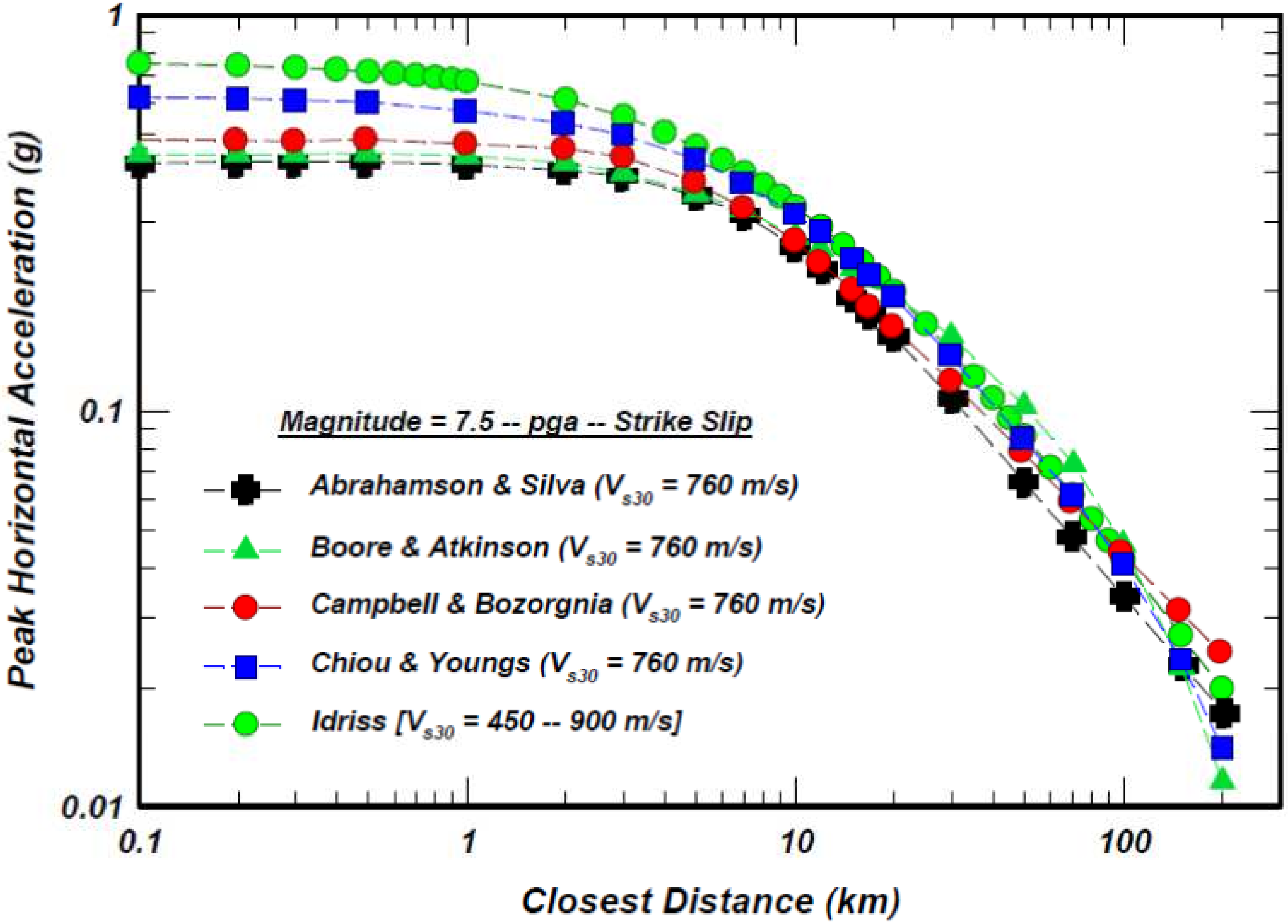
- for spectral accelerations
  - Campbell KW & Bozorgnia Y (2008)
  - Boore DM & Atkinson GM (2008)
  - Chiou BS-J & Youngs RR (2008)
  - Abrahamson NA & Silva W (2008)
  - Idriss (2008)

Next Generation of Attenuation (NGA) Relations Project

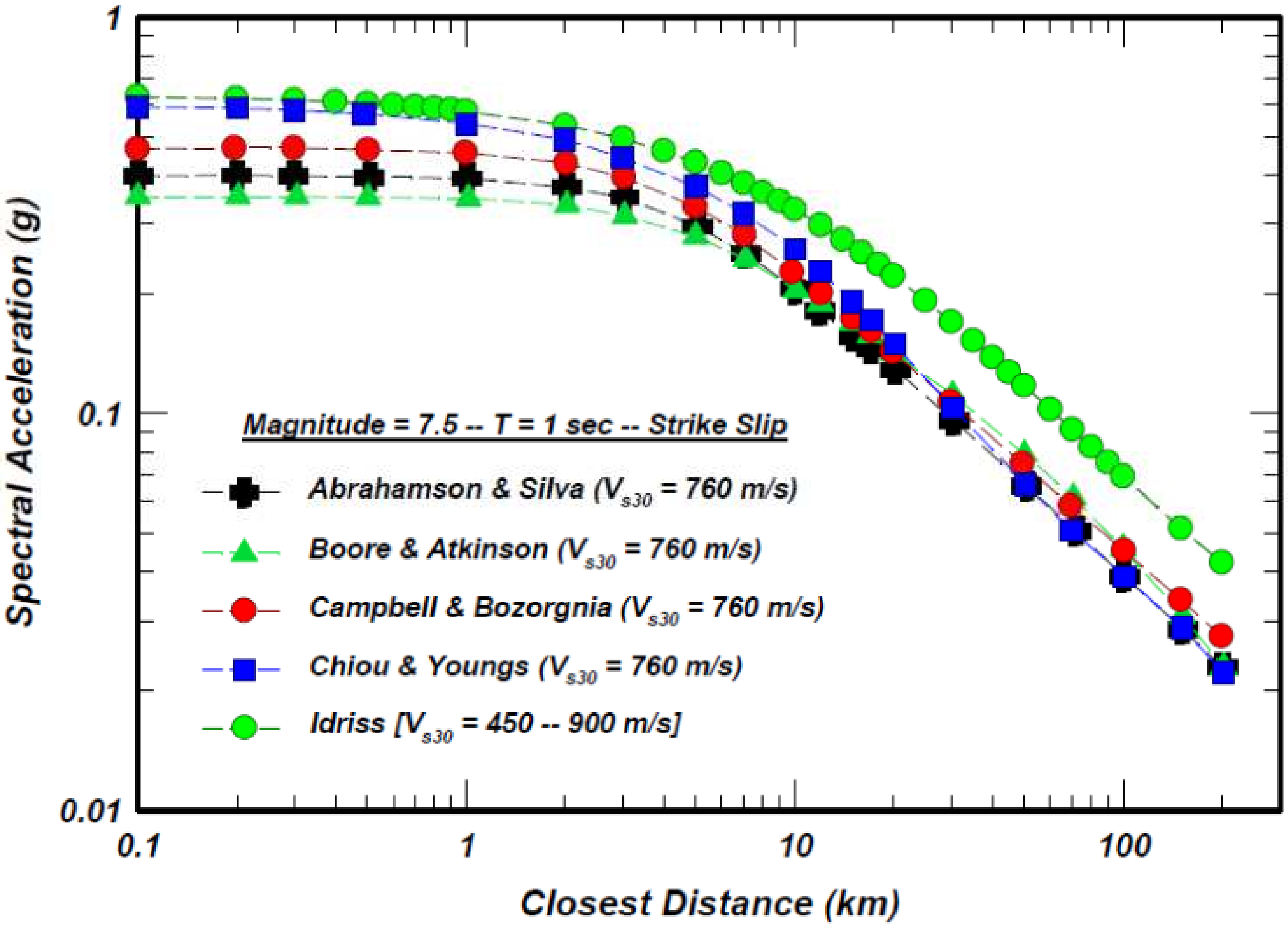
- for spectral displacement (more suitable for displacement-based design methodology)
  - Akkar S & Bommer JJ (2007)
  - Cauzzi & Faccioli (2008)

- for subduction zones (both interface and inslab events)
  - Youngs et al. (1997)
  - Atkinson and Boore (2003-2008)
  - Zhao et al (2006)
  - Kanno et al. (2006)
  - Lin and Lee (2008)
  - Youngs et al. (1997)

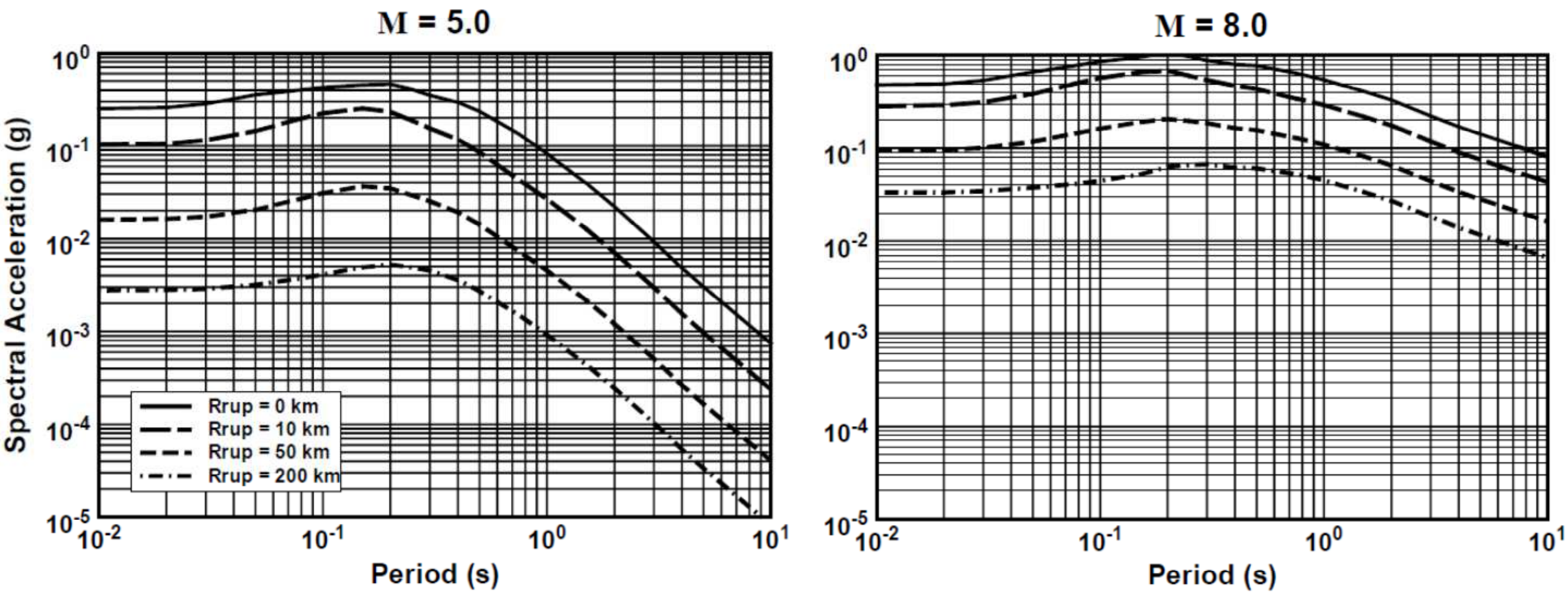
NGA GMPEs: comparison of median value of PGA



NGA GMPEs: comparison of median value of spectral acceleration at T=1.s

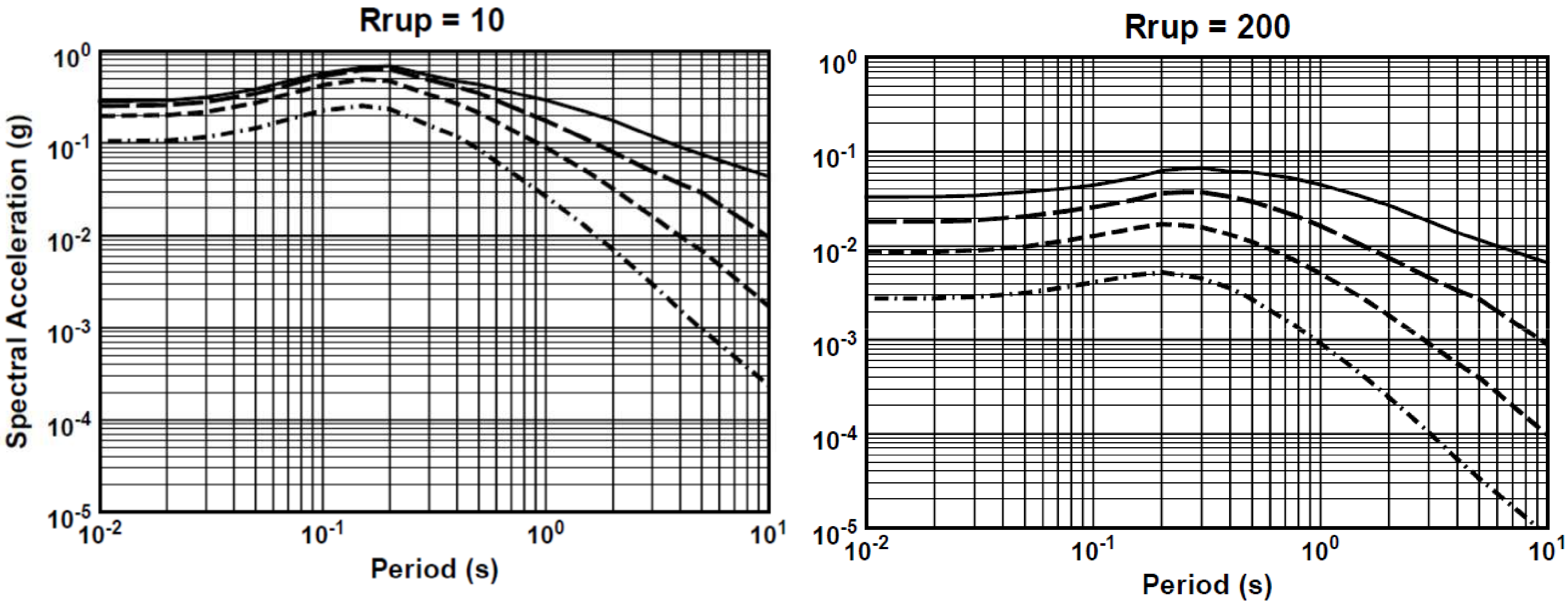


Spectral acceleration scaling with distance



(From Campbell and Bozorgnia, 2008)

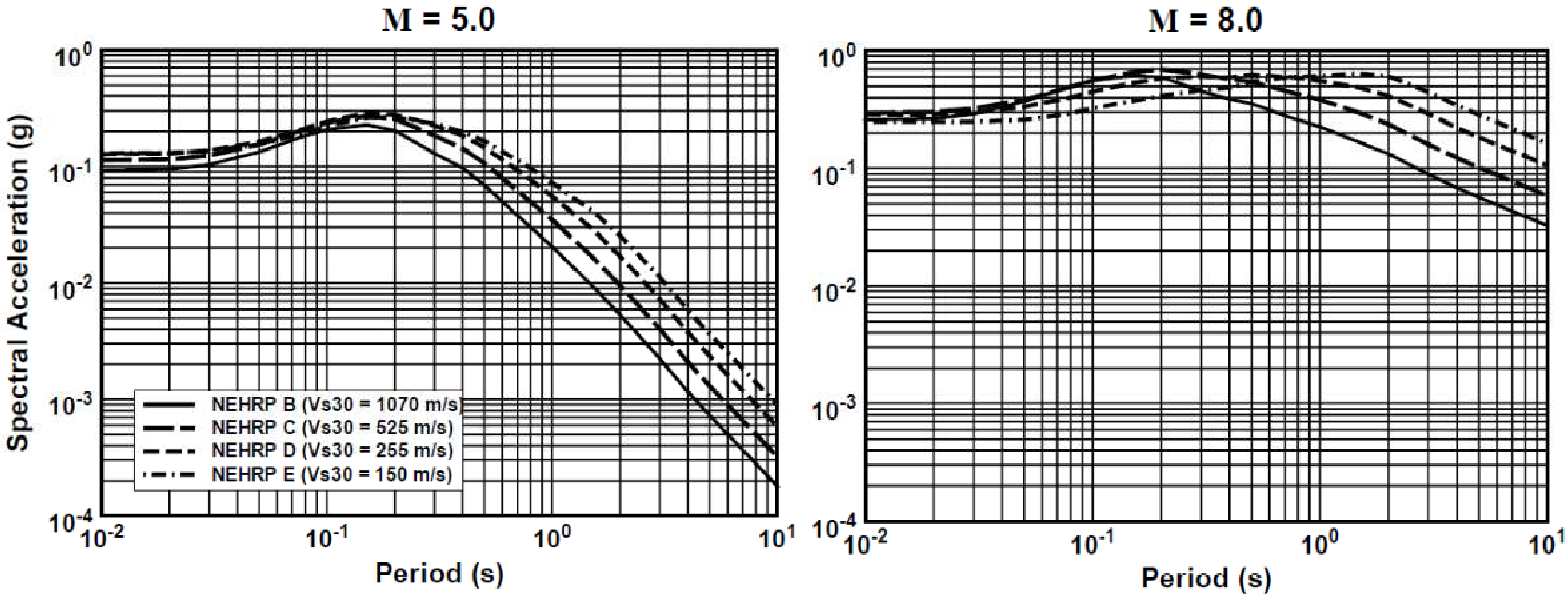
Spectral acceleration scaling with earthquake magnitude



(From Campbell and Bozorgnia, 2008)

Spectral acceleration scaling with site conditions

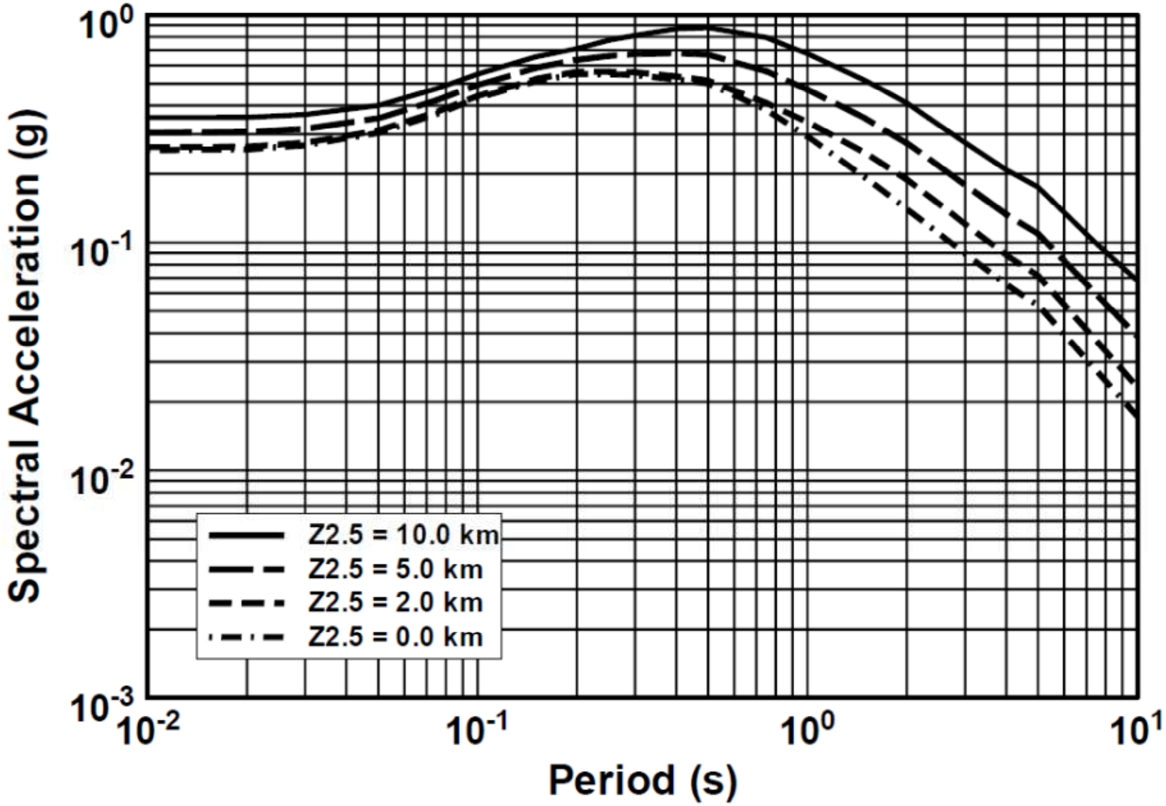
R_{rup}=10 km



(From Campbell and Bozorgnia, 2008)

Spectral acceleration scaling with sediments depth

(M=7, R_{rup}=10 km)



(From Campbell and Bozorgnia, 2008)

Seismic hazard in Palestina

Seismic hazard in Palestina

Earthquake hazard assessment for building codes (2007)

EARTHQUAKE HAZARD ASSESSMENTS FOR

BUILDING CODES

FINAL REPORT

April 2000 – March 2007
Proposal Number: M18-057
Grant No. PCE-G-00-99-00038

Principal Investigators:
Avi Shapira
Rami Hofstetter
Geophysical Institute of Israel

Collaborators:
Abdel-Qader F. Abdallah
Natural Resources Authority of the Hashemite Kingdom of Jordan

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University of North Carolina at Charlotte

Submitted to the U.S. Agency for International Development,
Bureau for Economic Growth, Agriculture and Trade

Seismic hazard in Palestina

Earthquake hazard assessment for building codes (2007)

Participants:

- The Geophysical Institute of Israel, Seismology Division
- Geological Survey of Israel
- Haifa Technion
- Israel Atomic Energy Commission
- Palestinian National Authority:
  An Najah National University, Nablus
- UNESCO (United Nations Educational, Scientific and Cultural Organization)
- Jordan:
  Natural Resources Authority of the Hashemite Kingdom of Jordan
  Royal Scientific Society
- USA USGS
- Lawrence Livermore National Laboratory

Seismic hazard in Palestina

Earthquake hazard assessment for building codes (2007)

Main product:

Newly developed regional probabilistic seismic hazard map of peak ground acceleration (PGA) levels that have a probability of 10% of being exceeded at least once within a period of 50 years

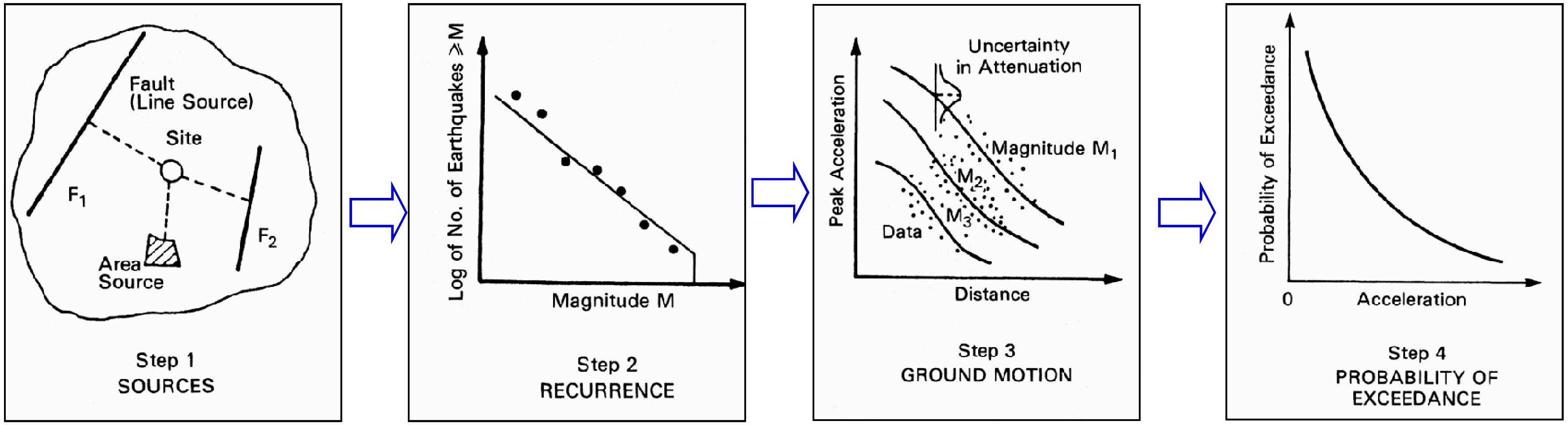
This map provides the basic seismic input parameter for use in the development and implementation of modern building codes and regulations in Jordan, Israel and the Palestinian National Authority

Seismic hazard in Palestina

Adopted method: Cornell-McGuire method (1968)

- it allows to take into account the uncertainties in location and magnitude of earthquakes, their process of time occurrence and ground motion attenuation
- it assumes exponential distribution of magnitude
- it assumes that earthquakes have “no memory” (*Poisson* occurrence model)

Steps of analysis:



Step 1: definition of earthquake sources characterized by uniform seismic potential

Step 2: definition of earthquake probability distribution (GR) for each source

Step 3: determination of earthquake effects through attenuation relationships (with ε)

Step 4: definition of seismic hazard integrating effects of all earthquakes of different sizes, occurring at different locations in different sources at different probabilities of occurrence

Seismic hazard in Palestina

Adopted method: Cornell-McGuire method (1968)

$$E(z)=\sum_{i=1}^{N}\alpha_i\int_{r=0}^{\infty}\int_{M_{\min}}^{M_{\max_i}} f_{m_i}(m)\,f_{r_i}(r)\,P(Z>z\,|\,m,r)\,dr\,dm$$

where

$E(z)$ = expected number of exceedances of ground motion level "*z*" during a specified time period "*t* "

α_i = mean rate of occurrence of earthquakes between lower/upper bounds magnitude being considered for the "i^{th}" source

$f_{mi}(m)$ = probability density distribution of magnitude within the "i^{th}" source

$f_{ri}(m)$ = probability density distribution of epicentral distance between various locations within source "i^{th}" and the site where hazard is estimated

$P(Z > z\,|m,r)$ = probability that a given earthquake of magnitude "*m*" and epicentral distance "*r*" will exceed ground motion level "*z*"

Seismic hazard in Palestina

Adopted method :

PROBABILISTIC SEISMIC HAZARD ANALYSIS

⇩

Classical Cornell-McGuire approach (1968)

⇩

EARTHQUAKE CATALOGUE AND PROCESSING

⇩

- Seismogenic zones
- Declustering
- Completeness periods
- Gutenberg-Richter recurrence relationships
- Ground motion prediction equations

⇩

PGA HAZARD MAP

Seismic hazard in Palestina

Creation of a unified earthquake catalogue for the period 1900-2004:

Used sources:

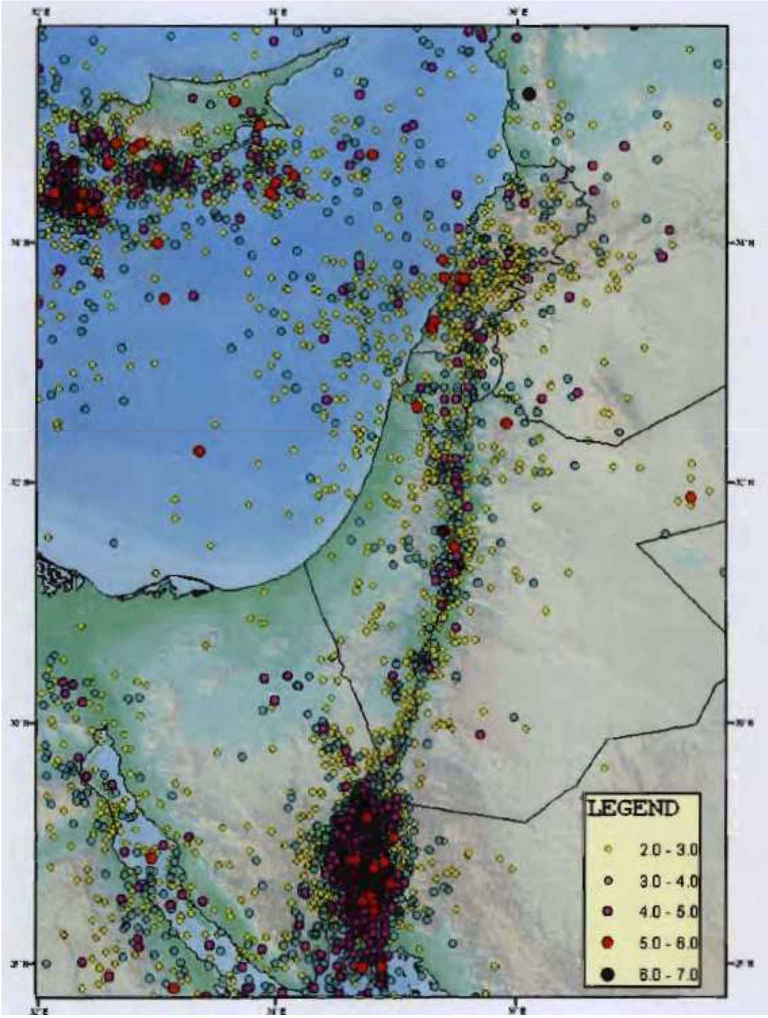
1) Historical earthquake information from different Arabic, Islamic, Jewish and Christian historians who assembled descriptions of earthquakes mentioned in ancient literature
2) Lists of historical earthquakes from revised earthquake catalogs of the region and added after cross checking the quality and the authenticity of the data sources published.
3) Instrumental data available from the beginning of the 20" century owing to the operation of seismic stations in Egypt (HLW), Lebanon (KSR), Israel (JER, EIL) and several tens of stations in Europe.

   Earthquake information for the period 1900-1982 from:

   - International Seismological Summary (ISS), England
   - International Seismological Center (ISC), England
   - National Earthquake Information Service (NEIS), USA
   - Compilation of Arieh et al. (1985).
4) Instrumental earthquake data from :
   - National seismic networks of Jordan and Israel (Since 1982)
   - Israel Seismic Network-ISN (since 1980)
   - Jordanian Seismic Network- JSN (since 1983)

Seismic hazard in Palestina

Regional seismicity:



Seismic hazard in Palestina

Homogenization of magnitude scales

- M_L values as determined by the ISN assigned to most of the events:

$$M_L = 0.7 + 1.54\,log(t) + 0.001R$$

$$M_L = -0.6 + 2\,log(t) + 0.0015R$$ (Modified since 1987)

t: duration
R: epicentral distance

- M_L of the earthquakes recorded by JSN:

$$M_L = 0.7 + 1.54\,log(t) + 0.001R$$

- M_L (Israel) correlation with seismic moment M_0

$$M_L = (0.97 \pm 0.2)\,log(M_0) - (16.9 \pm 0.36)$$

Correlation between the inferred M_L from M_0 estimations, the M_L (ISN) and M_L (JSN), used to unify the local magnitude :

$$\boxed{M_L = 1.01 + 0.66M \quad \text{Jordan} \qquad M_L = 1.11 + 0.61M \quad \text{Israel}}$$

✓ The correlations are valid for the magnitude range 1.0 to 5.0.
✓ For earthquakes which occurred prior to 1956 or for which local seismograms were not available, M_L equal to the given magnitude value (usually M_S) for M~>4.8 is assumed.

Seismic hazard in Palestina

Identification of seismogenic zones

Basis data:

- Studies of seismic zonation developed in the past
- Earthquake catalogue
- Compiled and integrated all relevant, existing geological and geophysical information for the region
- Improved seismic monitoring by the Jordanian and Israeli seismic networks, with the resolution of mapping and kinematical analysis of fault systems and the supporting studies in archaeo-seismology, palaeo-seismology and geodesy.
- Catalogue of young faults in Israel and active faults
- Mapped fault systems along the seismically active boundaries of the Israel-Sinai sub-plate, specifically in the Gulf of Eilat, the Gulf of Suez and in the Roum-Yamune fault system

Seismic zones classification:

A: Measurable seismicity clearly associated with active faults

B: Measurable seismicity associated with mapped geological structures, which have not been defined as active in post Pliocene times

C: Measurable seismicity with no apparent association with known geological structures

D: Active faults and sporadic seismicity with no coherent relation between them

E: Active faults with no recorded seismicity associated with them

Seismic hazard in Palestina

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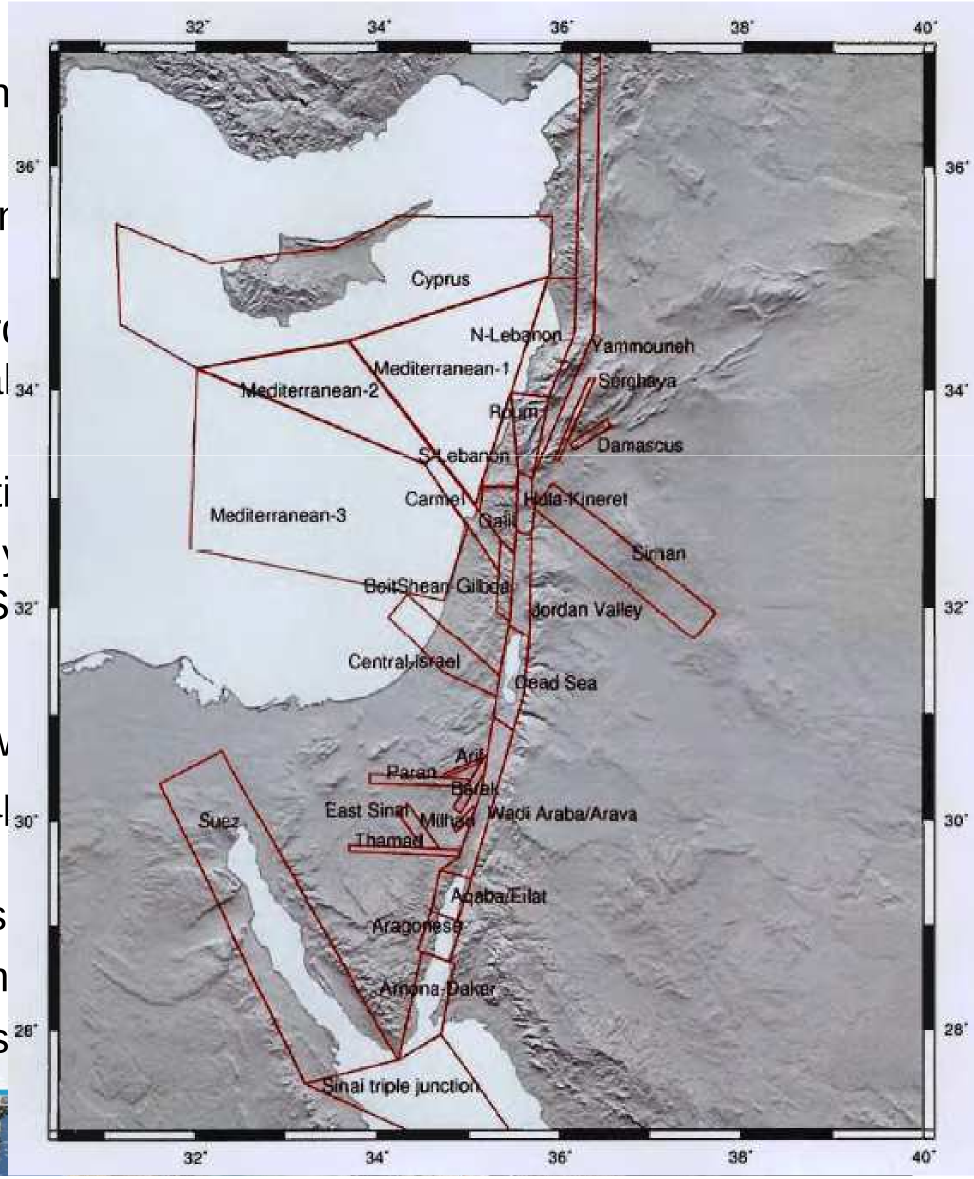

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Seismic hazard in Palestina

Completeness:

Based on the availability of earthquake data and following previous studies information available for the Dead Sea Rift system is complete for the following periods and magnitudes:

		Observed			Suggested	M>4		M>5		M>6	
Zone	km	N(M>2)	Mmax	N/km	Alfa	events/y	years	events/y	years	events/y	years
Arnona	100	25	7.5	0.25	20.00	0.241	4	0.026	38	0.003	358
Aragoneze	56	62	7.5	1.11	11.20	0.135	7	0.015	68	0.002	640
Elat	56	23	7.5	0.41	11.20	0.135	7	0.015	68	0.002	640
Arava	164	11	7.5	0.07	11.00	0.132	8	0.014	69	0.002	651
Dead sea	100	14	7.5	0.14	20.00	0.241	4	0.026	38	0.003	358
Jordan v.	100	15	7.5	0.15	20.00	0.241	4	0.026	38	0.003	358
Hula	60	9	7.5	0.15	12.00	0.144	7	0.016	63	0.002	597
Roum	80	10	7.5	0.13	16.00	0.192	5	0.021	48	0.002	448
Yamuneh	280	9	7.75	0.03	56.00	0.674	1	0.074	14	0.008	126

Estimation of mean annual rates of exceedance: $\lambda_{\bar{M}} = \dfrac{N.events\ M > \bar{M}}{completeness\ \ period}$

Seismic hazard in Palestina

Seismicity of seismogenic zones

The seismicity of each seismogenic zone is quantified by the standard Gutenberg-Richter recurrence relationship [Gutenberg and Richter, 1942], which hypothesizes the existence of an exponential correlation between the mean annual rate of exceedance λ_M of an earthquake with magnitude greater than or equal to M and the magnitude itself:

$$log\left(\lambda_M\right) = a - bM$$

$$\lambda_M = e^{\alpha - \beta M}$$ with $\alpha = 2.303a$ and $\beta = 2.303b$

Standard Gutenberg-Richter relationship predicts nonzero mean rates of exceedance for magnitude up to infinity, therefore a bounded Gutenberg-Richter can be used [Kramer, 1996]:

$$\lambda_M = \nu \cdot \frac{e^{-\beta\cdot(M - M_{min})} - e^{-\beta\cdot(M_{max} - M_{min})}}{1 - e^{-\beta\cdot(M_{max} - M_{min})}}$$

With $M_{min} \leq M \leq M_{max}$

$$\nu = e^{(\alpha - \beta\cdot M_{min})}$$

M_{min}, M_{max} are the lower and upper bound of magnitude M considered, while parameters α and β are the same of the standard Gutenberg-Richter

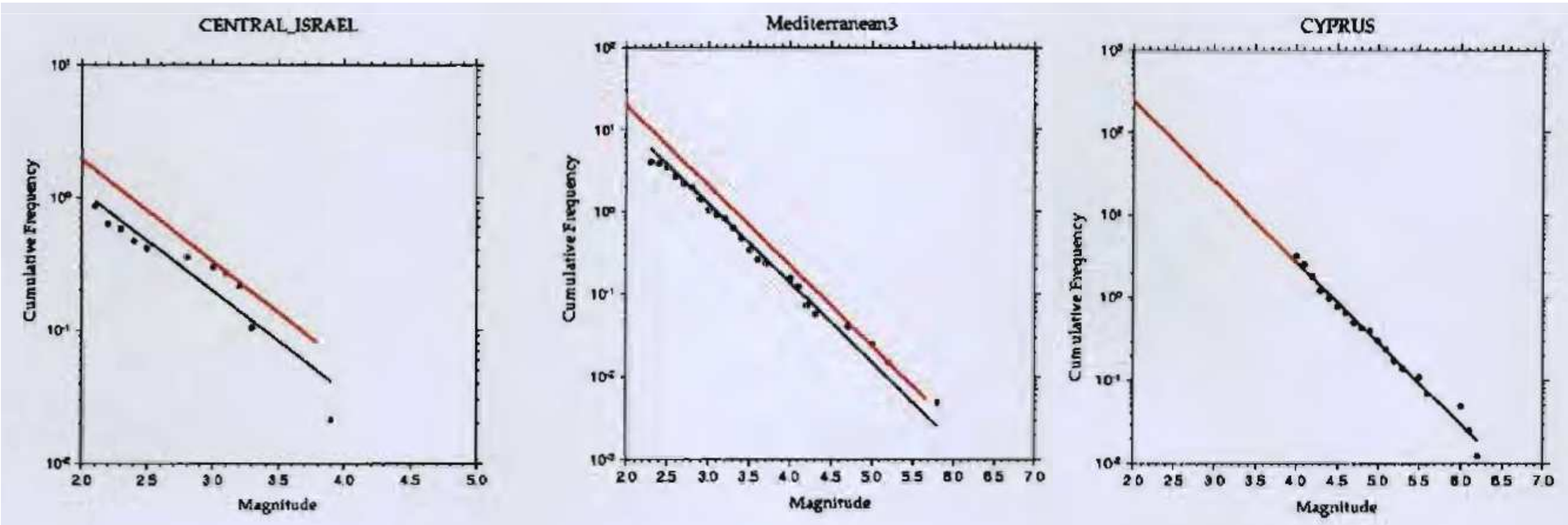
✓ The b value is indicative of the tectonic characteristics of a region

Seismic hazard in Palestina

Seismicity of seismogenic zones

b values:

- Area of the Dead Sea transform system. b =0.96
- Area along fault systems that are branching-off the Dead Sea Rift b =0.96
- Cyprus zone b =1.07
- Gulf of Suez zones b =0.98
- Background seismicity b =0.96



Seismic hazard in Palestina

Seismicity of seismogenic zones

Maximum magnitudes M_{max}:

- Area of the Dead Sea transform system M_{max} = 7.5
- Exception for the Yamouneh fault M_{max} = 7.75

Based on previous assessments

Zone	km	Observed N(M>2)	Mmax	N/km	Suggested Alfa	M>4 events/y	M>4 years	M>5 events/y	M>5 years	M>6 events/y	M>6 years
Arnona	100	25	7.5	0.25	20.00	0.241	4	0.026	38	0.003	358
Aragoneze	56	62	7.5	1.11	11.20	0.135	7	0.015	68	0.002	640
Elat	56	23	7.5	0.41	11.20	0.135	7	0.015	68	0.002	640
Arava	164	11	7.5	0.07	11.00	0.132	8	0.014	69	0.002	651
Dead sea	100	14	7.5	0.14	20.00	0.241	4	0.026	38	0.003	358
Jordan v.	100	15	7.5	0.15	20.00	0.241	4	0.026	38	0.003	358
Hula	60	9	7.5	0.15	12.00	0.144	7	0.016	63	0.002	597
Roum	80	10	7.5	0.13	16.00	0.192	5	0.021	48	0.002	448
Yamuneh	280	9	7.75	0.03	56.00	0.674	1	0.074	14	0.008	126

Average activity rates (N. events of M>2.0 per kilometer): 0.26 (excluding the Arnona, Aragonese, Arava and the Yamuneh seismogenic zones, which require further investigations)

Seismic hazard in Palestina

Seismicity of seismogenic zones

Maximum magnitudes M_{max}:

- Area of the Dead Sea transform system M_{max} = 7.5
- Exception for the Yamouneh fault M_{max} = 7.75
- Faults that are branches of the Dead Sea Rift M_{max} = 5.5, M_{max} = 6

Based mainly on the limited seismic history and partially on the length of the mapped fault

		Observed			Suggested	M>5		M>4	
Zone	km	N(M>2)	Mmax	N/km	alfa	N	Years	N	Year
Carmel	140	9	6.5	0.06	9.00	0.011	87	0.108	9
East Sinai	56	4	6	0.07	2.80	0.003	304	0.033	30
Thamad	108	3.5	6	0.03	5.40	0.006	158	0.064	16
Barak	64	3	5.5	0.05	3.20	0.003	354	0.037	27
Malhan	28	0.33	5.5	0.01	1.40	0.001	809	0.016	62
Arif	52	1.2	5.5	0.02	2.60	0.002	435	0.030	33
Paran	120	1.1	6	0.01	6.00	0.007	142	0.071	14
	280	11.7		0.04					

Average activity rates (N. events of M>2.0 per kilometer): 0.05

Seismic hazard in Palestina

Seismicity of seismogenic zones

Maximum magnitudes M_{max}:

- Area of the Dead Sea transform system M_{max} = 7.5
- Exception for the Yamouneh fault M_{max} = 7.75
- Faults that are branches of the Dead Sea Rift M_{max} = 5.5, M_{max} = 6
- Carmel fault M_{max} = 6.5
- Background seismicity

Based on seismicity record

Zone	KmSqr	Observed N(M>2)	Mmax	N/Km^2	Suggested Alfa	M>5 N	M>5 Years	M>4 N	M>4 Years
Suez	31774	65	7	2.05	65.00	0.085	12	0.78	1.28
Cyprus	40863	110	8	2.69	110.00	0.307	3	2.18	0.46
Bet She'an	1065	9	6.5	8.45	9.00	0.011	87	0.11	9.27
E. Med.	73799	38	6.5	0.51	36.90	0.047	21	0.44	2.26
Central Isr.	4093	2	5.5	0.49	2.05	0.002	553	0.02	42.12
N. Jordan	19404	5.2	5.5	0.27	9.70	0.009	117	0.11	8.88
Palmira	22164	11	6	0.50	11.08	0.013	77	0.13	7.59
W. Sirhan	28507	17	6	0.60	14.25	0.017	60	0.17	5.90
Galil	1930	1.1	5.5	0.57	0.96	0.001	1173	0.01	89.33

Seismic hazard in Palestina

Seismicity of seismogenic zones

Adopted seismic parameters for each zone:

