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Foreword

In the last twenty, years earthquakes and the destruction caused by earthquakes consequently increased the need for seismic retrofit and earthquake safety investigation. In particular, due to damage caused by the previous earthquakes, destruction and loss of life has increased in later earthquakes. In addition, since governments and authorities are not ready for disasters and emergencies of natural disasters, rapid damage assessment and crisis management is gaining importance. It is essential to obtain rapid damage assessment while decision making during a crisis control and effective planning. To do this, a new and improved integrated model will be developed and this model will use image processing of satellite images before and after the event by using GIS system that used in the innovation of RADATT. Also, the structural information will be integrated with image processing with this new model. At the end of the project, necessary training will be given to the target audience (engineers, managers, etc.). By using pre-disaster databases earthquake safety investigation of existing structures can also be made rapidly before an earthquake disaster.

In this book the main aspects of the engineering seismology and the structural response of the buildings is presented. In particular the composition of this book is the following:

- Chapter 1: Engineering seismology.
- Chapter 2: Geographic Information System. How GIS works.
- Chapter 3: An introduction to the remote sensing using satellite.
- Chapter 4: Seismic risk and risk mitigation. An overview.
- Chapter 5: Building response under earthquake event.
- Chapter 6: Safety evaluation methodology.
- Chapter 7: Building information database form (Annex A).
- Chapter 8: Form for seismic performance evaluation (Annex B).
- Chapter 9: Using QuantumGIS software (Annex C).
- Chapter10: Using OpenSEES software (AnnexD).
- Acknowledgement.

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Chapter 1.

Engineering Seismology

In this chapter, we will show, briefly, the principal concepts of the seismology that will be necessary to understand the following steps and chapters.

Keywords: Tectonics, Faults, Magnitude, Intensity, Attenuation, Strong and Ground Motion Parameters.

1.1 The Earthquake

In the simplest way, we can say that an earthquake is a sudden release of stored energy in the earth crust. This energy is due to the result of the tectonic forces within the earth. Clearly this energy is stored along a very long period of time.

Earthquakes result in a number of phenomena or agents namely seismic hazards which can cause significant damage to the built environment such as fault rupture, vibratory ground motion, tsunami, liquefaction, land sliding, fire and hazardous materials release. In an earthquake event, any particular hazard can dominate, and historically each has caused major damage and great loss of life in particular.

Earthquakes may result from a number of causes:

- tectonic ground motions,
- volcanism,
- landslides,
- human interactions (i.e. explosions).

Among these causes, probably tectonic-related earthquakes are the largest and most important. These are caused by the fracture and sliding of rock along faults within the Earth's crust (see Figure 2).

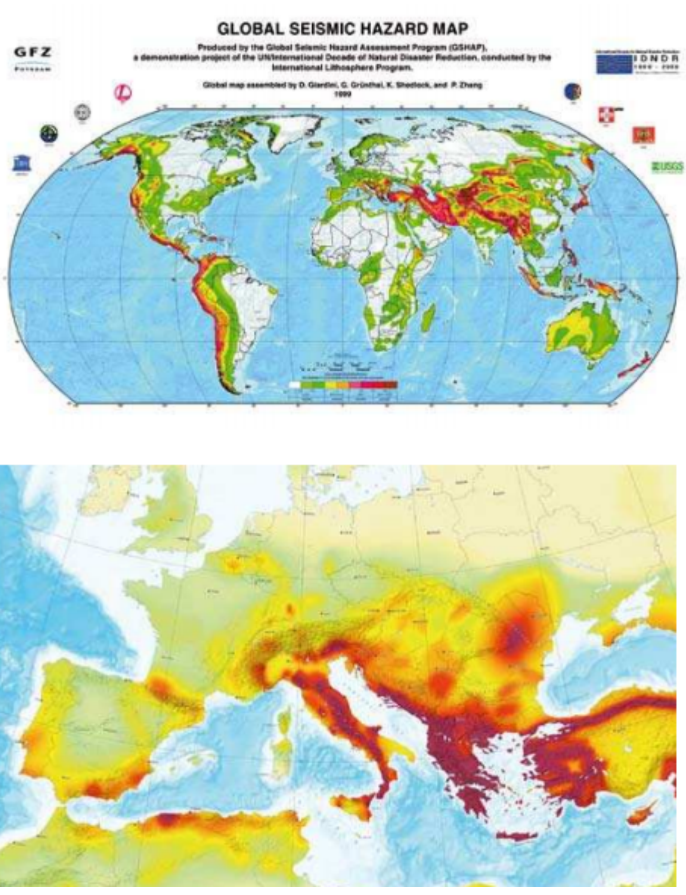


Figure 1 ***Global and Europe Seismic Hazard Map***

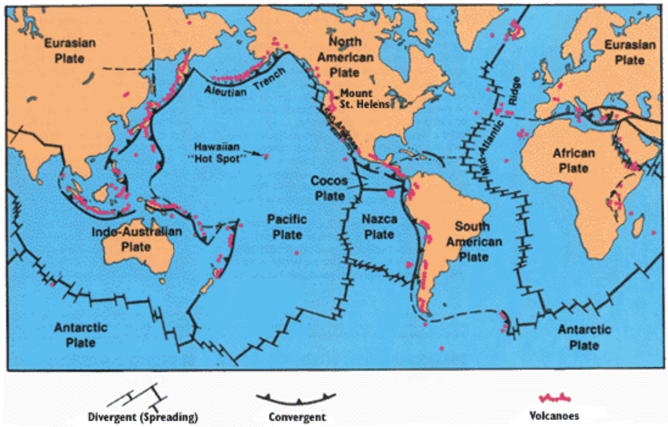


Figure 2 ***Active Volcanoes and Earthquake Fault***

A fault is a zone of the Earth's crust within which the two sides moved each other. This sudden motion causes seismic waves to radiate from their point of origin called the focus and travel through the earth up to the ground. It is seismic waves that can produce ground motion which people feel as an earthquake. In other words we can say that **tectonic earthquakes** are the result of the motion between number of large plates within the Earth's lithosphere (see Figure 3). These plates are driven by the convective motion of the material in the Earth's mantle, which in turn is driven by heat generated at the Earth's core. Relative plate motion at the fault interface is constrained by friction and/or asperities in this zone. If the energy stored in these plates overcomes the "rock's" resistance, the slip between the two sides of the fault become, and thus we have the earthquake.

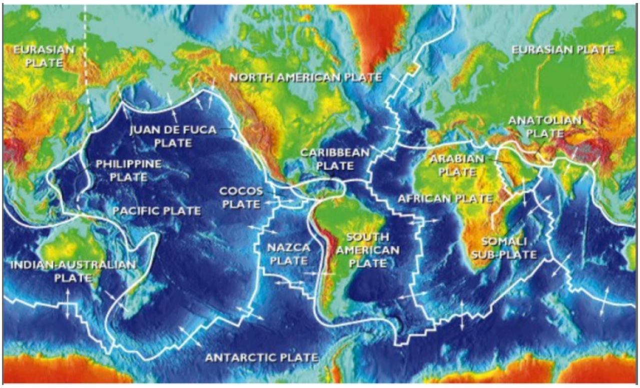


Figure 3 ***Tectonic Plates of the Earth***

As curiosity, it has been observed that each year there are thousands of earthquakes that can be felt by people and over one million that are strong enough to be recorded by instruments.

The study of the seismologists showed that the most part of earthquakes take place normally along faults in the upper 40 km of the Earth's surface. Earthquakes that happen along these zones can be divided into two groups:

- shallow focus earthquakes that have focal depths less than about 70 km;
- deep focus earthquakes that have focal depths between 75 and 700 km.

Within the first group we can classify these kinds of earthquakes (see Figure 4):

- **Earthquakes at Diverging Plate Boundaries.** Diverging plate boundaries are zones where two plates move away from each other, such as at oceanic ridges. In such areas the lithosphere is in a state of tensional stress and thus normal faults and rift valleys occur. Earthquakes that occur along such boundaries show normal fault motion and tend to be shallow focus earthquakes, with focal depths less than about 20 km. Such shallow focal depths indicate that the brittle lithosphere must be relatively thin along these diverging plate boundaries.

- **Earthquakes at Transform Fault Boundaries.** Transform fault boundaries are plate boundaries where lithospheric plates slide past one another in a horizontal fashion. The San Andreas Fault of California is one of the longer transform fault boundaries known. Earthquakes along these boundaries show strike-slip motion on the faults and tend to be shallow focus earthquakes with depths usually less than about 50 km.

* **Earthquakes at Converging Plate Boundaries.** Convergent plate boundaries are boundaries where two plates run into each other. Thus, they tend to be zones where compressional stresses are active and thus reverse faults or thrust faults are common. There are two types of converging plate boundaries:

  a) *Subduction boundaries* where oceanic lithosphere is pushed beneath either oceanic or continental lithosphere;

  b) *collision boundaries* where two plates with continental lithosphere collide.

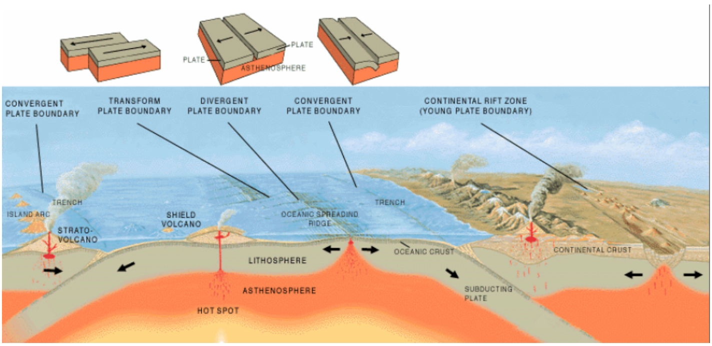


Figure 4 ***Plate Boundaries***

1.2 Faults and Types of Faulting

As said above, fault is a fracture, or zone of fractures, between two blocks of rock in the earth crust, where these blocks are moving relatively to each other. This movement may occur rapidly, in the form of an earthquake or may occur slowly, in the form of creep. Faults may range in length from a few meters to thousands of kilometers (see Figure 5 and Figure 6). Most faults produce repeated displacements over geologic time. During an earthquake, the rock on one side of the fault suddenly slips with respect to the other. The fault surface can be horizontal or vertical or some arbitrary angle in between.

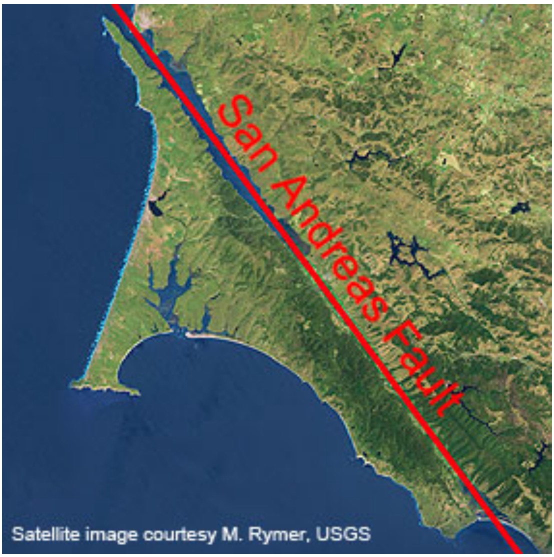


Figure 5 ***San Andreas Fault. Probably the Most Famous Fault in the World.***

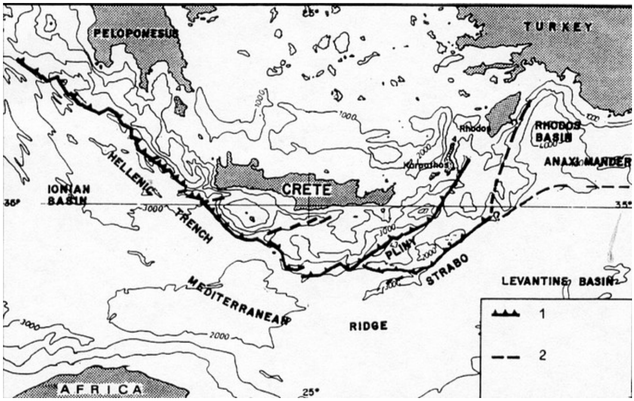


Figure 6 ***Main Tectonic Elements of the Eastern Mediterranean. Legend: 1) Trench, 2) Faults***

In engineering seismology it is possible to classify the fault using simply the angle of the fault with respect to the surface, and the direction of slip along the fault (dip angle, see Figure 7). Faults that move along the direction of the dip plane are dip-slip faults and described as either normal or reverse, depending on their motion. Faults that move horizontally are known as strike-slip faults and are classified as either right-lateral or left-lateral. Faults that show both dip-slip and strike-slip motion are known as oblique-slip faults.

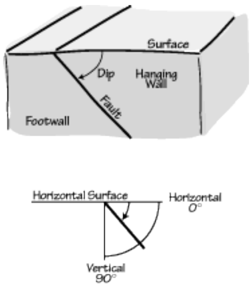
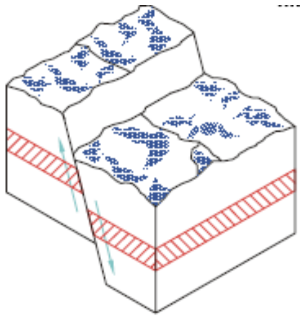
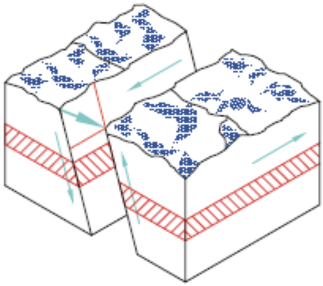


Figure 7 ***Dip Angle in a Fault***

The classification of faults depends only on the geometry and direction of relative slip. Various types are sketched in Figure 8. The dip of a fault is the angle that fault surface makes with a horizontal plane and the strike is the direction of the fault line exposed at the ground surface relative to the north. A strike-slip fault, sometimes called a transcurrent fault, involves displacements of rock laterally, parallel to the strike. If when we stand on one side of a fault and see the motion on the other side is from left to right, the fault is right-lateral strike-slip. Similarly, we can identify left-lateral strike-slip.



NORMAL FAULT



LEFT LATERAL NORMAL FAULT
(LEFT OBLIQUE NORMAL FAULT)

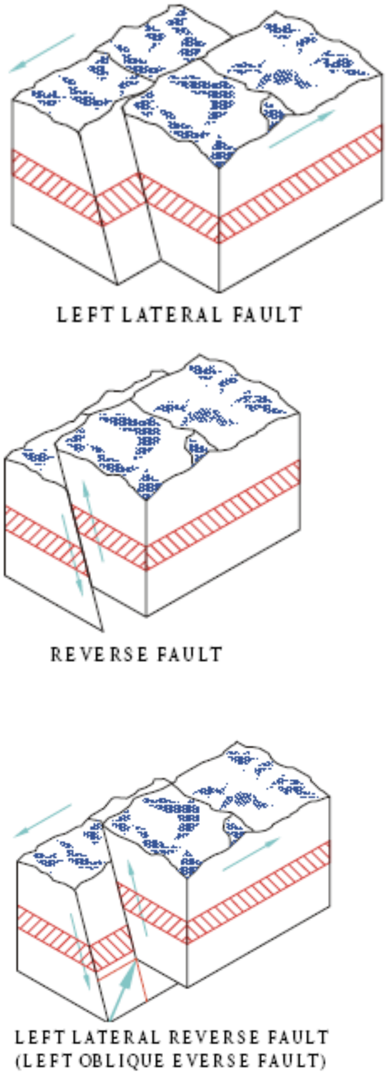


Figure 8 ***Types of Faulting***

1.3 Seismic Waves

Seismic waves represent the energy that travels through the earth due to the sudden breaking of earth crust. They are recorded by a particular instrument called seismographs (see Figure 9).