$\beta = 2.303b$

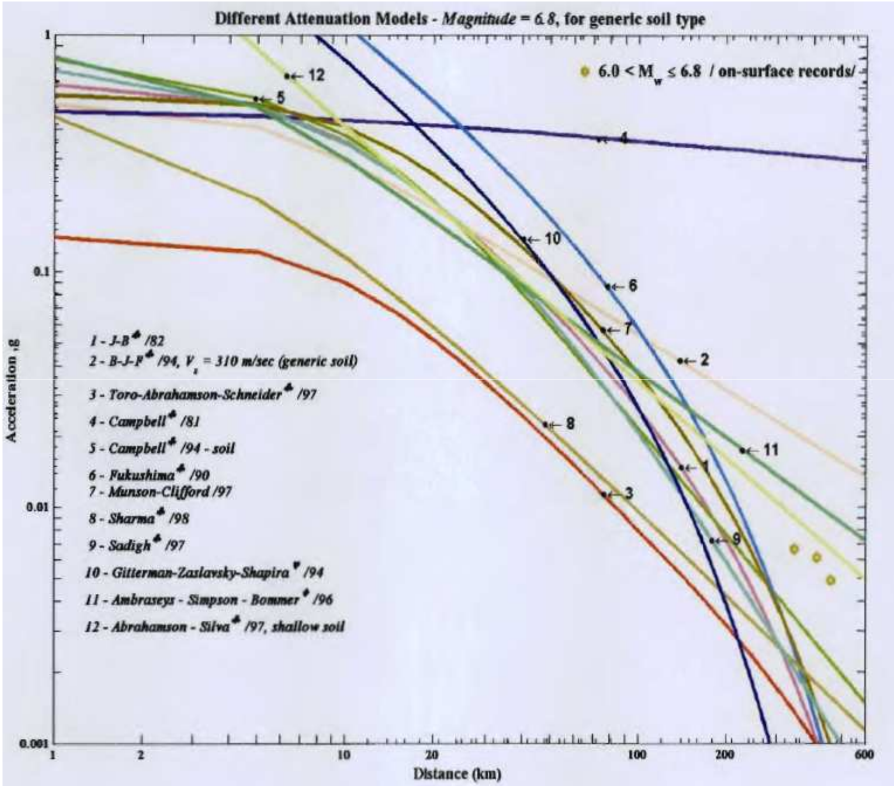
No	Source	Mmin	Mmax	beta	alfa
1	Aragonese	4	7.5	2.21	0.1925
2	Wadi-Araba/Arava	4	7.5	2.21	0.3007
3	Arif	4	5.5	2.21	0.0302
4	Arnona-Dakar	4	7.5	2.21	0.5654
5	Barak	4	5.5	2.21	0.0371
6	BeitShean-Gilboa	4	6.5	2.21	0.0599
7	Carmel	4	6.5	2.21	0.1199
8	Central-Israel	4	5.5	2.21	0.0232
9	Cyprus	4	8.0	2.25	2.7769
10	Dead-Sea	4	7.5	2.21	0.2887
11	Aqaba/Eilat	4	7.5	2.21	0.1925
12	Galilee	4	5.5	2.21	0.0348
13	Hula-Kineret	4	7.5	2.21	0.2526
14	Jordan-Valley	4	7.5	2.21	0.3729
15	Malhan	4	5.5	2.21	0.0162
16	Mediterranean-1	4	6.5	2.21	0.3956
17	Mediterranean-2	4	6.5	2.21	0.2277
18	Mediterranean-3	4	6.5	2.21	0.2158
19	Paran	4	6.0	2.21	0.0238
20	Roum	4	7.5	2.21	0.2887
21	Yammouneh	4	8.0	2.21	0.9144
22	Suez	4	7.0	2.46	2.0425
23	East-Sinai	4	6.0	2.21	0.0333
24	Thamad	4	6.0	2.21	0.0642
25	N-Lebanon	4	5.5	2.21	0.0903
26	S-Lebanon	4	6.5	2.21	0.0364
27	Sirhan	4	7.0	1.63	0.0500
28	Damascus	4	5.0	2.21	0.0641
29	Sergayha	4	7.5	2.21	0.0820
30	Sinai-T.J.	4	7.5	2.21	2.2726

α= N. of events/year ($M \geq M_{min}$)

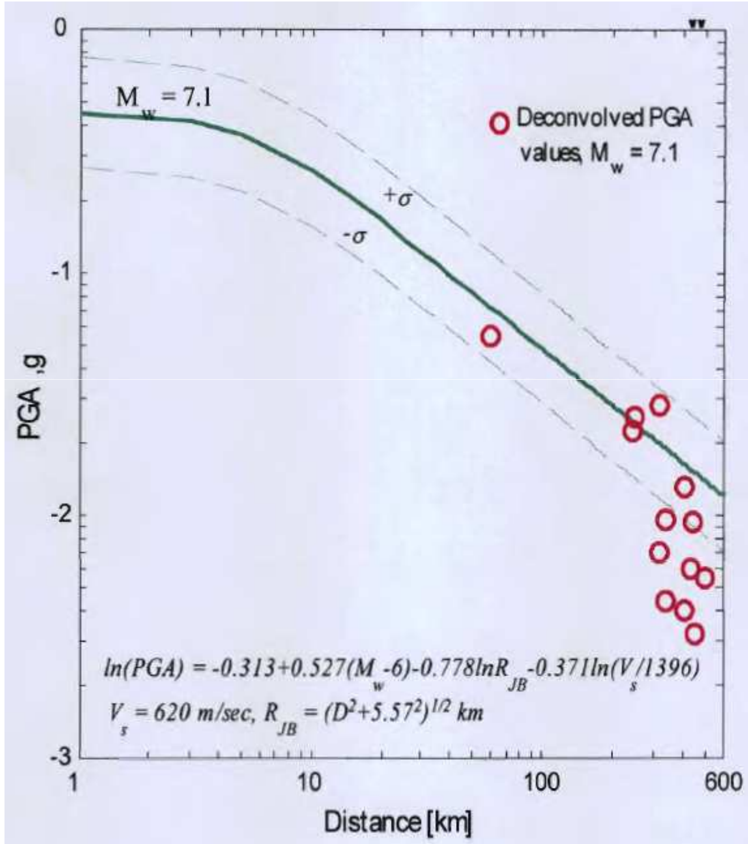
Seismic hazard in Palestina

Horizontal Peak Ground Acceleration Attenuation Relationship (GMPEs)

Considered GMPEs

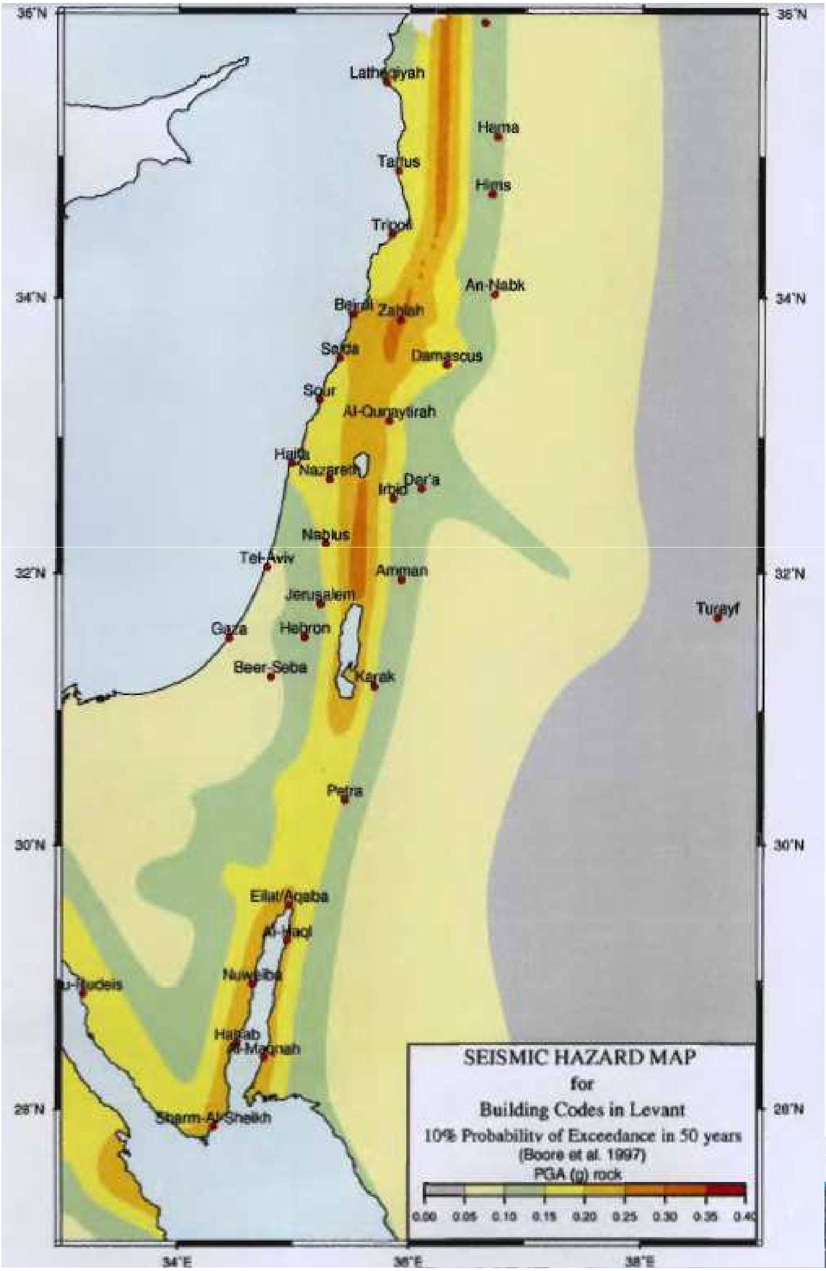


Adopted GMPE: Boore et al. (1997)



Following the comparison with recorded data, Boore et al. (1997) attenuation function for strike slip faults and for Vs=620 m/s ("Generic Rock") is adopted implemented into the Probabilistic Earthquake Hazard Analysis

Seismic hazard in Palestina



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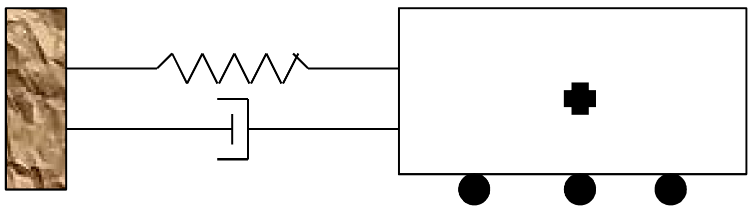
[Seismic hazard map:](#)

Horizontal Peak Ground Accelerations (***PGA***) with a probability of 10% of being exceeded at least once within an exposure time of 50 years.

PGA values are computed for rock site conditions.

Dynamics of SDOF System



Background

Fundamental equation of dynamic, assumption of: (i) non dissipative system; (ii) fixed boundary conditions

$$\frac{d}{dt}\frac{\partial L}{\partial \dot{q}_k} - \frac{\partial L}{\partial q_k} = Q_k \qquad k = 1...n$$

with:

n number of degree of freedom of the system

q free coordinates of the system

Q external forces

L Lagrange function L=T-V with T kinetic energy and V potential energy

$$T = \frac{1}{2}\sum_{k=1}^{n}\sum_{j=1}^{n} m_{jk}\dot{q}_j\dot{q}_k = \frac{1}{2}\dot{\mathbf{q}}^t\mathbf{M}\dot{\mathbf{q}} \qquad V = \frac{1}{2}\sum_{k=1}^{n}\sum_{j=1}^{n} k_{jk}q_j q_k = \frac{1}{2}\mathbf{q}^t\mathbf{K}\mathbf{q}$$

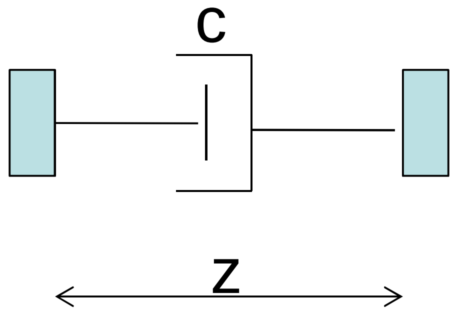
With **M** and **K** mass and stiffness matrixes that in the field of small displacements have constant components, are symmetric and positive defined

Hence the fundamental equation of dynamic become:

$$\frac{d}{dt}\frac{\partial T}{\partial \dot{q}_k} + \frac{\partial V}{\partial q_k} = Q_k \qquad k = 1...n \qquad \mathbf{M}\ddot{\mathbf{q}} + \mathbf{K}\mathbf{q} = \mathbf{Q}$$

Dissipative system:

To account for the dissipation of energy, the idea is to add to the system a viscous damper



Lagrange equations:

$$\underbrace{\frac{d}{dt}\frac{\partial L}{\partial \dot{q}_k}-\frac{\partial L}{\partial q_k}}_{\text{conservative world}}=\underbrace{Q_k+Q_{d_K}}_{\text{non conservative world}}\qquad k=1\ldots n$$

dissipative force

Releygh function:

$$D = \frac{1}{2}\dot{\mathbf{q}}^t \mathbf{C} \dot{\mathbf{q}}$$

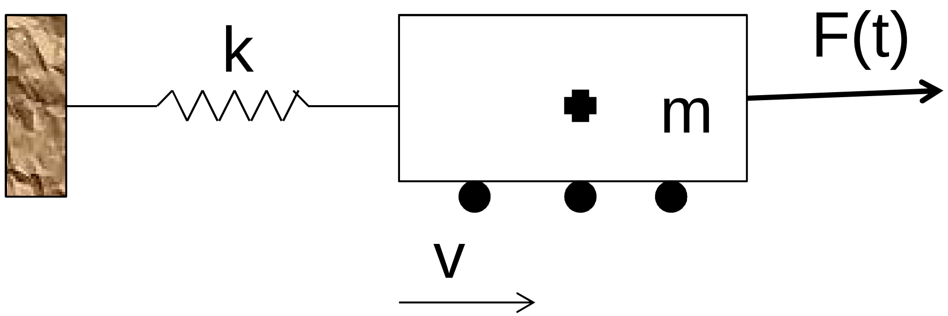
With C damping matrix

$$\frac{\partial D}{\partial \dot{q}_k} = -Q_{d_k}$$

Hence the Lagragian equation become:

$$\frac{d}{dt}\frac{\partial T}{\partial \dot{q}_k} + \frac{\partial D}{\partial \dot{q}_k} + \frac{\partial V}{\partial q_k} = Q_k \qquad k = 1 \ldots n \qquad \mathbf{M}\ddot{\mathbf{q}} + \mathbf{C}\dot{\mathbf{q}} + \mathbf{K}\mathbf{q} = \mathbf{Q}$$

Non dissipative SDOF system



Equation of motion:

$$T = \frac{1}{2} m \dot{v}^2 \; ; \qquad V = \frac{1}{2} k v^2$$

$$m\ddot{v} + kv = F(t)$$

Elastic force

Inertial force

Definitions

$$\omega_1 = \sqrt{\frac{k}{m}}$$ angular frequency/puls rad/sec

$$f_1 = \frac{\omega_1}{2\pi}$$ natural frequency of vibration Hz

$$T_1 = \frac{1}{f_1}$$ natural period of vibration sec

<u>Free vibrations:</u>

$$m\ddot{v} + kv = 0 \qquad \ddot{v} + \omega_1^2 v = 0$$

The solution could be in the form:

$$v(t) = A\cos(\omega_1 t) + B\sin(\omega_1 t)$$

With A and B defined through the initial conditions:

$$v(0) = A; \quad \dot{v}(0) = B\omega_1$$

<u>Vibrations due to an harmonic applied force:</u>

$$m\ddot{v} + kv = F\sin(\omega t) \qquad \ddot{v} + \omega_1^2 v = \frac{F}{m}\sin(\omega t) = \frac{F}{k}\omega_1^2 \sin(\omega t)$$

The solution could be in the form:

$$v(t) = \underbrace{A\cos(\omega_1 t) + B\sin(\omega_1 t)}_{\text{solution of associated homogeneous eq./free vibrations}} + \underbrace{v_p(t)}_{\text{response due to applied harmonic force}}$$

Assumption:

$$v_p(t) = V\sin(\omega t + \xi)$$
$$\dot{v}_p(t) = \omega V\cos(\omega t + \xi)$$
$$\ddot{v}_p(t) = -\omega^2 V\sin(\omega t + \xi)$$

Does the assumption satisfy the equation of motion?

$$-\omega^2 V\sin(\omega t + \xi) + \omega_1^2 V\sin(\omega t + \xi) = \frac{F}{k}\omega_1^2\sin(\omega t)$$

If $\xi=0$

$$-\omega^2 V\sin(\omega t) + \omega_1^2 V\sin(\omega t) = \frac{F}{k}\omega_1^2\sin(\omega t) \quad \rightarrow \quad V = \frac{F}{k}\frac{\omega_1^2}{\omega_1^2 - \omega^2} = \frac{F}{k}\frac{1}{1-\beta^2}; \beta = \frac{\omega^2}{\omega_1^2}$$

If $\xi=\pi$

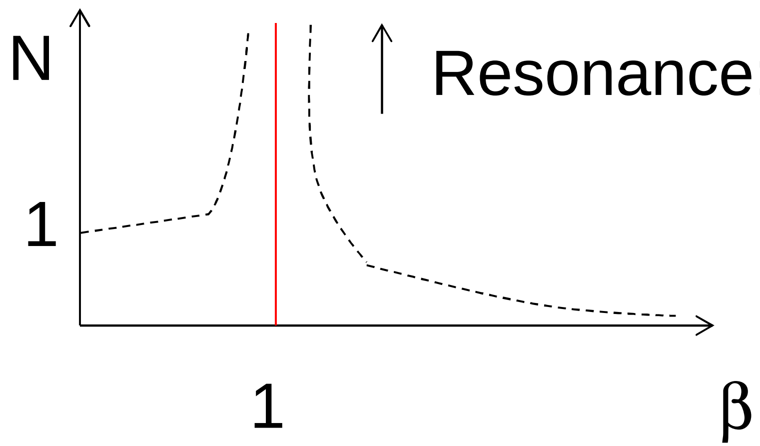
$$\omega^2 V\sin(\omega t) - \omega_1^2 V\sin(\omega t) = \frac{F}{k}\omega_1^2\sin(\omega t) \quad \rightarrow \quad V = -\frac{F}{k}\frac{\omega_1^2}{\omega_1^2 - \omega^2} = -\frac{F}{k}\frac{1}{1-\beta^2}$$

Hence:

$$\text{if } \beta < 1 \text{ (i.e. } \xi = 0) \rightarrow \text{response in phase}$$
$$\text{if } \beta > 1 \text{ (i.e. } \xi = \pi) \rightarrow \text{response in opposition of phase}$$
$$\text{if } \beta = 1 \qquad\qquad\qquad \rightarrow \text{resonance}$$

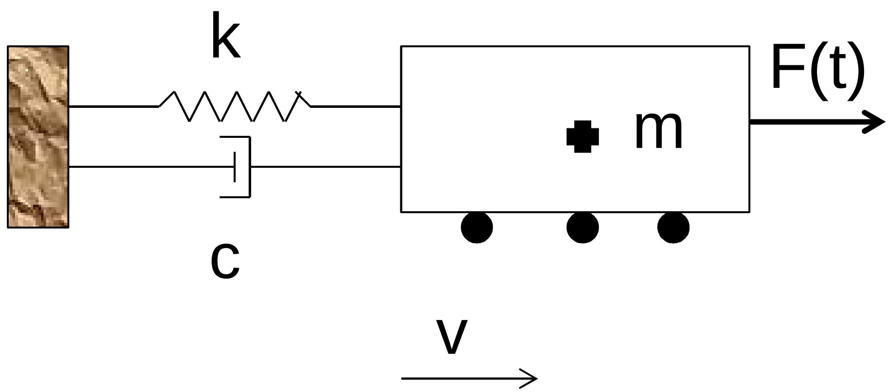
If we define the amplification factor like:

$$N = \frac{1}{\left|1-\beta^2\right|}$$



Resonance: N→ infinite

Dissipative SDOF system



Equation of motion:

$$T = \frac{1}{2} m\dot{v}^2 \; ; \qquad V = \frac{1}{2} k v^2 ; \qquad D = \frac{1}{2} c\dot{v}^2$$

$$m\ddot{v} + c\dot{v} + kv = F(t)$$

Elastic force

Inertial force

Dissipative force

Definition

$$\nu = \frac{c}{2\sqrt{km}}$$ damping coefficient

<u>Free vibrations:</u>

$$m\ddot{v} + c\dot{v} + kv = 0 \qquad \ddot{v} + 2\nu\omega_1\dot{v} + \omega_1^2 v = 0$$

Associated characteristic equation

$$\lambda^2 + 2\nu\omega_1\lambda + \omega_1^2 = 0$$

Solutions:

$$\lambda = -\nu\omega_1 \pm \omega_1\sqrt{\nu^2 - 1}$$

if $\nu > 1 \rightarrow$ solutions real and different between them

if $\nu = 1 \rightarrow$ solutions real and coincident

if $\nu < 1 \rightarrow$ solutions complex and coniugated

In earthquake engineering we always deal with systems that have ν<<1!

Assumptions:

$$\omega_d = \omega_1\sqrt{\nu^2 - 1} \quad \rightarrow \quad \lambda = -\nu\omega_1 \pm i\omega_d$$

Hence the solution is:

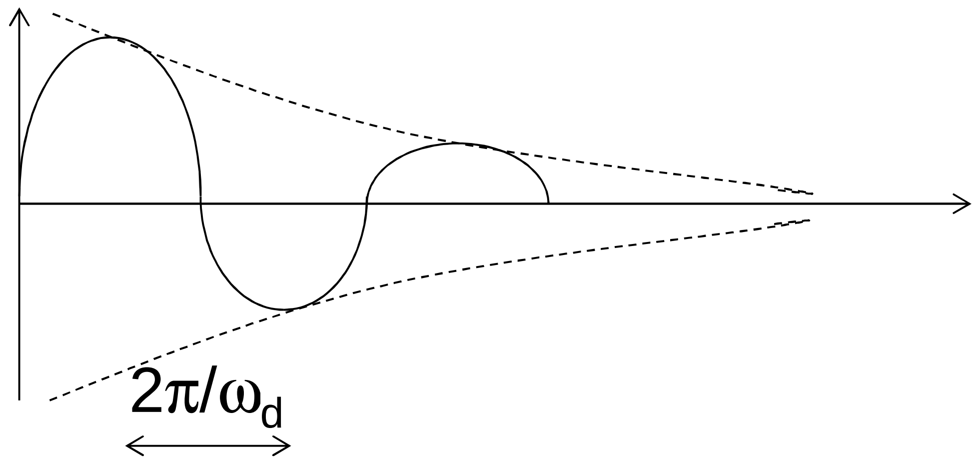
$$v(t) = Ae^{[(-\nu\omega_1 + i\omega_d)t]} + Be^{[(-\nu\omega_1 - i\omega_d)t]}$$

Since we do not like to deal with complex numbers, we use the rule that combinations of solutions is still a solution. Therefore:

$$v(t) = e^{(-\nu\omega_1)t}\left[C\cos(\omega_d t) + D\sin(\omega_d t)\right]$$

With C and D defined through the initial conditions:

Shape of the solution:



The amplitude of free vibration decreases exponentially and the distance between the peaks is constant and equal to $2\pi/\omega_d$

→ The influence of free vibration after a short interval of time is attenuated in dissipative system, also if the dissipation is small (ν<<1)!

Vibrations due to an harmonic applied force:

$$\text{if } F(t) = \sin(\omega t) \quad \rightarrow \quad \ddot{v} + 2\nu\omega_1\dot{v} + \omega_1^2 v = \frac{F}{k}\omega_1^2 \sin(\omega t)$$

$$\text{if } F(t) = \cos(\omega t) \quad \rightarrow \quad \ddot{s} + 2\nu\omega_1\dot{s} + \omega_1^2 s = \frac{F}{k}\omega_1^2 \cos(\omega t)$$

If we consider z=s+iv, the equation to study become:

$$\ddot{z} + 2\nu\omega_1\dot{z} + \omega_1^2 z = \frac{F}{k}\omega_1^2 e^{-i\omega t}$$

Where:

(i) the real part of z is the response of the system to cosen excitation

(ii) the imaginary part of z is the response of the system to sin excitation

Assumption:

$$z_p(t) = F\widetilde{H}(\omega)e^{i\omega t}$$
$$\dot{z}_p(t) = i\omega F\widetilde{H}(\omega)e^{i\omega t}$$
$$\ddot{z}_p(t) = -\omega^2 F\widetilde{H}(\omega)e^{i\omega t}$$

Does the assumption satisfy the equation of motion?

$$-\omega^2 F\widetilde{H}(\omega)e^{i\omega t} + 2i\nu\omega_1\omega F\widetilde{H}(\omega)e^{i\omega t} + \omega_1^2 F\widetilde{H}(\omega)e^{i\omega t} = \frac{F}{k}\omega_1^2 e^{i\omega t}$$

$$\downarrow$$

$$\widetilde{H}(\omega) = \frac{1}{k}\frac{\omega_1^2}{\omega_1^2 - \omega^2 + 2i\nu\omega_1\omega} = \frac{1}{k}\frac{1}{1-\beta^2+2i\nu\beta};$$ $\widetilde{H}(\omega)$ is defined as the transfer function

$$\left|\widetilde{H}(\omega)\right| = N(\omega) = \frac{1}{k}\frac{1}{\sqrt{(1-\beta^2)^2 + 4\nu^2\beta^2}};$$ $N(\omega)$ is the amplification factor

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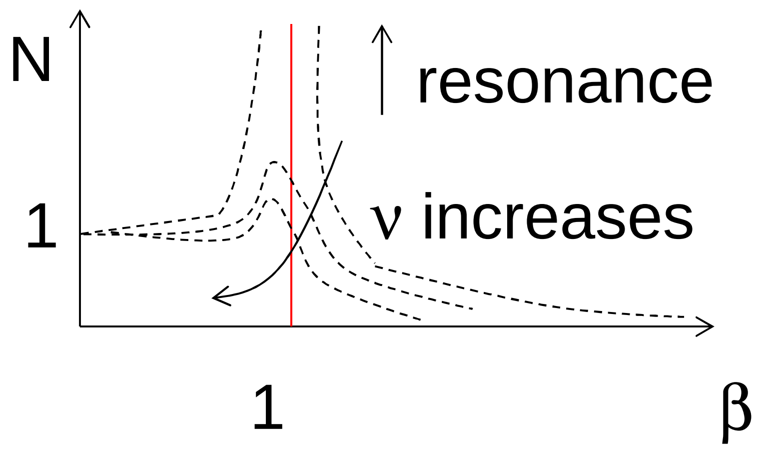


For damped system, also at resonance (applied frequency correspondent to natural vibration frequency) the amplification does not go to infinite

Generic applied force:

$$\ddot{v} + 2\nu\omega_1\dot{v} + \omega_1^2 v = \frac{F}{m}(t)$$

Solutions of homogenic associated equation:

$$v_1(t) = e^{-\nu\omega_1 t}\cos(\omega_d t)$$
$$v_2(t) = e^{-\nu\omega_1 t}\sin(\omega_d t)$$

Assumption 1:

Instead of multiplying v_1 and v_2 by constant values, we multiply them by function of time:

$$v_p(t) = r(t)v_1(t) + s(t)v_2(t)$$

$$\dot{v}_p(t) = \cancel{\dot{r}(t)v_1(t) + \dot{s}(t)v_2(t)} + r(t)\dot{v}_1(t) + s(t)\dot{v}_2(t)$$

Assumption 2 :

$$\dot{r}(t)v_1(t) + \dot{s}(t)v_2(t) = 0$$

$$\dot{v}_p(t) = r(t)\dot{v}_1(t) + s(t)\dot{v}_2(t)$$

$$\ddot{v}_p(t) = \dot{r}(t)\dot{v}_1(t) + \dot{s}(t)\dot{v}_2(t) + r(t)\ddot{v}_1(t) + s(t)\ddot{v}_2(t)$$

Substituting in the equation of motion:

$$r\underbrace{\left(\ddot{v}_1 + 2\nu\omega_1\dot{v}_1 + \omega_1^2 v_1\right)} + s\underbrace{\left(\ddot{v}_2 + 2\nu\omega_1\dot{v}_2 + \omega_1^2 v_2\right)} + \dot{r}\dot{v}_1 + \dot{s}\dot{v}_2 = \frac{F}{m}(t)$$

= 0 because solution of homogeneous equation associated

$$\begin{cases} \dot{r}\dot{v}_1 + \dot{s}\dot{v}_2 = \dfrac{F}{m}(t) \\ \dot{r}v_1 + \dot{s}v_2 = 0 \end{cases}$$

Solving the system:

$$\dot{r}(t) = -\frac{1}{m\omega_d} e^{-\nu\omega_1 t} \sin(\omega_d t)\, F(t)$$

$$\dot{s}(t) = \frac{1}{m\omega_d} e^{-\nu\omega_1 t} \cos(\omega_d t)\, F(t)$$

We want that r and s satisfy the initial condition:

$$v_p(0) = 0; \;\; \dot{v}_p(0) = 0$$

$$v_p = e^{-\nu\omega_1 t}\cos(\omega_d t)\int_0^t -\frac{1}{m\omega_d} e^{-\nu\omega_1\tau}\sin(\omega_d\tau)F(\tau)d\tau +$$

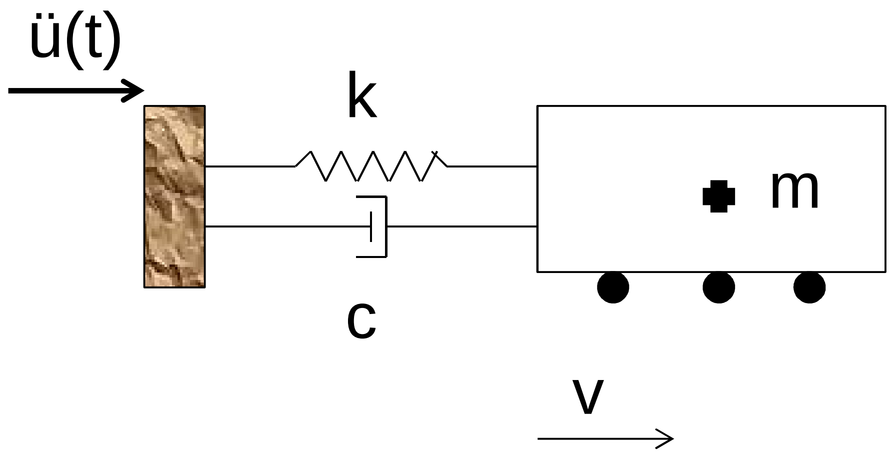
$$e^{-\nu\omega_1 t}\sin(\omega_d t)\int_0^t \frac{1}{m\omega_d} e^{-\nu\omega_1\tau}\cos(\omega_d\tau)F(\tau)d\tau$$

$$\downarrow$$

$$\boxed{v_p = \frac{1}{m\omega_d}\int_0^t e^{-\nu\omega_1(t-\tau)}\sin[\omega_d(t-\tau)]F(\tau)d\tau}$$

This integral is known as Duhamel integral or convolution integral.
By solving numerically the convolution integral, the response of the system to a generic excitation can be found.

Case of SDOF system subjected to seismic exitation



Equation of motion:

$$m\ddot{v} + c\dot{v} + kv = -m\ddot{u} \qquad \ddot{v} + 2\nu\omega_1\dot{v} + \omega_1^2 v = \ddot{u}$$

The peak of relative displacement, relative velocity and absolute acceleration of and SOF system with period of vibration T_1, is the displacement, velocity and acceleration spectral ordinate of ü(t) in $T=T_1$

Elastic response spectrum

Elastic response spectrum

The simplest model used in the seismic assessment of structures is the ***single degree of freedom (SDOF) oscillator***

Equation of motion $\left(F = m\ddot{x}\right)$:

$$-ky - c\dot{y} = m\ddot{x}$$

m: Oscillator mass

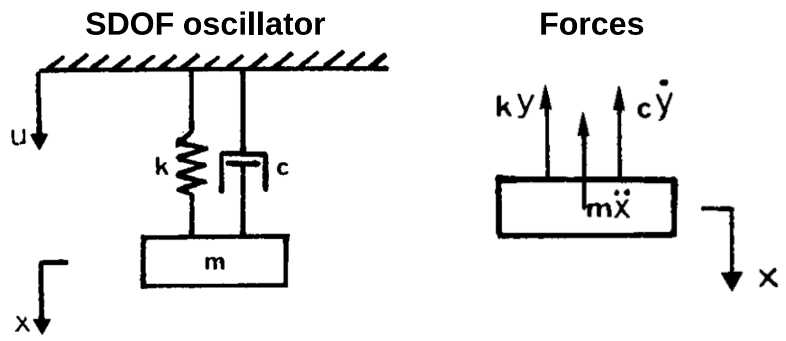
x: Mass absolute displacement

u: Ground absolute displacement

y: x-u Relative displacement

k: Recalling constant of the elastic suspension

c: Viscous constant of the damping



$$\Rightarrow \quad m\ddot{x} + c\dot{y} + ky = -m\ddot{u}$$

Introducing:

$$\omega_n = \sqrt{k/m}$$ Natural vibration of the system

$$\zeta = \frac{c}{2m\omega_n} = \frac{c}{c_{cr}}$$ Damping coefficient with respect to the critic one

$$\Rightarrow \quad \boxed{\ddot{y} + 2\zeta\omega_n\dot{y} + \omega_n^2 y = -\ddot{u}}$$

Elastic response spectrum

Oscillator motion:

For damping less than the critic one $\left(\zeta < 1\right)$

$$y(t) = e^{-\zeta\omega_n t}\left[y(0)\cos\omega_d t + \frac{\dot{y}(0)+\zeta\omega_n y(0)}{\omega_d} sen\omega_d t \right] - \frac{1}{\omega_d}\int_0^t \ddot{u}(\tau) e^{-\zeta\omega_n(t-\tau)} sen\omega_d (t-\tau)\, d\tau$$

With: $\omega_d = \omega_n\sqrt{1-\zeta^2}$ Damping vibration of the oscillator

Assuming starting conditions:

$$y(0)=0, \quad \dot{y}(0)=0$$

RELATIVE DISPLACEMENT

$$\Rightarrow \boxed{y(t) = \underbrace{\frac{1}{\omega_d}\int_0^t \ddot{u}(\tau) e^{-\zeta\omega_n(t-\tau)} sen\omega_d (t-\tau)\, d\tau}_{\text{Duhamel integral}}}$$

For structures design the maximum response value is of interest, thus, for a given accelerogram, curves of the maximum responses of the oscillator varying ζ and ω_n can be developed:

RESPONSE SPECTRA

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Elastic response spectrum

- Response spectra are developed as function of the oscillator natural period T_n, for a given ζ
- Maximum response can be calculated in terms of relative displacement, velocity or absolute acceleration:

RELATIVE VELOCITY

$$\boxed{\dot{y}(t) = -\int_0^t \ddot{u}(\tau) e^{-\zeta\omega_n(t-\tau)} \cos\omega_d (t-\tau) d\tau - \zeta\omega_n y(t)}$$

ABSOLUTE ACCELERATION

$$\boxed{\ddot{x}(t) = -\omega_n^2 y(t) - 2\zeta\omega_n \dot{y}(t)}$$

RELATIVE DISPLACEMENT RESPONSE SPECTRUM

$$\boxed{D(T_n,\zeta) = \max_t \left|y(t)\right|}$$

RELATIVE VELOCITY RESPONSE SPECTRUM

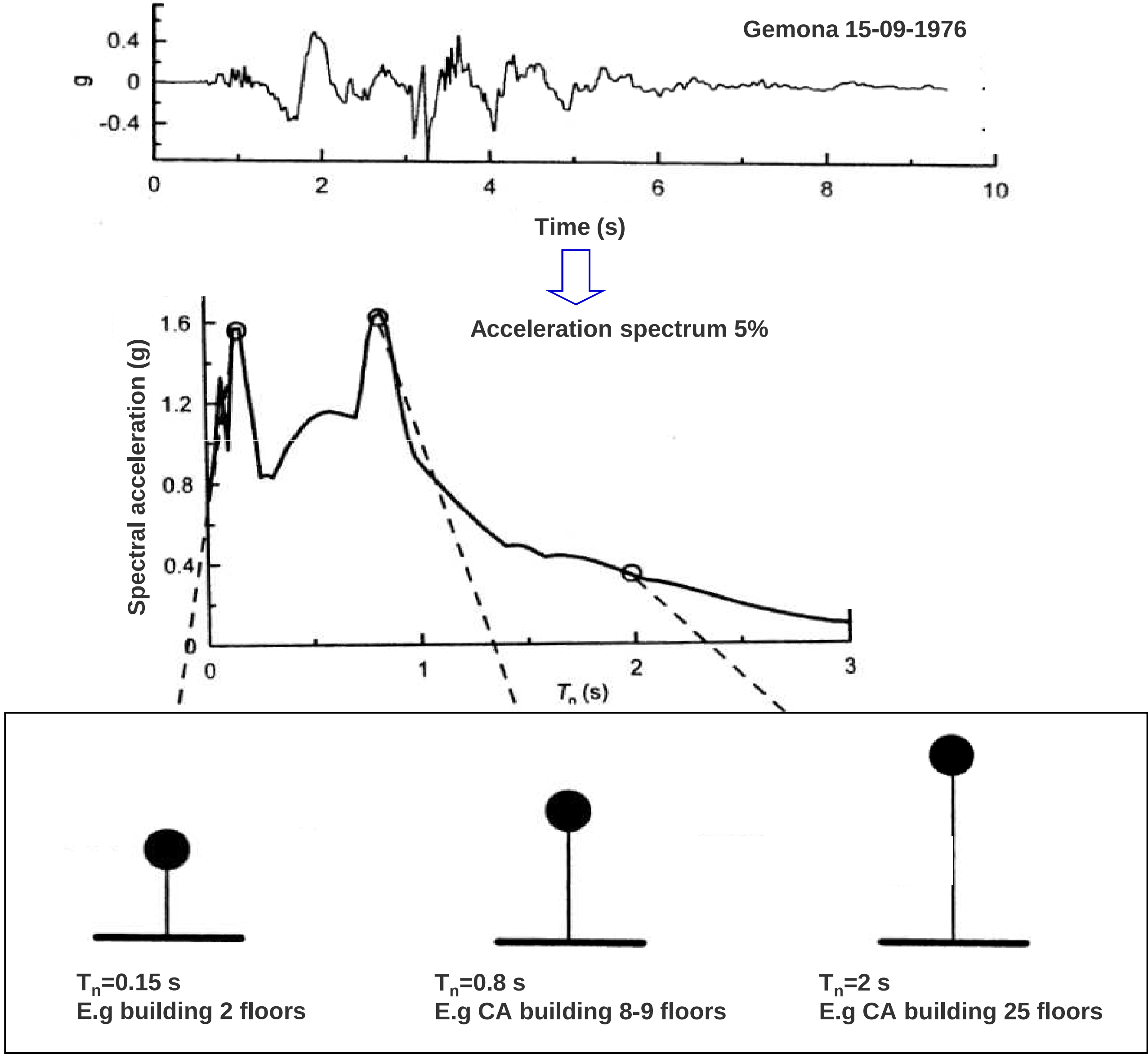
$$\boxed{V(T_n,\zeta) = \max_t \left|\dot{y}(t)\right|}$$

ABSOLUTE ACCELERATION RESPONSE SPECTRUM

$$\boxed{A(T_n,\zeta) = \max_t \left|\ddot{y}(t)\right|}$$

Elastic response spectrum

Construction of the acceleration response spectrum:



(from Faccioli, 2005)

Elastic response spectrum

Approximation neglecting ζ:

Equation of motion: $\ddot{y}+\omega_n^2 y=-\ddot{u} \quad \Rightarrow \quad \omega_n^2 y=-\ddot{x}$

$$A(T_n)=\omega_n^2 D(T_n)=\left(\frac{2\pi}{T_n}\right)^2 D(T_n)$$

PSEUDO ACCELERATION RESPONSE SPECTRUM

$$\boxed{PSA(T_n,\zeta)=\left(\frac{2\pi}{T_n}\right)^2 D(T_n,\zeta)}$$

PSEUDO VELOCITY RESPONSE SPECTRUM

$$\boxed{PSV(T_n,\zeta)=\left(\frac{2\pi}{T_n}\right) D(T_n,\zeta)}$$

For common engineering values of T_n and ζ: $PSV \cong V,\ PSA \cong A$

Elastic response spectrum

Notes:

- For too **flexible structures**: $T_n \to \infty$ $\Rightarrow$ Mass is not affected by soil displacement

$$|y(t)| \cong |u(t)|, \quad |\dot{y}(t)| \cong |\dot{u}(t)|, \quad |\ddot{x}(t)| = 0$$

$$\lim_{t\to\infty} D = |s_{max}|, \quad \lim_{t\to\infty} V = |v_{max}|, \quad \lim_{t\to\infty} A = 0$$

Soil max displacement and velocity

$PSV = 0$ $\Rightarrow$ Approximation not valid for flexible structures

- For too **rigid structures**: $T_n \to 0$ $\Rightarrow$ No mass-soil relative displacement

$$|y(t)| = |\dot{y}(t)| \cong 0, \quad |\ddot{x}(t)| \cong |\ddot{u}(t)|$$

$$D = V = 0, \quad A = |a_{max}|$$

Soil max acceleration

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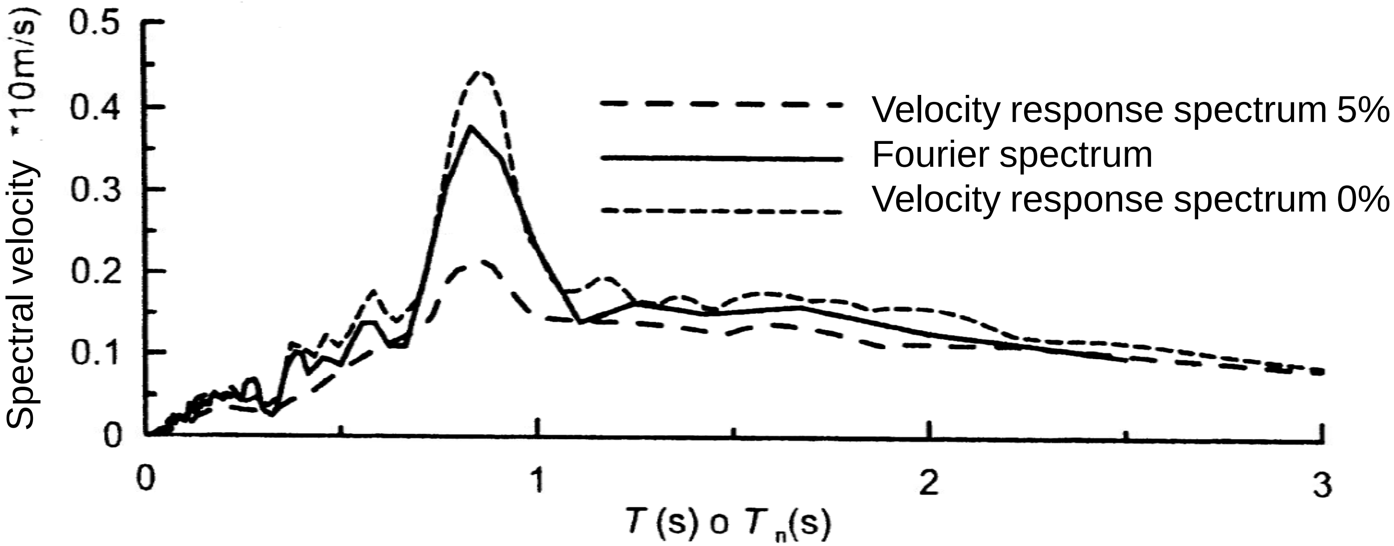
Elastic response spectrum

Notes:

- Velocity response spectrum with no damping represents the upper limit of the Fourier acceleration spectrum, thus these two spectra approximately show the peaks at the same frequencies. The meaning of the abscissa is different: oscillator natural period T_n, in the first case, period T of the single harmonic of the signal in the second one

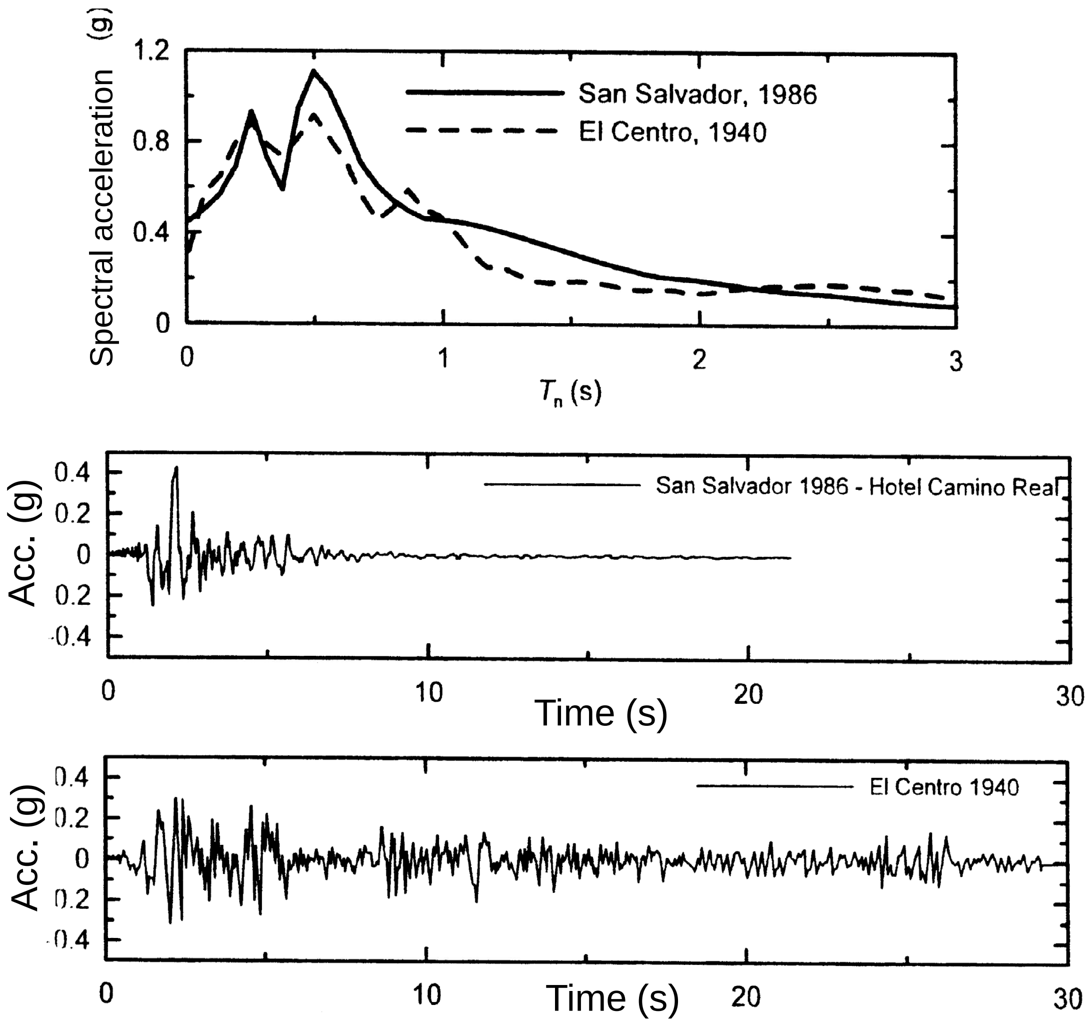


(From Faccioli, 2005)