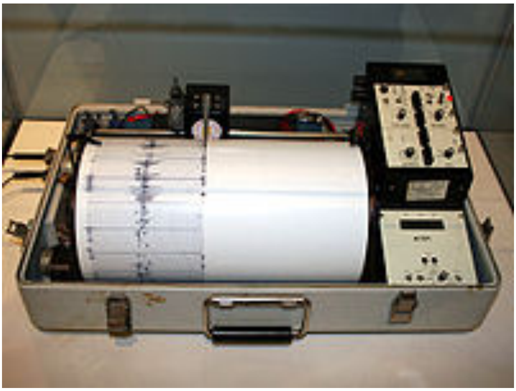


Figure 9 ***Seismograph***

There are several different kinds of seismic waves, and they all move in different ways. The two main types of waves are **body waves** and **surface waves**. The first one can travel through the earth's inner layers, while the second one can only move along the surface of the planet. Speaking about the earthquake there are two important “physical point” to keep in your mind:

- **hypocenter or focus** that is the point where an earthquake or underground explosion originates
- **epicenter** that is the point on the Earth's surface that is directly above the hypocenter.

You can see that just said shown in the picture below (see Figure 10).

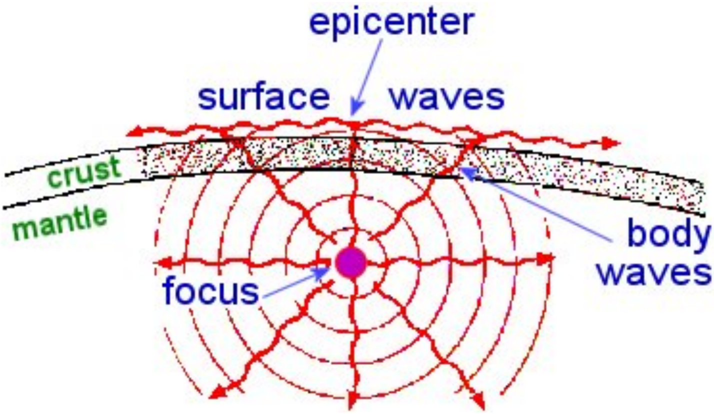


Figure 10 ***Body and Surface Waves***

Traveling through the interior of the earth, **body waves** arrive before the surface waves emitted by an earthquake. These waves are of a higher frequency than surface waves. Within these kind of waves we have:

- ***P - Waves***

The first kind of body wave is the P wave or primary wave. This is the fastest kind of seismic wave, and, consequently, the first to 'arrive' at a seismic station. The P wave can move through solid rock and fluids, like water or the liquid layers of the earth. It pushes and pulls the rock it moves through just like sound waves push and pull the air. P waves are also known as **compressional waves**, because of the pushing and pulling they do while traveling through the earth crust.

- ***S–Waves***

The second type of body wave is the S wave or secondary wave, which is the second wave you feel in an earthquake. An S wave is slower than a P wave and can only move through solid rock, not through any liquid medium. It is property of S waves that led seismologists to conclude that the Earth's **outer core** is a liquid. S waves move rock particles up and down, or side-to-side-perpendicular to the direction that the wave is traveling in the direction of wave propagation.

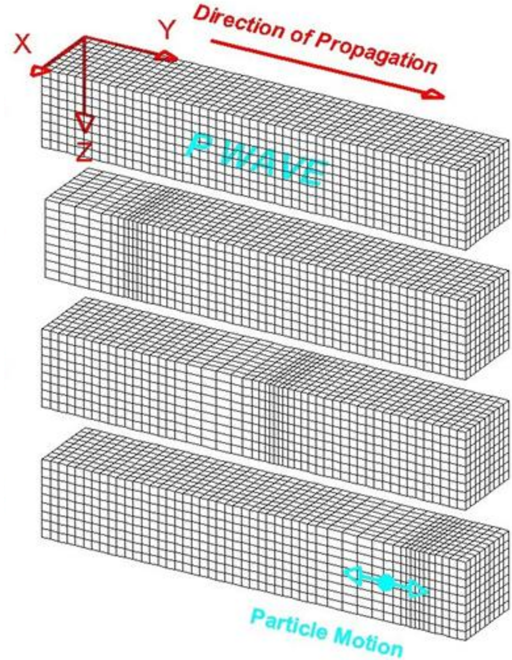


Figure 11 ***Propagation of P-Waves***

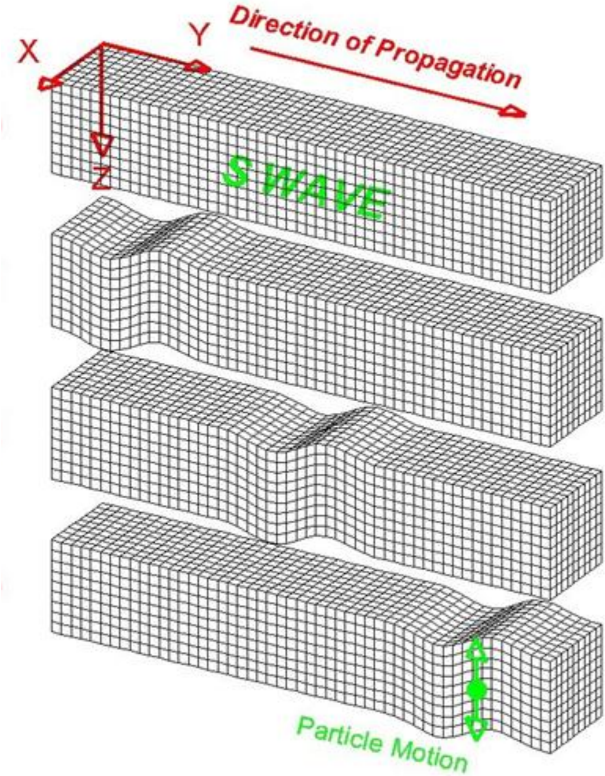


Figure 12 ***Propagation of S-Waves***

As said above, **Surface waves** travel only through the crust and they are of a lower frequency than body waves, and are easily distinguished on a seismogram as a result. Though they arrive after body waves, these waves are almost entirely responsible for the damage and destruction associated with earthquakes. This damage and the strength of the surface waves are reduced in deeper earthquakes. Within these kind of waves we have:

- ***Love Waves***

The first kind of surface wave is called a **Love wave**, named after A.E.H. Love, a British mathematician who worked out the mathematical model for this kind of wave in 1911 (Figure 13). It's the fastest surface wave and moves the ground from side-to-side. Confined to the surface of the crust, Love waves produce entirely horizontal motion.

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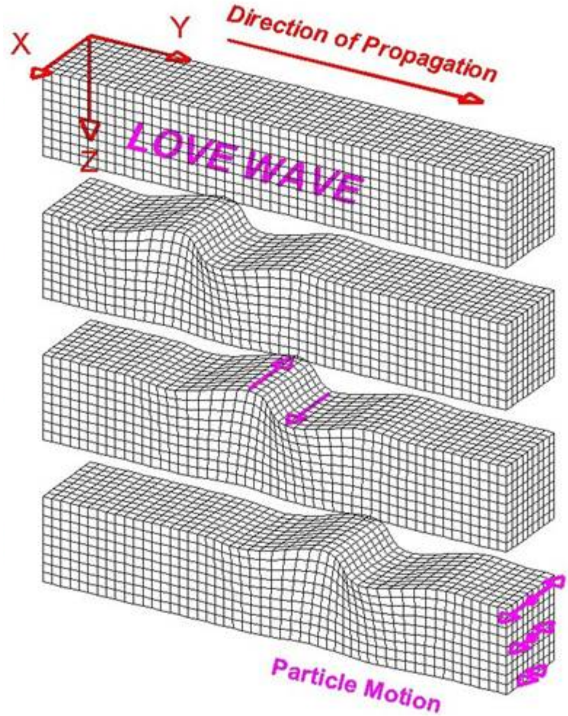


Figure 13 ***Propagation of Love Waves***

- ***Rayleigh Waves***

The other kind of surface wave is the Rayleigh wave, named for John William Strutt, Lord Rayleigh, who mathematically predicted the existence of this kind of wave in 1885 (see Figure 14). A Rayleigh wave rolls along the ground just like a wave rolls across a lake or an ocean. Because it rolls, it moves the ground up and down, and side-to-side in the same direction that the wave is moving. Most of the shaking felt from an earthquake is due to the Rayleigh wave, which can be much larger than the other waves.