Elastic response spectrum

Notes:

- The elastic response spectrum is not directly related to the duration of the signal; accelerograms with different duration can be characterized by similar response spectra



(modified From Faccioli, 2005)

Elastic response spectrum

D, PSV, PSA Three-log representation

$$\log PSA = 2\log\left(\frac{2\pi}{T_n}\right) + \log D$$

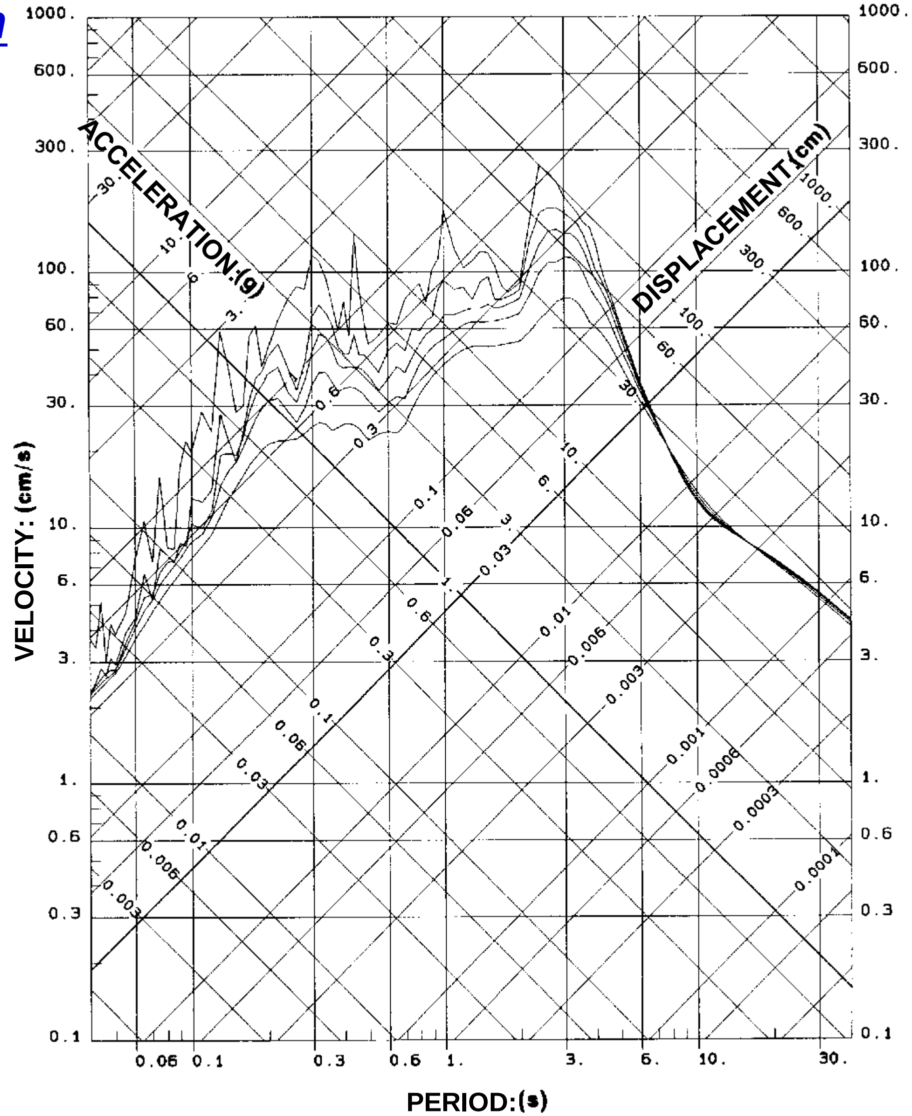
$$\log PSV = \log\left(\frac{2\pi}{T_n}\right) + \log D$$

$$\Downarrow$$

$$\log PSV = \log\left(PSA\right) + \log\frac{T_n}{2\pi}$$

$$\log PSV = \log D - \log\frac{T_n}{2\pi}$$

$$\Downarrow$$



- Lines with inclination +1 are points location with constant acceleration

- Lines with inclination -1 are points location with constant displacement

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Elastic response spectrum

Design elastic response spectra:

In the practice, especially for normative aims, envelope spectra from statistical analyses are used

Eurocode 8 (EC8):

- Earthquake motion is represented by an elastic acceleration response spectrum
- Two shapes are prescribed for two levels of seismic action:
  - Type 1: no collapse
  - Type 2: damage limitation requirements
- Two horizontal components are assumed independent and represented by the same response spectrum; a different form is prescribed for the vertical component

Elastic response spectrum

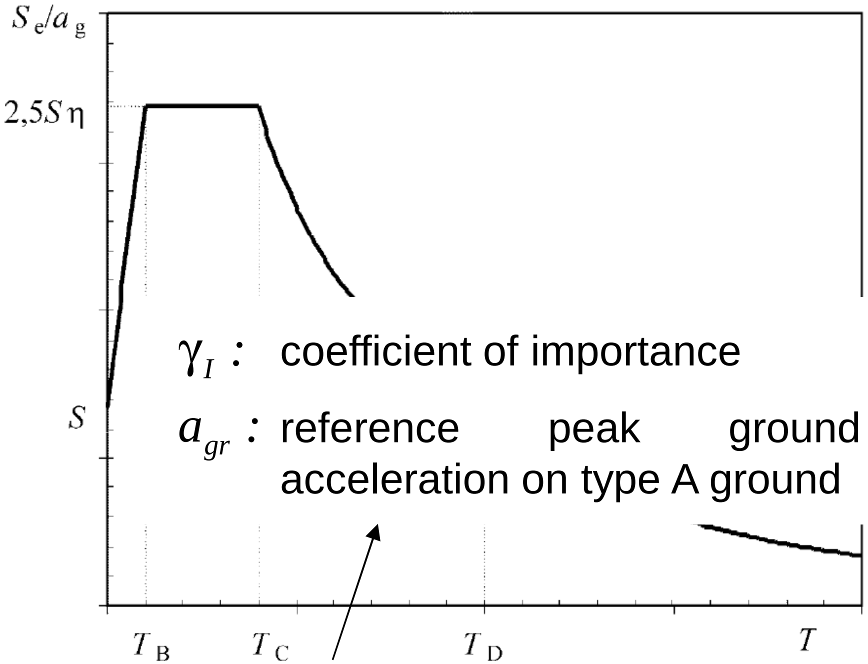
EC8 elastic acceleration response spectrum for horizontal components:

$$S_e(T) = a_g S\left[1 + \frac{T}{T_B}(\eta 2.5 - 1)\right] \qquad 0 \le T \le T_B$$

$$S_e(T) = a_g S \eta 2.5 \qquad T_B \le T \le T_C$$

$$S_e(T) = a_g S \eta 2.5 \frac{T_C}{T} \qquad T_C \le T \le T_D$$

$$S_e(T) = a_g S \eta 2.5 \frac{T_C T_D}{T^2} \qquad T_D \le T \le 4s$$



γ_I : coefficient of importance

a_{gr} : reference peak ground acceleration on type A ground

With:

T : vibration period of a linear SDOF system

a_g : design ground acceleration on ground of type A ($a_g = \gamma_I a_{gr}$)

T_B, T_C : lower and upper limits of the period of the constant spectra acceleration branch

T_D : value defining the beginning of the constant displacement response range

S : soil factor to take into account non-linear soil response effects

Depend on ground type

η : damping (η=1 for 5% viscous damping)

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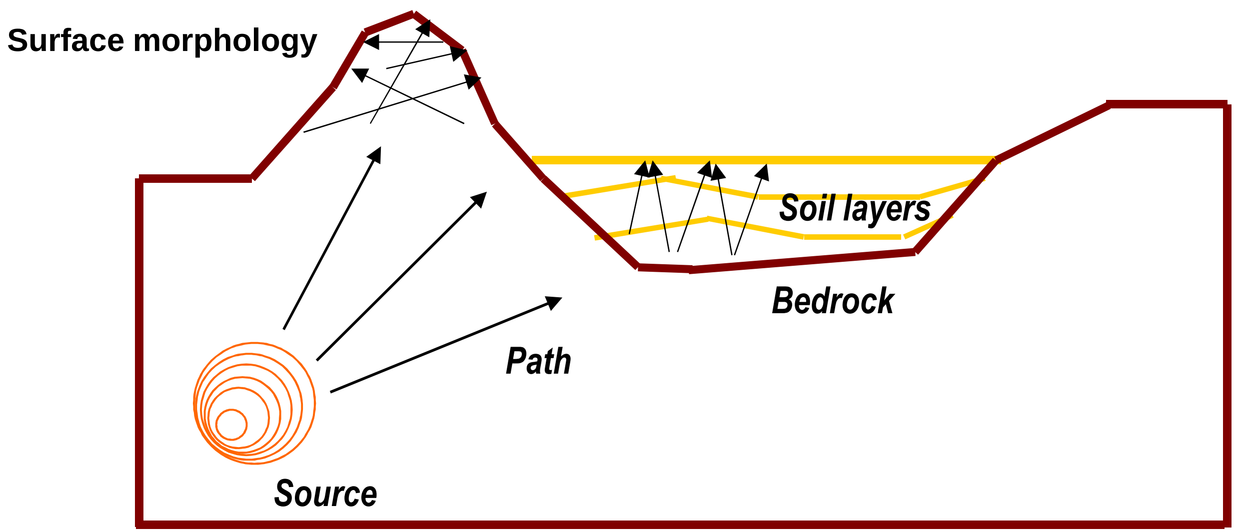
Elastic response spectrum

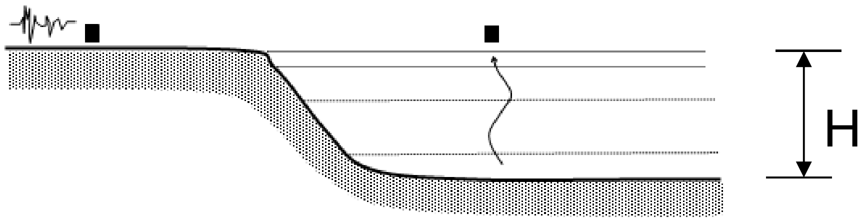
Local site effects:

- Local geological-geomorphological-geotechnical conditions modify the characteristics of ground motion
- Local site effects include:
  - Lithostratigrafic amplification
  - Topographic amplification



Elastic response spectrum

Lithostratigrafic amplification



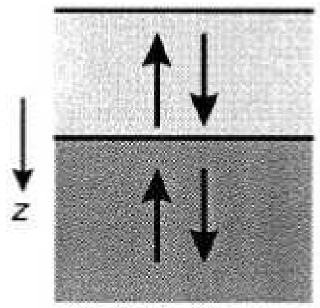
- Impedance effect
  - Seismic Impedance depend on shear-wave velocity and density, which increase with increasing depth
  - Seismic waves are amplified by impedance effects as they travel to the surface
  - Amplification due to impedance effects is frequency dependent
  - Since shear-wave velocity is lowest near the surface, the impedance amplification increases with increasing frequency
- Resonance effects
  - Trapped waves reverberate due to multiple reflections
  - Constructive interference causes resonance, which depends on thickness of layers and elastic properties
  - Resonance can occur even if there are not discontinuities in seismic impedances
  - Resonance frequency for one layer over bedrock: $f=V_s/4H$
- Basin effects
  - Basins are filled by sediments, within body waves are trapped
  - Surface waves are generated at basin edges
  - Duration of shaking in basins is greatly increased

Elastic response spectrum

Lithostratigrafic amplification

- 1D Ground Response Analysis

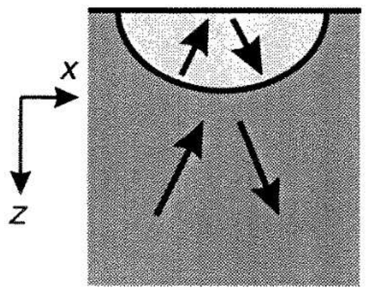
  Vertical propagation of waves in soil deposits constituted by plane and parallel layers over bedrock



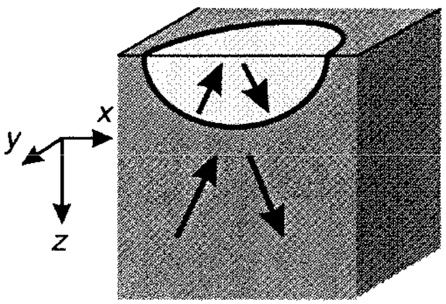
1-1D

- 2D/3D Ground Response Analysis

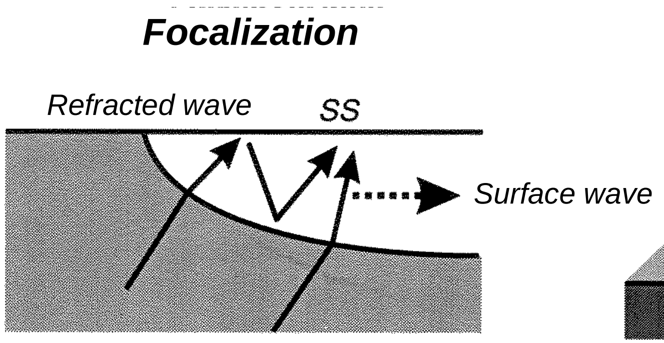
  Propagation of waves with arbitrary incident angle in complex geological configurations with generation of diffractive/scattering effects

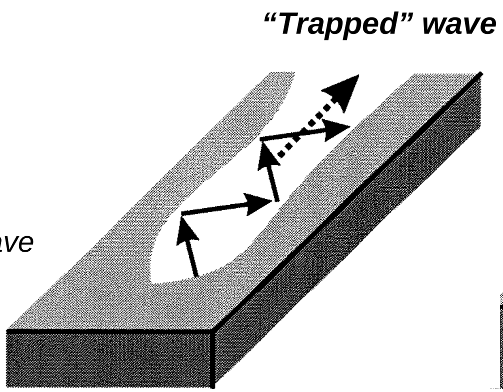


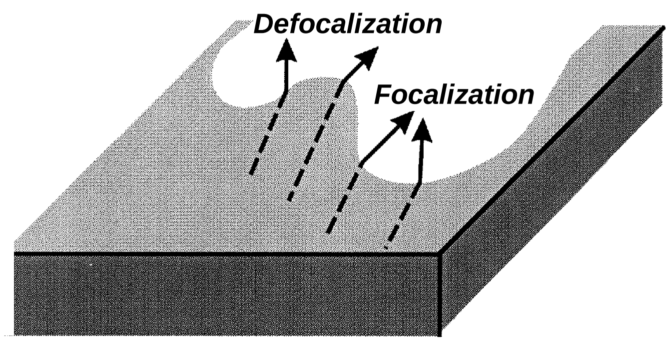
2-2D



3D





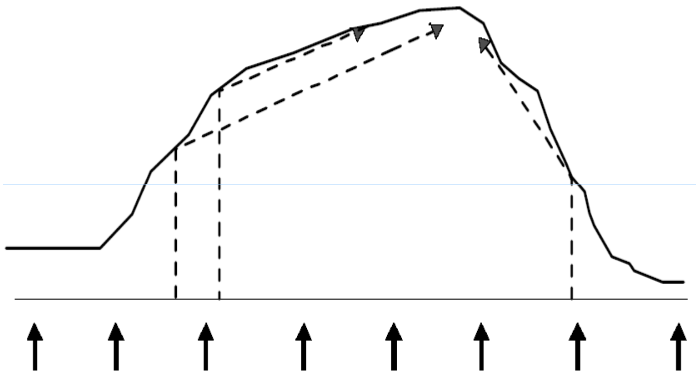


(*from* Faccioli, 2005)

Elastic response spectrum

Topographic amplification

- Effects due to abrupt variations of earth surface
- Surface morphologies as slopes, trenches, ridges, crests and canyons can influence waves propagation path and induce relevant amplifications of the seismic motion



- Focalization/de-focalization of the trajectories of propagation of seismic waves due to reflection of free surface in the proximity of crests
- Interaction between the incident wave field and the refracted one by the topographic irregularity

Elastic response spectrum

Ground types: used to account for local ground conditions

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s> 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

$$V_{S,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{v_i}}$$

with:

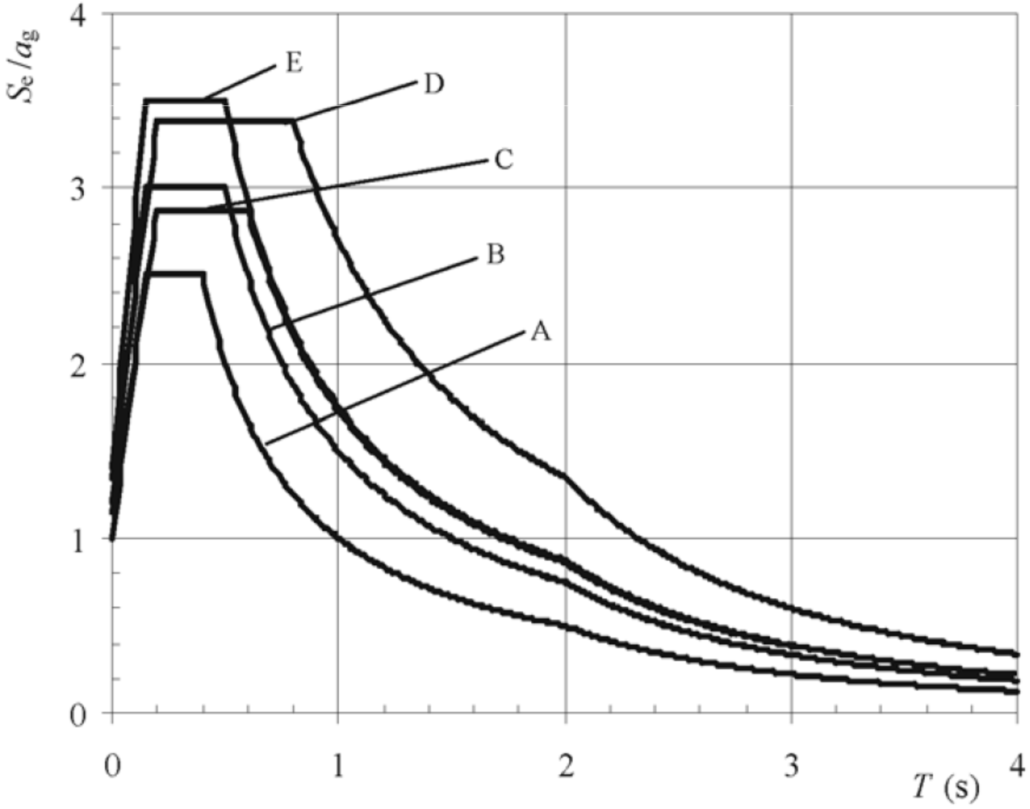
h_i, v_i : Thickness and shear wave velocity of i layer in a total of N in the top 30 m.

For S_1 and S_2 special studies are required

Elastic response spectrum

- Type 1 ($M_s > 5.5$)

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0



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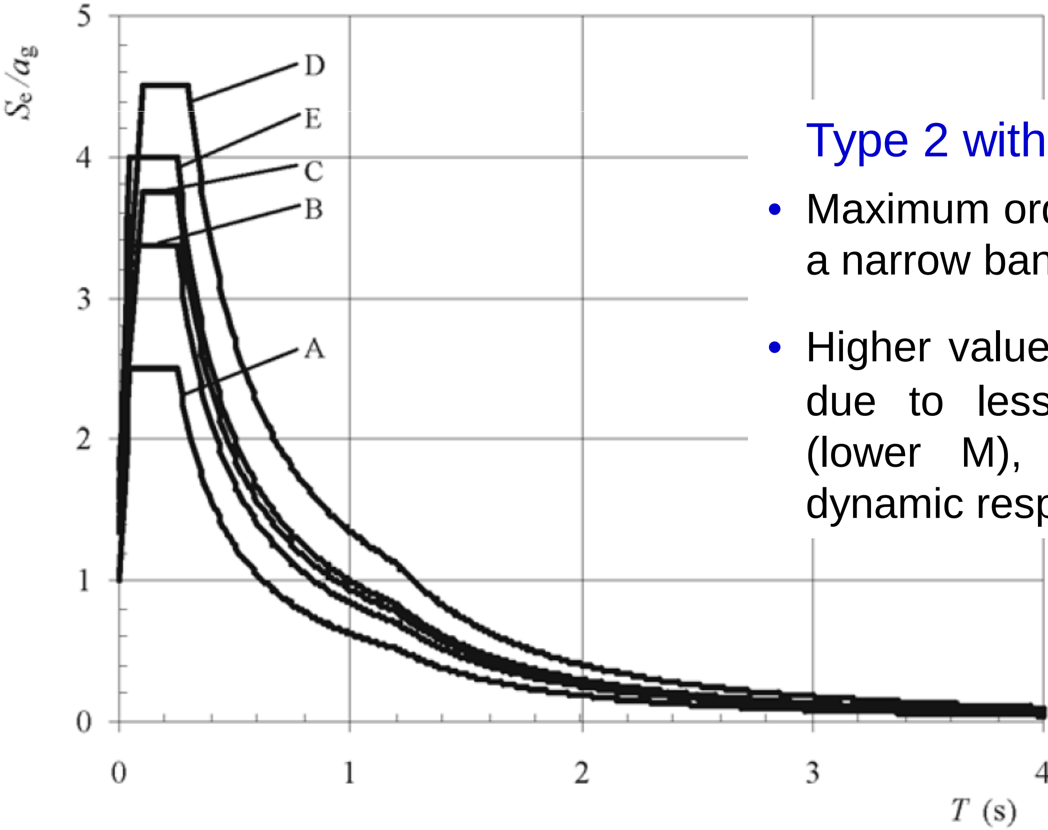
Elastic response spectrum

- Type 2 ($M_s < 5.5$)

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2



Type 2 with respect to type 1:

- Maximum ordinates concentrated in a narrow band at low periods
- Higher values of S for classes B-E due to lesser cyclic deformation (lower M), thus a more linear dynamic response

Elastic response spectrum

EC8 elastic acceleration response spectrum for vertical component:

$$S_{ve}(T) = a_{vg}\left[1 + \frac{T}{T_B}(\eta 3.0 - 1)\right] \qquad 0 \le T \le T_B$$

$$S_{ve}(T) = a_{vg}\eta 3.0 \qquad T_B \le T \le T_C$$

$$S_{ve}(T) = a_{vg}\eta 3.0\frac{T_C}{T} \qquad T_C \le T \le T_D$$

$$S_{ve}(T) = a_{vg}\eta 3.0\frac{T_C T_D}{T^2} \qquad T_D \le T \le 4s$$

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,90	0,05	0,15	1,0
Type 2	0,45	0,05	0,15	1,0

Elastic response spectrum

EC8 topographic amplification factors S_T:

- S_T Should be taken into account for important structures (γ_I > 1.0)
- S_T is considered independent of the fundamental period of vibration, hence, multiply as a constant scaling factor the ordinates of the elastic design response spectrum
- S_T should be applied when the slopes belong to two-dimensional topographic irregularities, such as long ridges and cliffs of height greater than about 30 m.
- S_T is recommended for slope angles >15º:
    a) isolated cliffs and slopes: S_T > 1.2 for sites near the top edge
    b) ridges with crest width significantly less than the base width: S_T > 1.4 near the top of the slopes for average slope angles greater then 30º, S_T > 1,2 for smaller slope angles
    c) presence of a loose surface layer: S_T given in a) and b) increased by at least 20%
    d) spatial variation of amplification factor: S_T may be assumed to decrease as a linear function of the height above the base of the cliff or ridge, and to be unity at the base
- In general, seismic amplification also decreases rapidly with depth within the ridge. Therefore, topographic effects to be reckoned with in stability analyses are largest and mostly superficial along ridge crests, and much smaller on deep seated landslides where the failure surface passes near to the base

Elastic response spectrum

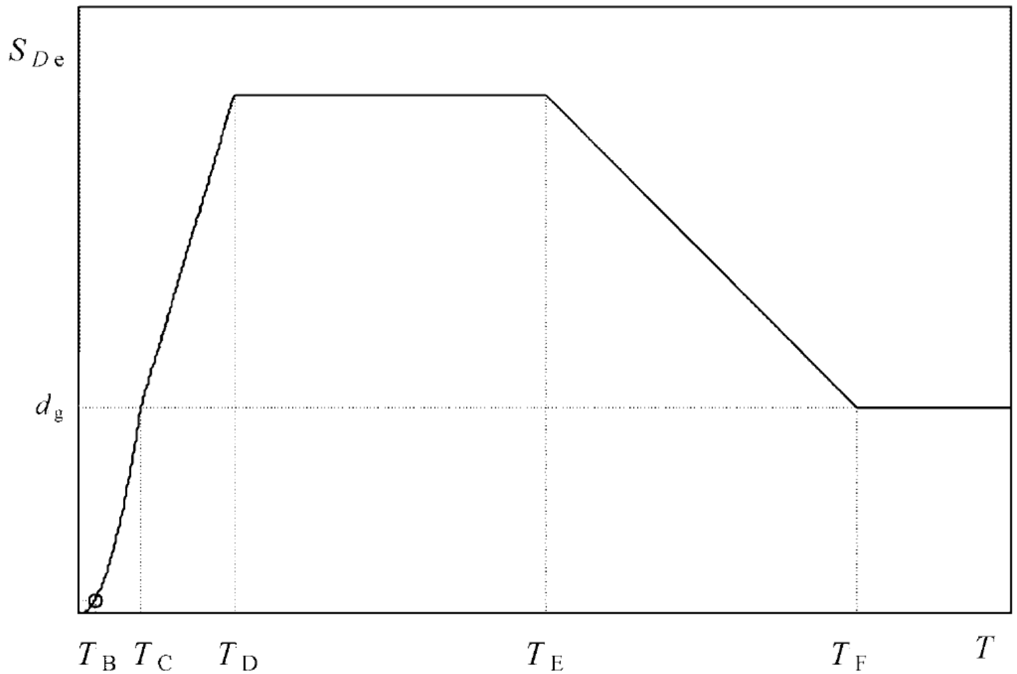
EC8 elastic displacement response spectrum for horizontal component :

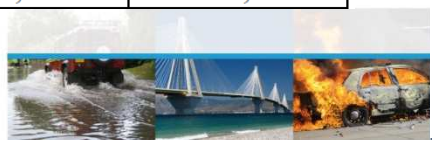
$$S_{De}(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \qquad T < T_E$$

$$S_{De}(T) = 0.025 a_g S T_C T_D \left[2.5\eta + \left(\frac{T - T_E}{T_F - T_E}\right)(1 - 2.5\eta)\right] \qquad T_E \le T \le T_F$$

$$S_{De}(T) = 0.025 a_g S T_C T_D \qquad T > T_F$$

Ground type	T_E (s)	T_F (s)
A	4,5	10,0
B	5,0	10,0
C	6,0	10,0
D	6,0	10,0
E	6,0	10,0

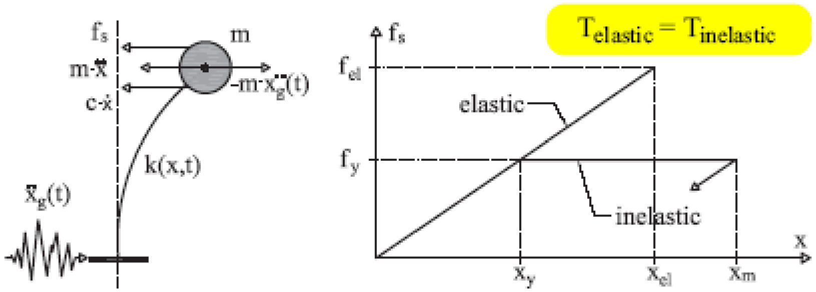




Fundamental of Ductility

Illustrative Example

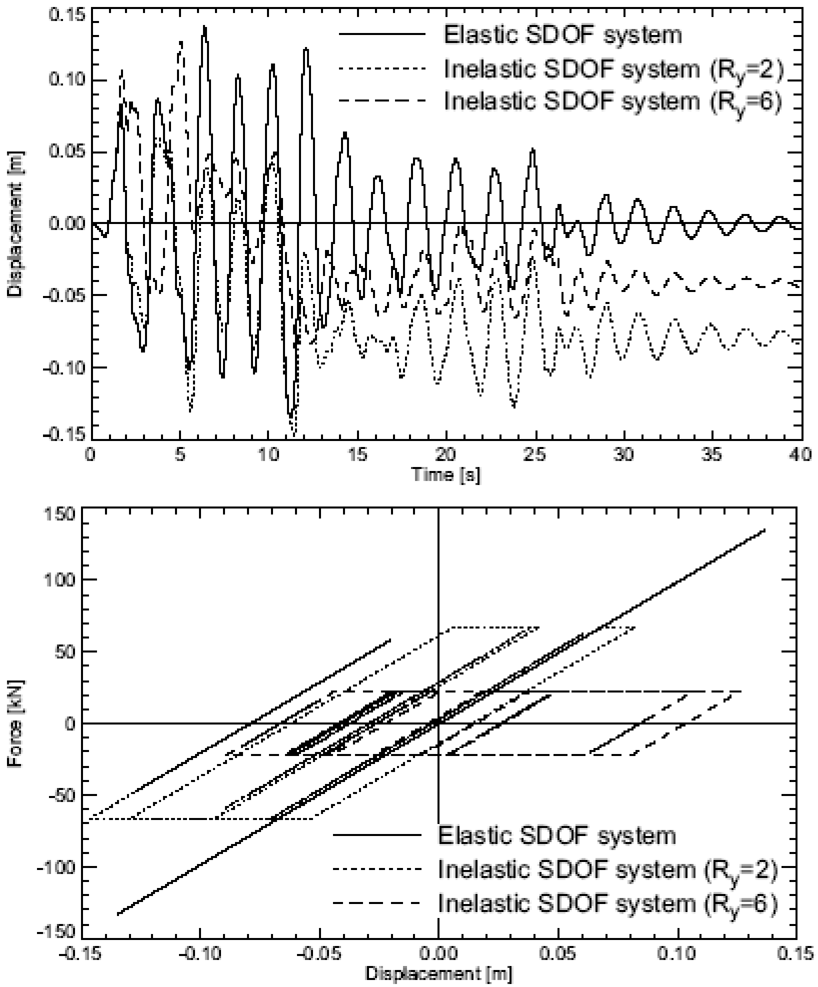
Comparison of the time history analyses of an elastic and an inelastic SDOF system



Where:

$R_y = f_{el}/f_y$: Force reduction factor (R_y in USA codes and q in EU codes)
f_{el} : Maximum restoring force that the elastic SDOF system reaches over the course of elastic exitation
f_y : Yield force of the inelastic SDOF system
$\mu_\Delta = x_m/x_y$: Displacement ductility
x_m : Maximum displacement that the inelastic SDOF system reaches over the course of elastic exitation
x_y : Yield displacement of the inelastic SDOF system

Results:


Elastic SDOF system
Inelastic SDOF system (Ry=2)
Inelastic SDOF system (Ry=6)
Displacement [m]
Time [s]
Force [kN]
Displacement [m]
Elastic SDOF system
Inelastic SDOF system (Ry=2)
Inelastic SDOF system (Ry=6)

Comparison:

Quantity	Elastic SDOF	Inela. SDOF R_y=2	Inela. SDOF R_y=6
T [s]	2.0	2.0	2.0
F_{max} [kN]	134.70	67.35	22.45
R_y [-]	–	2.0	6.0
x_y [m]	–	0.068	0.023
x_m [m]	0.136	0.147	0.126
μ_Δ [-]	–	2.16	5.54

Comments:

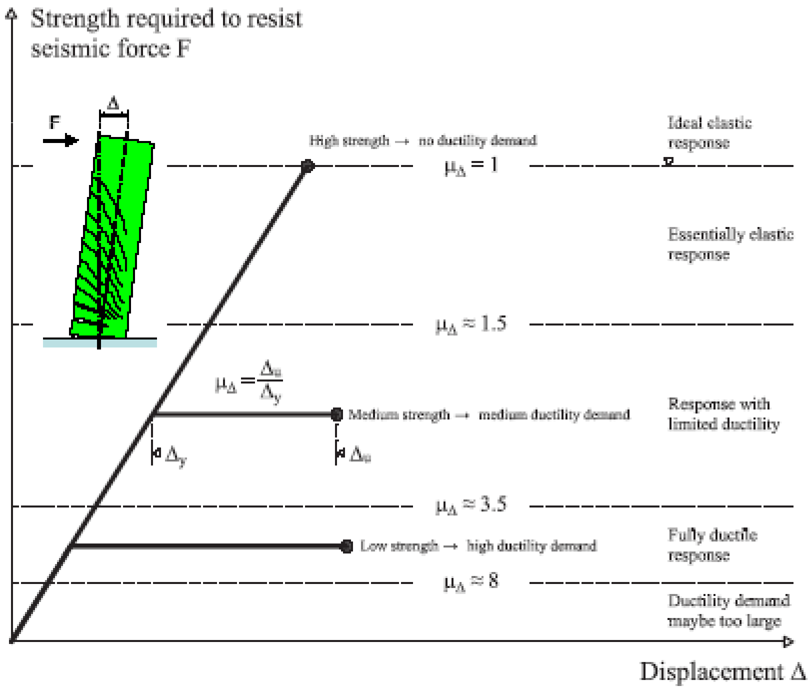
We could define an inelastic response spectrum integrating the equation of motion of an SDOF system that is inelastic instead of being elastic. But we would rather define rules to pass from the elastic spectrum to the inelastic one, since for an Engineer the starting point is the elastic response spectrum of the design codes.

"Seismic behaviour equations":

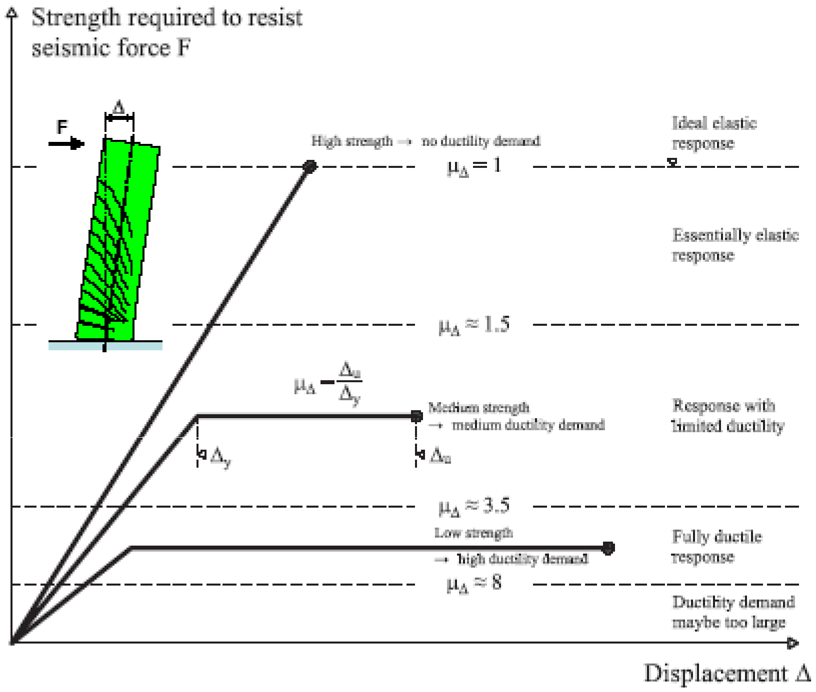
For seismic collapse prevention, the following approximate relationship applies:

″quality″ of seismic behaviour ≈ strength × ductility

To survive an earthquake different combination of strength and ductility are possible:

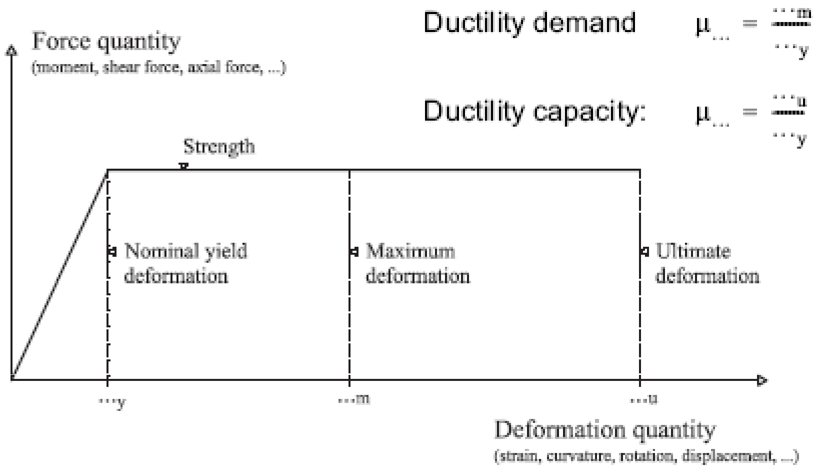


More realistic representation of the decision possibilities



- If the strength of the structure reduces, the stiffness typically reduces too;
- If the masses do not change significantly (which is typically the case), the fundamental period T of the softer structure is longer;
- Structures with a longer fundamental period T are typically subjected to larger deformations, i.e., the deformation demand is larger.

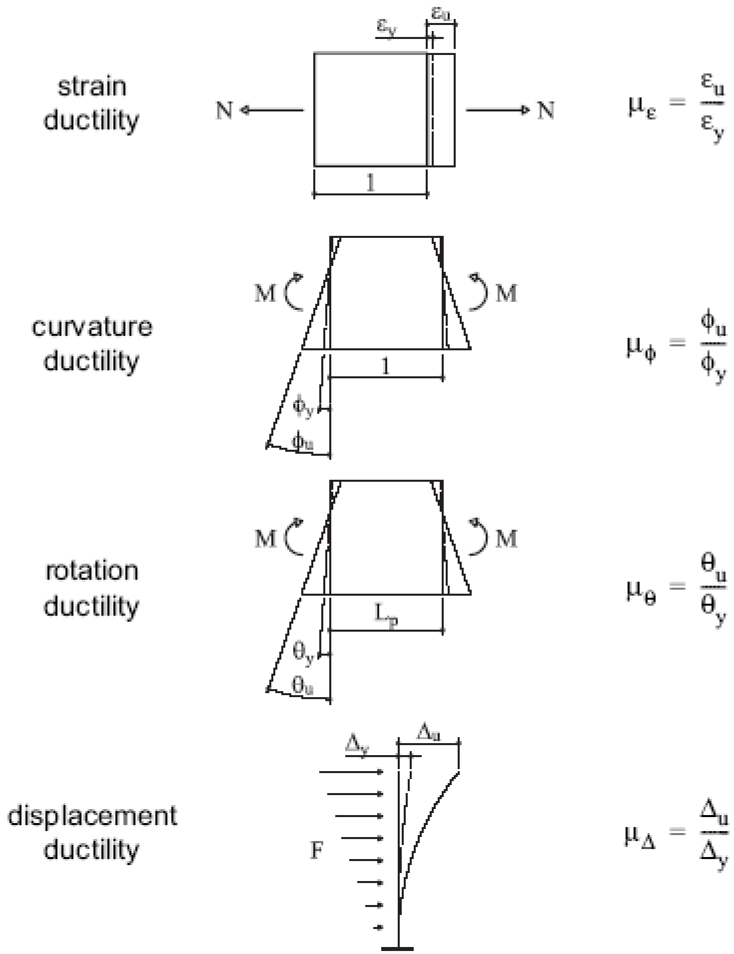
General definition of ductility



Comments

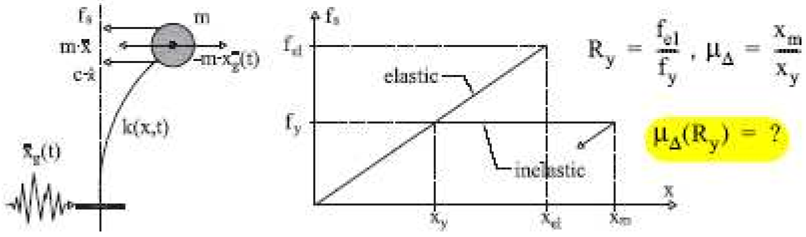
- The **ductility capacity is a property of the structural member;**
- The **ductility demand is a result of the seismic excitation and also** a function of the dynamic properties of the structure;
- A structural member survives the earthquake if: Ductility capacity ≥ Ductility demand;
- The structural member collapse when locally the deformation capacity of the structural materials (i.e., their strain capacities) are reached and exceeded. The ductility capacity is therefore exhausted.

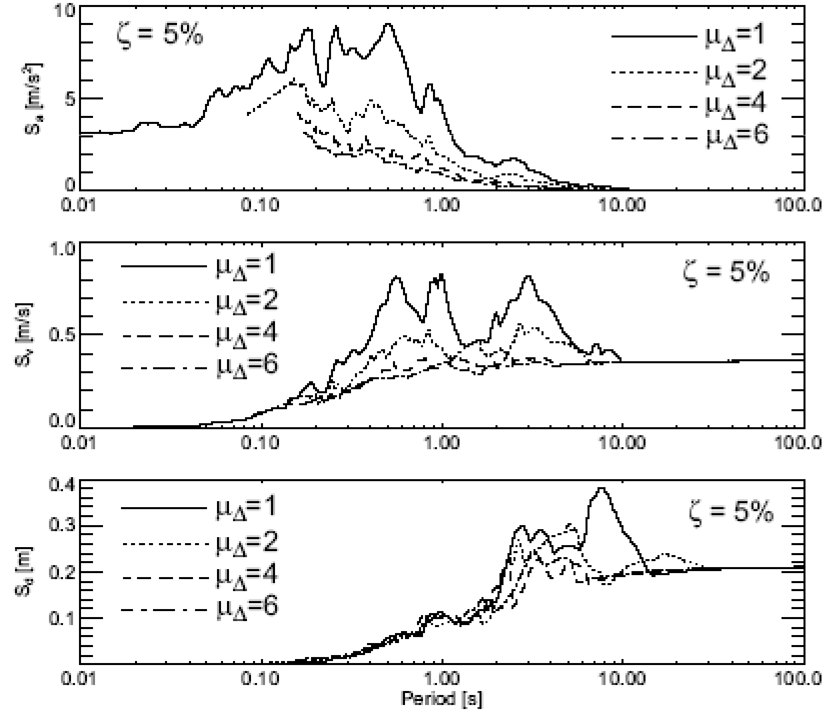
Types of ductility



Inleastic Responce Spectra

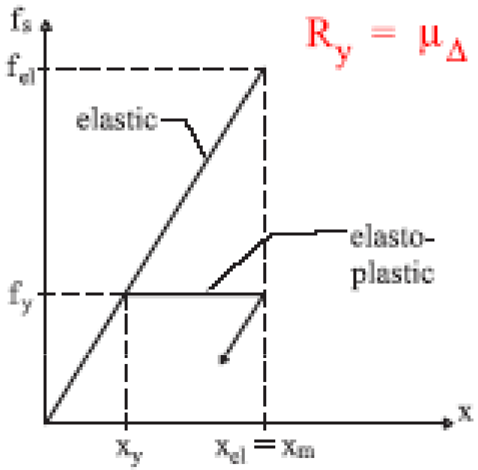
Inelastic responce spectra





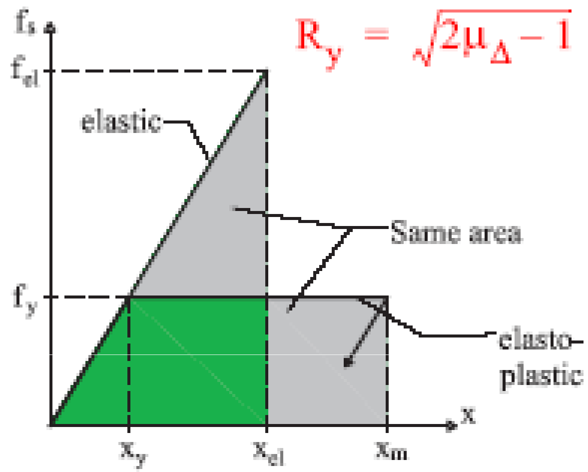
Force reduction factor R_y

Equal displacement principle



Suitable in the long period range

Equal energy principle

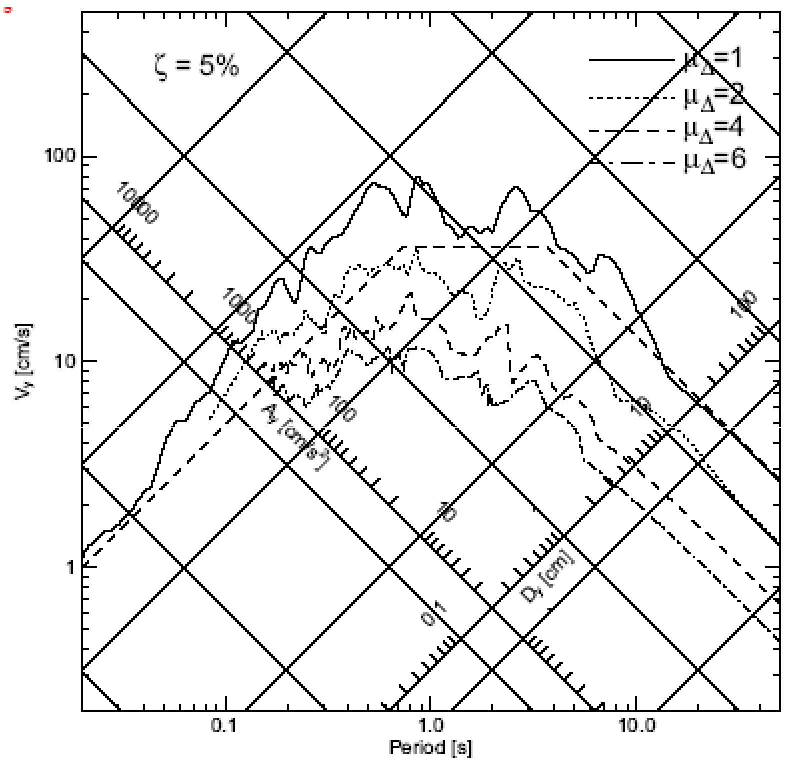


Suitable in the short period range

In the “equal displacement principle” and the “equal energy principle” there are “historical” R_y, μ_Δ-T_n relationships. Also in recent years a lot of research has been done to come up with more accurate relationships.

R_y, μ_Δ-T_n relationships lead to the definition of behaviour factor (R_y in US codes and q in European codes)

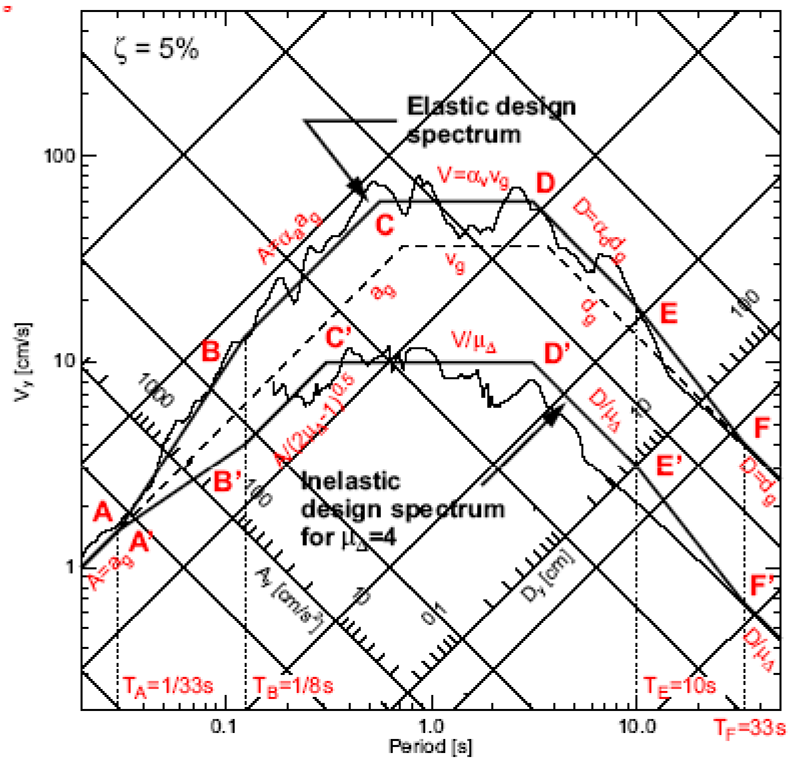
Inelastic design spectra in D (displacement) - V(velocity) – A (acceleration):



If x_y is the yield displacement:

$D_y = x_y$; $V_y = \omega_n x_y$; $A_y = \omega_n^2 x_y$

Newmark's inelastic design spectra (NH82):



Maximum displacement of inelastic SDOF system:

$x_m = \mu_\Delta D_y$

Yeld strenght of the inelastic SDOF system:

$f_y = m A_y$

Construction of the R_y, μ_Δ-T_n relationships:

$A_y = S_{a,\ inelastic} = S_{a,\ elastic} / R_y$

$D = S_{d,\ inelastic} = \mu_\Delta S_{d,\ elastic} / R_y$

It should be noted that:

$S_{a,\ inelastic} \neq \omega^2\ S_{d,\ inelastic}$

R_y, μ_Δ-T_n relationships according to NH82:

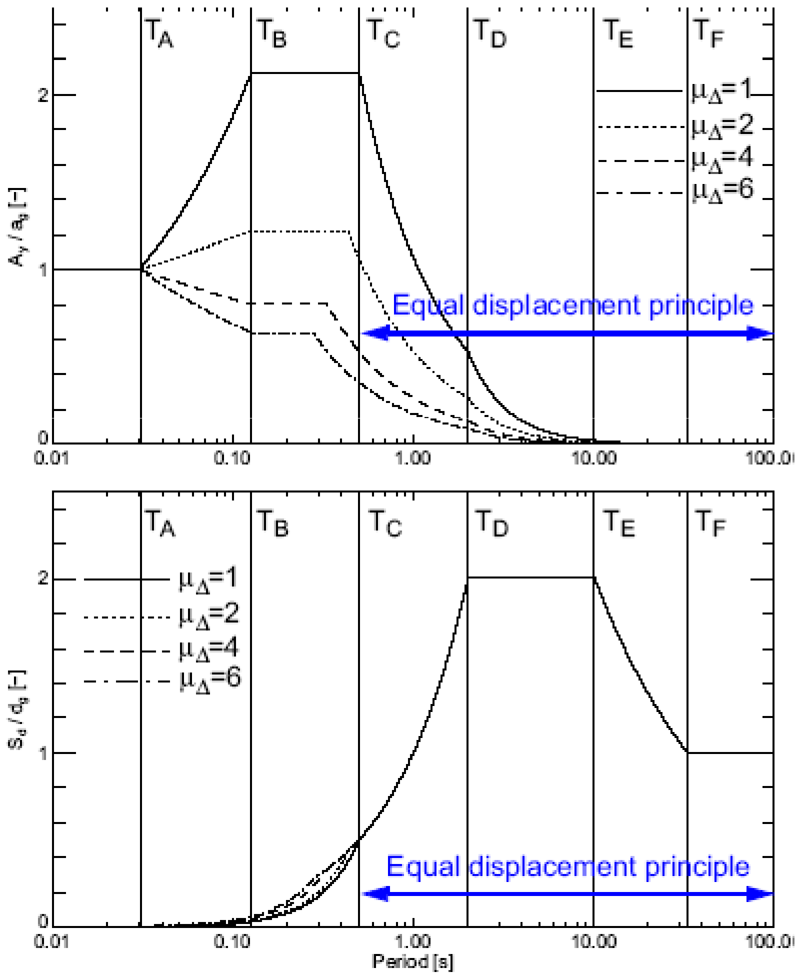
$$R_y = \begin{cases} 1 & T_n < T_a \\ (2\mu_\Delta - 1)^{\beta/2} & T_a < T_n < T_b \\ \sqrt{2\mu_\Delta - 1} & T_b < T_n < T_{c'} \quad \text{(equal energy principle)} \\ \dfrac{T_n}{T_c}\mu_\Delta & T_{c'} < T_n < T_c \\ \mu_\Delta & T_n > T_c \quad \text{(equal displacement principle)} \end{cases}$$

Where:

$\beta = \log(T_n/T_a)/\log(T_b/T_a)$

$T_a = 1/33$ sec
$T_b = 1/8$ sec
T_c corner period between constant $S_{a,\ elastic}$ region and constant $S_{v,\ elastic}$ region
$T_{c'}$ corner period between constant $S_{a,\ inelastic}$ region and constant $S_{v,inelastic}$ region

Inelastic design spectra according to NH82:



R_y, μ_Δ-T_n relationships according to VFF94:

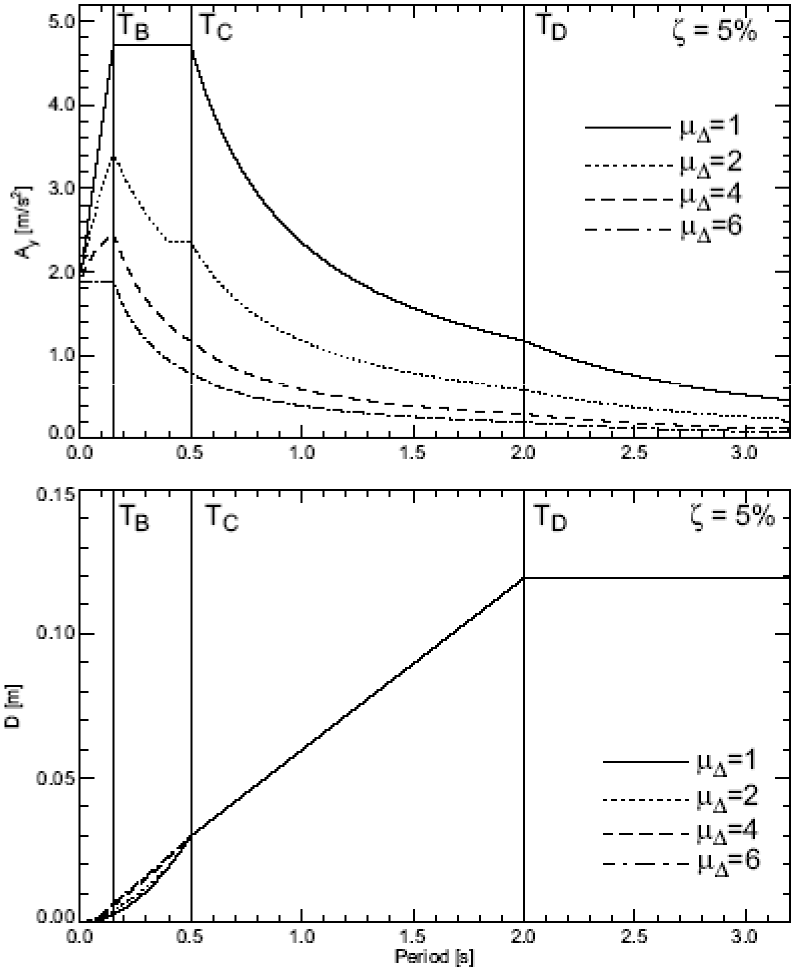
$$R_y = \begin{cases} (\mu_\Delta - 1)\dfrac{T_n}{T_0} + 1 & T_n \le T_0 \\ \mu_\Delta & T_n > T_0 \text{(equal displacement principle)} \end{cases}$$

Where:

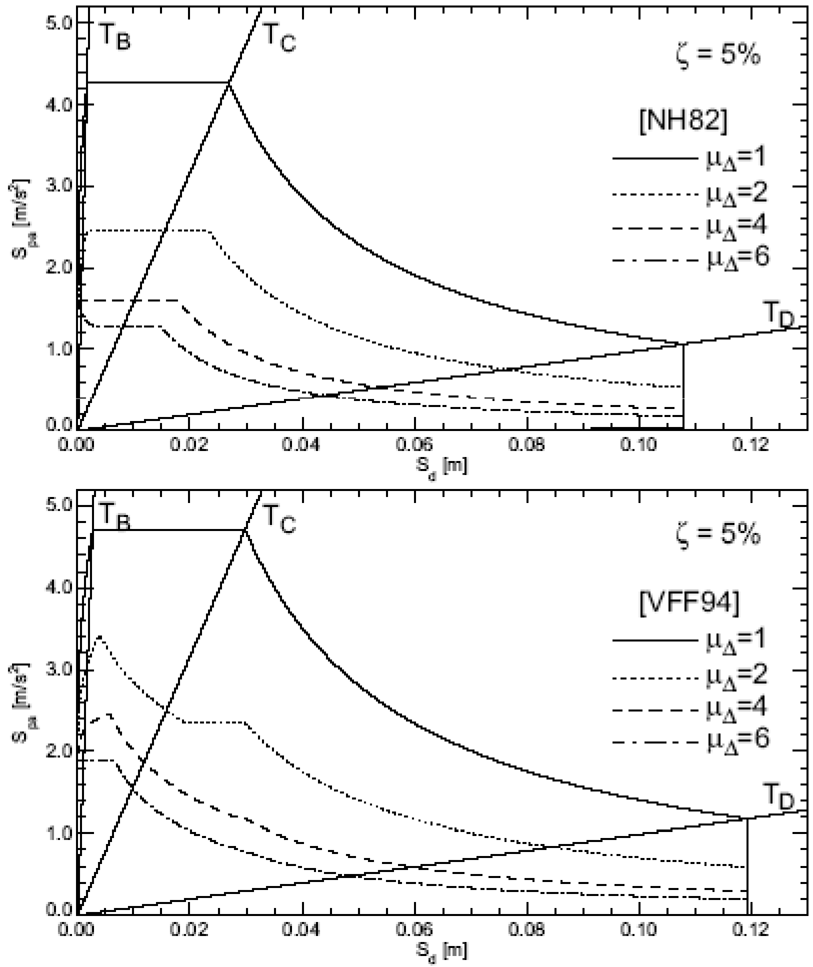
$T_0 = 0.65\ \mu_\Delta^{0.3}\ T_c \le T_c$

T_c corner period between constant $S_{a,\,elastic}$ region and constant $S_{v,\,elastic}$ region

Inelastic design spectra according to VFF94:



Inelastic design spectra in ADRS format (Acceleration – Displacement Response Spectra):

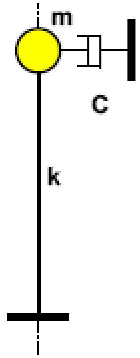


Determining the response of an inelastic SDOF system by means of inelastic design spectra in ADRS format

In this section the response of two example inelastic SDOF systems is determined by means of inelastic design spectra in ADRS-format:

- SDOF system 1 with $T_n = 0.9$ s
- SDOF system 2 with $T_n = 0.3$ s
- R_y, μ_Δ-T_n relationships according to [VFF94] will be used
- $T_c = 0.5$ sec

SDOF system 1



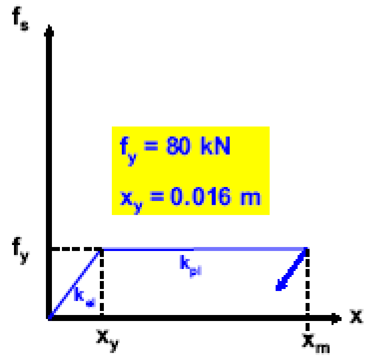
properties:

m = 100 t

k_{el} = 4874 kN/m

k_{pl} = 0 kN/m

ζ = 5%



Responce of the elastic SDOF system 1

$$T_n = 2\pi\sqrt{\frac{m}{k}} = 2\pi\sqrt{\frac{100}{4874}} = 0.9\,\text{sec}$$

$$S_a = 2.62\,\text{m}/\text{sec}^2$$

$$S_d = 0.054\,\text{m}$$

$$f_{el} = 261.7\,\text{kN}$$

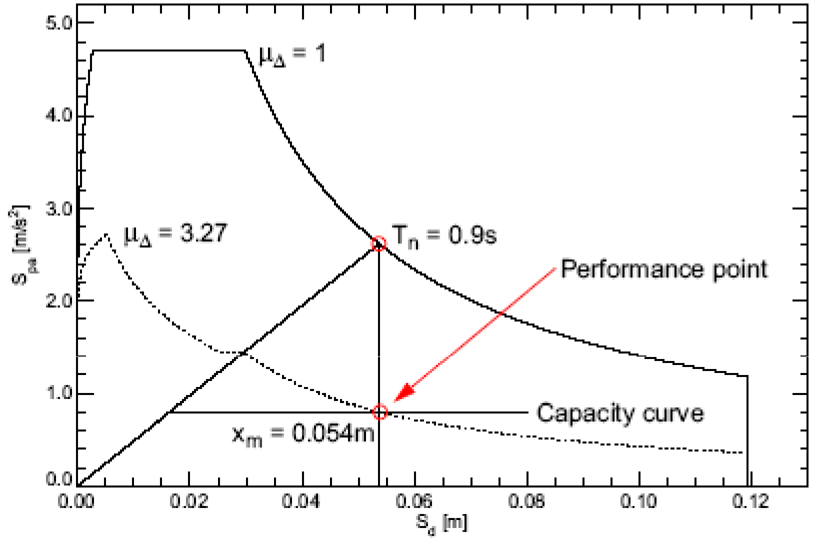
Responce of the inelastic SDOF system 1

$$R_y = \frac{f_{el}}{f_y} = 3.27$$

$$\mu_\Delta = R_y = 3.27 \quad \text{because } T_n > T_c = 0.5\,\text{sec}$$

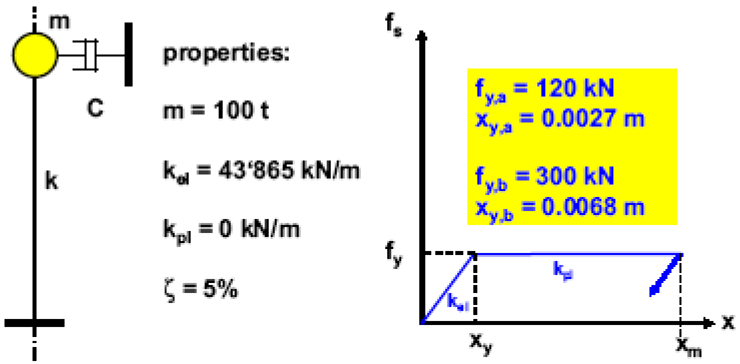
$$x_m = x_y \mu_\Delta = 0.016\ 3.27 = 0.054$$

Representation of the inelastic SDOF system 1 in the inelastic design spectrum in ADRS-format:



- If the force-deformation relationship of the inelastic SDOF system is divided by its mass m, the “capacity curve” is obtained, which can be plotted on top of the spectrum in ADRS-format
- The capacity curve and the inelastic spectrum intersect in the “performance point”

SDOF system 2



Responce of the elastic SDOF system 2

$$T_n = 2\pi\sqrt{\frac{m}{k}} = 2\pi\sqrt{\frac{100}{43865}} = 0.3\,\text{sec}$$

$$S_a = 4.71\,\text{m}/\text{sec}^2$$

$$S_d = 0.011\,\text{m}$$

$$f_{el} = 471\,\text{kN}$$

In this second example two different inelastic SDOF systems will be considered:

(a) a SDOF system with a rather low f_y

(b) a SDOF system with a rather high f_y

Responce of the inelastic SDOF system 2 a

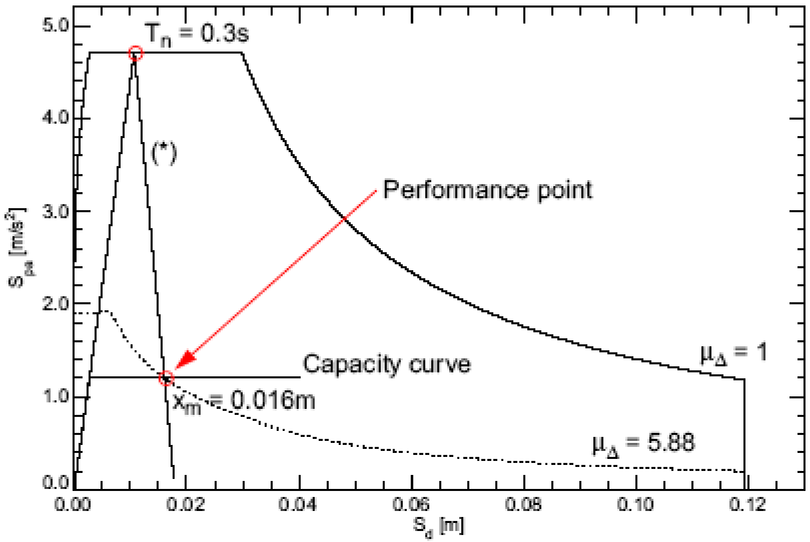
$$R_y = \frac{f_{el}}{f_y} = 3.93 \rightarrow \mu_\Delta \text{ is so large that } T_0 = T_c = 0.5 \text{ sec}$$

$$\mu_\Delta = (R_y - 1)\frac{T_c}{T_n} + 1 = (3.93 - 1)\frac{0.5}{0.3} + 1 = 5.88$$

check if $T_0 > T_c$

$$T_0 = 0.65\mu_\Delta^{0.3}T_c = 0.65\,5.88^{0.3}0.5 = 0.553 \text{ sec} > T_c$$

$$x_m = \mu_\Delta x_y = 5.88\,0.0027 = 0.016 > S_d$$



Responce of the inelastic SDOF system 2 b

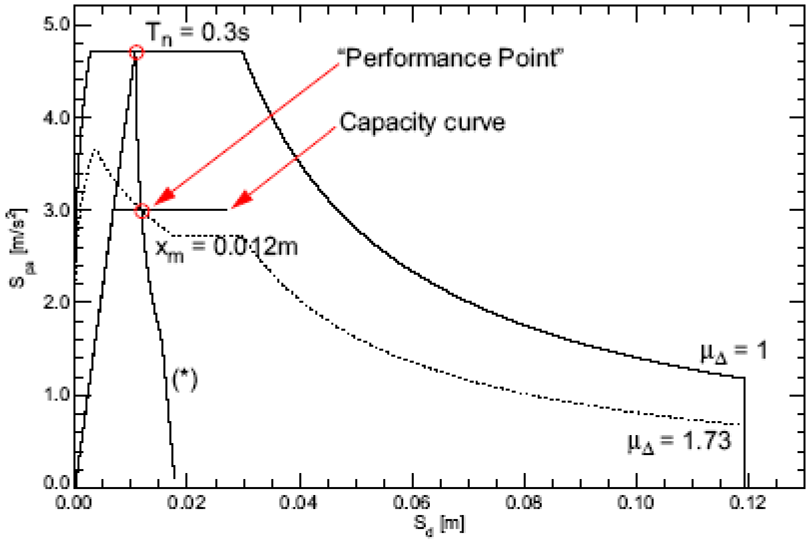
$$R_y = \frac{f_{el}}{f_y} = 1.57 \rightarrow \mu_\Delta \text{ is small, hence } T_0 < T_c = 0.5 \text{ sec}$$

$$(\mu_\Delta - 1)\frac{T_n}{0.65\mu_\Delta^{0.3}T_c} + 1 = R_y \rightarrow \mu_\Delta = 1.73$$

check if $T_0 < T_c$

$$T_0 = 0.65\mu_\Delta^{0.3}T_c = 0.65\,1.73^{0.3}0.5 = 0.383 \text{ sec} < T_c$$

$$x_m = \mu_\Delta x_y = 1.73\,0.0068 = 0.012 > S_d$$



Comments:

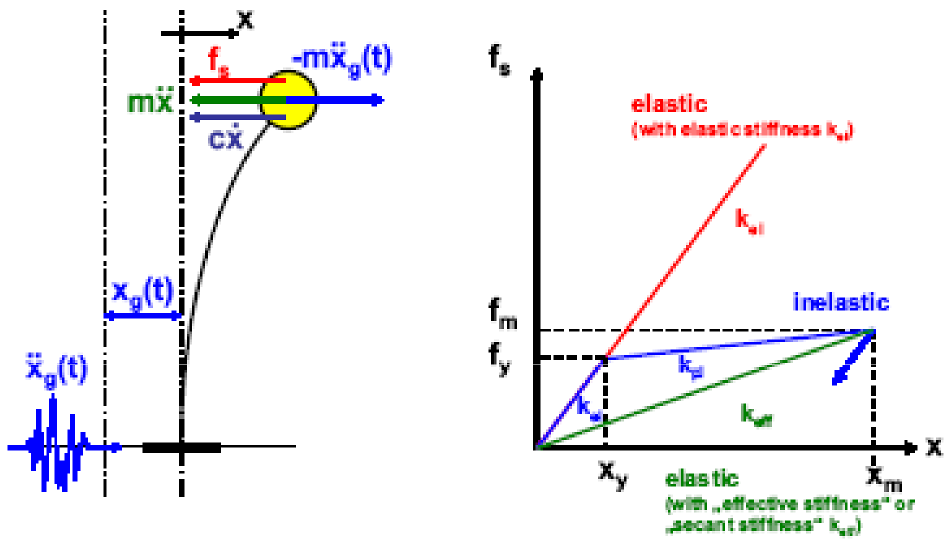
- R_y-μ_Δ-T_n relationships should only be used in conjunction with smoothed spectra. They should not be used to derive the inelastic response spectra of a single ground motion

- Design spectra are very useful tools to design structures for the expected seismic demand. Design spectra represent the average effect of an earthquake with design intensity

- If a single earthquake is considered, the spectra may under or over estimate the seismic demand for a certain period range

- This characteristic of design spectra should be considered when designing structures: The seismic design should aim at structures that are as robust as possible

Linear Equivalent Elastic SDOF System



Where:

The equivalent elastic system in red is used in conjunction to the behaviour factor R_y in force based design methods

The equivalent system in green is used in conjunction to the equivalent viscous damping ξ_{eq} in displacement based design methods

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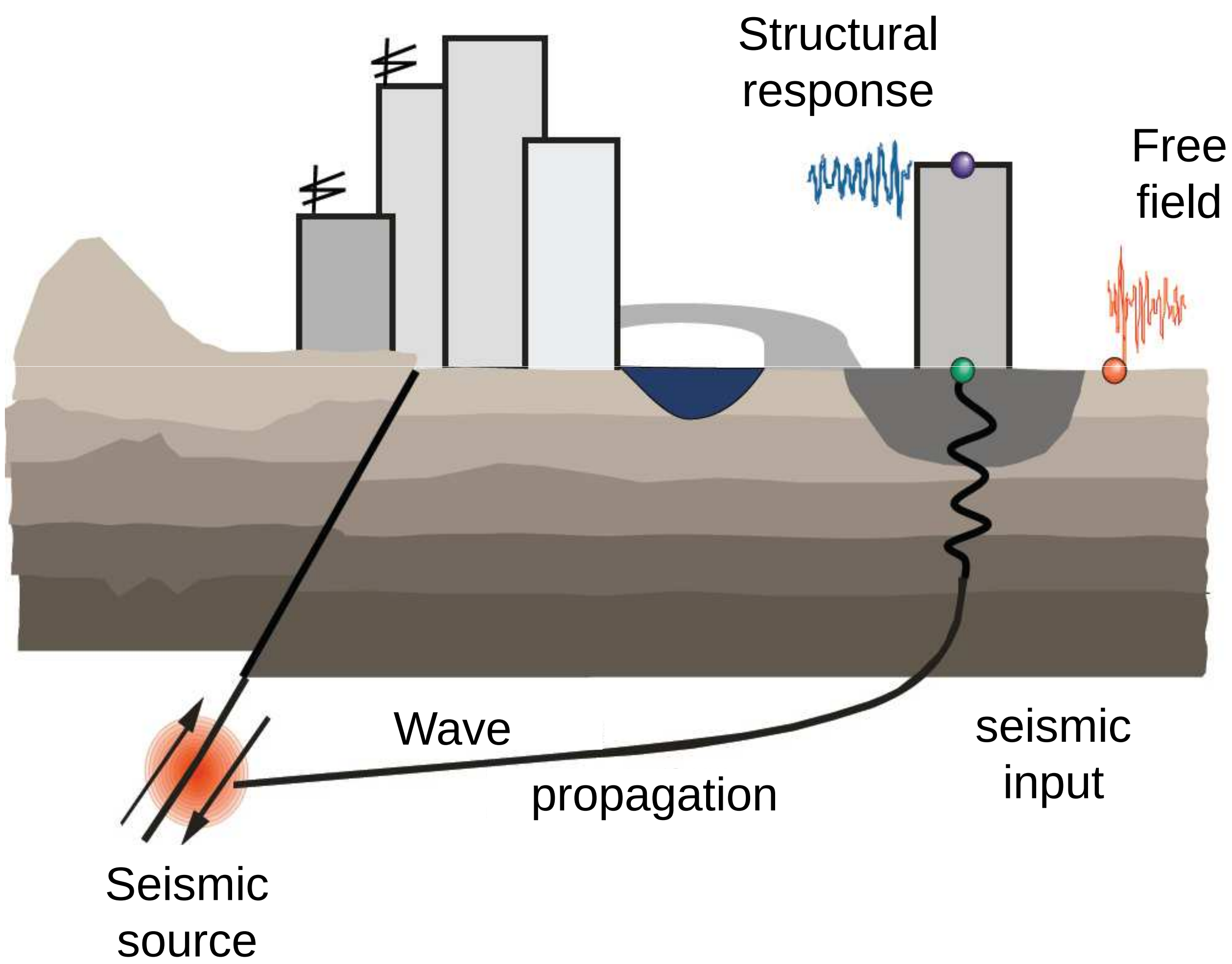
Conceptual Seismic Design

Uncertainty of the seismic action



Earthquake's characteristics
↓
Site effect
↓
Site response
↓
Structural response
↓
Damage
↓
Consequences

General characteristics of the buildings

- It is almost impossible to exactly predict which seismic action a structure will undergo during its life time. For this reason the structure should be conceived to allow significant variations of the loading function without failing.
- To ensure a good seismic performance, a building should be ductile and easily transfer the lateral forces to the ground without reaching excessive deformations.

The conceptual design is crucial in earthquake engineering

General characteristics of the buildings

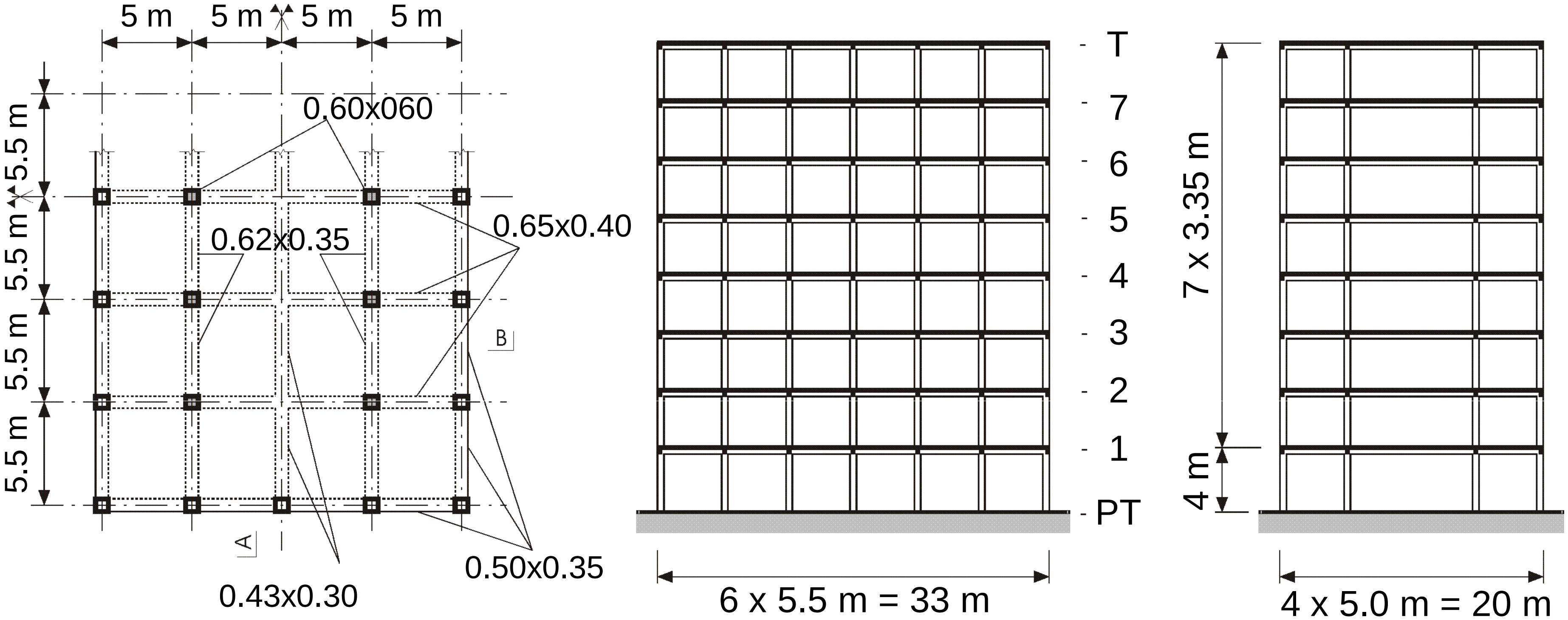
- Structural simplicity
  - easier to understand, predict and build
- Regularity, symmetry and continuity
  - in plan and elevation
- Static overdetermination
- Flexural strength and stiffness along two orthogonal directions
- Torsional strength and stiffness
- In-plane strength and stiffness of the floors
- Adequate foundation

A good seismic building has to be robust!

Adequate structural systems

- Moment resisting frames
  - System comprising beams rigidly connected to columns. Applicable to steel and reinforced concrete structures.
- System with single or coupled walls
  - The horizontal forces are totally carried by reinforced concrete or masonry (reinforced!) walls. Other structural elements carry vertical loads only.
- Dual systems
  - Reinforced concrete frames coupled with reinforced concrete or masonry (reinforced!) walls. The horizontal forces are shared by the different structural elements.
- Trusses with centric or eccentric braces
  - Popular for tall steel structures

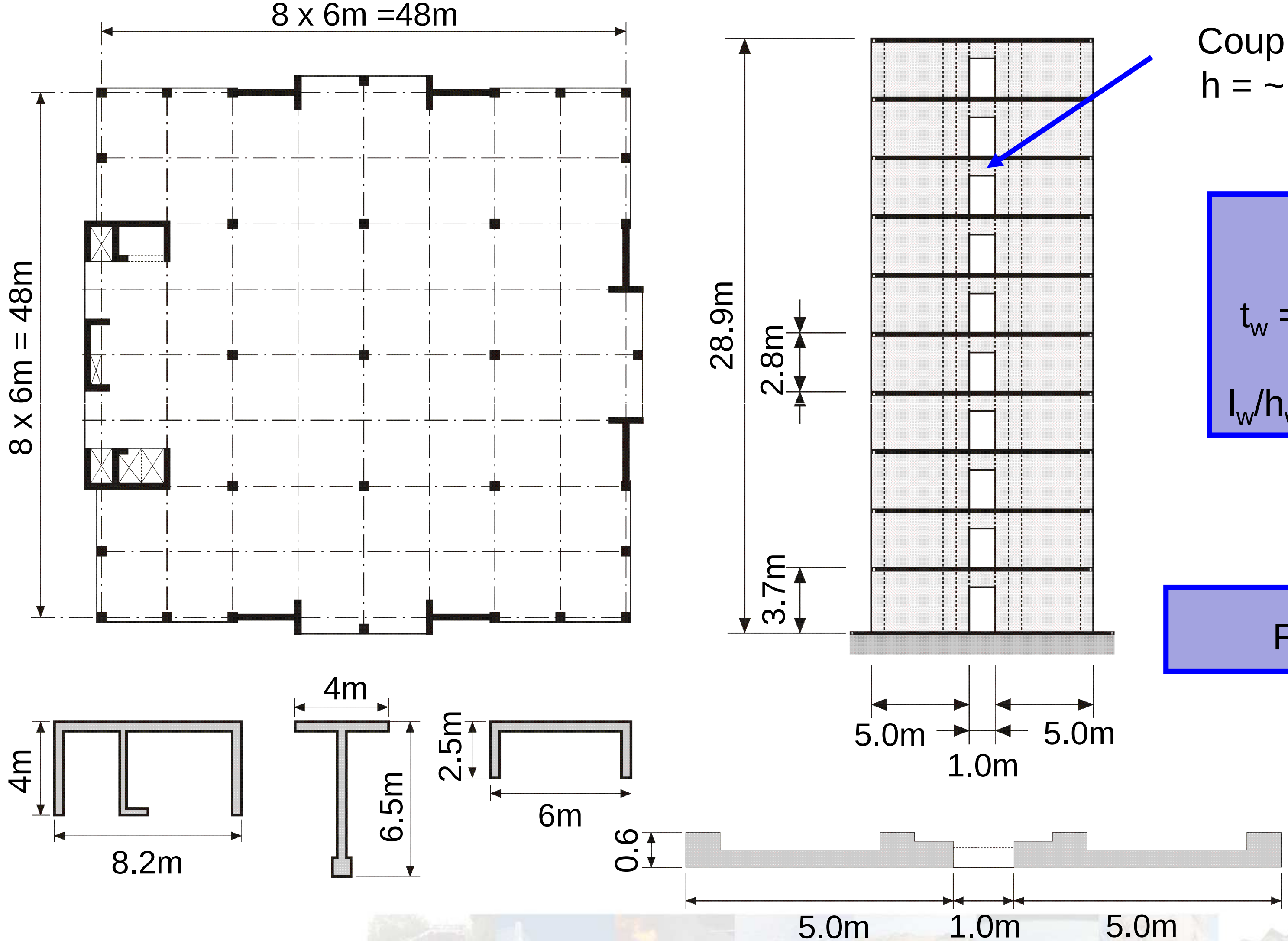
Example of moment resisting frames



Columns	Beams
No. story = 1 - 3: bxh > ca. 40x40 cm	Span 6x6 m: hxb = ca. 50x35 cm
No. story = 3 - 6: bxh > ca. 50x50 cm	Span 8x8 m: hxb = ca. 75x40 cm
No. story = 6 - 10: bxh > ca. 60x60 cm	h including the floor

Example of moment resisting frames

Example of system with single or coupled walls



Coupling beams
h = ~ 60-100cm

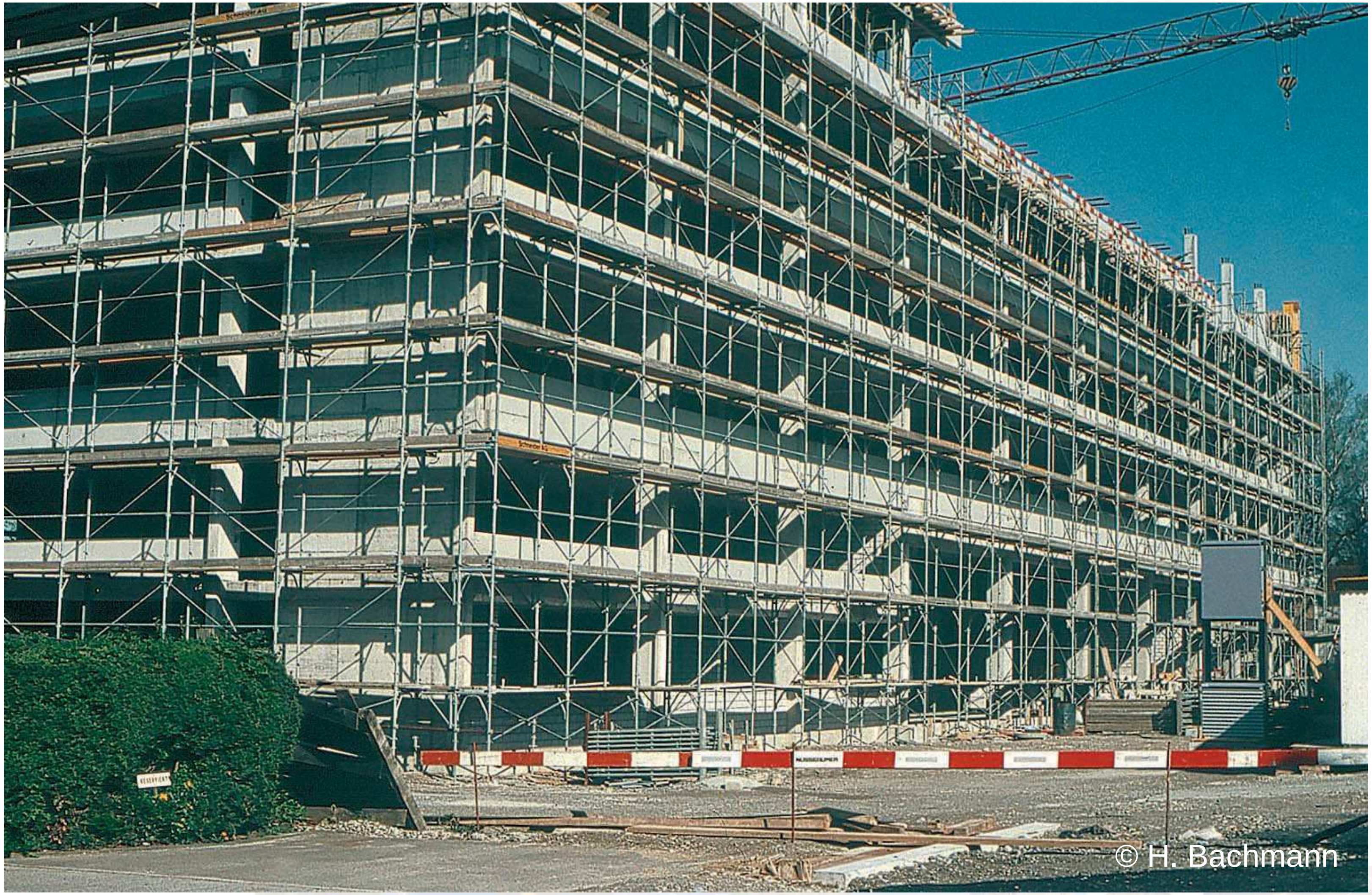
Dimensions

t_w = 0.25 – 0.40 m

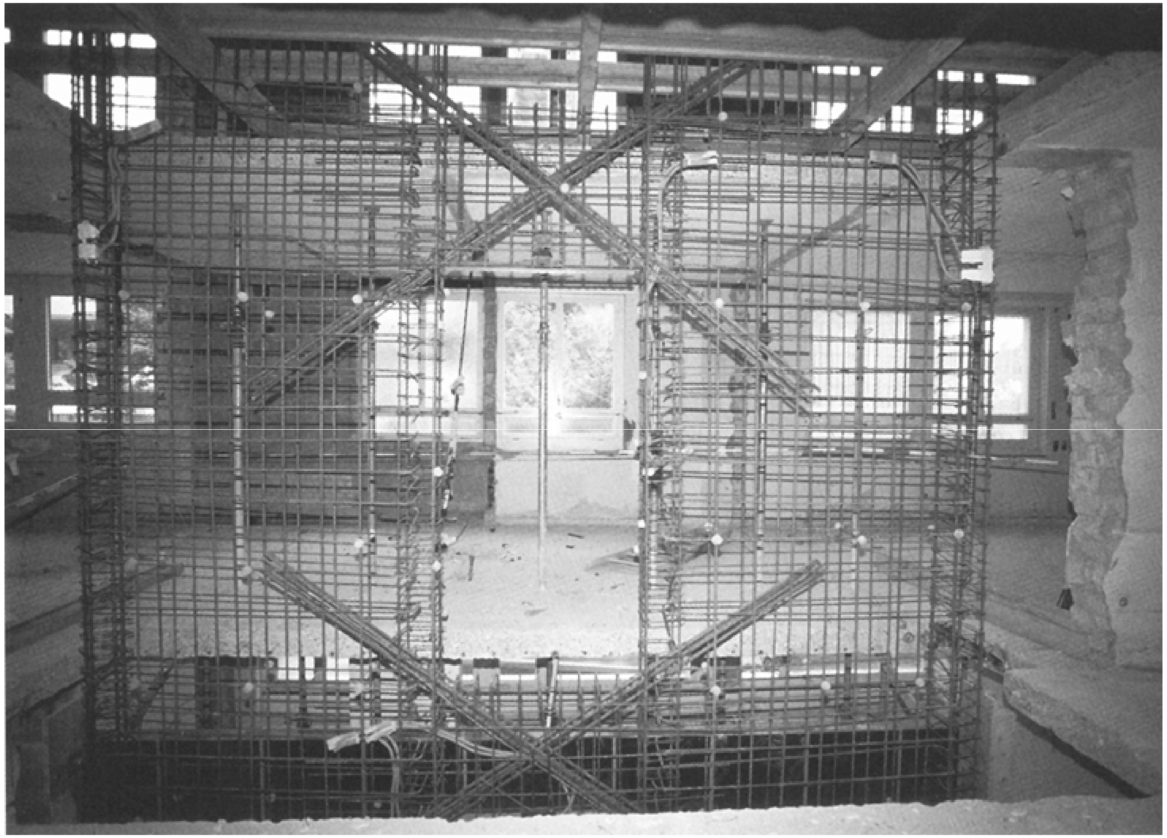
l_w/h_w = ca. 1/3 – 1/5

Foundation!!!