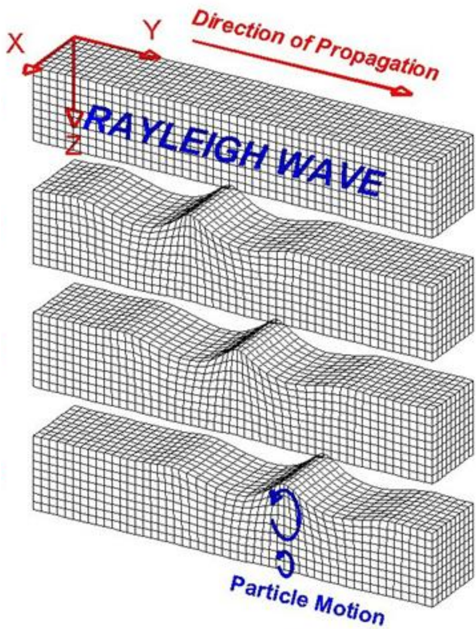


Figure 14 ***Propagation of Rayleigh Waves***

1.4 Measurement of Earthquakes

Once defined some general properties of the earthquake, and the related seismic waves, now it is time to define how it is possible to measure the “strength” of an earthquake. The oldest and simple way is called the **earthquake intensity**, where the “intensity” is the *measure of damage* to works of man, to the ground surface, and of human reaction to the shaking and so forth. It is important to underling that this kind of measure do not depend on instrument, but only on the observation of effects in the seismic zone. Historically, the first intensity scale, with values from I to X, was developed by De Rossi (Italy) and Forel (Switzerland) in the 1880s. A more refined scale was devised in 1902 by the Italian volcanologist and seismologist Mercalli with a twelve-degree range from I to XII. A modified version by H.O. Wood is given below.

Modified Mercalli Intensity Scale (MMI) - 1931

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

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.

III. Felt quite noticeably indoors, especially on upper floors or buildings, but many people do not recognize it as an earthquake. Duration estimated.

IV. During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make cracking sound.

V. Felt by nearly everyone, many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop.

VI. Felt by all, many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.

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

VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings, with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. San and mud ejected in small amounts. Changes in well water.

IX. Damage considerable in specially designed structures; well designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep sloes. Shifted sand and mud. Water splashed (slopped) over banks.

XI. Few, if any, (masonry) structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rail bent greatly.

XII. Damage total. Practically all works of construction are damaged greatly or destroyed. Waves seen on ground surface. Lines of sight and level are distorted. Objects are thrown into the air.

Also if this scale in useful for classify the strength of an earthquake, as already said above it is not depending to a physical quantities, but it is related to a “subjective” variables that depending for example from the type of construction, the constructions techniques employed and the related technological developments in the field of the earthquake’s preventions, the density of the population and so on. So we need to define another kind of quantities that is able to classify the earthquake from an “objective” point of view in way that an earthquake that occurred can be classified in the same kind in all part of the world, independently from where it happened. A strictly quantitative scale that can be applied to earthquakes in both inhabited and uninhabited regions was originated in 1931 by Wadati in Japan and developed by Charles Richter in 1935 in California. **Richter** defined the magnitude of a local earthquake as the logarithm to base ten of the maximum seismic wave amplitude in microns (10-4 centimeters) recorded on a Wood-Anderson seismograph located at a distance of 100 kilometers from the earthquake epicenter. This means that every time the magnitude goes up by one unit, the amplitude of the earthquakes waves increase 10 times. Since the fundamental period of the Wood-Anderson seismograph is 0.8 second, it selectively amplifies those seismic waves with a period ranging approximately from 0.5 to 1.5 seconds. Because the natural period of many building structures are within this range, the local Richter magnitude remains of value to engineers. In other word the Richter magnitude tends to measure the energy of the seismic phenomenon based on purely instrumental measurement.

It follows from the definition of the Richter magnitude, that it has no theoretical upper or lower limits. However, the size of an earthquake is limited at the upper end by the strength of the rocks of the Earth's crust. Since 1935, only a few earthquakes have been recorded on seismographs that have had a magnitude over 8.0. At the other extreme, highly sensitive seismographs can record earthquakes with a magnitude of less than minus two. In table below for an average number of world-wide earthquakes of various magnitudes are given. Generally speaking, shallow earthquakes have to obtain Richter magnitudes of more than 5.5 before significant widespread damage occurs near the source of the waves. At its inception, the idea behind the **Richter Magnitude** or **Local Magnitude** scale (**M_L**) was a modest one. It was defined for Southern California, shallow earthquakes, and epicenter distances less than about 600 kilometers.

Today, the method has been extended to apply to a number of types of seismographs throughout the world. Consequently, a variety of magnitude scales based on different formulas for epicenter distance and ways of choosing an appropriate wave amplitude emerged. For example we have:

- **Surface Wave Magnitude (M_s) :** Surface waves with a period around 20 seconds are often dominant on the seismograph records of distant earthquakes (epicentral distances of more than 2000 kilometers). To quantify these earthquakes, Gutenberg defined a magnitude scale (Ms) which is based on measuring the amplitude of surface waves with a period of 20 seconds.

World-wide Earthquakes per Year

Magnitude M_s	**Average No. > M_s**
8	1
7	20
6	200
5	3,000
4	15,000
3 >	100,000

- **Body Wave Magnitude (M_b) :** Deep focus earthquakes have only small or insignificant trains of surface waves. Hence, it has become routine in seismology to measure the amplitude of the P wave, which is not affected by the focal depth of the source, and thereby determine a P wave magnitude (M_b). This magnitude type has also been found useful in continental regions like the eastern United States where no Wood-Anderson instruments have operated historically.
- **Moment Magnitude (M_W) :** Because of significant shortcomings of M_L, M_b, and to a lesser degree M_s in distinguishing between great earthquakes, the moment magnitude scale was devised. This scale assigns a magnitude to the earthquake in accordance with its seismic moment (M_0) which is directly related to the size of the earthquake source. The moment-magnitude scale (MW) is the only magnitude scale which does not suffer from the problems of other magnitude types for great earthquakes. The reason is that it is directly based on the forces that work at the fault rupture to produce the earthquake and not the recorded amplitude of specific types of seismic waves.

In light of the above information, application of different scales have been suggested for measuring shallow earthquakes of various magnitudes:

M_D	for magnitudes less than 3
M_L or M_b	for magnitudes between 3 and 7
M_s	for magnitudes between 5 and 7.5
M_W	for all magnitudes

1.5 Ground Motion Parameters

Some parameters can be useful to characterize the ground motion within the earthquake engineering applications. These parameters are:

- **Peak ground motion** (peak ground acceleration, peak ground velocity, and peak ground displacement);
- **Duration of motion**;
- **Frequency content**.

These quantities are important in the earthquake engineering applications because they influences the response of a structure. Peak ground motion primarily influences the vibration amplitudes. Duration of motion has a pronounced effect on the severity of shaking. It is shown that a ground motion with a moderate peak acceleration and a long duration may cause more damage than a ground motion with a larger acceleration and a shorter duration. Frequency content strongly affects the response characteristics of a structure. In a structure, ground motion is more amplified when the frequency content of the motion and the natural frequencies of the structure are close to each other.

More in details the **peak ground motion parameters** are respectively peak ground acceleration, velocity and displacement. There is no clear relationship between the earthquake magnitude, epicenter distance, and site description and these parameters. A list of earthquakes with their ground motion parameters are given in the table below (see
Figure **15**). Peak ground acceleration had been widely used to scale earthquake design spectra and acceleration time histories. Later studies recommended that in addition to peak ground acceleration, peak ground velocity and displacement should also be used for scaling purposes.