Example of system with single or coupled walls

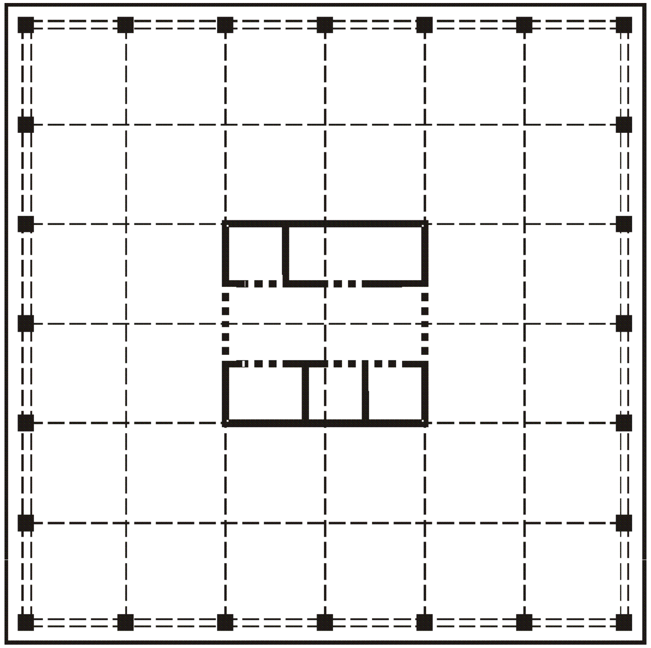


Example of system with single or coupled walls



Seismic retrofit of the children's hospital in Aarau (a_g=0.1g) using RC coupled walls with diagonal reinforcement in the coupling beams

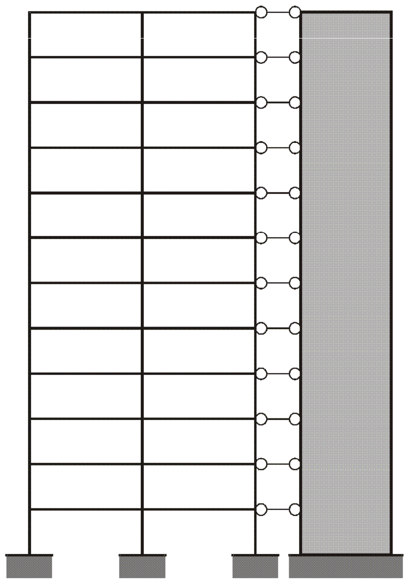
Dual systems



„Tube in Tube“ system

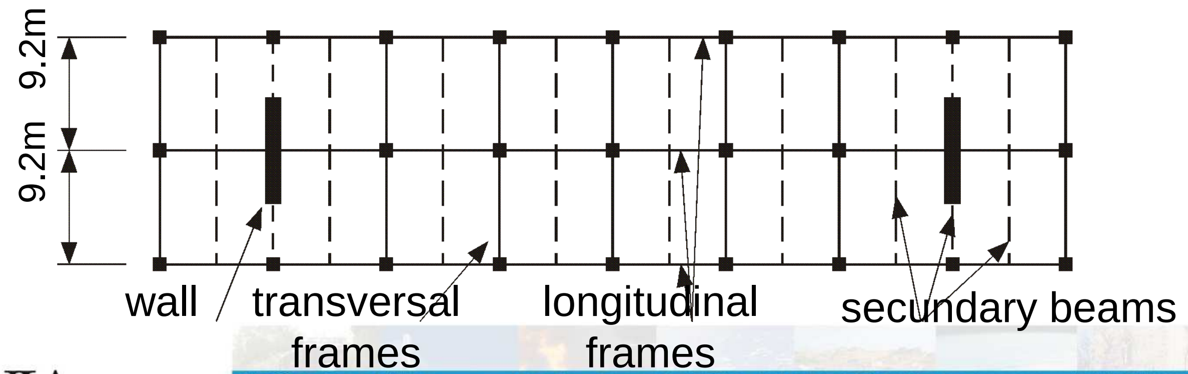
Classic dual system

Elevation



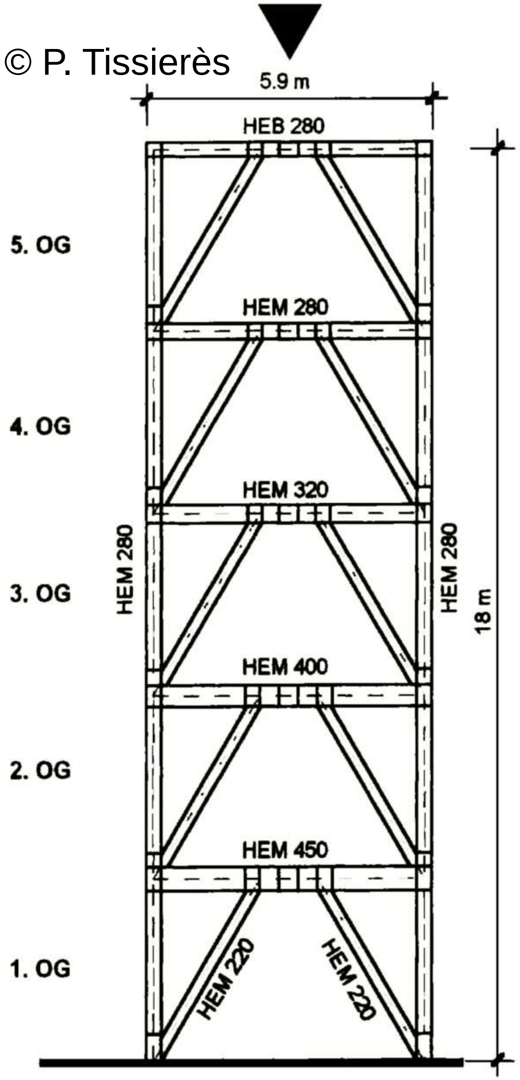
7 frames + 2 walls

Plan



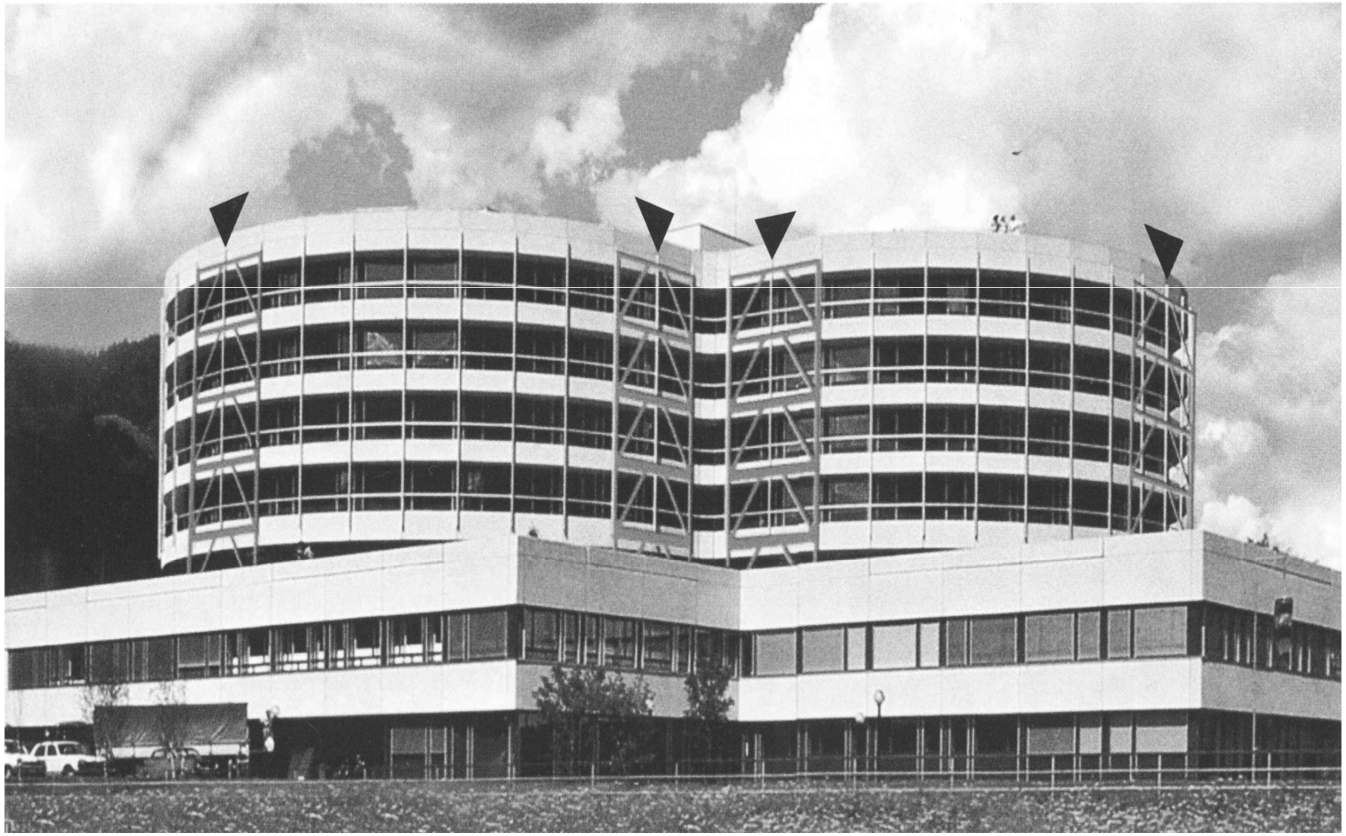
Trusses with centric or eccentric braces

© P. Tissierès



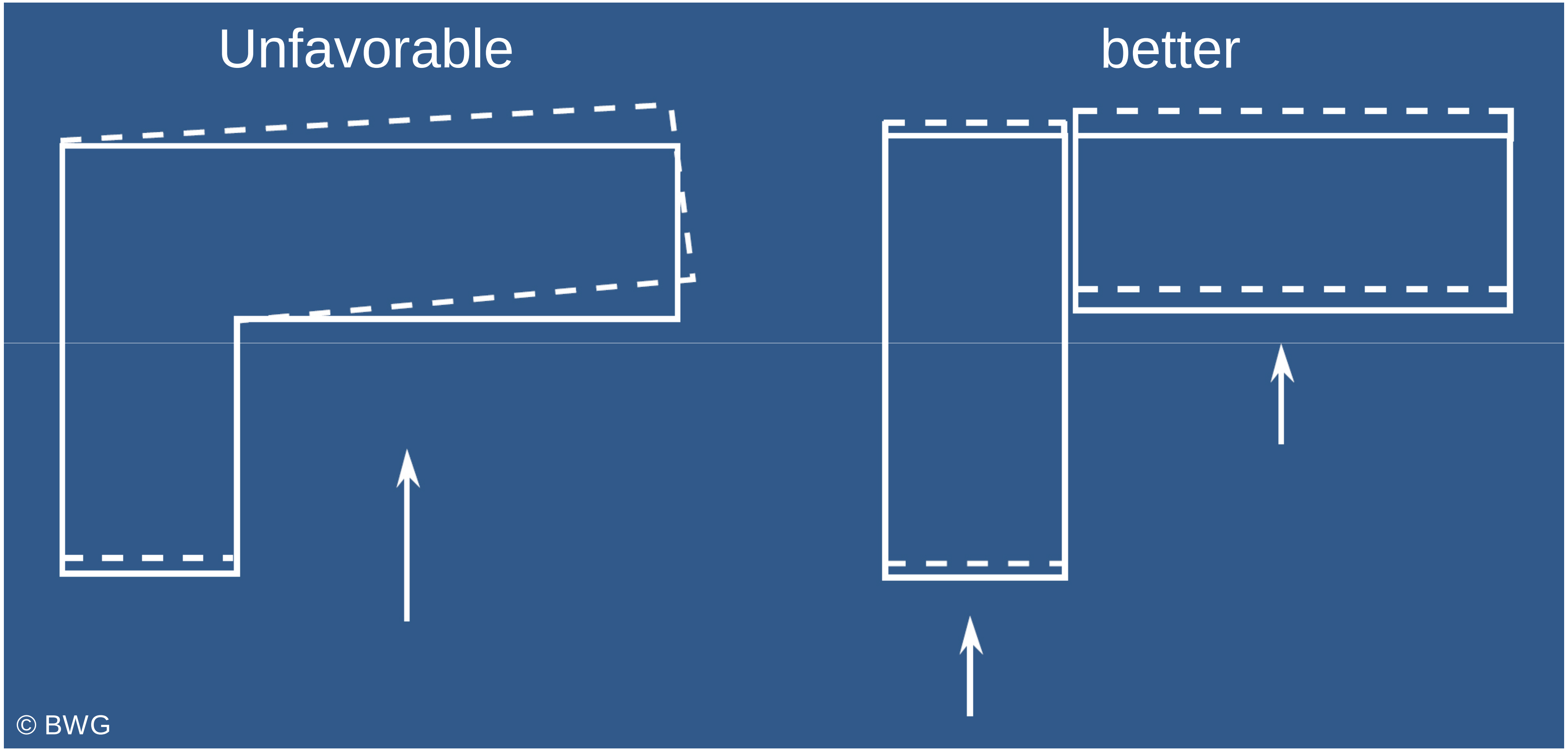
Children's hospital in Brig (ag=0.2g)

Seismic retrofit using ductile steel trusses.



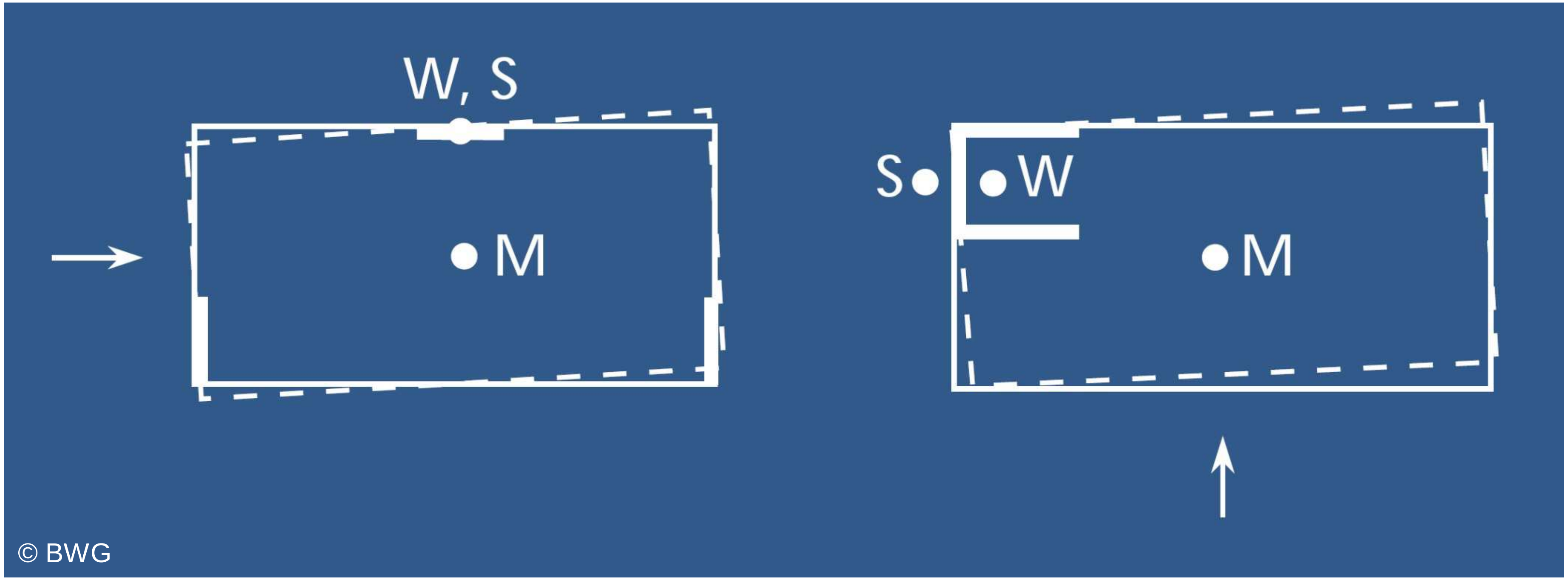
Eccentric joint with shear connectors

Use compact plan configurations!



Use compact plan configurations!

Avoid eccentric bracing systems!



M = Center of mass

W = Center of strength

S = Center of stiffness

Avoid eccentric bracing systems!

© T. Wenk

Past earthquakes have shown that structures with eccentric bracing systems perfom poorly, however ...

© BWG

... structures with eccentric bracing systems keep being built!

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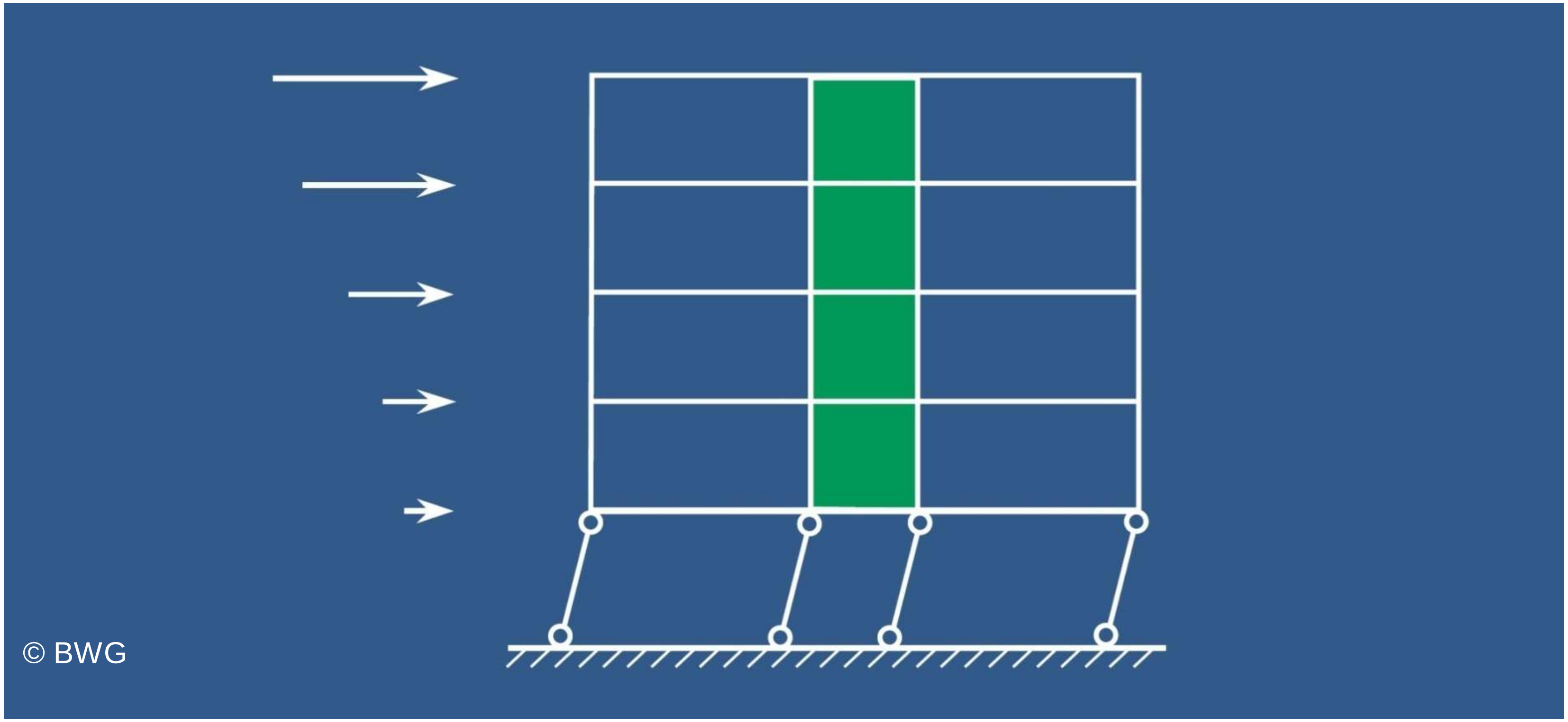
Avoid ground soft-storey!



!!! MOST POPULAR CAUSE OF SEISMIC FAILURE !!!

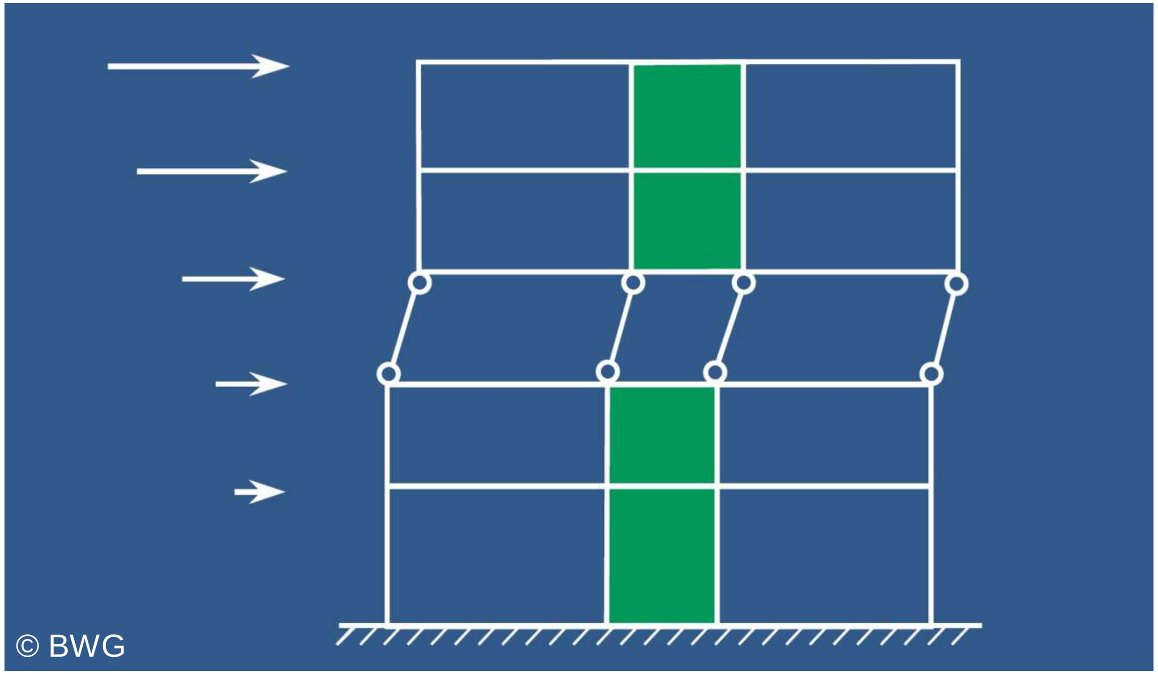
The structural system is “reduced” at the ground floor to increase use friendliness of commercial spaces.

Avoid ground soft-storey!

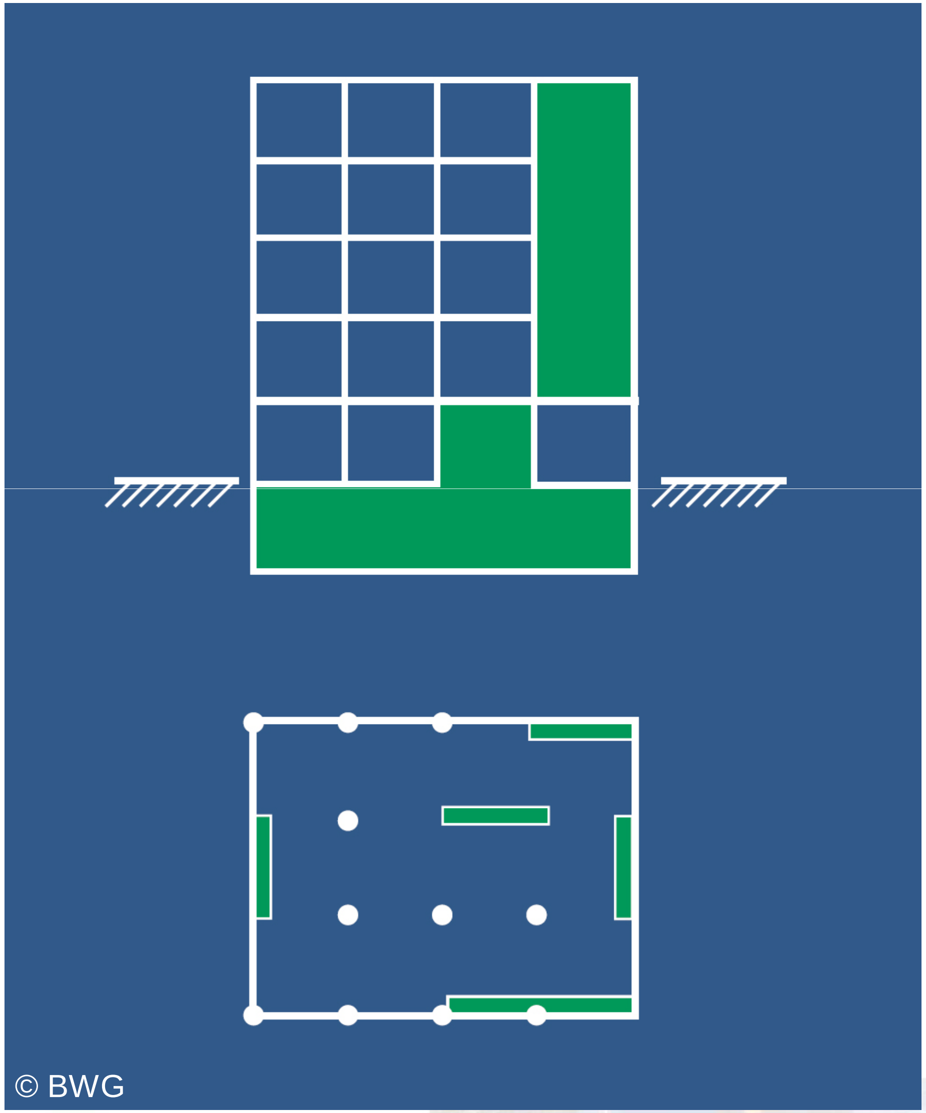
Avoid ground soft-storey!

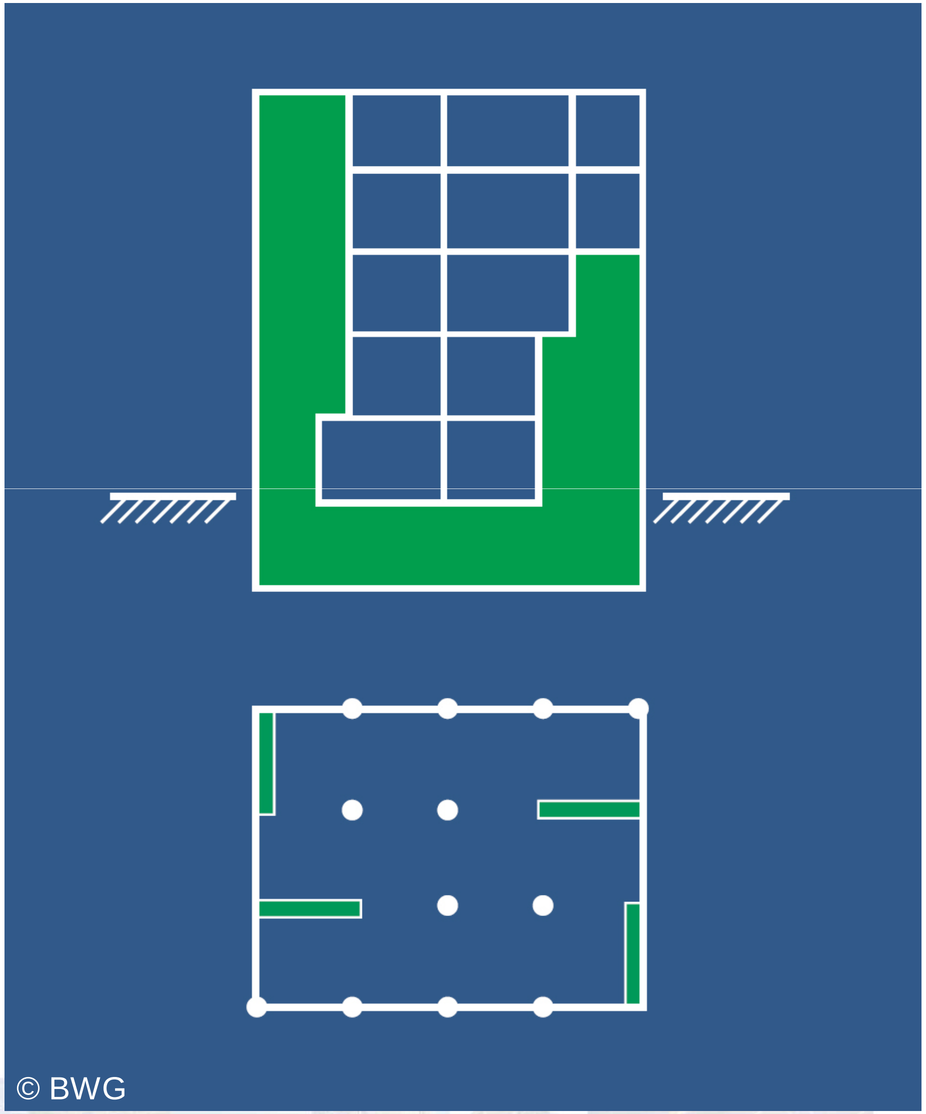
© EERI, Olive View Hospital, San Fernando Earthquake, February, 9 1971

Avoid soft-storey!

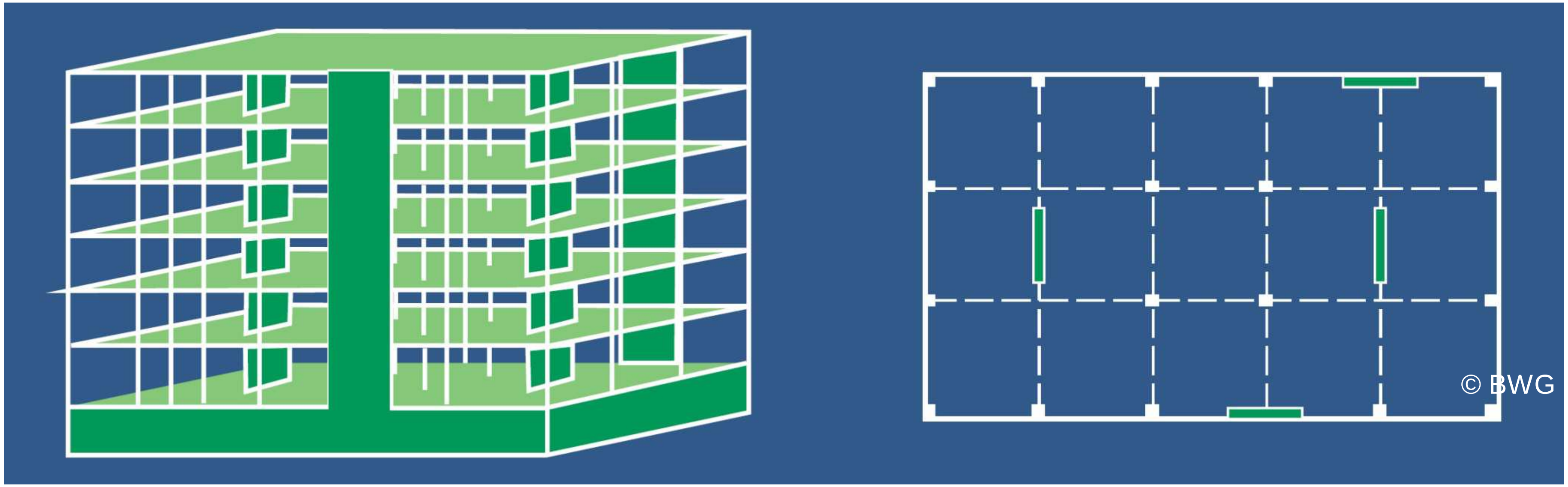


Avoid irregular systems in elevation





Use slender RC structural walls

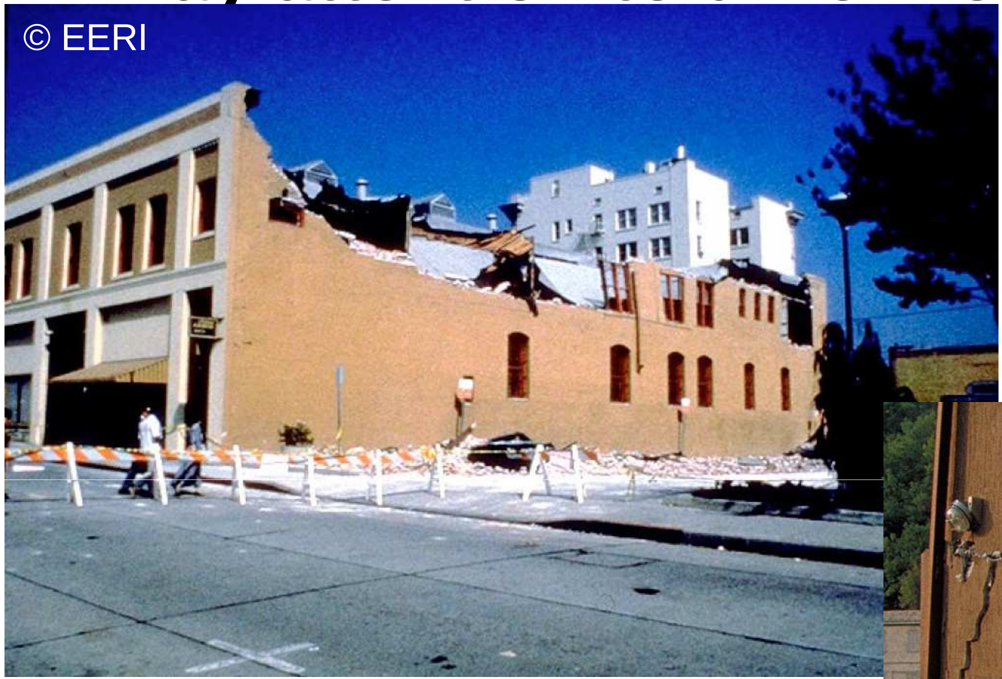


In most cases two slender walls per orthogonal direction ensure:

- a predictable and dependable seismic behavior (economic)
- high structural safety (capacity design)
- high torsional stiffness (especially when walls on the perimeter)
- reduced deformations (less damage to non-structural elements)
- optimum use of the available space

Foundation!!!

Pay attention to unreinforced masonry (URM)!

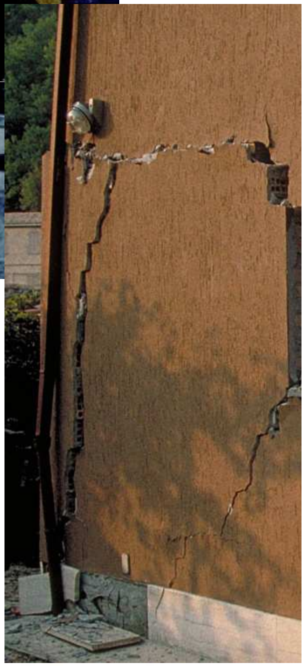


1987 Whittier Narrows Earthquake M = 5.9

Already moderate earthquakes can cause severe damage to unreinforced masonry!

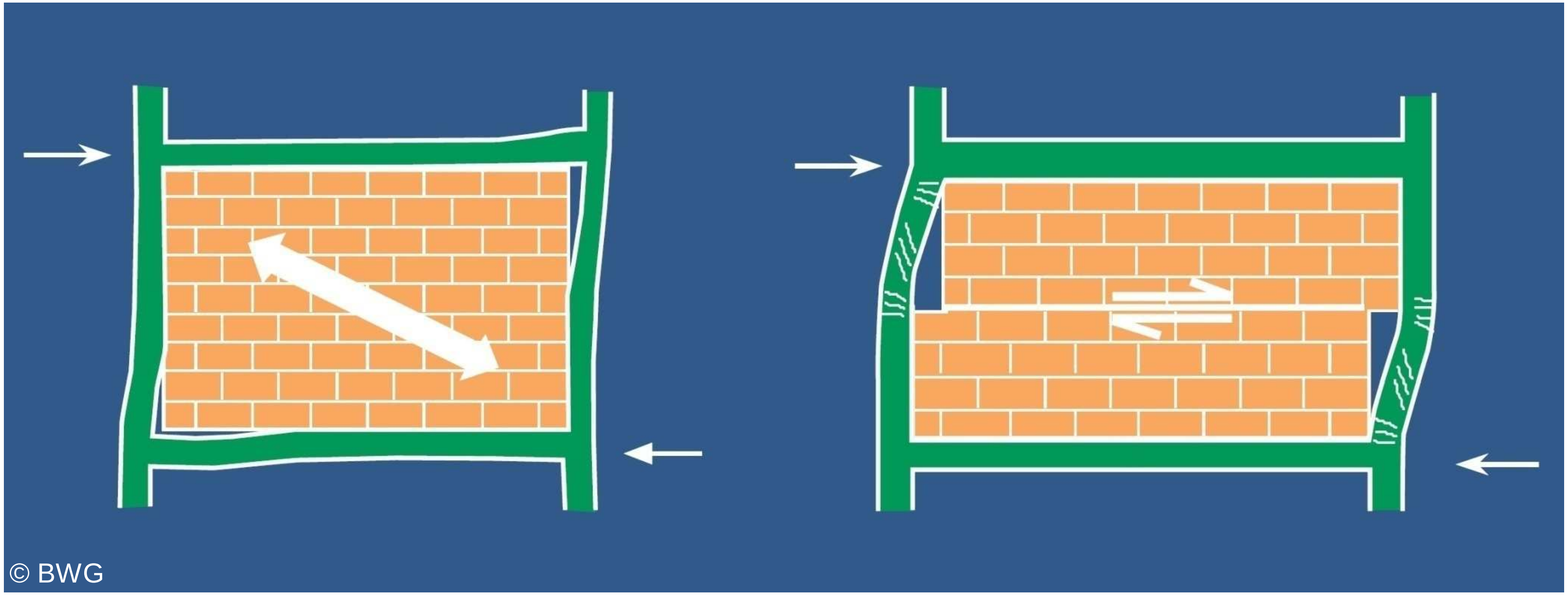
Solutions

- Carry horizontal forces with RC structural walls (stiffness!)
- Reinforced masonry (behave basically like RC)
- Avoid local collapse failure mechanisms



1997 Umbria-Marche Earthquake M = 5.5

Pay attention to masonry infills!



- Structural or non-structural infills?
- Design of the frames!

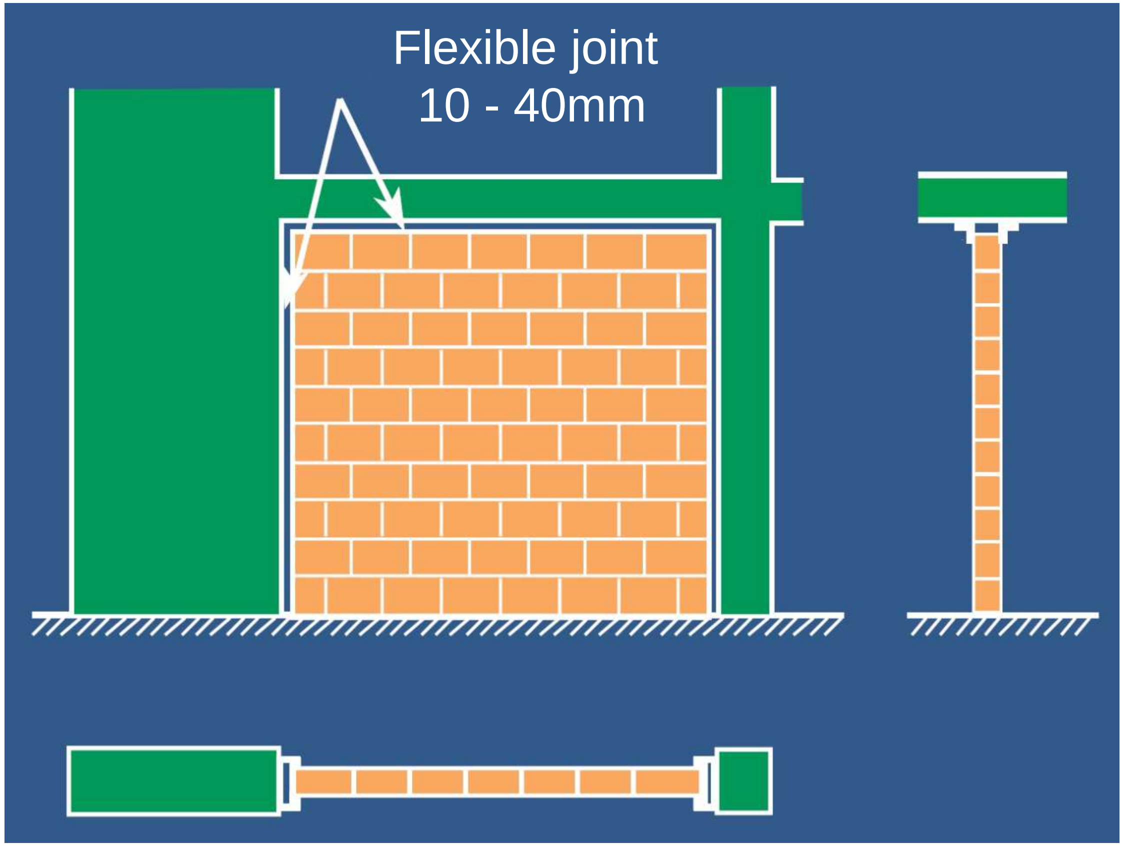
Pay attention to masonry infills!

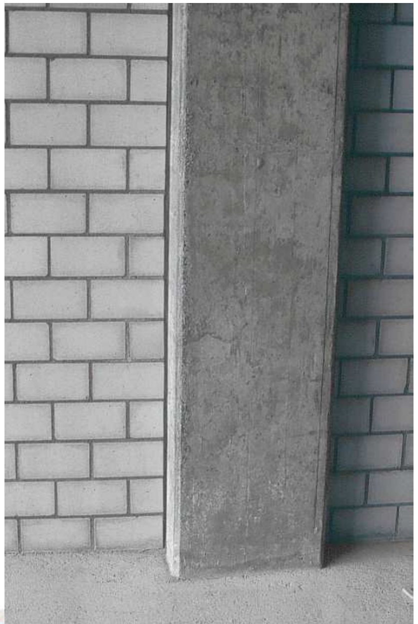
“Strong frame – weak infill”

Structural infills

“Weak frame – Strong infill”

Pay attention to masonry infills!