Earthquake and location	Mag.	Epicentral distance (km)	Comp.	Peak Acc. (g)	Peak Vel. (in/sec)	Peak Disp. (in)	Site Description
Helena, 10/31/1935	6.0	6.3	S00W	0.146	2.89	0.56	Rock
Helena, Montana Carroll College			S90W	0.145	5.25	1.47	
			Vert	0.089	3.82	1.11	
Imperial Valley, 5/18/1940	6.9	11.5	S00E	0.348	13.17	4.28	Alluvium, several 1000 ft
El Centro site			S90W	0.214	14.54	7.79	
			Vert	0.210	4.27	2.19	
Western Washington, 4/13/1949	7.1	16.9	N04W	0.165	8.43	3.38	Deep cohesionless soil,
Olympia, Washington Highway Test			N86E	0.280	6.73	4.09	420 ft
Lab			Vert	0.092	2.77	1.59	
Northwest California, 10/7/1951	5.8	56.2	S44W	0.104	1.89	0.94	Deep cohesionless soil,
Ferndale City Hall			N46W	0.112	2.91	1.08	500 ft
			Vert	0.027	0.87	0.64	
Kern County, 7/21/1952	7.2	41.4	N21E	0.156	6.19	2.64	40 ft of alluvium over
Taft Lincoln School Tunnel			S69E	0.179	6.97	3.60	poorly cemented
			Vert	0.105	2.63	1.98	sandstone
Eureka, 12/21/1954	6.5	24.0	N11W	0.168	12.44	4.89	Deep cohesionless soil,
Eureka Federal Building			N79E	0.258	11.57	5.53	250 ft deep
			Vert	0.083	3.23	1.83	
Eurkea, 12/21/1954	6.5	40.0	N44W	0.159	14.04	5.58	Deep cohesionless soil,
Ferndale City Hall			N46E	0.201	10.25	3.79	500 ft deep
			Vert	0.043	2.99	1.54	
San Francisco, 3/22/1957	5.3	11.5	N10E	0.083	1.94	0.89	Rock
San Francisco Golden Gate Park			S80E	0.105	1.82	0.33	
			Vert	0.038	0.48	0.27	
Hollister, 4/8/1961	5.7	22.2	S01W	0.065	3.06	1.12	Unconsolidated alluvium
Hollister City Hall			N89W	0.179	6.75	1.51	over partly consolidated
			Vert	0.050	1.85	0.85	gravel
Parkfield, 6/27/1966	5.6	56.1	N05W	0.355	9.12	2.09	Alluvium
Cholame Shandon, California Array			N89E	0.434	10.02	2.80	
No. 5			Vert	0.119	2.87	1.35	
Borrego Mountain, 4/8/1968	6.4	67.3	S00W	0.355	9.12	2.09	Alluvium
El Centro site			S90W	0.434	10.02	2.80	

Figure 15 ***Earthquakes Versus Ground Motion Parameters***

We can also define:

- **Peak Horizontal Acceleration (PHA):** Simply the maximum absolute value of acceleration in the ground motion record. Two component accelerations can be used along with vector addition to determine a single horizontal acceleration estimate.
- **Peak Vertical Acceleration (PVA):** Often assumed to be 2/3 of the PHA (Newmark and Hall 1982), but the ratio is quite variable. PVA can be quite large and can not be ignored.
- **Peak Horizontal Velocity (PHV):** Likely more accurate than PHA to characterize ground motion amplitude more accurately for intermediate frequencies.

An example set of accelerations from a large earthquake that occurred near the coast of Mexico in September of 1985, are shown in the picture below (see Figure 16).

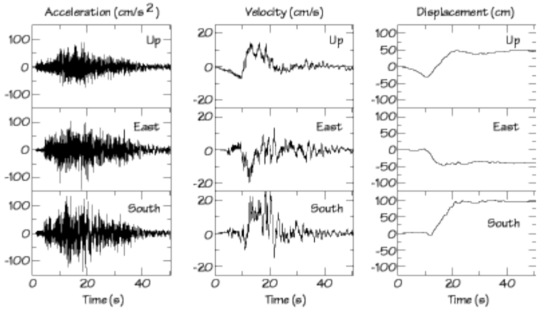


Figure 16 Example for Acceleration, Velocity and Displacement Time History of a Recorded Ground Motion for Three Components

Once showed the parameters that characterize the earthquake ground motion, is necessary underling the parameters that influence this motion. Simply attenuation can be defined as the rate at which earthquake ground motion decreases with distance due to topography and soil properties on the travel path of the seismic waves. The most important factors are:

- earthquake magnitude;
- distance from the source of energy release (epicenter distance or distance from the causative fault);
- local soil conditions;
- variation in geology and propagation velocity along the travel path;

- earthquake source conditions and mechanism (fault type, slip rate, stress conditions, stress drop, etc.).

Past earthquake records have been used to study some of these influences. While the effect of some of these parameters such as local soil conditions and distance from the source of energy release are fairly well understood and documented, the influence of source mechanism is under investigation and the variation of geology along the travel path is complex and difficult to quantify. It should be noted that several of these influences are interrelated, consequently, it is difficult to discuss them individually without incorporating the others.

Chapter 2.

Geographic Information System:

How GIS Works

In this chapter we will show, briefly, what is it a GIS, and what are its potentiality and principal functionality.

Keywords: GIS, Vector Model, Raster Model, Comparison Pre- and Post-event

2.1 What is GIS ?

A **geographic information system** (GIS) integrates hardware, software, and data for capturing, managing, analyzing, and displaying all forms of geographically referenced information (see Figure 17).

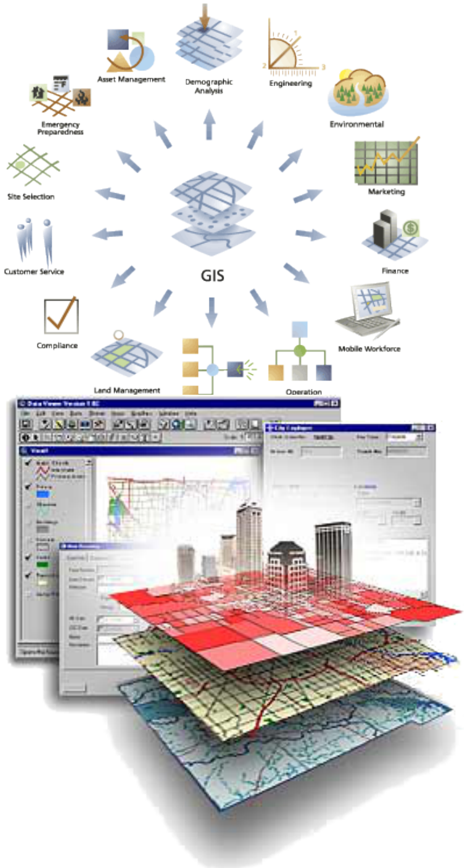


Figure 17 ***Example of GIS System***

GIS allows us to view, understand, question, interpret, and visualize data in many ways that reveal relationships, patterns, and trends in the form of maps, globes, reports, and charts.

A GIS helps you answer questions and solve problems by looking at your data in a way that is quickly understood and easily shared.

Finally GIS technology can be integrated into any enterprise information system framework.

2.2 How GIS Works

The GIS stores geographic information as a collection of theme layers that can be related to each other through connection and geographical overlap.

Geographical information can be acquired in two basic ways: by using the *vector* model or by using the *raster* model. In the **vector** model, as we will discuss later, information of points, lines and polygons are encoded and stored as series of coordinates *(x, y)*, while the **raster** model is constituted by a grid of regular cells (pixels) which represents a specific value. Each cell in a raster is characterized by its grid position (row number and column); each object can be represented by a single cell, or from a set of close cells.

Actually, GISs allow you to do the following operations:

- *Input*: different information are provided to the system in the form of digital data; the data may be graphic elements (points, arcs, polygons), images (aerial, by satellite, scan) or tabular data related to the previous items.

- *Handling*: the data must often be manipulated so as to make them compatible with your own geographical system, then you are geocoding operations and ortho-referencing.
- *Management*: when the number of data is high we use a relational database to store, manage and organize them; the data is stored as a collection of tables, the common fields between tables serve as a key relationship.
- *Questioning and analysis*: GIS allows both to make simple interactive queries (such as “point and click”) and to perform sophisticated analyses related to the contents of their specific disciplines territorial.
- *Display*: showing geographic information you can print maps or charts; these are useful tools that allow an approach to the information managed by the GIS to users who are not insiders.