Non-structural infills
=
Non structural elements

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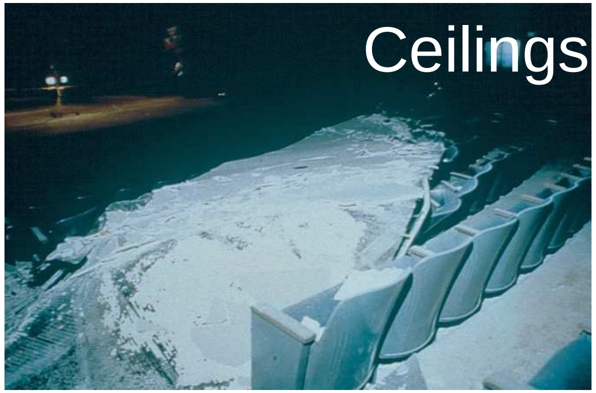
Proper design of non-structural elements

- Non-structural elements are expensive and can be damaged very easily
  - Because of the seismic acceleration
  - Because of the seismic deformations
- The failure of non-structural elements can be dangerous
  - Mechanical effects
  - Emissions
  - Indirect consequences

Damage due to acceleration



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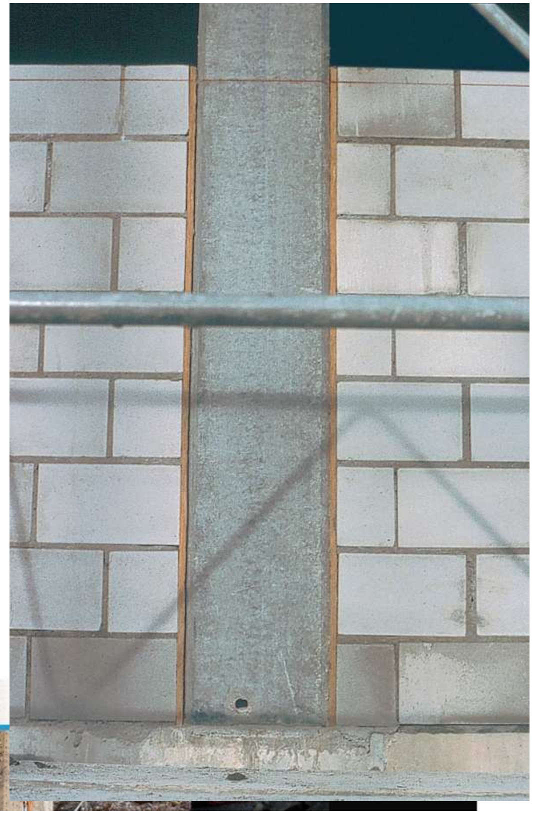
Damage due to deformations

- Limit structural deformations
- Use flexible non-structural elements
- Separate structural and non-structural elements with joints



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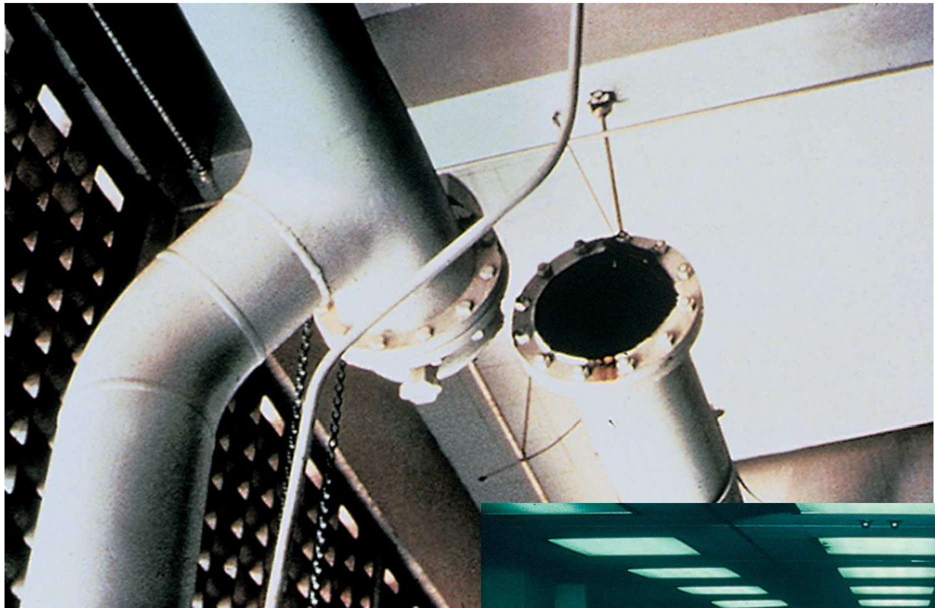
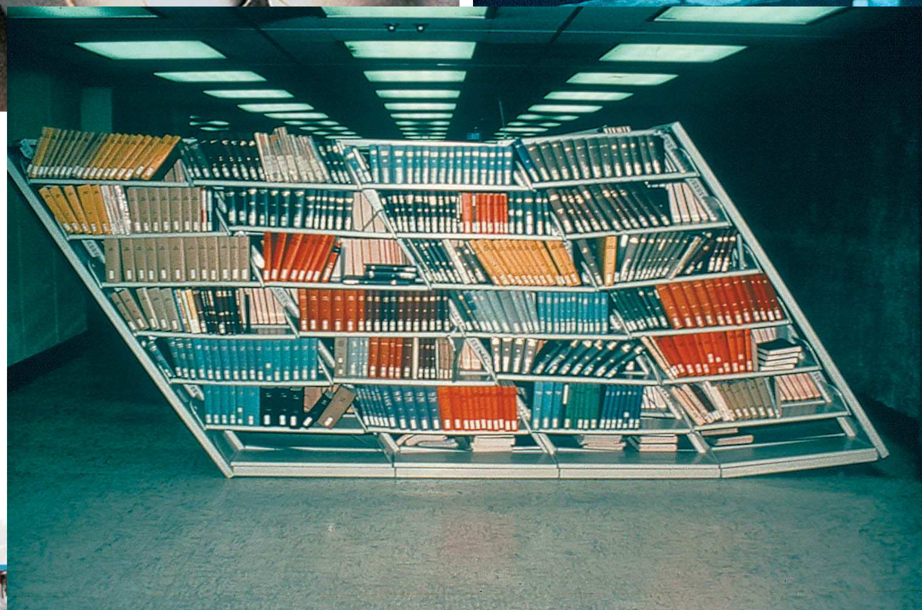
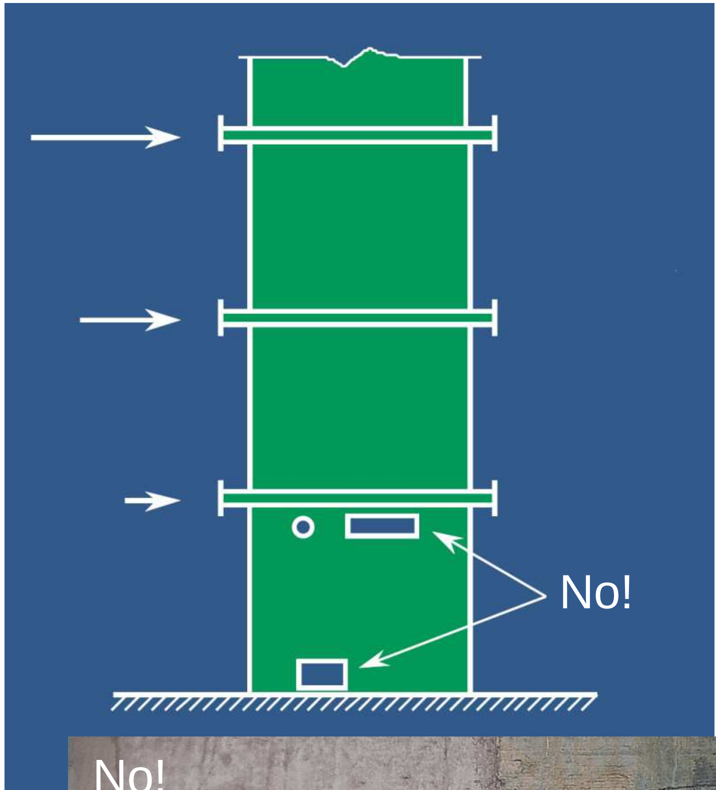
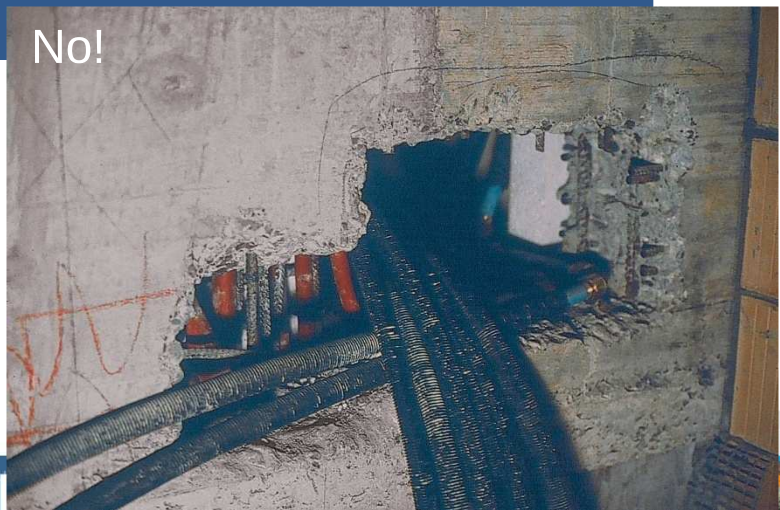
Fasten installations and equipment



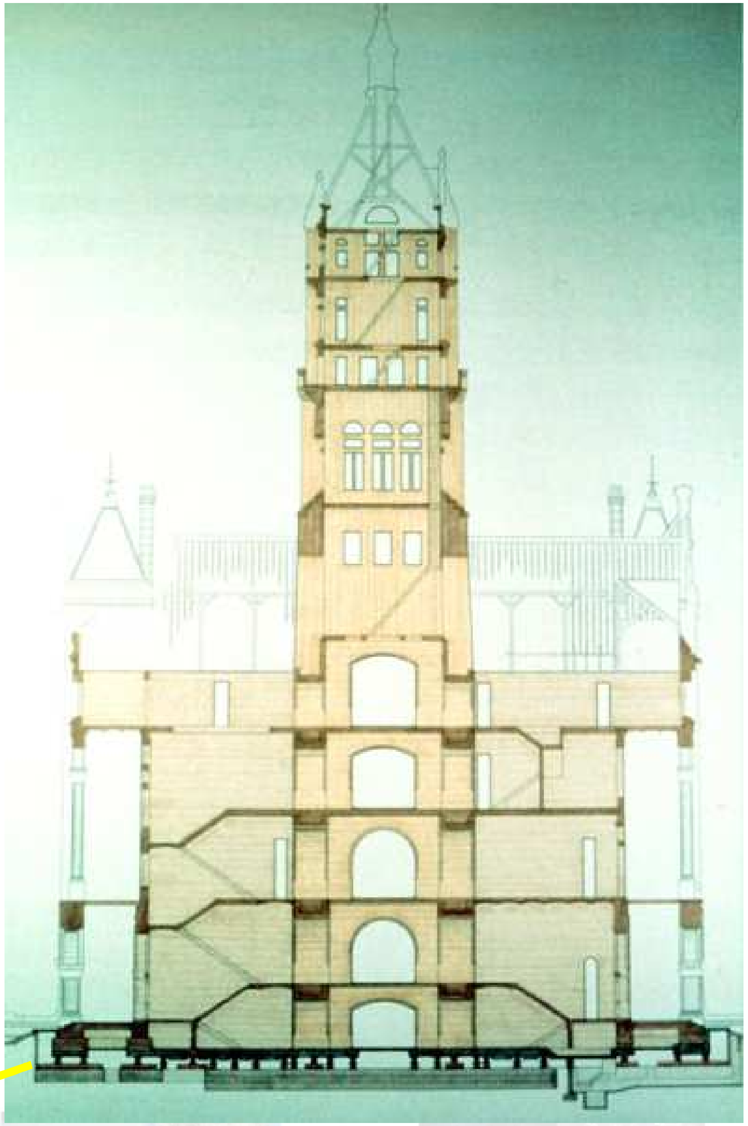


Design installations on time





Important structures can be isolated

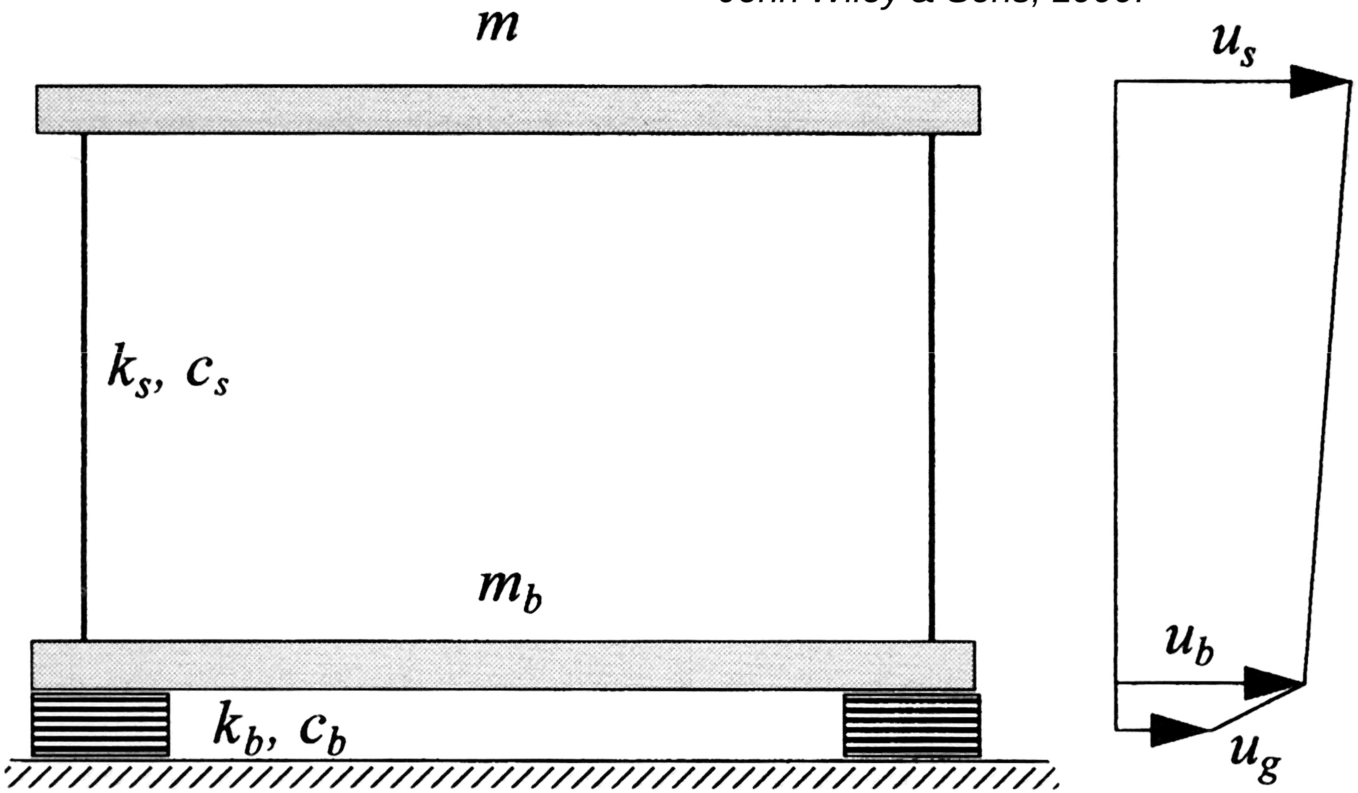


Separate the building

from the ground

Dynamic Model for 2-DoF Isolated System

Naeim F., Kelly J.: Design of Seismic Isolated Structures.
John Wiley & Sons, 1999.



Frequencies

$$\omega_s^2 = \frac{k_s}{m}$$

$$\omega_b^2 = \frac{k_b}{m + m_b}$$

Important Ratios:

$$\boxed{\gamma = \frac{m}{m + m_b} = \frac{m}{M}}$$

$$\boxed{\varepsilon = \frac{\omega_b^2}{\omega_s^2}}$$

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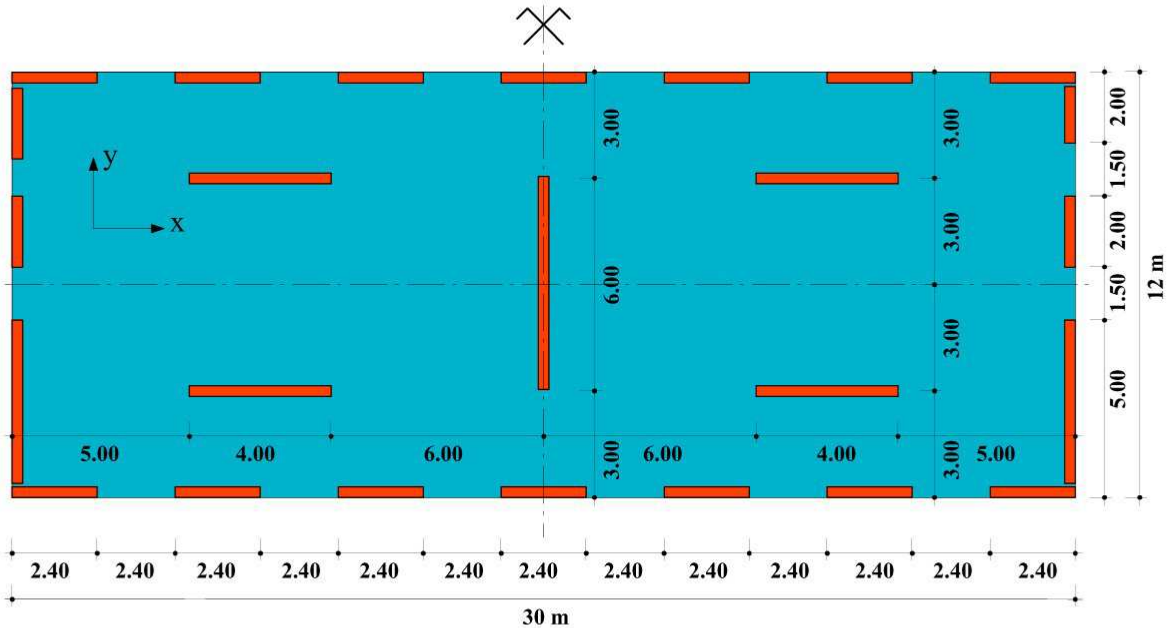
Example masonry bldg

Postgraduated course “Seismic Design of Building Structures”
Dr. Alessandro Dazio

Building

Modal mass	893 t
Mass basement	250 t
Stiffness k_s	130‘589 kN/m
Period T_1	0.52 s (f_1=1.92Hz)

$\gamma = 0.78$

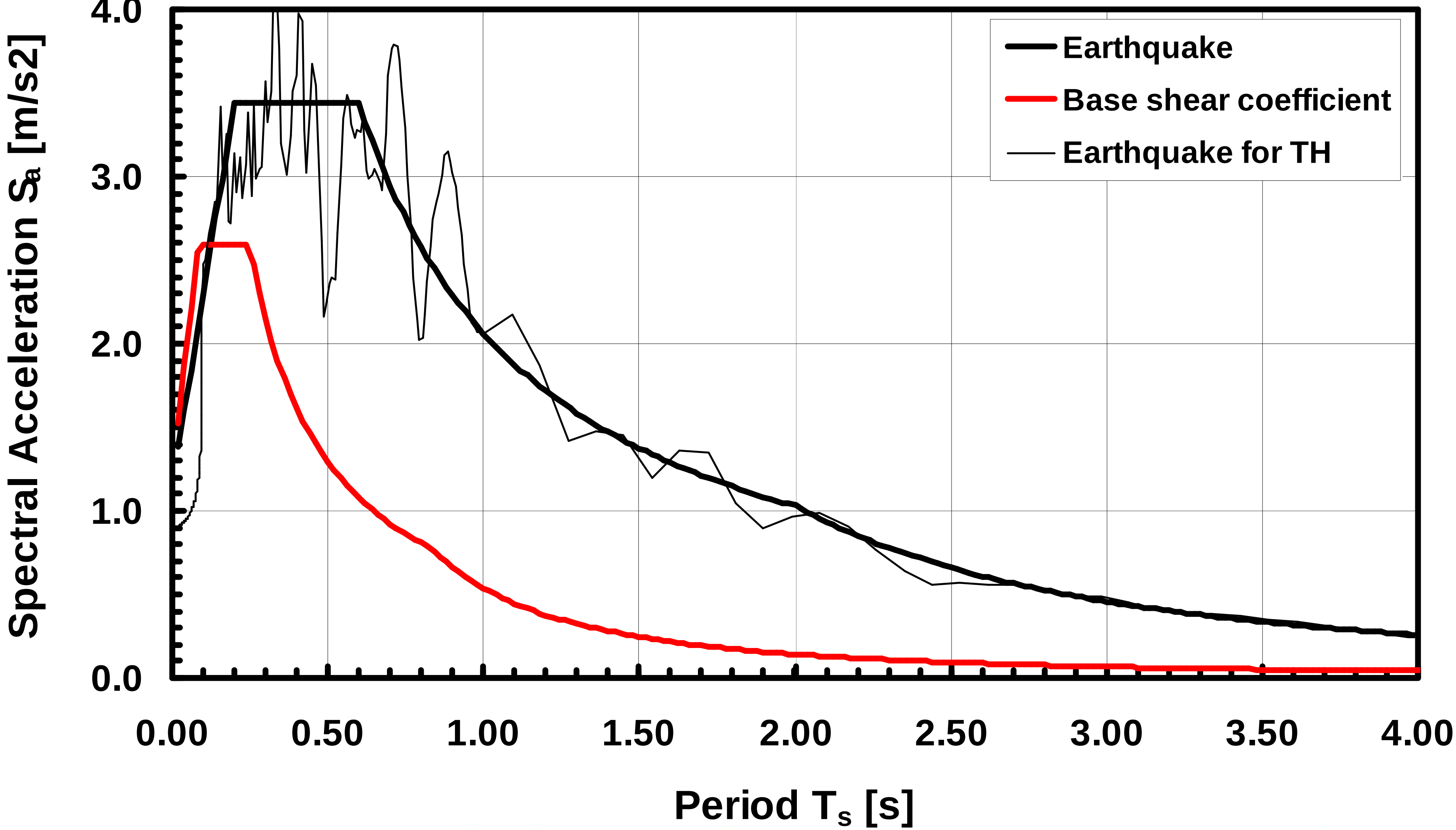


Dampers:

40 pieces: 30x30x10 cm

Stiffness k_b	36‘000kN/m
ε	0.2
Damping β_b	0.10
Damping c_b	1281 t/s

Acceleration spectra ($\gamma = 0.8$, $\varepsilon = 0.2$)



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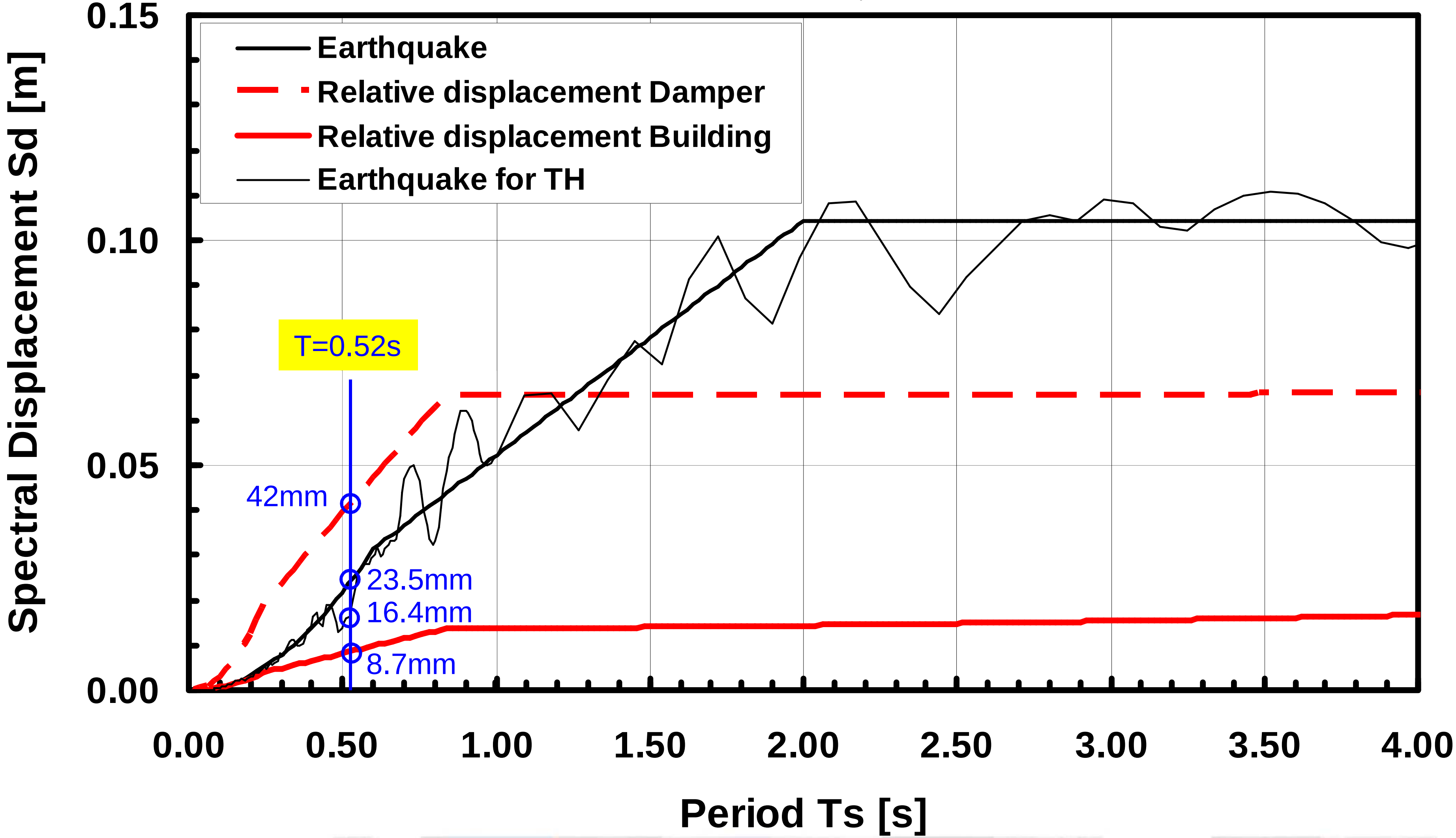
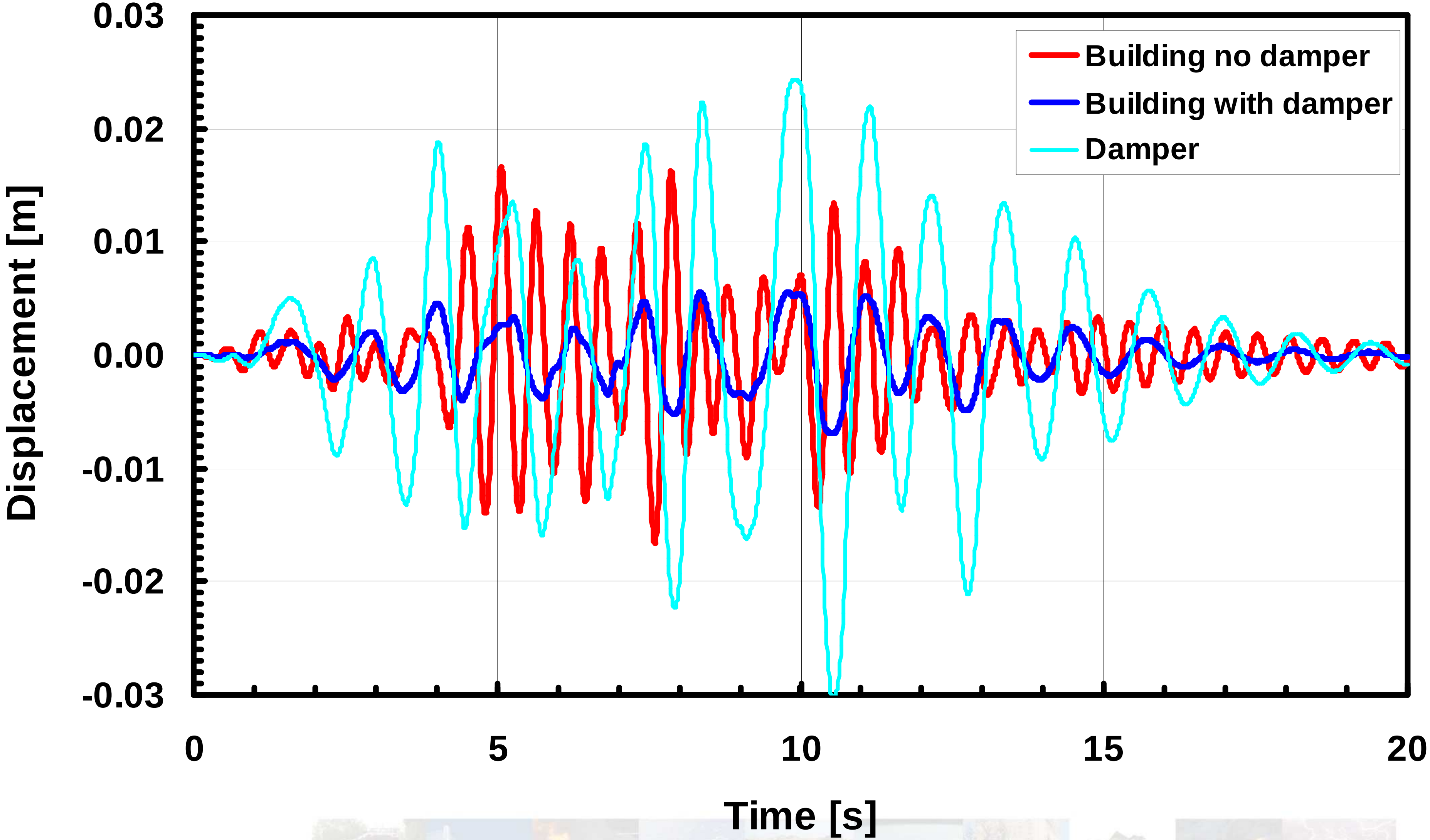
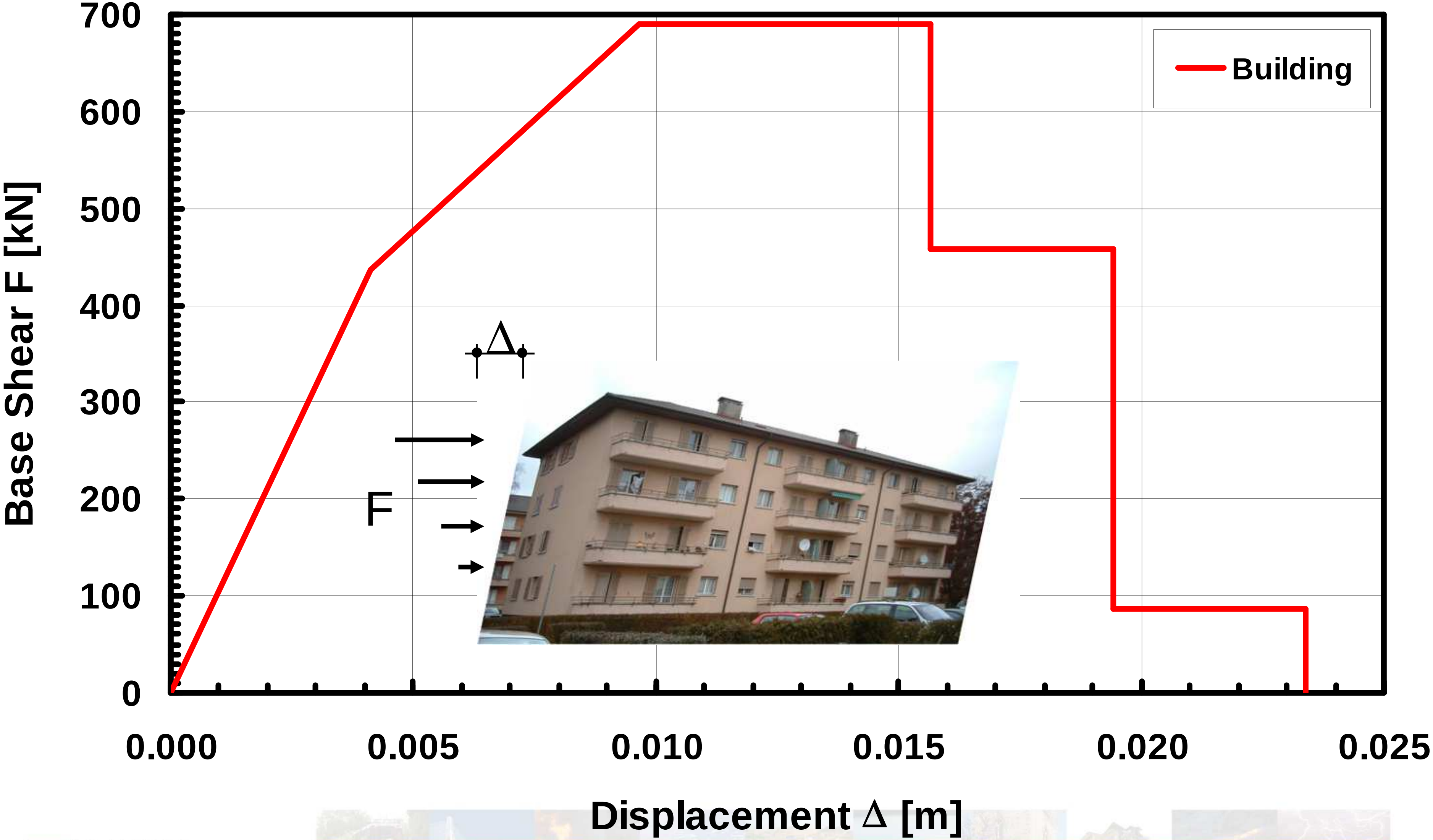
Displacement spectra (γ = 0.8 , ε = 0.2)



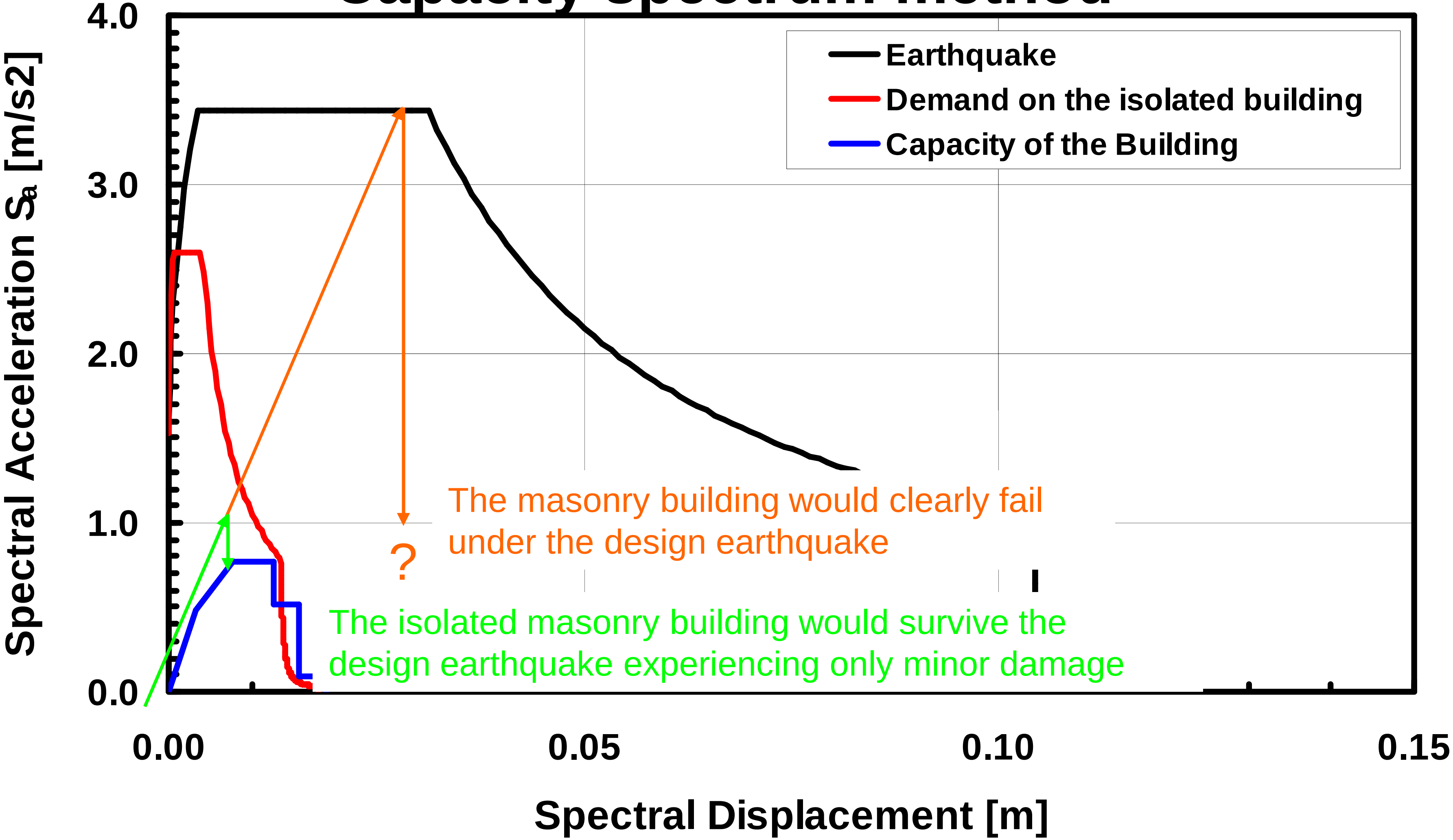
Time-history analysis (SAP2000®)



Pushover curve of the masonry building



Capacity spectrum method



Fajfar P.: CAPACITY SPECTRUM METHOD BASED ON INELASTIC DEMAND SPECTRA. Earthquake Engng. Struct. Dyn. 28, 979-993 (1999)

Close collaboration between Architect and engineer from the earliest planning stage!

- The seismic behavior of structures is extremely complex (cyclic, non-linear, dynamic) and the forces in play are huge!
- Even the most sophisticated design methods can't compensate for conceptual mistakes!
- Capacity design and displacement based design principle can't be use on conceptually wrong structures!
- The "serial design" is typically inefficient because it yields unsatisfactory solutions (structural and non-structural) at extra costs!

Seismic Analysis

Introduction

There are 2 philosophy for seismic analysis:

Force-based design methods

- Force-based procedures are currently the most used design methods in practice, and corresponding provisions are widely available in almost all design codes worldwide.
- For this reason in most of this section force-based procedures are discussed in order to ensure their meaningful application in design practice.
- However, force-based procedures are affected by several drawbacks that should be known in order to avoid serious mistakes.

Displacement-based design methods

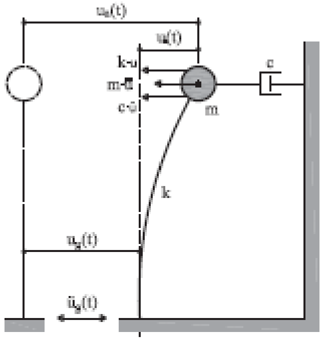
- The intensive development of displacement-based design procedures started about 10 to 15 years ago.
- Such procedures are very promising, however they are still mostly confined to the academic environment.
- In many cases the use of displacement-based design procedures in practice require the engineer to deviate from current code provisions and take full responsibility for that.
- However, in the near future they will find more and more space in design codes. Hence it is important to be aware of some principles of displacement based design method and they will be given in this lecture

Force Based Design Method

Overview of Force Based Design Method

	Equivalent lateral force method	Response spectrum method	Non linear static analysis	Non lineal time history analysis
Dynamic Model	Linear SDOF system	Linear MDOF system	Non linear SDOF system	Non linear MDOF system
Material Model	Linear	Linear	Non linear	Non linear
Modes of vibration considered	Fundamental mode only	All	Fundamental mode only	---
Consideration of material non linearities	Q-factor	Q-factor	Non lineal model	Non linear model
Seismic action	Design spectrum	Design spectrum	Design spectrum	Time history
Typical application	Design	Design	Assessment	Design/Assessment
Effort	Low	Medium	Medium	High

Overview of Structural Dynamic
SDOF System



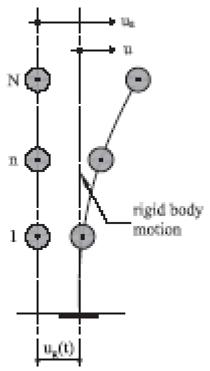
u_a absolute displacement
u relative displacement
u_g ground displacement
k stiffness
c damping
m mass

The equilibrium equation is:

$$m\ddot{u}_a + c\dot{u} + ku = 0 \quad \rightarrow \underbrace{m\ddot{u} + c\dot{u} + ku}_{\text{Relative equations}} = \underbrace{-m\ddot{u}_g}_{\text{Effective earthquake force}}$$

This equation is a second order inhomogeneous differential equation and can be solved analytically for simple excitations (harmonic). For seismic excitation, it is typically solved numerically either solving the Duhamel integral or integrating with a numerical method the equation of motion

MDOF System



M mass matrix
K stiffness matrix
C damping matrix (usually defined as a combination of **M** and **K**)
1 unitary vector

The equilibrium equation is:

$$\mathbf{M}\ddot{\mathbf{u}}_{\mathbf{a}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{K}\mathbf{u} = \mathbf{0} \quad \text{if } \ddot{\mathbf{u}}_{\mathbf{a}} = \ddot{\mathbf{u}} + \mathbf{1}\ddot{u}_g \rightarrow \mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{K}\mathbf{u} = -\mathbf{M}\mathbf{1}\ddot{u}_g$$

Background on mode of vibration

Free vibration of non dissipative MDOF systems:

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{K}\mathbf{u} = \mathbf{0}$$

Let us express the solution as:

$$u_i(t) = \boldsymbol{\Phi}_i \, q(t) \qquad i = 1..n$$

Substituting in the equation of motion:

$$\mathbf{M}\,\boldsymbol{\varphi}\,\ddot{q}(t) + \mathbf{K}\boldsymbol{\varphi}\,q(t) = \mathbf{0} \rightarrow \ \mathbf{M}\,\boldsymbol{\varphi}\frac{\ddot{q}(t)}{q(t)} = -\mathbf{K}\boldsymbol{\varphi}$$

$$\text{posing } \ \frac{\ddot{q}(t)}{q(t)} = -\omega^2 \qquad \rightarrow \ddot{q}(t) + \omega^2 q(t) = 0$$

The solution like for the SDOF system is of the type:

$$q(t) = R\sin(\omega t + \vartheta)$$

Substituting in the equation of motion:

$$(\mathbf{K} - \omega^2\mathbf{M})\boldsymbol{\varphi} = \mathbf{0}$$

if $\left|\mathbf{K} - \omega^2\mathbf{M}\right| \neq 0 \rightarrow \boldsymbol{\varphi} = \mathbf{0}$ that is a solution but not a very interesting one!

We look for the solution of:

$$\left|\mathbf{K} - \omega^2\mathbf{M}\right| = 0$$

ω_i^2 are the eigenvalue of the matrix above and are the pulsation of the MDOF system
$\boldsymbol{\Phi}_i$ are the eigenvector of the matrix above and are the represent the vibration shape of the MDOF system. They are not defined in term of magnitude (if $\boldsymbol{\Phi}_i$ is a solution, also $\alpha\boldsymbol{\Phi}_i$ is a solution)

If we define:

$$\mathbf{M}^* = \boldsymbol{\varphi}^t\mathbf{M}\boldsymbol{\varphi}$$

M* and **K*** are diagonal matrixes because the eigenvector are orthogonal

$$\mathbf{K}^* = \boldsymbol{\varphi}^t\mathbf{K}\boldsymbol{\varphi}$$

The equation of motion becomes:

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{K}\mathbf{u} = -\mathbf{M}\,\mathbf{1}\ddot{u}_g \rightarrow \boldsymbol{\varphi}^t\mathbf{M}\boldsymbol{\varphi}\ddot{\mathbf{q}} + \boldsymbol{\varphi}^t\mathbf{K}\boldsymbol{\varphi}\boldsymbol{\varphi} = -\boldsymbol{\varphi}^t\mathbf{M}\,\mathbf{1}\ddot{u}_g \rightarrow \mathbf{M}^*\ddot{\mathbf{q}} + \mathbf{K}^*\mathbf{q} = -\boldsymbol{\varphi}^t\mathbf{M}\,\mathbf{1}\ddot{u}_g$$

Important outcome!

- The equation of motion of the original system is transformed in n equation of motion of SDOF systems, with n the number of degree of freedom of the MDOF system
- If the system is dissipative, but $\mathbf{\Phi}^t\mathbf{C}\mathbf{\Phi}$ is diagonal (this is certainly the case when **C** is expressed as **C**=α**M**+β**K**) what is true for the non dissipative MDOF system is valid also for the dissipative one
- n can be less than the number of degree of freedom of the system, because if n is big enough, the modes above n do not contribute to the response of the system

Modal equation of motion:

$$\ddot{q}_i + 2\nu_i\omega_i\dot{q}_i + \omega_i^2 q_i = \frac{-\phi_i^t\mathbf{M}\,\mathbf{1}}{\phi_i^t\mathbf{M}\phi_i}\ddot{u}_g \xrightarrow[\substack{\Gamma_i = \frac{\phi_i^t\mathbf{M}\,\mathbf{1}}{\phi_i^t\mathbf{M}\phi_i} \\ \text{modal partecipation facor}}]{} \ddot{q}_i + 2\nu_i\omega_i\dot{q}_i + \omega_i^2 q_i = -\Gamma_i\ddot{u}_g$$

If we define the" effective modal mass" like:

$$\boxed{m^*_{i,eff} = \Gamma_i^2 m^*_i \xrightarrow[\text{if n is big enough}]{} \sum_i^n m^*_{i,eff} \rightarrow m_{tot} = \text{total mass of the system}}$$

Criterion to select
the n of vibration modes to consider

Equivalent lateral forces

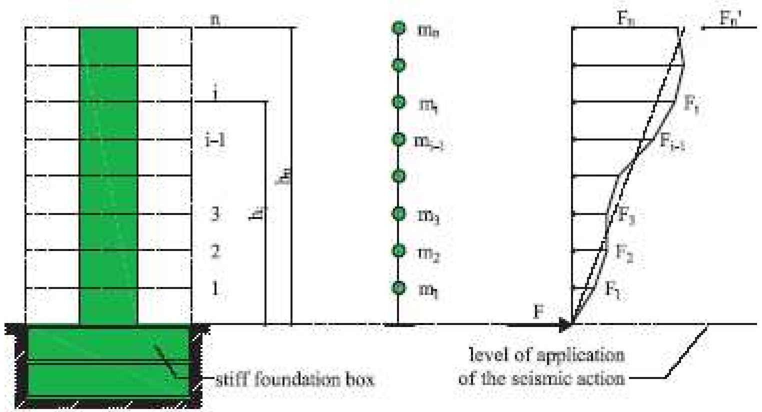
- It is a simple linear static analysis method allowed by almost all design codes worldwide.
- The dynamic action of the earthquake on the structure is replaced by lateral static forces called “equivalent lateral forces”.
- The methods can be applied to structural systems which can be represented by two 2D structural models, and which behaviour is not substantially influenced by higher modes of vibration.
  - Criteria for regularity shall be met
  - The fundamental period should be smaller than 2 seconds
  - Higher vibration modes are neglected.
- The fundamental period of vibration of the building can be estimated by means of simple equations or by means of a proper structural model. The second option is recommended.
- Torsional effects are taken into account in an approximate way (e.i. increasing the forces to account for torsion)
- The inelastic behaviour of the structure is also taken into account in an approximate way through inelastic design spectra.

$F = \lambda\, m_{tot}\, S_a\, (T_1, \nu, q, \gamma_f) \rightarrow$ total equivalent lateral force

m_{tot} : total mass of the building
S_a : spectral value taken from the design acceleration spectrum at the fundamental period of the building T_1; the spectrum is computed assuming a damping ratio ν and a behaviour factor q. Additional parameters, like e.g. the importance factor γ_f can be taken into account while computing the spectral ordinate
λ : Correction factor ranging between 0.85 and 1.0. Some codes (e.g. EC8) use to account for the fact that in buildings with at least three storeys and translational degrees of freedom in each horizontal direction, the effective modal mass is about 85% of the total mass.

F shall be distributed along the building height

$$F_i = \left(F - F_n^{'}\right)\frac{m_i h_i}{\sum_{j=1}^{i=n} m_j h_j}$$



F_i , m_i, h_i: equivalent lateral force, mass and height on floor i
F'_n: special single force acting at the top of the building. Some codes use this force to increase the shear force in the upper storeys and the bending moment at the base

Modeling issues

1) Substitute beam

The simplest structural model which allows the analysis of the seismic action on a building is the cantilever substitute beam. The substitute beam runs through the centres of stiffness of all storeys.

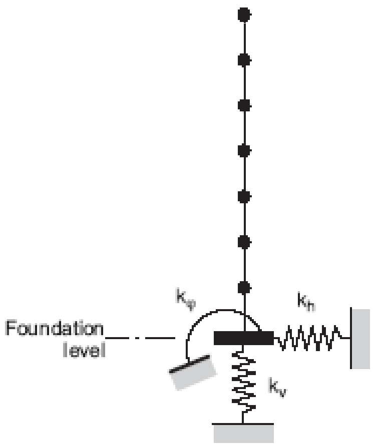
The clamping of the substitute beam is set in correspondence of the fix horizon. The fix horizon corresponds to the location where for the first time a storey floor provides a relatively stiff horizontal bearing. In cases where the foundation of the building features a stiff basement, the fix horizon normally corresponds to the level of the ground storey floor. Independently of the location of the fix horizon, internal forces must be tracked and considered until they are introduced into the ground.

The total mass of the building is distributed among the storey masses acting at the level of the storey floors.

2) Underground

The flexibility of the underground in many cases is not considered. With soft soils, the clamping of the substitute beam can be modelled by means of springs. In this case the fix horizon shall be taken at the foundation level.

For additional information on soil-structure-interaction see e.g.[Cho07] and [Kra96]



Spring stiffness for circular foundation :

$$\text{Vertical} \quad \rightarrow k_v = \frac{4\,G\,r}{1-\nu}$$

$$\text{Horizontal} \quad \rightarrow k_h = \frac{32(1-\nu)\,G\,r}{7-8\nu}$$

$$\text{Rotation} \quad \rightarrow k_\varphi = \frac{8G\,r^3}{3(1-\nu)}$$

G shear modulus

ν Poisson ratio

r Radius of foundation

3) Regularity

To use the method of equivalent lateral force, the building is supposed to be regular in plan and elevation. The design codes give the criteria to consider a building as regular in plan and elevation.

4) Stiffness

The members used to assemble the structural model should be characterized by a realistic stiffness up to yielding. For members made of masonry or reinforced concrete the effect of cracking should be properly taken into account.
Computations carried out using a stiffness based on the properties of the uncracked sections result normally in a gross overestimation of the sectional forces and in a gross underestimation of the deformations.

4) Fundamental period

Crude formula:

Plenty of crude formula exists to estimate the fundamental period of vibration of structures

T_1= 0.1 sec x number of storey

Eurocode 8:

$T_1 = C\,h^{0.75}$

With C=0.085 for steel frame buildings, C=0.075 for RC frame buildings, C=0.05 for all the other buildings

Rayleigh formula:

The natural vibration frequency of structures can be estimated by means of the Rayleigh method. The method is based on the principle of conservation of energy (see [Cho07]). If d_i is the horizontal displacement of the floor i:

$$\frac{1}{T_1} = f_1 = \frac{1}{2\pi}\sqrt{\frac{\sum_i^n F_i d_i}{\sum_i^n m_i d_i^2}}$$

Please not that the shape of the distribution of F_i does have an influence on T_1, while the magnitute of the forces does not.

Finite element analysis:

Perform a modal analysis to define the first vibration mode.

Responce spectrum analysis

If the maximum response only and not the response to the entire time history is of interest, the response spectrum method can be applied.
The method is based on the combination of the contribution of each vibration mode to the seismic performance of the building.
The response spectrum can be computed for the considered seismic excitation and the maximum value of the modal coordinate can be determined as follows:

$$q_{n,max} = \Gamma_n S_d(\omega_n, \nu_n) = \Gamma_n \frac{1}{\omega_n^2} S_a(\omega_n, \nu_n) \xrightarrow[\substack{\text{contribution of n-mode} \\ \text{to total displacement}}]{} u_{n,max} = \phi_n q_{n,max}$$

Where:
Γ_n : modal participation factor of the n-th mode
S_d : Spectral **displacement for the circular eigenfrequency** and the modal damping rate
S_a : Spectral **acceleration for the circular eigenfrequency** and the modal damping rate

The maxima of different modes do not occur at the same instant. An exact computation of the total maximum response on the basis of the maximum modal responses is hence impossible. Different methods have been developed to estimate the total maximum response from the maximum modal responses.