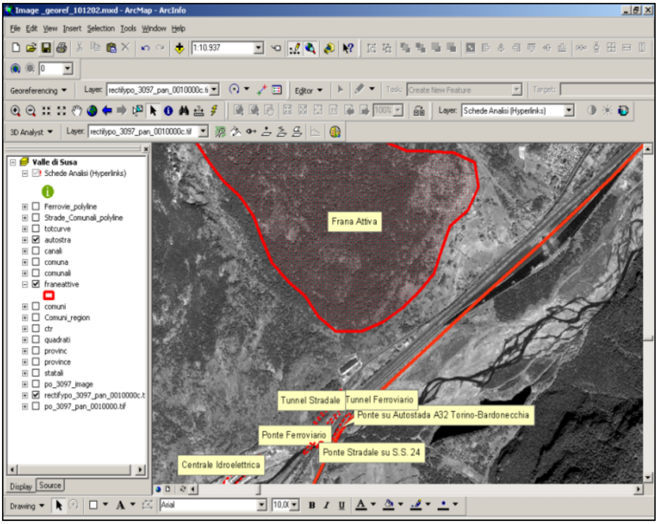


Figure 18 *"Landslide GIS" using ArcMap – ESRI. From RADATT Project – Val Susa Valley (ITA)*

2.3 Comparison Between Pre- and Post-Event Images

Once all the data have been collected on the territory within the GIS, you can make it refreshable time (almost) real. In particular, the assessment of landslide hazard, which is continually updated, helps to identify risk areas, with all the benefits that it offers. However, to do this it is necessary to have a series of images in time sequence and with a certain level of accuracy. The identification procedure of the effects caused by catastrophic events through the use of satellite images, has been tested in several studies.

The techniques used for the comparison between the pre- and post-event are implemented in two algorithms:

- comparison based on the identification of segments and contours;
- comparison based on the analysis of the forms and perceptual grouping.

The two algorithms are integrated within the GIS allow to identify the changes induced in the territory by any catastrophic event, in a time interval reduced by using a compromise between the accuracy of the image and the need to provide an estimate of damage in a quick time.

Assuming to be able to have a pair of images taken at the turn of a particular event which caused an alteration of the geometry of the slope (such as a mild shock seismic event or a particularly intense meteor), it is possible to enter the code calculation and update the parameters of the simulation. In this way, one is able to control the degree of danger of the slope. In Figure 19 you can see the two satellite images for comparison.

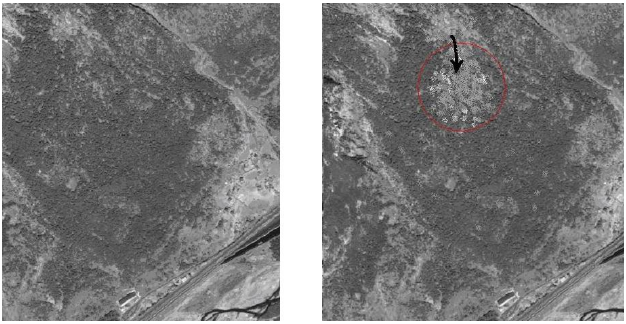


Figure 19 ***Comparison Satellite Images Pre- and Post-Event. From RADATT Project – Val Susa Valley (ITA)***

Following the event there was a landslide that caused the detachment of a certain quantity of material. The soil thus collapsed has reached a slope area that is in a condition of unstable equilibrium, and in which even a small increase in load on the surface of rupture may trigger a landslide of considerable magnitude.

An analysis of satellite images can determine the extent of soil collapsed and then make a quantitative estimate (and qualitative) of the material in question. At this point it is thus able to update the simulation with the new parameters thus obtained, and to determine whether or not there is an actual danger.

The analysis of the updated and integrated into the GIS, thus allows the **real-time monitoring of the situation**.

2.4 Data Model

In a GIS all objects on the Earth's surface are represented by three essential features: geometry, topology and attributes. The geometry reproduces the shape of objects and is based on three basic elements: point, line (or arc) and polygon (or area). A point is used to reproduce point elements, such as a spot dimension, a well, or the position of a meteorological station. The line defines a linear elements like a road, a power line or a watercourse. The polygon defines a confined area such as a building, a lake or a geological outcrop.

The topology is the set of information about the mutual spatial relations between the different elements such as wireless, adjacency or inclusion. For example, specifies whether an arc is common to two adjacent polygons, or if a polygon is completely enclosed within another. The attributes are descriptive data of individual real objects. For an element representing a point well, such attributes can be set up from the depths, the year of drilling and the owner, a weather station air temperature or precipitation, a road width, the category or type of flooring.

2.5 Data Presentation

As already said above, GIS can be used in two different techniques of data representation: *vector* or *raster*. In **vector** representation a point is defined by a pair of coordinates while a line or a polygon from coordinates of a set of points which when connected to each other with straight segments, forming the graphical representation of the object (Figure 20). Generally the two points at the ends of a line are called nodes, the intermediate points of a broken line are defined vertices.

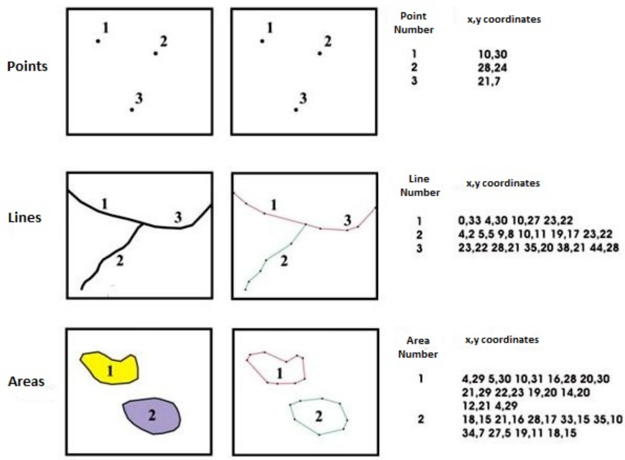


Figure 20 ***Vector Representation: points, lines (arcs), and areas (polygons)***

A vector image is constituted by a set of geometrical characteristics and attributes. The geometry is stored in a specific vector format. All attributes of vector images are saved in a database table and connected to the geometric characteristics through a DBMI (Data Base Management Interface). The vector model is the functional description of the elements that have a specific dimension and a geographical space, but it is less convenient to describe the information that varies continuously as the morphology of the soil.

In the representation **raster** area considered is divided into a set of cells, generally square in shape, in each of which is recorded the attribute (or category) present. Each cell is then assigned a numerical value (see Figure 21). For each object in a raster map can possibly be attributed, in addition to the value category, a descriptive label. The vector and raster formats are saved in different directors, and are managed by different commands, so you can assign the same name to a card vector and a raster without problems of conflict.

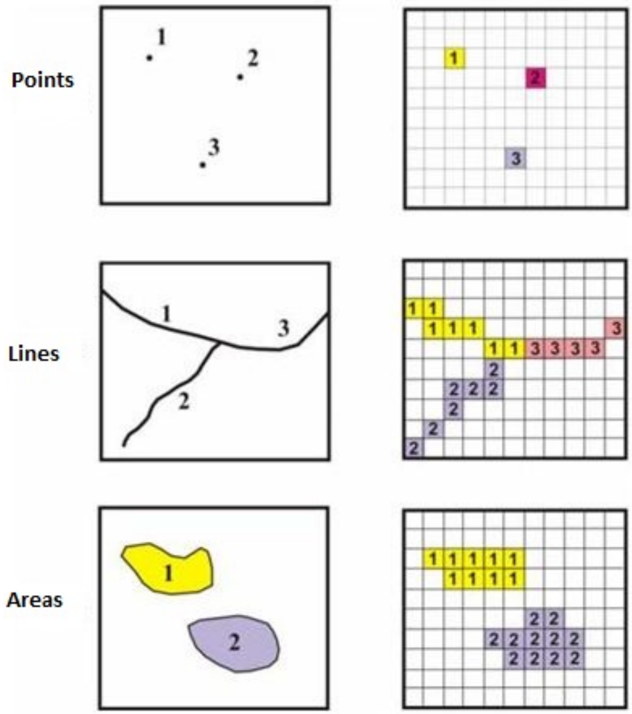


Figure 21 ***Raster representation: points, lines (arcs), and areas (polygons)***

In a GIS the different categories of objects on the earth's surface is divided into several processed, or cards. Each paper thus contains a different characteristic or thematic, such as hydrography, the altitude, etc.. (Figure 22). It’s also, possible to insert different themes in a same card by dividing them into separate layers. The data in raster form generally occupy more memory data in vector form as to each cell is assigned an attribute, even if there are techniques of compaction data which limit this drawback. The advantage is that geographic space is uniformly defined in a simple and predictable. In this way the raster systems generally have more analytical power of vector systems in the analysis of the continuous space and are therefore suitable for the analysis of data that exhibit a continuous variability in space, such as temperature, rainfall, etc. altimetry. These systems are therefore an optimal evaluation of problems that include many mathematical combinations of data from different themes. They are therefore excellent in the evaluation of environmental models. Finally, since the images by satellite employ a raster structure, most of these systems can easily incorporate and process data of this type.

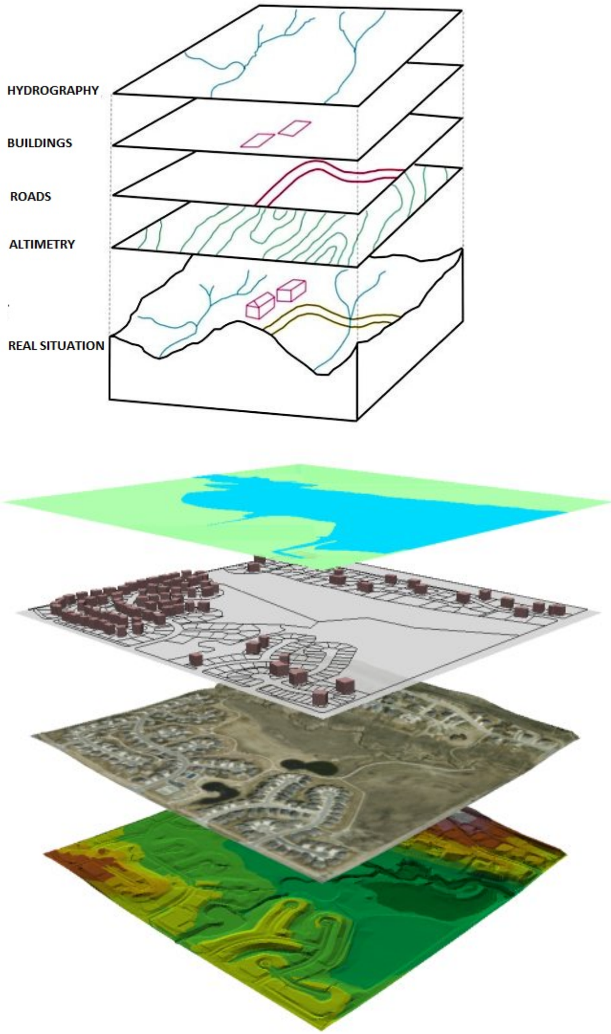


Figure 22 ***Two Representation of the Territory Through Different Themes, Subject to Processing by a GIS***

2.6 GIS Component

As the user a GIS may appear as a single computation program, in reality it is typically divided into a number of components, or elements, with different functions. In most GIS can commonly identify the following essential components:

(a) *a spatial database and attribute*: consists of a set of cards and associated information, in digital form. Since the database contains objects, or elements, of the earth's surface, it is possible to distinguish a spatial database that describes the geography (shape and position) of objects, and a database of attributes describing the characteristics or quality of the same objects. Thus, for example, it is possible to have the perimeter of a portion of polygonal surface defined in the spatial database using the coordinates of the vertices and some of its characteristics, such as lithology, the soil type, the average gradient, contained in the database attributes.

(b) *a display system*: contains those components that allow the display of items in the database to produce cards on screen and on paper by a printer or plotter. Generally a GIS does not produce sophisticated representations, delegating to other systems specifically dedicated to the production of high quality processed.

(c) *a digitizing system*: is constituted by a program to convert existing map data on paper in digital form, and then subject to processing by the GIS. The digitization is commonly performed using a graphics tablet (or digitizer) or directly from the screen of scanned images.

(d) *a system of geographic analysis*: a fundamental characteristic of a GIS, which distinguishes it from traditional Management Systems Database (DBMS Data Base Management System), is the ability to compare different entities according to their topology. Imagine for example that a certain area has either two processed digitized, representing the first distribution of various plants, according to the distribution of the various lithology. The two themes, vegetation and lithology, of course, do not exhibit the same topology in other words the plots with the different types of vegetation are in the shape and distribution different from the portions of land occupied by the various lithological types. With a GIS is possible to identify those areas in which a particular type plant is associated with a particular lithology. This type of operation, much used in GIS, is defined overlay, as it is equivalent to the operation manual of overlapping transparent papers containing different themes. And operation not possible through the common DBMS for the lack of topological information of the objects analyzed.

We can conclude this briefly, introduction to the GIS with this note: the potentiality of a GIS system is enclosed into is core. In other words its potentiality is due to the possibility to store, manage, and query, not a simple data base, but a data base that include within it both alphanumeric and geographic information. So the potentiality of management in different filed can be reach with this instruments. Naturally this system is good only if the data base is correctly fill-in with all the data for that purpose, and periodically up-dated.

With the last development of the technology, the GIS system could be used not only from professionals, but also by everyone (see Figure 23). Moreover it is possible to have always your GIS system thanks the mobile smartphone (see Figure 24).

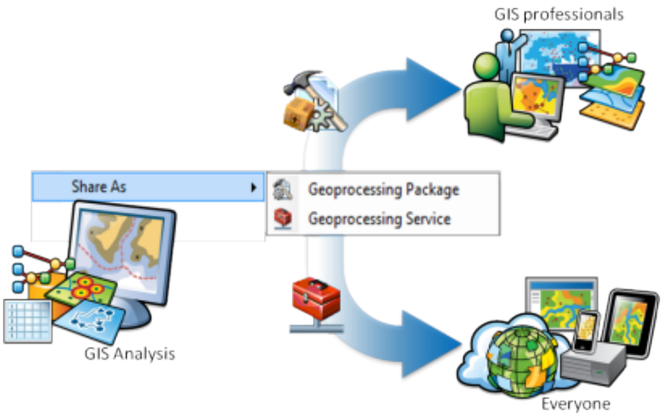


Figure 23 ***GIS Applications***

Finally, GIS system can be very useful only if the

The linked image cannot be displayed. The file may have been moved, renamed, or deleted. Verify that the link points to the correct file and location.

Figure 24 ***GIS on Mobile***

Finally, GIS system can be very useful only if the user is able to formulate the best query for that specific issue.

2.7 Georeference

As already said above the potentiality of GIS system is possibility to work with a "cartographic data base". When we have a satellite or aircraft photo, there in not cartographic information. So the first think to do when we want to create this kind of system is try to get the above information. This process is called ***georeference***.

In the simplest way we can said that to georeference something means to define its existence in physical space. That is, establishing its location in terms of map projections or coordinate systems. In our field this he term is used both when establishing the relation between raster or vector images and coordinates, and when determining the spatial location of other geographical features. Examples would include establishing the correct position of an aerial photograph within a map or finding the geographical coordinates of a place name or street address.

Thus, as said above, this procedure is imperative to data modeling in the field of geographic information systems (GIS) and other cartographic methods. When data from different sources need to be combined and then used in a GIS application, it becomes essential to have a common referencing system. This is brought about by using various georeferencing techniques.

To georeference for example an image, one first needs to establish **control points** (**CP**). These point, that must be at least three to solve the problem of gereferencing, are point on field with known geographic coordinates. Moreover these points must be visible from the picture that you want to georeference. Inserting these known coordinates within the equations that represent the mathematical model[1] of this transformation, it is possible to evaluate the coordinates of the correlated points on the picture. To do this, nowadays many software have been implemented in order to help us, and the process of georeferencing can simply reduced into two step (see Figure 25):

- enter a coordinates on CP on field;
- click the corresponding point on the picture.