Combination rules:

"Absolute Sum (ABSSUM)" Combination Rule

$$u_{i,max} \leq \sum_{1}^{n} \phi_{ij} q_{j,max}$$

The assumption that all maxima occur at the same instant and in the same direction yields an upper bound value for the response quantity. This assumption is commonly too conservative.

"Square-Root-of Sum-of-Squares (SRSS)" Combination Rule

$$u_{i,max} = \sqrt{\sum_{1}^{n} \left(\phi_{ij} q_{j,max}\right)^2}$$

This rule is often used as the standard combination method and yields very good estimates of the total maximum response if the modes of the system are well separated. If the system has several modes with similar frequencies the SRSS rule might yield estimates which are significantly lower than the actual total maximum response.

• “Complete Quadratic Combination (CQC)” Combination Rule

$$u_{i,max} = \sqrt{\sum_{j=1}^{n}\sum_{k=1}^{n} u_{i,max}^{(j)} u_{i,max}^{(k)} \rho_{jk}}$$

where :

$u_{i,max}^{(j)}$ and $u_{i,max}^{(k)}$ are the max modal responce of mode j and mode k

ρ_{jk} is the modal correlation coefficient between mode j and mode k

$$\rho_{jk} = \frac{8\sqrt{\nu_i \nu_k}\left(\nu_i + r\nu_k\right) r^{3/2}}{\left(1-r^2\right)^2 + 4\nu_i\nu_k r\left(1-r^2\right) + 4\left(\nu_i^2 + \nu_k^2\right) r^2}$$

This method based on random vibration theory gives exact results if the excitation is represented by a white noise. If the frequencies of the modes are well spaced apart, the result converge to those of the SRSS rule. More detailed information on this and other combination rules can be found in [Cho07]

Internal forces: the combination rules used showed for displacements are valid also for internal forces. **It is wrong to compute the maximum internal forces directly by the maximum displacement!**

Number of modes to be considered:

All modes which contribute to the dynamic response of the system should be considered. In practical applications, however, only those modes are considered which contribution to the total response is above a certain threshold. It should be noted that in order to achieve the same accuracy for different response measures (e.g. displacements, shear forces, bending moments, etc.) different numbers of modes might need to be considered in the computation. For a regular building the top displacement can be estimated fairly well on the basis of the fundamental mode only. To estimate the internal forces, however, higher modes need to be considered too.

According to Eurocode 8 “Design of Structures for Earthquake Resistance” [CEN04] all modes should be considered (starting from the lowest) until the sum of the effective modal masses of all considered modes corresponds to at least 90% of the total mass . As an alternative, Eurocode 8 allows to consider all modes where the effective modal mass is higher than 5%.

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Step by step procedure:

The maximum response of a N-storey building can be estimated according to the following procedure:

1) Determine the properties of the MDOF system
- Choose DOFs
- Determine mass matrix **M** and stiffness matrix **K**
- Estimate modal damping ratios ν_n

2) Carry out modal analysis of the MDOF system
- Determine circular eigenfrequencies ω_n and eigenvectors ϕ_n
- Compute the modal properties of the MDOF system (**M***, **K***)
- Compute the modal participation factor Γ_n

3) The maximum response of the n-th mode should be determined as described in the following. This should be done for all modes which require consideration.
- For all periods T_n and for the corresponding damping ratios ν_n, the spectral response $S_a(T_n, \nu_n)$ should be determined from the response spectrum for pseudo-accelerations. (The spectral displacement should be determined in the same manner)

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- Compute the maximum displacement:

$u_{n,max} = \phi_n \Gamma_n S_d(T_n, \nu_n)$

- Compute the maximum equivalent static force:

$F_{n,max} = \Gamma_n \mathbf{M} \phi_n S_a(T_n, \nu_n)$

- Computation of the maximum internal forces on the basis of the forces $F_{n,max}$

4) Estimate the total response in terms of displacements and internal forces by means of suitable combination rules. Different combination rules might be applied (ABSSUM, SRSS, CQC).

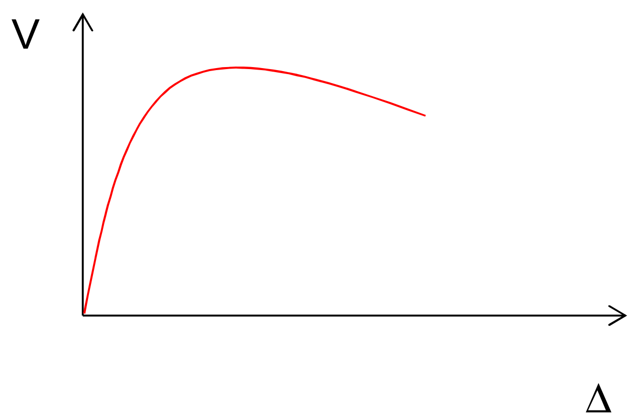
Comment

In order to consider the non-linear behaviour of the structure the equivalent lateral static forces can be determined from the spectral ordinate of the **design spectrum for pseudo** accelerations:

$$F_{n,max} = \Gamma_n \mathbf{M} \phi_n S_a(T_n, \nu_n, q)$$

Static to collapse analysis

Starting point: A distribution of lateral forces is applied to the structure
The lateral forces are increased up to up to when the structure reaches the collapse limit condition.



Where:
V base shear force
Δ displacement in a control point of the structure

A curve base shear force – versus displacement is known as pushover curve

Please note that in case of softening behaviour, after the peak of the pushover curve, the analysis must follow till the collapse limit condition is achieved in control of displacement an not in control of forces. Hence, the displacements increase, while the applied forces reduce in magnitude

From the capacity curve it is possible to define the properties of an equivalent SDOF system, which is equivalent to the original MDOF system in terms of:

1. Vibration period;
2. Displacement capacity;
3. Amount of dissipated energy.

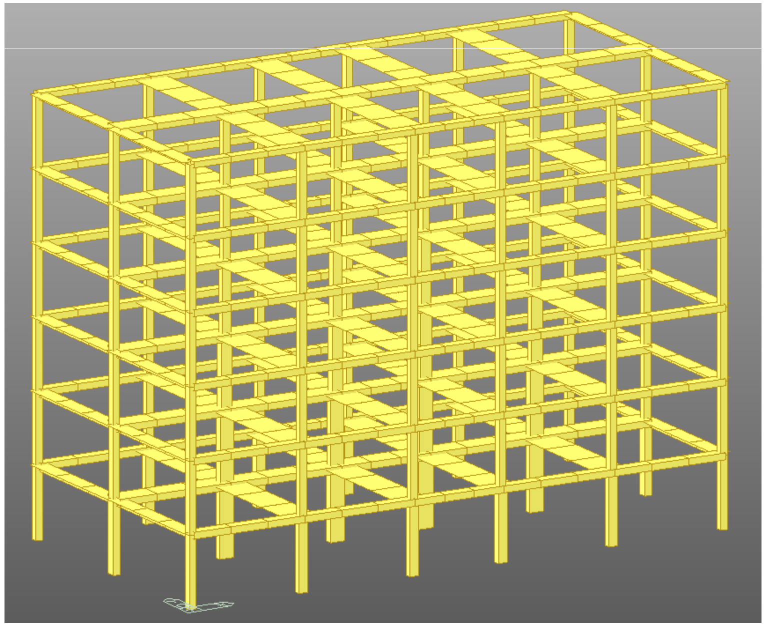
To undertake a pushover analysis, the following choices must be undertaken:

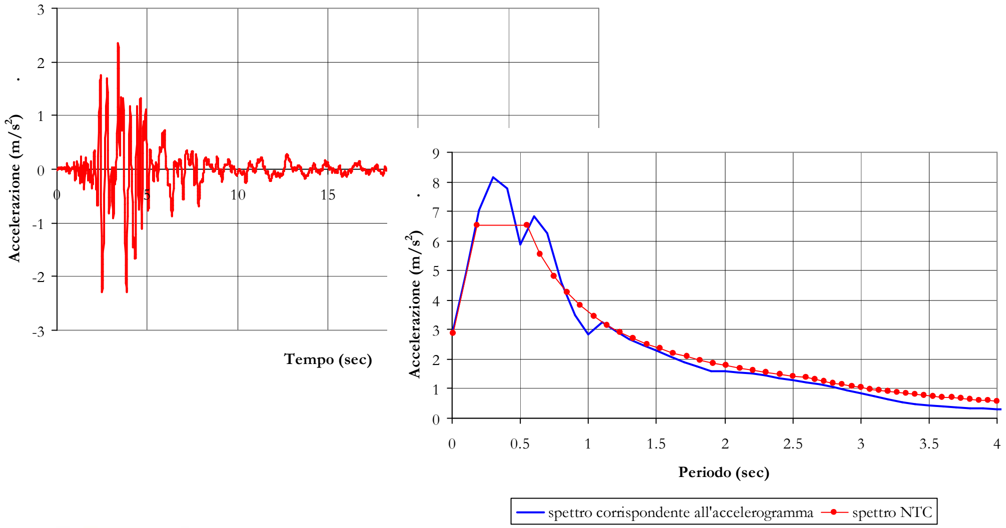
- The distribution of lateral forces;
- Set of rules to pass from the pushover curve of the MDOF system to the corresponding pushover curve of the SDOF system. Such curve is then used to define the properties of the equivalent systems.

The choices mentioned above are driven by the design codes

Non linenar dynamic analysis

Starting point: A non linear FE model of the structure is undertaken.
Such model is subjected to an input motion described by a set of accelerograms. More than one accelerogram need to be taken into account, since the variability of the input motion must be considered. The number of accelerograms and how they must comply in term of spectral compatibility with the elastic spectrum is defined by design codes.
In nonlinear dynamic analyses the equation of motion are integrated through numerical integration method





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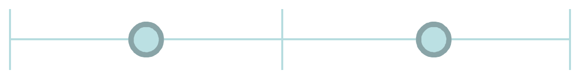
Basics of numerical integration methods: instead of imposing the equilibrium along a continuum time axes, the equilibrium is imposed only in selected times, separated by an interval ΔT, which represent the integration time step. Within the integration time step, assumptions are undertaken to define how the velocity and the acceleration vary.

Numerical methods can be conditionally or unconditionally stable. If the method is conditionally stable, ΔT shall be small enough to guarantee the numerical stability of the solution.

$$\dot{\mathbf{u}} = \dot{\mathbf{u}}\left(t_{\mathbf{k}} + \frac{\Delta t_{\mathbf{k}}}{2}\right)$$

$$\ddot{\mathbf{u}} = \ddot{\mathbf{u}}(t_{\mathbf{k}})$$



$t_k - \Delta t_{k-1}$ t_k $t_k + \Delta t_k$

Example of central difference integration method on the assumption of variability of accelerationwithin the integration time step (e.i. the acceleration is constant from $t_k - \Delta t_{k-1}/2$ and $t_k + \Delta t_k/2$

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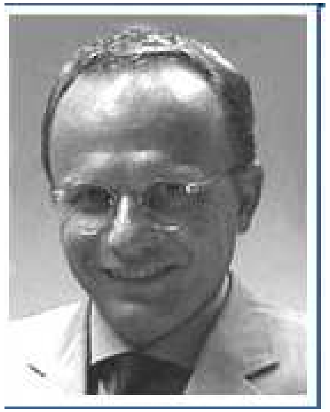
Direct Displacement Based Design Method

- **Key players**


- **Main references**
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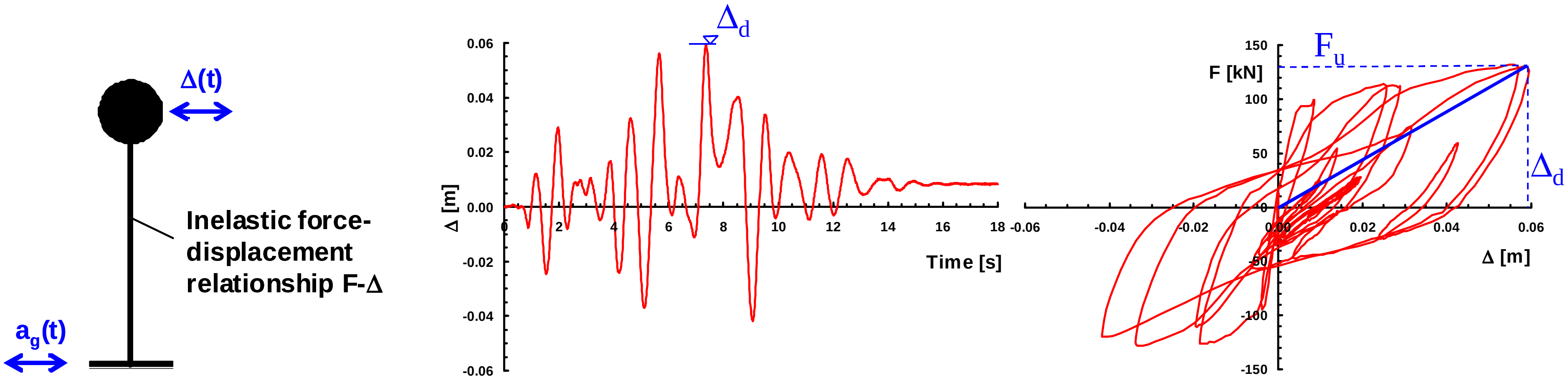
  - Priestley MJN (2003) “Myths and Fallacies in Earthquake Engineering, Revisited,” The Mallet Milne Lecture 2003, IUSS Press, Pavia, Italy.
  - Priestley MJN, Calvi GM, Kowalsky MJ (2007) *Displacement-Based Seismic Design of Structures*, IUSS Press, Pavia, Italy.
  - Priestley MJN (2004) Handouts and PPT Presentations to the Course “Fundamentals of Seismic Design” at the ROSE School, Pavia, Italy.

Basic Idea of DDBD

- Substitute SDOF system: Characterize structure at peak displacement response.



- Characterizes the structure by
  - the secant stiffness to the maximum displacement response of the substitute SDOF system
  - an equivalent viscous damping that represents both initial elastic damping, and the inelastic hysteretic damping.

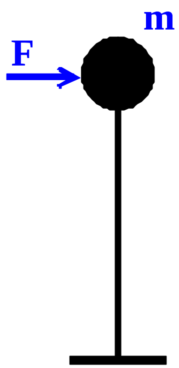
Objective of DDBD

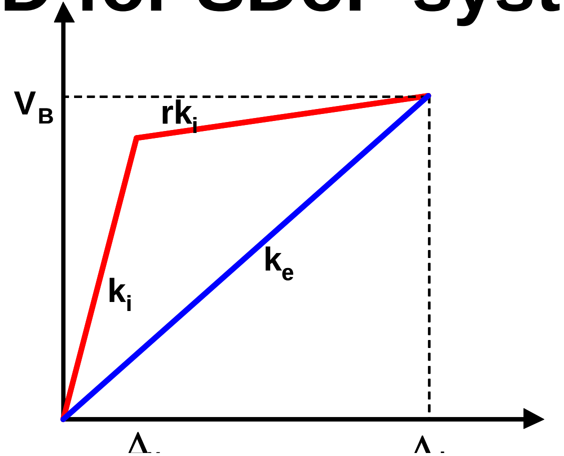
- Objective of design approach: Design a structure which achieves a chosen performance limit (= peak displacement) → Results in structures with uniform vulnerability.

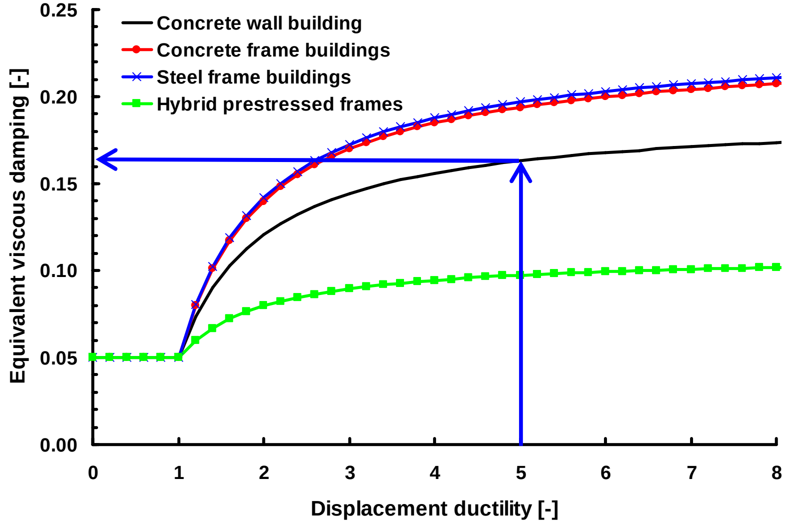
- Design procedure determines strength required at plastic hinge locations to achieve defined displacement for a given seismic intensity.

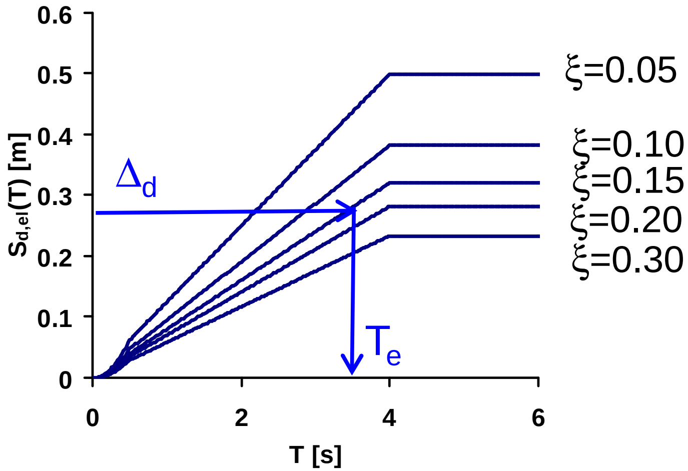
- Combine DDBD with capacity design principles to ensure that the chosen mechanism develops.

Basic formulation of DDBD for SDoF systems





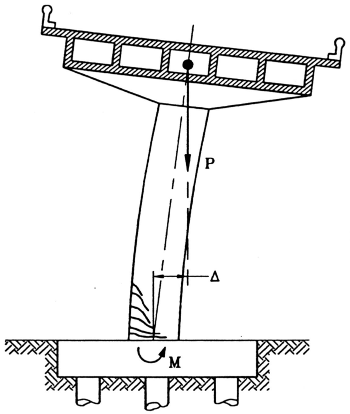


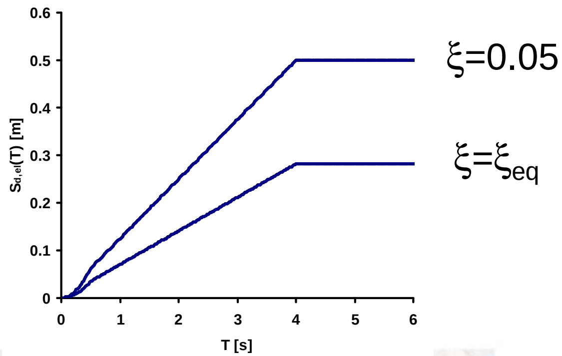


- If more than one limit state needs to be considered (e.g. SLS and ULS): Determine for each the design displacement and using the DDBD method the required base shear. The highest base shear governs the design.

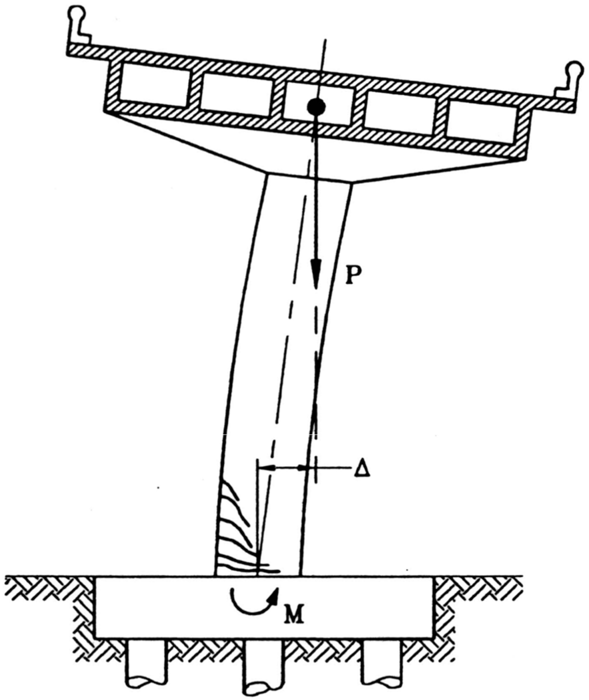
Design input:

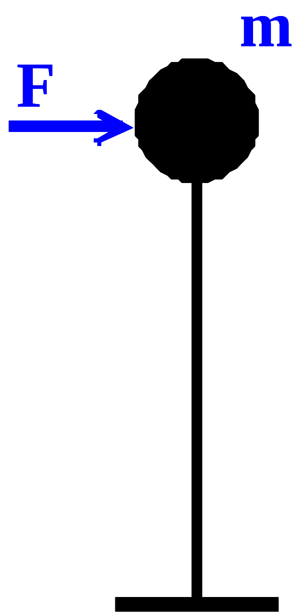
- Geometry: Dimensions of structural elements
- Basic material properties: ε_{sy}, ε_{su}, ε_{cu}, ε_{ccu}
- Seismic input (Displacement spectra + Equations for transforming 5% spectrum into spectrum with $\xi=\xi_e$)
  - Without velocity pulse: $$S_{d,\xi} = S_{d,5\%}\left(\frac{0.07}{0.02+\xi}\right)^{0.5}$$
  - With forward directivity velocity pulse: $$S_{d,\xi} = S_{d,5\%}\left(\frac{0.07}{0.02+\xi}\right)^{0.25}$$
- Example: Bridge column with superstructure = SDoF system

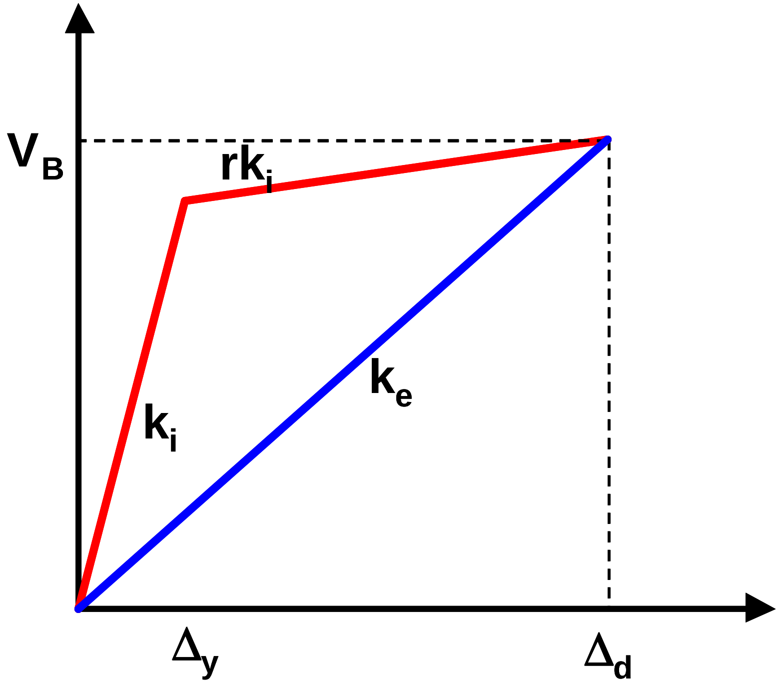




Step 1: Determine design displacement







- Design displacement governed by:
  - Structural displacement limits: Strain limits
  - Non-structural displacement limits: Drift limits

  →Choose critical of structural and non-structural displacement limits

  Often: Design for a drift limit specified by the code and detail the section in such a way that the strain limits are satisfied.

Step 2: Determine yield displacement and design displacement ductility

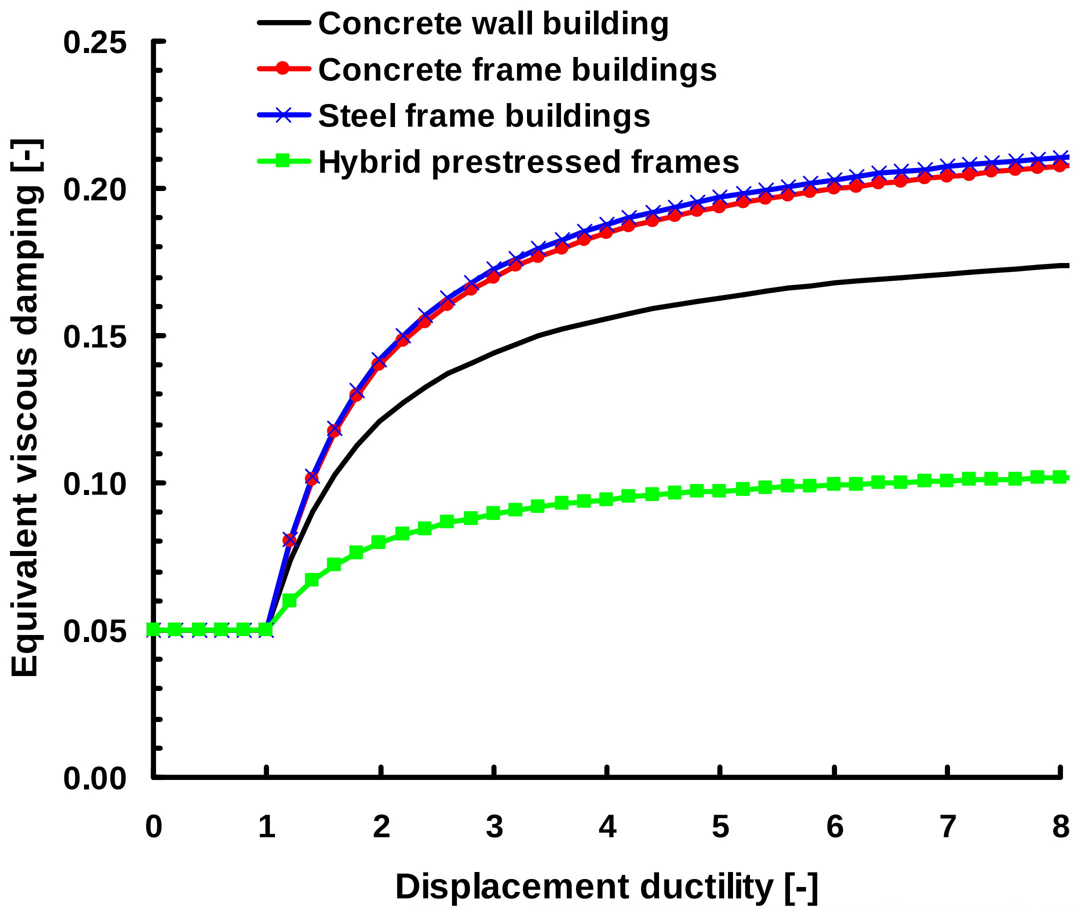
- Yield displacement Δ_y is (approximately) only dependent on the geometry of the members and the material properties

  → known at the beginning of the design [PCK07].

- Design displacement ductility

$$\mu_\Delta = \frac{\Delta_d}{\Delta_y}$$

Step 3: From charts determine the equivalent damping as a function of μ_Δ and the structural type



Concrete wall buildings

$$\xi_{eq} = 0.05 + 0.444\left(\frac{\mu_\Delta - 1}{\mu_\Delta \pi}\right)$$

Concrete frame buildings

$$\xi_{eq} = 0.05 + 0.565\left(\frac{\mu_\Delta - 1}{\mu_\Delta \pi}\right)$$

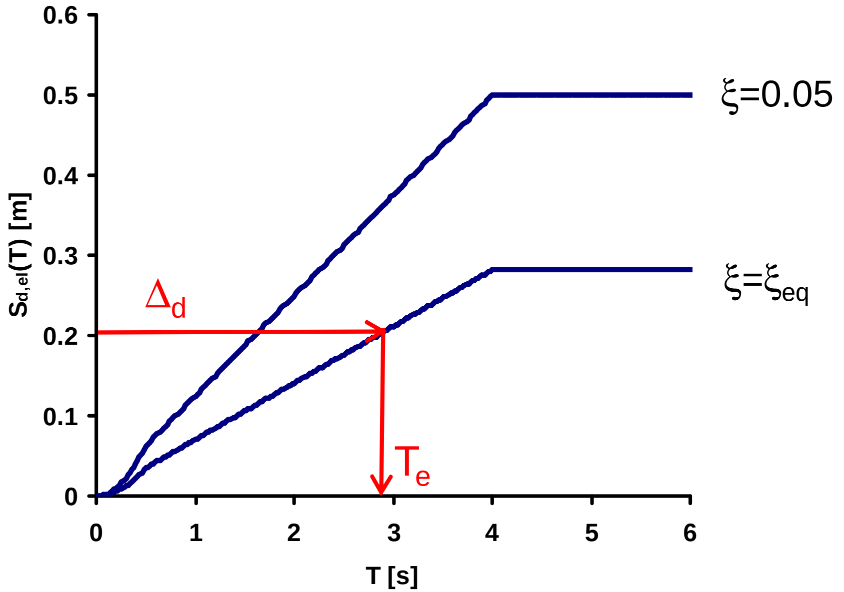
Steel frame buildings

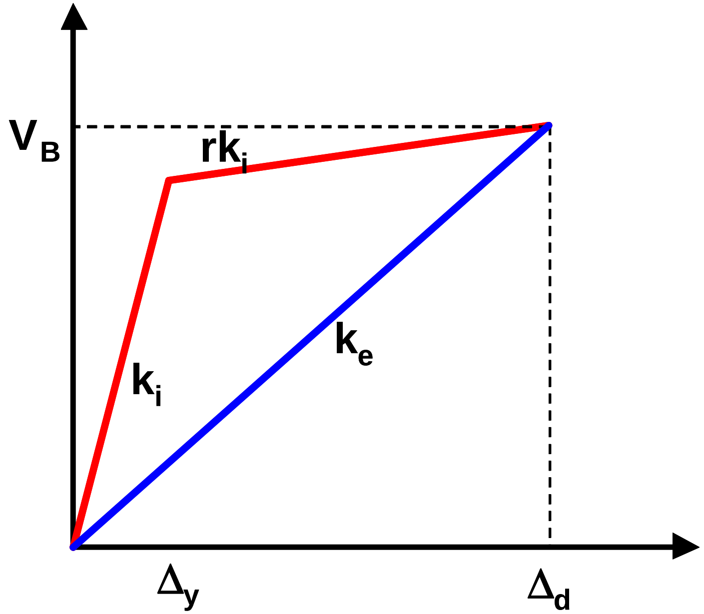
$$\xi_{eq} = 0.05 + 0.577\left(\frac{\mu_\Delta - 1}{\mu_\Delta \pi}\right)$$

Hybrid prestressed frame

$$\xi_{eq} = 0.05 + 0.186\left(\frac{\mu_\Delta - 1}{\mu_\Delta \pi}\right)$$

Step 4: Compute for the effective damping the displacement spectra. Determine T_e as the period where $\Delta = \Delta_d$





$$T_e = 2\pi\sqrt{\frac{m_e}{k_e}} \longrightarrow k_e = \frac{4\pi^2 m_e}{T_e^{\,2}} \longrightarrow V_B = k_e \Delta_d$$

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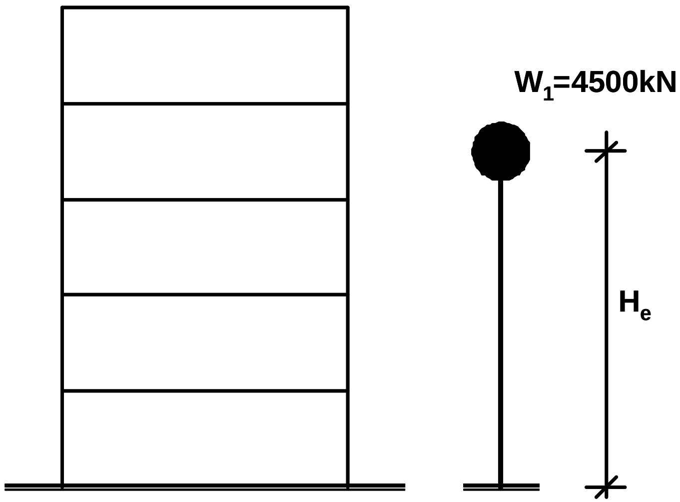
Example 1: Basic DDBD

5 Storey RC Frame Building ([PCK07] Example 3.1, p. 67)

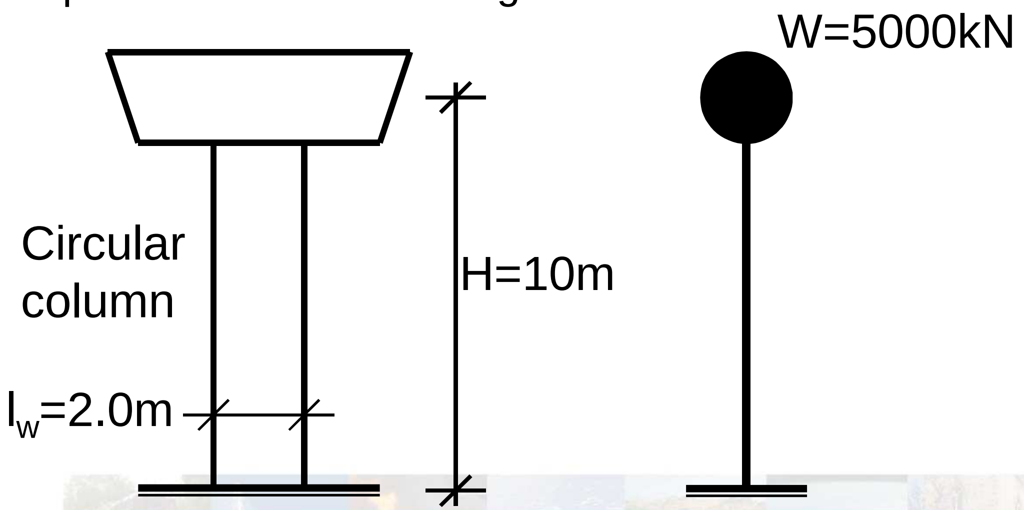
- Design displacement 0.185m (μ_Δ=3.25)
- Weight of first mode: W_1=4500kN
- Displacement spectrum:
  - T_D = 4s, $S_{d,5\%}(T_D)$=0.5m
  - no directivity effects
- Determine required base shear strength



Example 2: Basic DDBD

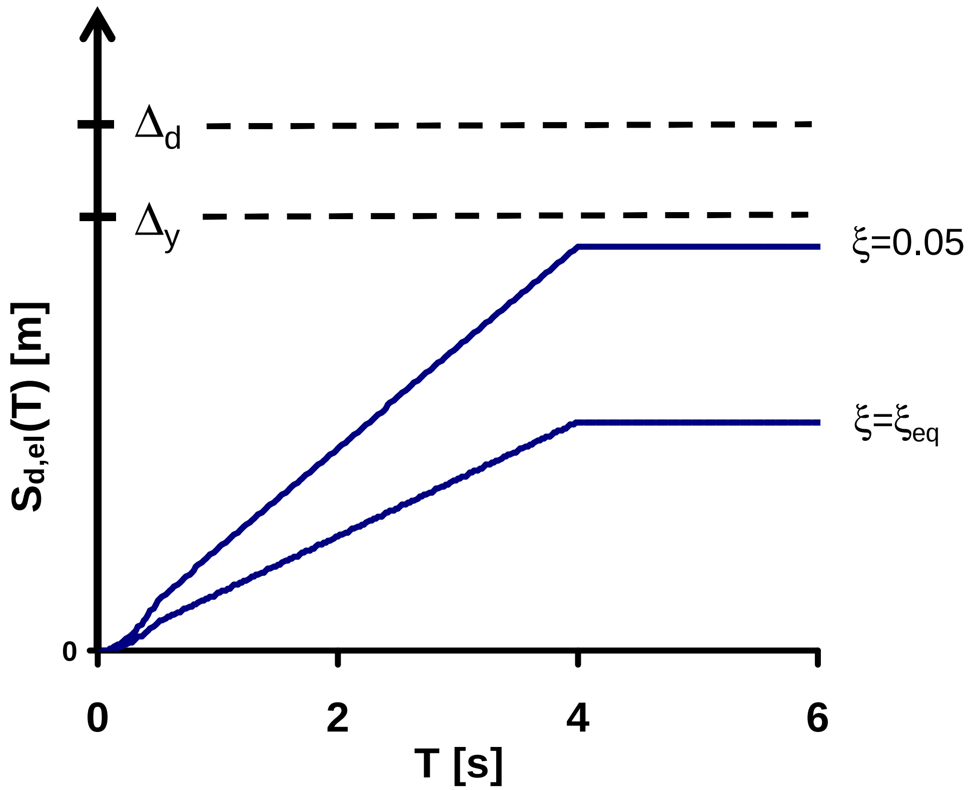
Flexible Bridge Pier ([PCK07] Example 3.4, p. 93)

- Consider 2 limit states:
  - $\mu_{\Delta}=4.0$
  - $\theta_d=0.035$
- Displacement spectrum:
  - T_D = 4s, $S_{d,5\%}(T_D)$=0.875m
  - With and without directivity effects
- Determine required base shear strength



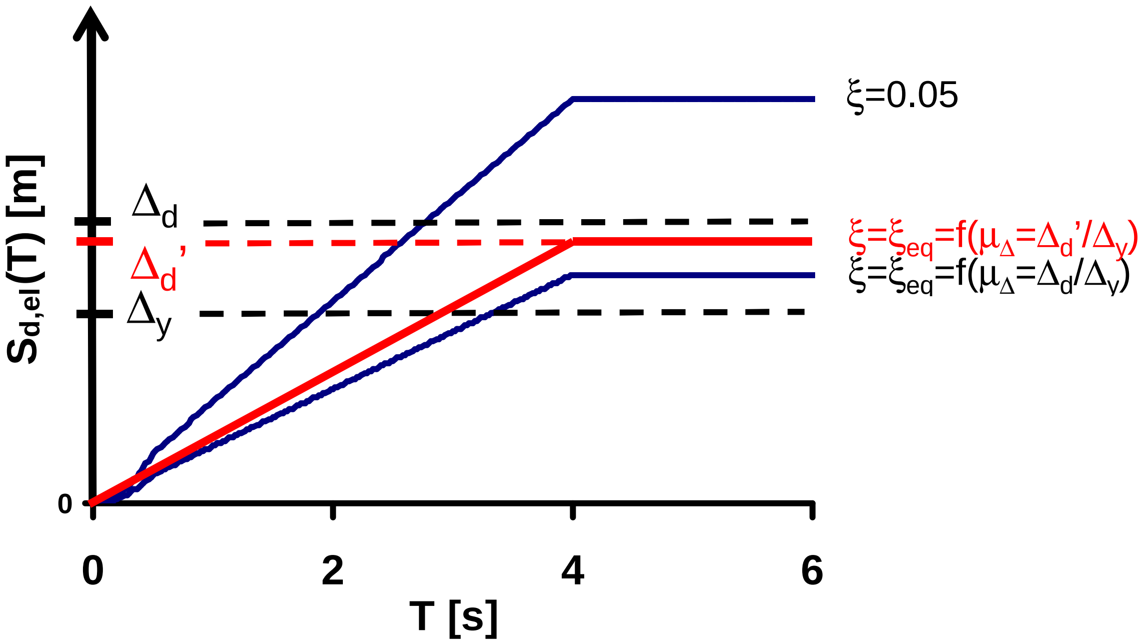
Example 3: Basic DDBD

Very tall, flexible structure



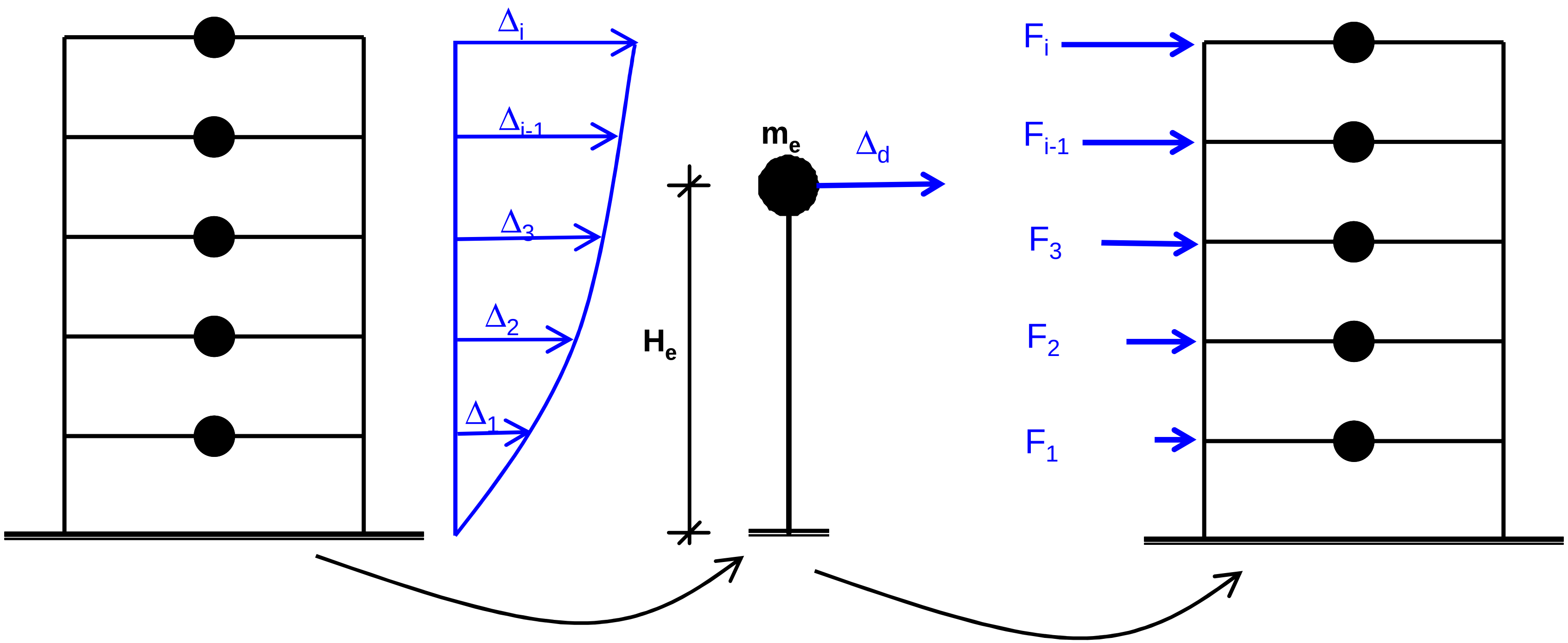
Example 4: Basic DDBD

Medium tall, flexible structure



DDBD for MDoF systems

- Real buildings are MDoF systems
- DDBD approach is based on SDoF system



Reduce MDoF system to equivalent SDoF system based on inelastic first mode shape

DDBD for $SDOF_e$

-Distribute forces over height and in plan.

-Account for higher mode effects.

-Apply capacity design

Steps involved in the DDBD of MDoF systems:

- Yield displacement of MDoF System
- Design displacement of MDoF system
- Substitute SDoF structure
- Effective damping of substitute SDoF structure
- (Determine system base shear based on DDBD approach)
- Distribution of base shear forces between structural elements
- Distribution of base shear forces over the height
- Capacity design for DDBD

Note: Structural wall buildings will be used as an example.

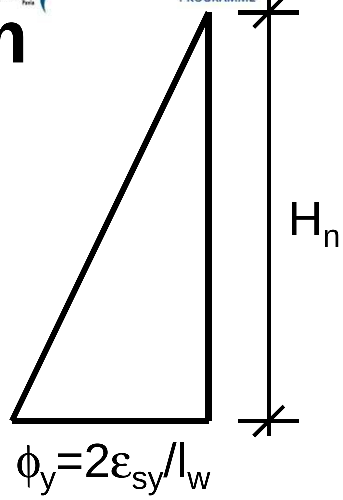
Yield displacement of MDoF system

Cantilever walls:

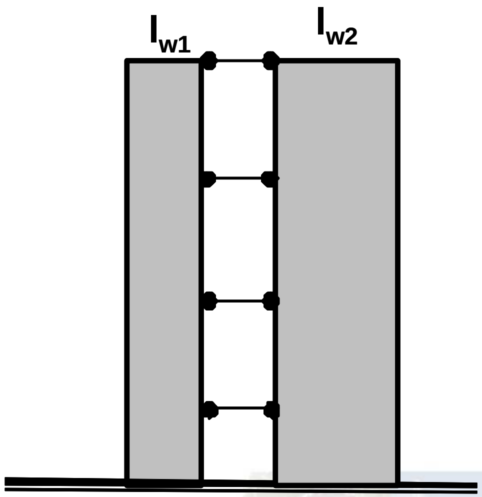
- For each wall assume a triangular curvature distribution at yield.

$$\Delta_{yi} = \frac{\phi_y}{2} H_i^2 \left(1 - \frac{H_i}{3H_n}\right) = \frac{\varepsilon_{sy}}{l_w} H_i^2 \left(1 - \frac{H_i}{3H_n}\right)$$

- Assumption of triangular curvature distribution accounts for tension shift effects on displacement.



System of walls of different length:



$$\Delta_{ys} = \Delta_{y1}\frac{V_1}{V_B} + \Delta_{y2}\frac{V_2}{V_B}$$

Δ_{ys}: yield displacement of system at effective height H_e

Δ_{yi} : yield displacement of wall i at H_e

V_i: base shear capacities of wall i

Design displacement of MDoF system

"Inelastic mode shape"; shape dependent on

- structural type
- mechanism
- height of the structure
- displacement ductility

Cantilever wall buildings

- Limit state can be drift or strain controlled.
- For a single wall: If the roof drift θ_{dn} governs the design:

$$\theta_{dn} = \theta_{yn} + \theta_{pn} = \frac{\phi_y H_n}{2} + \left(\phi_m - \phi_y\right) L_p \le \theta_c$$

- If roof drift is less than θ_c, ϕ_m is determined from strain limits.

Design displacement of MDoF system

Cantilever wall buildings

- Displacement profile:

$$\Delta_i = \Delta_{yi} + \Delta_{pi} = \frac{\varepsilon_{sy}}{l_w} H_i^2 \left(1 - \frac{H_i}{3H_n}\right) + \left(\phi_m - \phi_y\right) L_p H_i$$

- System of walls of different length: Longer and stronger walls govern the response

→ Adopt their displacement profile for design. Alternatively: Use average profile chosen to have equal displacements at the effective height and weighted by wall flexural strength.

Design displacement of MDoF system

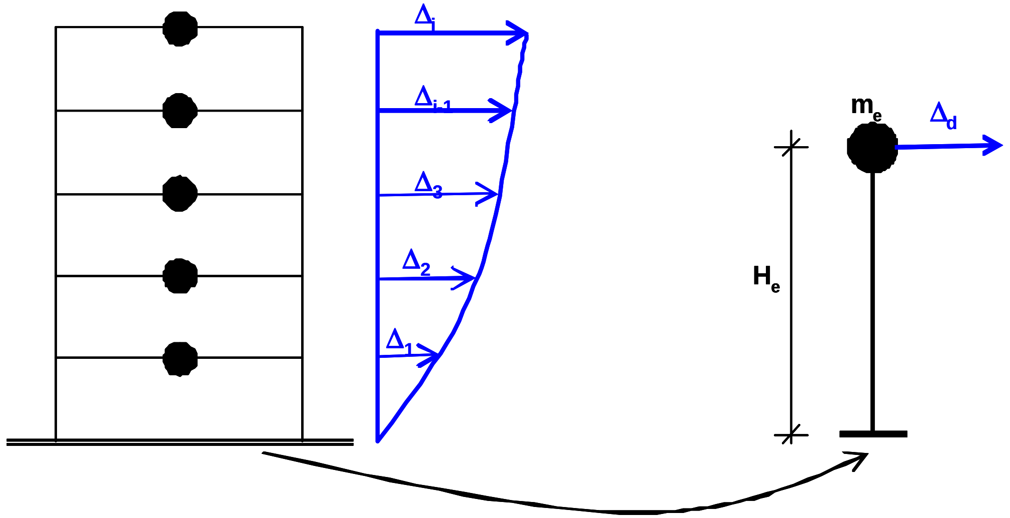
Serviceability and damage control curvatures for RC walls

- From section analysis of rectangular walls with
  - Axial load ratios ν = [0 ; 0.15]
  - Flexural reinforcement ratios ρ_f = [0.005 ; 0.2]
  - Uniformly distributed reinforcement

- Limit curvatures are dominated by the steel strain limits
  - Serviceability curvature ($\varepsilon_{c,max} = 0.004$, $\varepsilon_{s,max} = 0.015$)
    - $\phi_s l_w = 0.0175$
  - Damage-control curvature ($\varepsilon_{c,max} = 0.018$, $\varepsilon_{s,max} = 0.06$)
    - $\phi_{dc} l_w = 0.072$
  - Equivalent to $\phi_{ls} l_w = 1.2\varepsilon_{s,max}$

- For sections with concentrated end reinforcement the limit curvatures are slightly lower.

Substitute SDoF structure



- Effective displacement (design displacement)

$$\Delta_d = \sum_{i=1}^{n}\left(m_i\Delta_i^{\,2}\right) / \sum_{i=1}^{n}\left(m_i\Delta_i\right)$$

- Effective mass

$$m_e = \sum_{i=1}^{n}\left(m_i\Delta_i\right) / \Delta_d$$

- Effective height

$$H_e = \sum_{i=1}^{n}\left(m_i\Delta_i H_i\right) / \sum_{i=1}^{n}\left(m_i\Delta_i\right)$$

- Yield displacement

$$\Delta_y = \Delta_{yi}\left(H_e\right)$$

Equivalent viscous damping

- Determine ductility demand $\mu_{\Delta j}$ on each wall j

- Equivalent effective damping of each wall $\xi_{ej}=\xi_{ej}(\mu_{\Delta j})$

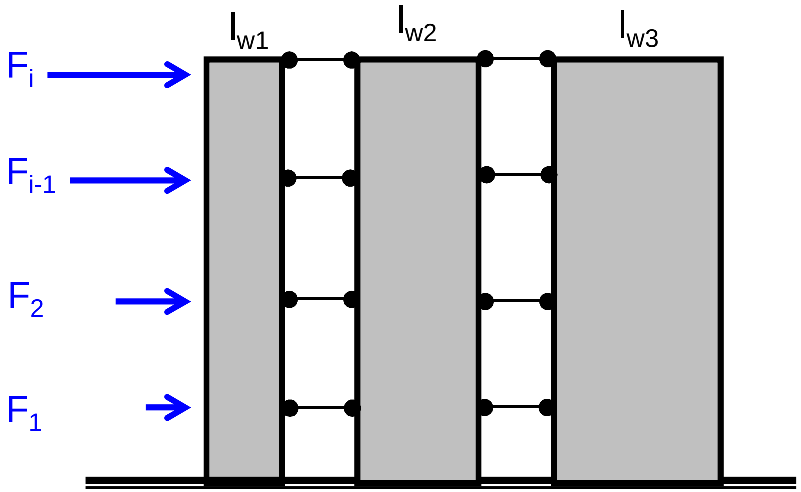
- Equivalent effective damping of system = weighted average based on the energy dissipated by each structural element

$$\xi_e = \frac{\sum_{j=1}^{m} V_j \Delta_j \xi_j}{\sum_{j=1}^{m} V_j \Delta_j} = \frac{\sum_{j=1}^{m} V_j \xi_j}{V_B}$$

- Wall base shears are at this point still unknown. Reasonable assumption: Apportion V_B between the walls in proportion to the square of the wall length l_w

$$\xi_e = \frac{\sum_{j=1}^{m} {l_{wj}}^2 \xi_j}{\sum_{j=1}^{m} {l_{wj}}^2}$$

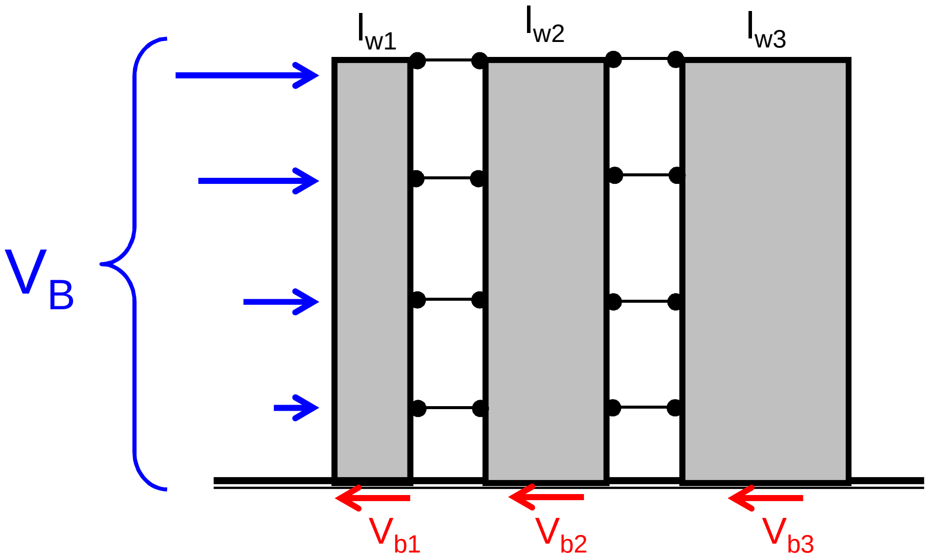
Distribution of design base shear force over height



- The base shear should be distributed over the height in proportion to mass and displacement at the discretized mass locations. The lateral force acting at mass m_i is therefore:

$$F_i = V_B \frac{m_i \Delta_i}{\sum_{i=1}^{n} m_i \Delta_i}$$

Distribution of base shear forces between structural elements



- Traditionally: Proportional to elastic element stiffnesses.
- New idea: Designer is free to decide on the distribution of strength between the elements.

Distribution of base shear forces between structural elements

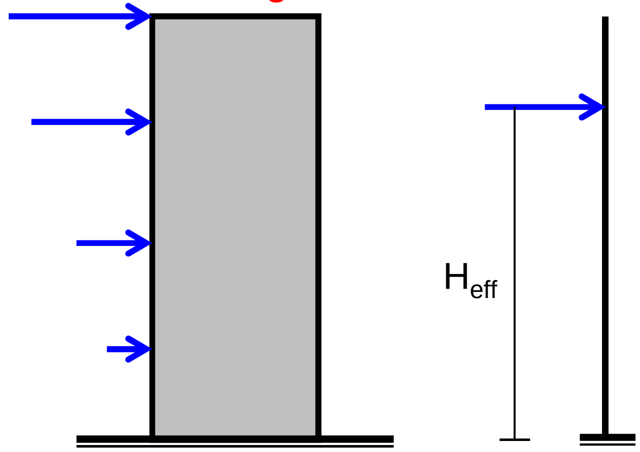
- Issues to consider:
  - Reinforcement contents (equal reinforcement contents lead approximately to moment capacities proportional to l_w^2)
  - Shear demand (avoid excessive shear demand → choose smaller reinforcement ratios for longer walls)
  - Strength distribution within the structure → Torsional response of building

- References:

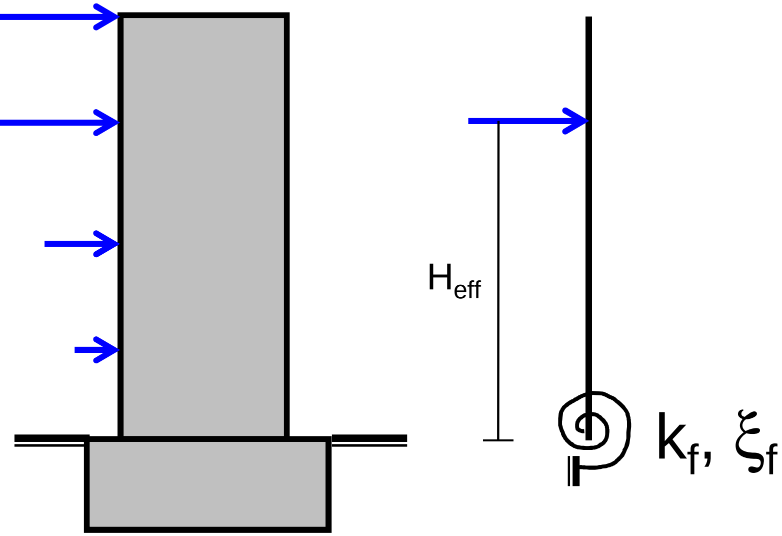
  - Paulay T, Restrepo JI (1998) “Displacement and ductility compatibility in buildings with mixed structural systems,” SESOC Journal 11(1):7-12.
  - Paulay T (2001) “The freedom in choosing the seismic strength of components,” SESOC Journal 14(2):51-56.

Effect of foundation flexibility

Often assumed: Rigid foundation



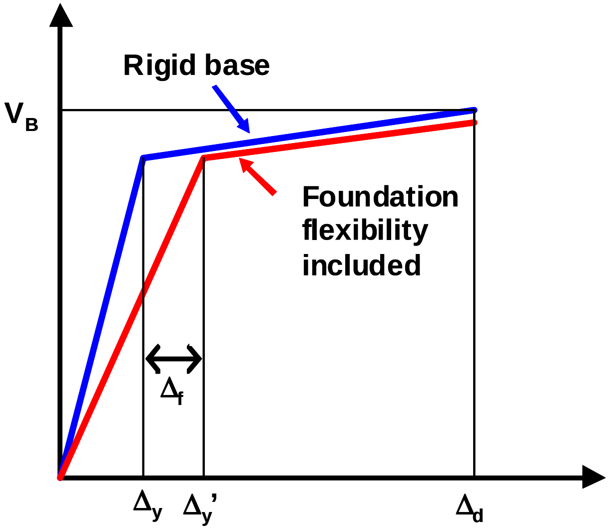
More realistic: Flexible foundation

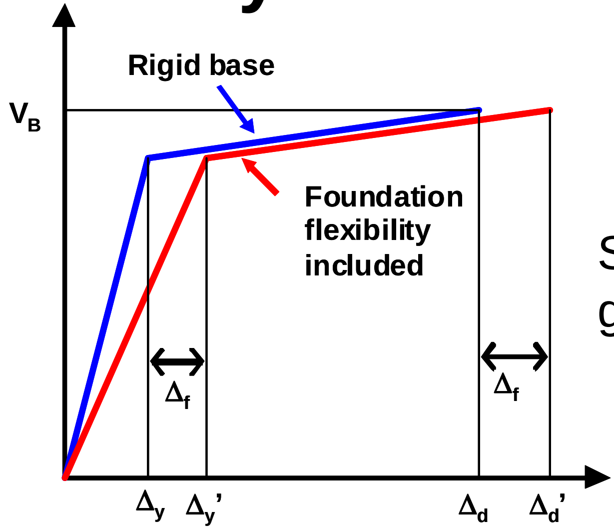


- Foundation flexibility: Assume the foundation flexibility can be modelled as elastic rotational spring with stiffness k_f and with an effective damping ξ_f.
- Foundation stiffness: Uplift of the footing (rocking) is not prohibited but needs to be accounted for when determining the foundation stiffness.
- Codes often require that uplift is limited to 50% of the foundation length.
- Rocking can be beneficial as additional damping is provided.

Effect of foundation flexibility on DDBD

Drift limit governs





Strain limit governs

Effect of considering foundation flexibility:

- The yield displacement is increased by the displacement due to foundation rotation.

$$\Delta_y' = \Delta_y + \Delta_f$$

- If the design displacement is driven by a drift limit, the design displacement will not change and thus implying reduced structural deformations.

$$\Delta_D' = \Delta_D$$

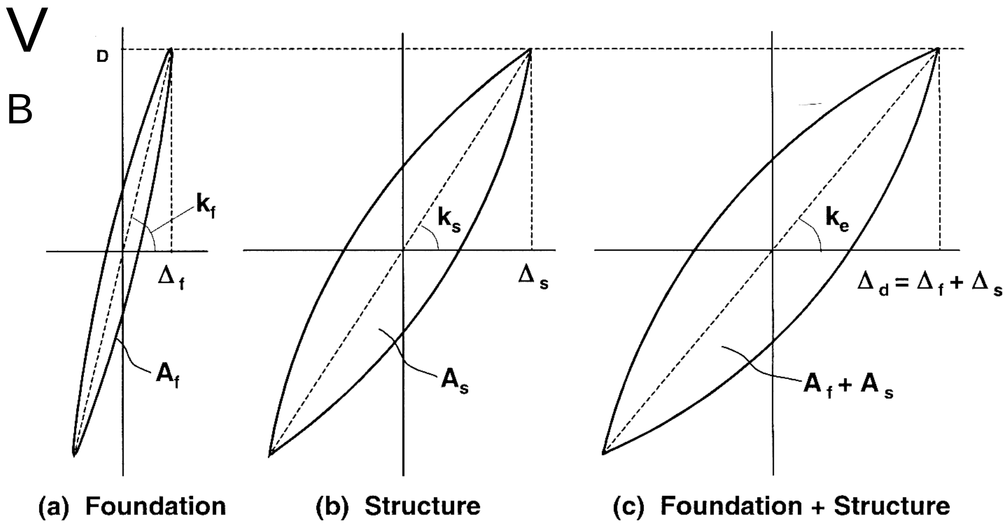
- If design displacement is driven by a strain limit, then the design displacement will be increased.

$$\Delta_D' = \Delta_D + \Delta_f$$

- In both cases the system ductility capacity is reduced.

Effect of foundation flexibility

Effective damping of the structure accounting for foundation flexibility:



$$\xi_e = \frac{\sum_{j=1}^{m} V_j \Delta_j \xi_j}{\sum_{j=1}^{m} V_j \Delta_j} = \frac{V_f \Delta_f \xi_f + V_B \Delta_s \xi_s}{V_f \Delta_f + V_B \Delta_s} = \frac{\Delta_f \xi_f + \Delta_s \xi_s}{\Delta_f + \Delta_s}$$

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[ATC05] Applied Technology Council: Improvement of Nonlinear Static Seismic Analysis Procedures. Document prepared for the Federal Emergency Management Agency. **FEMA 440. Redwood** City, California, 2005.

[Cas04] Castillo R: “Seismic design of asymmetric ductile systems”. PhD thesis, University of Canterbury, Christchurch, New Zealand, 2004.

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[Pau01a] Paulay T.: “The freedom in choosing the seismic strength of components”. SESOC Journal 14(2):51-56, 2001.

[Pau01b] Paulay T.: “Some design principles relevant to torsional phenomena in ductile buildings”. Journal of Earthquake Engineering, Vol. 5, No 3, pp. 273-308, 2001.

[PCK07] Priestley M.J.N., Calvi G.M., Kowalsky M.J.: “Displacement-Based Seismic Design of Structures”. IUSS Press, Pavia, Italy, 2007.
[PR88] Paulay T., Restrepo J.I.:“Displacement and ductility compatibility in buildings with mixed structural systems”. SESOC Journal 11(1):7-12, 1998.
[Pri93] Priestley M.J.N.: “Myths and Fallacies in Earthquake Engineering”. Bulletin of the NZSEE, Vol. 26, No. 3, pp. 329-341, 1993.
[Pri03] Priestley M.J.N.: “Myths and Fallacies in Earthquake Engineering, Revisited”. The Mallet Milne Lecture 2003, IUSS Press, Pavia, Italy, 2003.
[Pri04] Priestley M.J.N.:” Handouts and PPT presentations to the course *Fundamentals of Seismic Design at the ROSE* School, Pavia, Italy, 2004.
[Sch71] Schrefler B.: “Zur Berechnung aussteifender Systeme allgemeiner Art von Hochhäusern”. Beton- und Stahlbetonbau 66, S. 213-219, 1971. (In German)

Capacity Design of Buildings

Seismic Demand

Particularities of response due to seismic excitation:

- Response in inelastic domain
- Cyclic demand
-“Mass proportional”

For comparison: response due to gravity and wind loads:

- Response in elastic domain
- Mono-directional demand
- “Area/Surface proportional”

Implication for RC Elements:

Tension/Compression zone due to bending

- Alternation between tension and compression zone. Reinforcement bars yielding in tension have to yield in compression before cracks are closing and concrete acts in compression

Shear demand

- Cracks in both diagonal directions: High demand on concrete in flange and shear reinforcement
- Shear reinforcement is always in tension

Bond between concrete and reinforcement

- Reduced bond stress due to cyclic demand and loss of cover concrete

Overstrength

- Mobilized by large deformations within the inelastic range

Definition of Capacity Design

Objective

Under the design earthquake the structure should be ductile enough to respond in the inelastic range without failing

Design Procedure

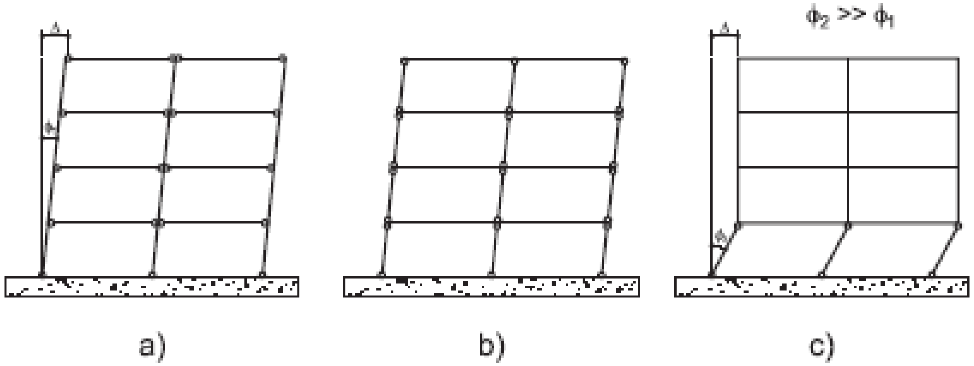
- Choose a suitable mechanism and identify the regions which will undergo inelastic deformations (plastic hinges)
- The plastic hinges should be designed and detailed in order to attain the required ductility/deformation capacity
- All other regions should be designed for an increased capacity to ensure that these regions remain elastic when the overstrength in the plastic hinges is developed

Comparison of Performance Under Seismic Excitation

Performance under seismic excitation	
Conventionally designed structures	Capacity designed structures
• Plastic hinges could develop anywhere. • The plastic mechanism is arbitrary and not identified. • The local ductility of the plasticized regions varies significantly and the global ductility of the structure is in general small and not known. • The performance under seismic excitation is not really known.	• Plastic deformations are only possible within clearly identified regions. • The plastic mechanism is suitable and know. • The local ductility within the plastic hinges is adapted to the global ductility which in return is chosen in accordance with the design class. • The behaviour under seismic excitation is well known.
Limited safety against collapse	High safety against collapse

Suitable and Unsuitable mechanisms

Example of frames under seismic excitation



suitable:

a) Beam mechanism

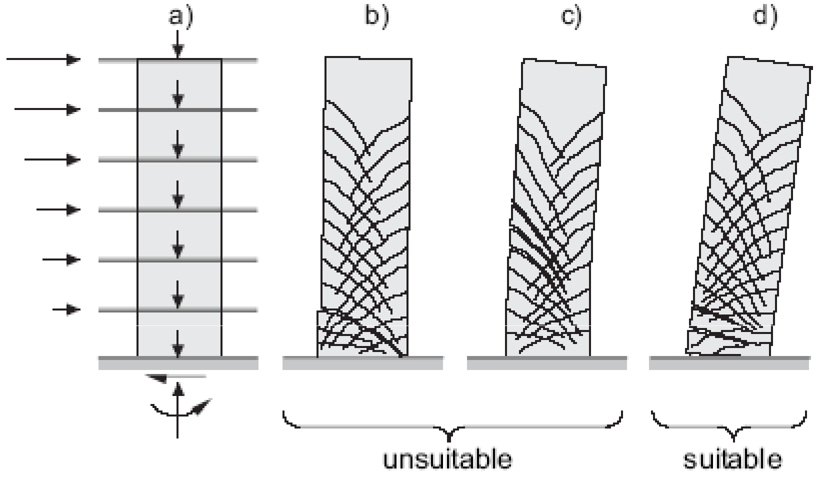
unsuitable:

b) Column mechanism:
- Plastic hinges in columns unsuitable (axial force!)
- Danger: soft storey mechanism

c) Soft storey mechanism
- Frequent cause for collapse

Suitable and Unsuitable mechanisms

Example of slender walls



suitable:

d) Plastic hinge at wall base

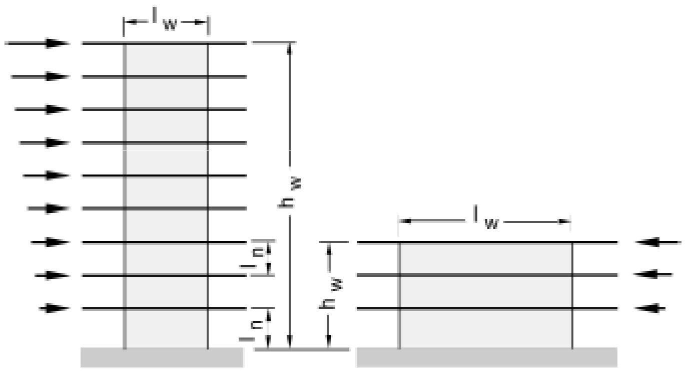
unsuitable:

c) Yielding of longitudinal reinforcement in upper storeys where no appropriate detailing has been provided.
b) Shear failure at wall base before required ductility has developed.

Capacity Design of RC Structural Walls

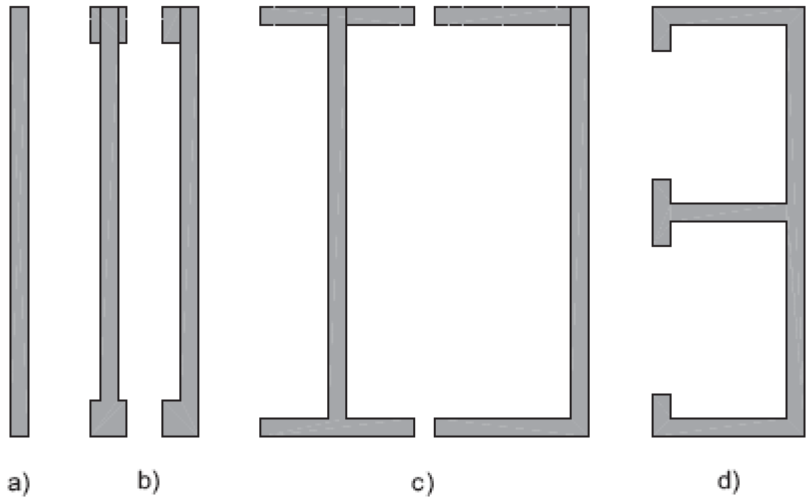
To undertand the capacity design principle, we will consider the case of RC structural walls

Type of RC structural walls



- Slender walls (left figure) → $h_w / l_w \geq 3$ → Bending governs
- Squat walls (right figure) → $h_w / l_w < 3$ → Shear governs
- Connected walls → Coupling through floors (very little bending and shear stiffness)
- Coupled walls → Coupling through squat beams (stiff in bending and shear)

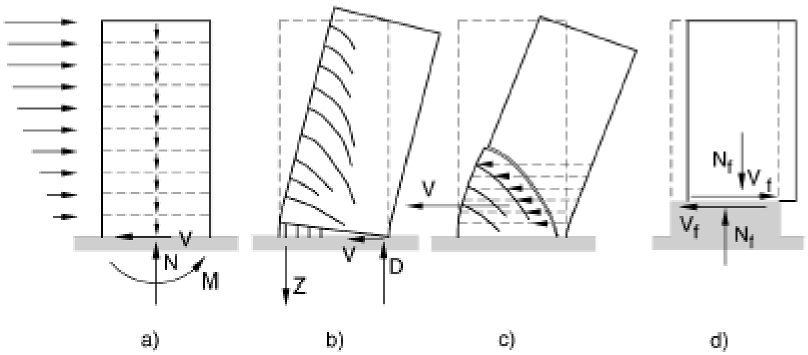
Cross section for RC structural walls:



a) rectangular
b) with boundary elements on one or two ends
c) with flanges elements on one or two ends
d) cross section of service cores
Note

- High shear stresses in the web of walls with boundary elements as a consequence of the high flexural capacity!
- Non-symmetric cross section can be tricky and they should be designed carefully (The properties and the behaviour are different depending on the loading direction)

Failure modes for RC structural walls:



a) Forces and reactions
b) Flexural failure
c) Tensile shear failure (stirrups!)
d) Sliding shear failure

Goal:
Provide enough deformation and energy dissipation capacity

Capacity design for RC structural walls:

Step-by-step procedure

1. Choice of the plastic mechanism and of the height of the plastic hinge region
2. Design for flexural strength in the plastic hinge region
3. Check of the curvature ductility capacity and of the need for confinement in the plastic hinge region
4. Check of the stability of the vertical reinforcement and of the need for stabilizing reinforcement in the plastic hinge region
5. Design for shear strength in the plastic hinge region
6. Detailing of the transverse reinforcement in the plastic hinge region
7. Design an detailing of the elastic region
8. Design of the foundation

The step-by-step design procedure presented in different references like e.g. [PP92], [Cen04], [NZS95], [PCK07] and [SIA03b] is typically very similar. However, the single equations to verify the principles within the steps can be quite different!

Step 1: Choice of the plastic mechanism

• A plastic hinge should form at the base of the wall
• Height of the plastic hinge region:

$L_p \geq \max(l_w, h_w/6) \leq 2\, l_w$

In the plastic hinge zone the detailing of the transverse reinforcement is very important because such reinforcement should:

1) stabilize the longitudinal reinforcement
2) increase the curvature capacity (confinement, if needed)
3) carry shear

Step 2: Flexural design of the plastic hinge region

- Flexural design: Design of the wall section for the sectional forces , due to earthquake load case
- Check of the location of the vertical reinforcement (e.g. according to [PP92]):

$s_{max} \leq 200 \div 250$ mm (in confined zone)

$s_{max} \leq \min(3\, b_w, 450 \text{ mm})$ (in the web region)

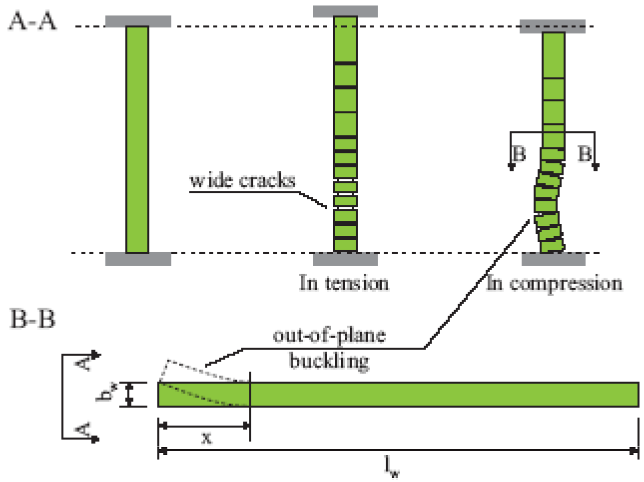
$\phi_{l,max} \leq b_w/10$

Where: s_{max} horizontal distance of the vertical reinforcement and $\phi_{l,max}$ maximum diameter of longitudinal reinforcement

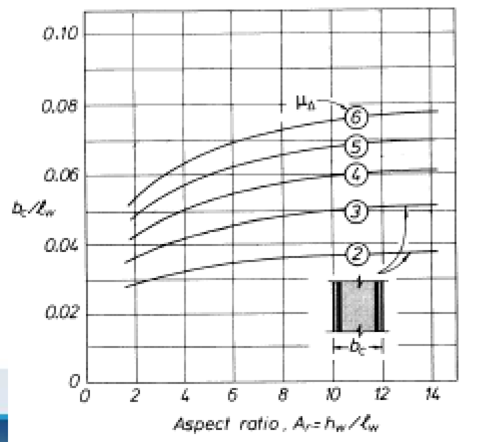
- Check the reinforcement recommendations:

Web reinforcement content → $0.30\% \leq \rho_w \leq 0.50\%$
Reinf. content of boundary region → $\rho_c \leq 4\%$
Total reinforcement content → $0.30\% \leq \rho_t \leq 1\%$

- Check of wall stability



As a function of the ductility that we want to get, we should respect some aspect ration to make sure we are not going to have out of plane buckling



Step 3: Check of the curvature ductility capacity and of the need for confinement in the plastic hinge region

Goal of this step is to ensure that in the plastic zone the cross section of the wall posses an adequate curvature ductility capacity in order to make sure that the assumed displacement ductility of the wall can develop

In this step two activities are carried out:

- Computation of the flexural overstrength of the section
- Actual check of the curvature ductility capacity and of the need for confinement

A sufficient curvature ductility capacity is normally ensured – if needed – by confining the boundary zones of the cross-section. The equations used to check the curvature ductility capacity and to design the confinement reinforcement varies among different design codes. In order to present an example, the following the NZS95 will be considered

If the following equation is fulfilled the curvature ductility capacity is enough and no confinement is required

$$x \le x_c \quad \text{where} \quad x_c = \left(0.3\frac{\Phi_0}{\mu_\Delta}\right)$$

$$x \rightarrow \text{depth of compression zone at overstrength}$$

If the equation above is NOT fulfilled, confinement is required and the confinement length is:

$$\alpha = 1 - 0.7\frac{x_c}{x} \ge 0.5$$

And the reinforcement quantity is:

$$\frac{A_{sh}}{s_h h''} = \left(\frac{\mu_\Delta}{40} + 0.1\right)\frac{A_g^*}{A_c^*}\frac{f_c^{'}}{f_{yb}}\left(\frac{x}{l_w} - 0.07\right)$$

Where

A_{sh}: section area of the confinement reinforcement in the considered direction
s_h: vertical distance of the confinement reinforcement
h'': dimension of the confined concrete core perpendicular to the considered direction
A_g^* : gross area of confined region $A_g^* = b_w\,(\alpha\, x)$
A_c^* : area of confined region
f_{yh}: yield strength of the confinement reinforcement

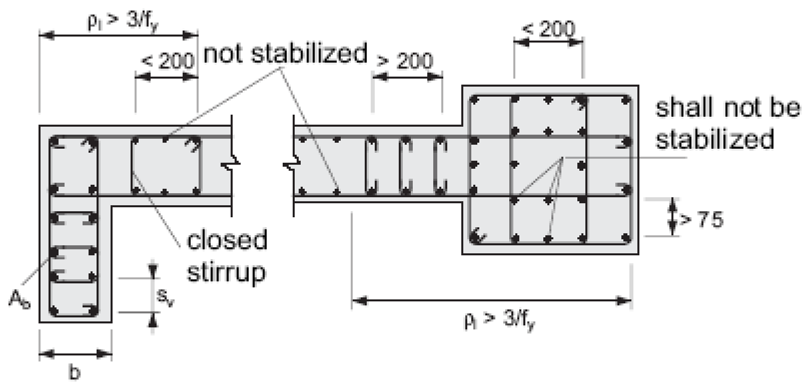
Step 4: Check of the stability of the vertical reinforcement and of the need for stabilizing reinforcement in the plastic hinge region

• Phenomenon
Under cyclic action the vertical reinforcement yields in compression. The phenomenon is known as **Buckling**.

• Zone where the reinforcement has to be stabilized (according to [PP92]):
The vertical reinforcement needs to be stabilized within all zones where:

$$\rho_l \geq (3\,\text{MPa})/f_{yd} \approx 0.6\%$$

$$\rho_l \rightarrow \text{local reinforcement content}$$



• Stabilizing reinforcement:

• All vertical rebar close to the concrete surface should be stabilized by a 90° -bent or a 135° -seismic hook.
• Vertical distance of the stabilizing reinforcement:

$s_h \leq \min(6\ \Phi_l, 150\ \text{mm})$ → Φ_l maximum diameter of the vertical reinforcement

• Minimum diameter Φ_{sl} of the stabilizing reinforcement:

$\Phi_{sl} \geq 0.35\ \Phi_l$

Step 5: Design of shear strength in the plastic hinge region

Ductility dependent shear strength: e.g. the modified UCSmodel [KP00] (see also [PCK07])

$V_{design} = 0.85\ (V_c+V_p+V_s)$

Where

V_c: Concrete shear resisting mechanism
V_p: Axial load component
V_s: Transverse reinforcement truss shear resisting mechanism

These quantities are computed as follows

$$V_c = \alpha\beta\gamma\sqrt{f_c^{'}}(0.8A_g)$$

$$V_p = 0.85\,P\tan\zeta$$

$$V_s = \frac{A_v f_{yh}(D - c - c_0)\cot\vartheta}{s}$$

where :

$$1 \le \alpha = 3 - \frac{L}{D} \le 1.5$$

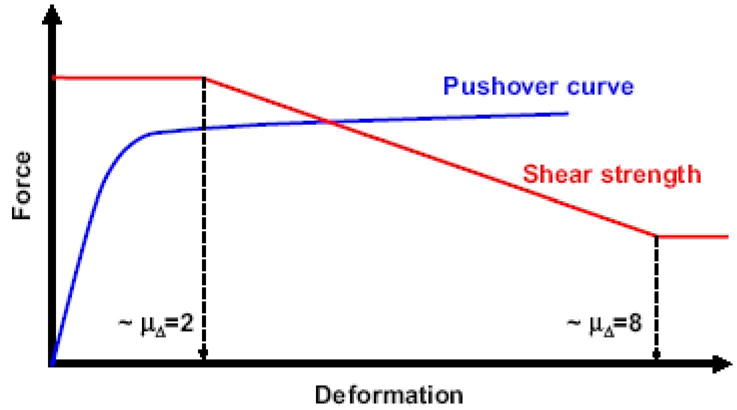
$$\beta = 0.5 + 20\rho_l \le 1$$

$$\gamma = \begin{cases} 0.25 & \text{if } \mu_\phi \le 2 \\ 0.3025 - 0.0175\mu_\phi & \text{if } 2 < \mu_\phi \le 15 \\ 0.04 & \text{if } \mu_\phi > 15 \end{cases}$$

with:

L: Shear span
D: Height of the section (L/D = slenderness)
ρ_l: Longitudinal reinforcement content
μ_ϕ : Curvature ductility
A_g: Gross area of section
f_c': Cylinder compressive strength of concrete
P: Axial load (compression positive)
ζ: Inclination of the compression strut
c: Location of the neutral axis
A_v: Area of the shear reinforcement
f_{yh}: Yield strength of the shear reinforcement
c_0: Thickness of the concrete cover
s: Spacing of the shear reinforcement (in longitudinal direction)
θ: Inclination of the shear cracks ($\theta \approx 35°$)

The equation above shows that and hence reduce with increasing curvature ductility . The typical design check representation is:


Pushover curve
Shear strength
Force
~ μ_Δ=2
~ μ_Δ=8
Deformation

Step 6: Detailing of the transverse reinforcement in the plastic hinge region

The goals of the transverse reinforcement are:
• stabilize the longitudinal reinforcement
• confine the boundary regions
• transfer shear

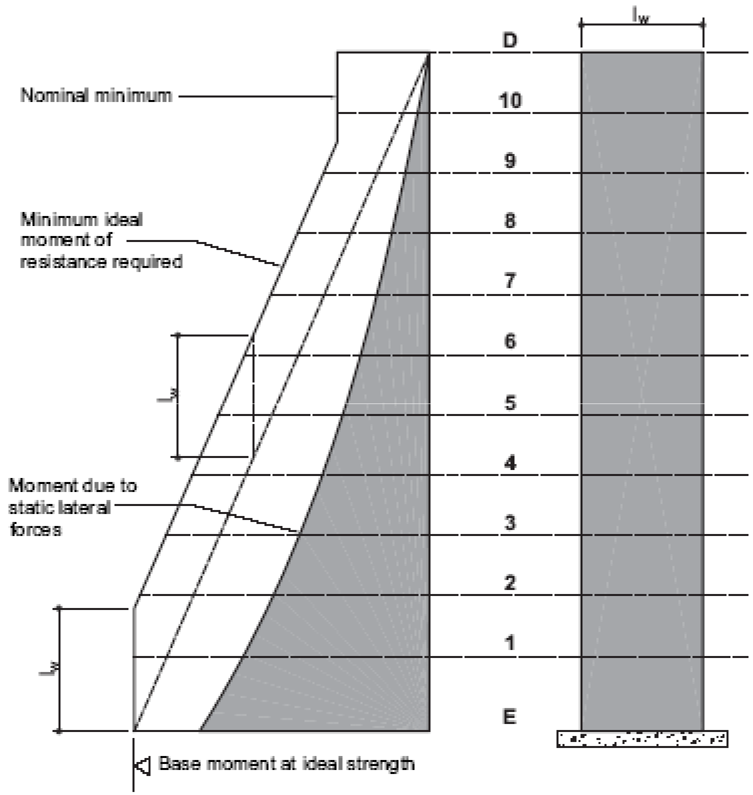
↓

The same transverse reinforcement can be used to fulfill all three requirement, i.e. the needed reinforcement quantity corresponds to the maximum of the single requirements and not to the sum of them (according to [PP92]).

In detail following requirements shall be met:
• Stabilizing and confining reinforcement
$s_h \leq b_w/2$ or $s_h \leq 6\ \Theta_l$ or $s_h \leq 150$ mm
• Shear reinforcement
$s \leq 2.5\ b_w$ or $s \leq 450$ mm

Step 7: Design an detailing of the elastic region

Flexural design



Shear design: should account for overstrength

Step 8: Design of the foundation

Conventional design with sectional forces (moment, shear, axial force) at overstrength

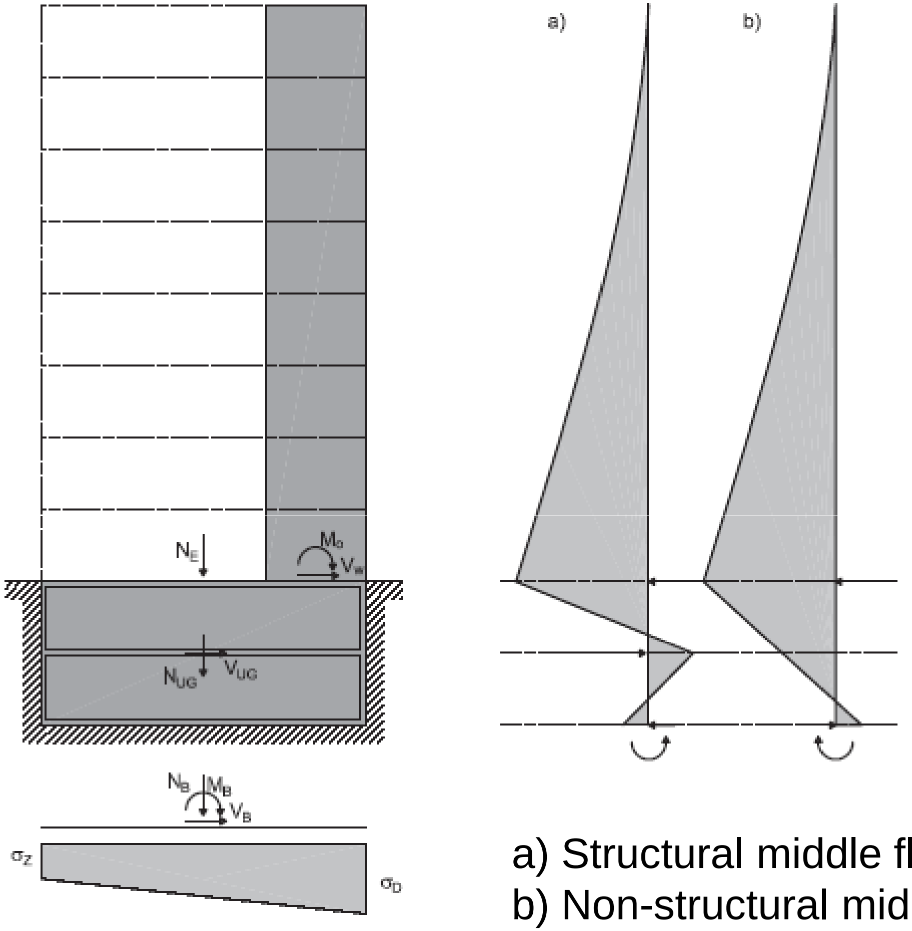
Requirements for the foundation

- Avoid differential deformations of the foundation
- Avoid foundations on strongly inhomogenous ground
- No yielding of the foundation (difficult to repair, the foundation should be capacity designed)
- Follow the forces from the top of the building to the ground
- Design for stability, sliding, soil pressure
- The behaviour of the foundation affects the dynamic properties of the building. Therefore the behaviour of the foundation shall be carefully assessed.
- RC structural walls are very efficient element and it could be difficult to provide enough strength to the foundation. For this reason the foundation should be checked already during early stages of the design process.

↓

In many cases a RC box foundation is needed

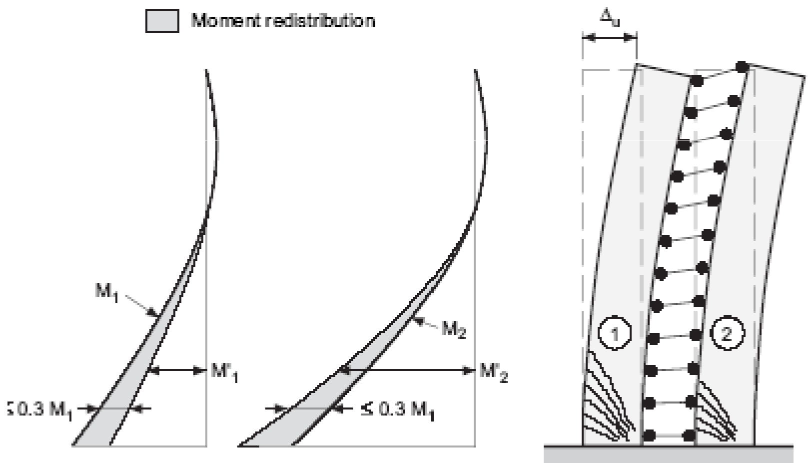
Furthermore the soil pressure should be checked

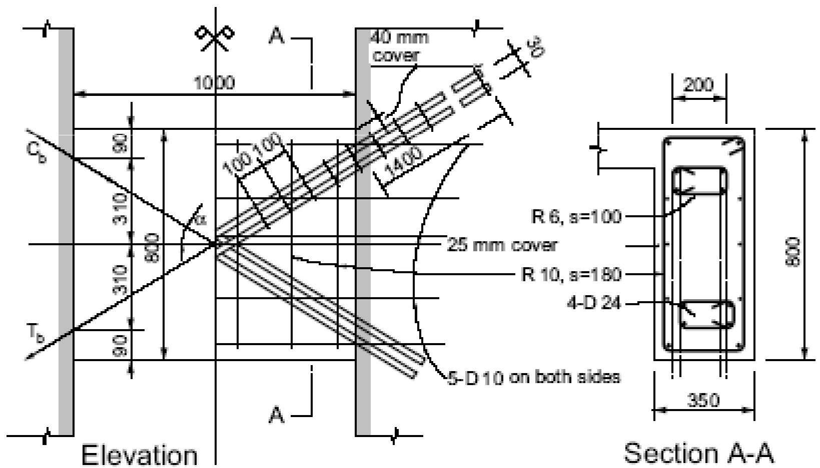


a) Structural middle floor
b) Non-structural middle floor (for horizontal forces)

Capacity design of coupled structural walls

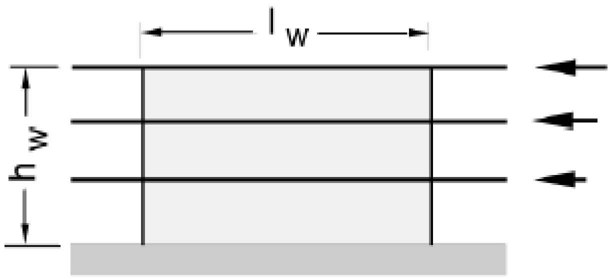
- Additionally to the two plastic hinge at the bottom of the walls, many other plastic hinges will form within the coupling beams increasing energy dissipation capacity
- Shear forces in the coupling beams change the axial load in the walls.
- The wall in compression is stiffer than the one in tension, hence attracting more forces.





Detailing for coupling beams

Squat walls



Three different types of walls with different behaviour:

- Elastic squat walls
- Uplifting squat walls
- Ductile squat walls (Very difficult to acheive!!!!)

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