In geometry, an affine transformation or affine map or an affinity is a transformation which preserves straight lines and ratios of distances between points lying on a straight line. Examples of affine transformations include translation, geometric
contraction, expansion, homothety, reflection, rotation, shear mapping, similarity transformation, and spiral similarities and compositions of them. An affine transformation is equivalent to a linear transformation followed by a translation.

Of course it is not difficult to understand as this process has to be iterative, in other words to minimize the residual between the coordinates of CP on field and on the picture, we have to repeat the above step several time. More in details, residuals are the difference between the actual coordinates of the control points and the coordinates predicted by the geographic model created using the control points. Thus, they provide a method of determining the level of accuracy of the georeferencing process.

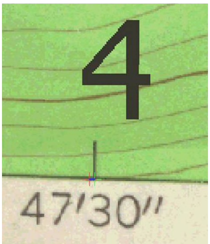


Figure 25 ***Images Showing the Control Points (green) and the Georeferenced Coordinates (red) of Point 4***

As said, there are different software or code/tool able to georeference a picture. Sometimes this tools are included within some software. For example, as for **QuantumGIS** (see Figure 26), PCI Geomatica, ERDAS Imagine or other software.

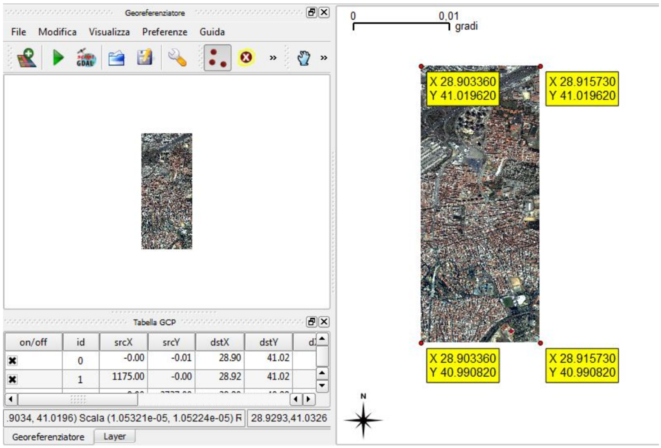


Figure 26 ***Georeferencing Toolbar on QuantumGIS***

Chapter 3.

An Introduction to the Sensing Using Satellite

In this chapter, we will show the principal concepts of the remote sensing, useful for the next purpose of this course.

Keywords: Remote sensing, Satellite, Sensor, Picture

3.1 The Remote Sensing

Remote sensing is a way to obtain information from objects that is founded on collection and on data analysis without the instrument used collect the data is directly linked with the studied object.

In remote sensing (see Figure 27 *An Example of Satellite Able to Take Pictures of the EARTH*), we have four essential elements:

- a **platform** must be up to support the instrument;
- an **object** to observe;
- an instrument or a **sensor** to observe the object;
- **information** which obtain data of image and how they are used and stored.

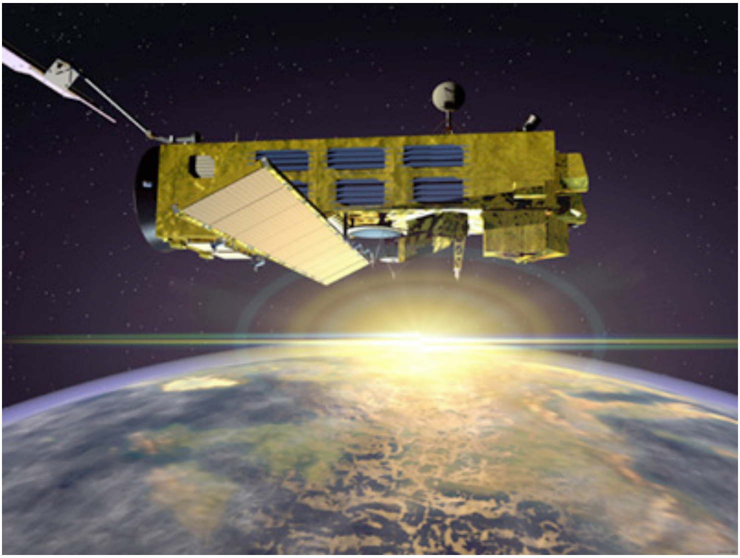


Figure 27 ***An Example of Satellite Able to Take Pictures of the Earth***

In scientists opinion, platforms are all the means of transportation which determine *the distance* from the surface of the planet (e.g.: satellites and aircrafts).

The goal is your own Planet, sensors are all the instruments used to observe Earth (e.g.: cameras, scanners, radars, etc.) and the information obtained are all necessary to increase our knowledge of the planet (the layer of clouds over Europe, the evolution of the ozone hole, the advance of deserts, the increasing of deforestation and more).

Hence remote sensing is a science which aims primarily to observe and discover what happens on the Earth's surface.

3.2 Platforms

As sophisticated the tools used for remote sensing, without a suitable means to fly over the Earth is not possible to acquire a reasonable overview from above.
Before the invention of balloons and airplanes, it wasn't possible to acquire systematic vertical photographs or pictures of the Earth's surface.

3.2.1 Aircrafts

Currently, the easiest way to *tape* Earth at a distance is by aircrafts. To take photographs, planes are equipped with cameras.

An example of aircraft used for this purpose is shown in Figure 28.

Figure 28 ***The COPTERCAM 8 - Aerial Camera System***

An example of a aircrafts' photo is shown below (see Figure 29 *A Shot by an Aircraft*).

Figure 29 ***A Shot by an Aircraft***

A peculiarity of planes is that they can fly at a relatively low altitude (few kilometers on the Earth's surface) and so they can take photographs of limited areas, even if these photographs help us to recognize lots of details (e.g.: cars, people, trees, etc.). To ensure such aircrafts can make the flying tracking, weather conditions should be good enough to ensure the ability to capture a lot of photos; hence the photographs taken by the equipment on board of these aircrafts are often quite sharp (i.e. in absence of clouds).

3.2.2 Satellites

In 1957 an important event marked the beginning of a new era in observing Earth, when Soviet Union launched the first satellite (Sputnik) in history.

A satellite is a celestial body which revolves around a planet; the best-known satellite is the Moon. Moon is a *natural satellite* because it is not a man-made, as *artificial satellites* are those man-made object in orbit around a planet. A satellite can not "fly" and it is not able to leave the Earth's surface on its own: it must be transported by a rocket. An example is the *Ariane*, the European carrier to launch satellites. Satellites also follow an orbit that is the path followed by a celestial body around another one larger and it usually has an almost circular shape.

Figure 30 ***A Rocket Launches a Satellite***

Today, satellites are the main platforms used in remote sensing; they are able to carry a wide range of sensors, some to study the weather, some to study landscapes or natural disasters, some to study the vegetation, some are even be capable to “see” through clouds or to acquire images during night. Compared to aircrafts, once launched and after the reaching of its orbit, a **satellite is always available** so we don’t need to plan a flight to capture images.

Another great advantage of satellites is that they are able to acquire images of great areas, but they have their own limits. For example, when a satellite is over Australia, it cannot acquire an image of Europe and hence we have to wait that it fly over the Europe or we have to use another satellite. This is one of the reasons which we need so many satellites to have a complete covering of the Earth's surface.

3.3 The Object Under Control

When scientists perform a remote sensing, the goal of their observations is the Earth. But sometimes Earth is too large to be viewed all at once and to have a picture of the whole Earth, the satellite must move away from the planet. A satellite on its orbit, however, can not suddenly change its course, so when we need a complete picture of the planet, we use a different satellite on a further orbit.

As the Earth is a sphere, it's impossible to see the whole planet and we can only see a side one by one.

From time to time, if the scientists want to have a picture of a larger part of the Earth's survey, they set up the sensor on the satellite to take several images next to each other.

Performing a mosaic of pictures, in addition, we can obtain a vision of a larger area of the ground and even the entire planet. Consequently, according to the required image and the orbit of the satellite, we have two main typology of pictures. In Figure 31, the satellite is placed in an orbit relatively far from Earth (about 36,000 kilometers). For this reason we can see the *whole* planet.

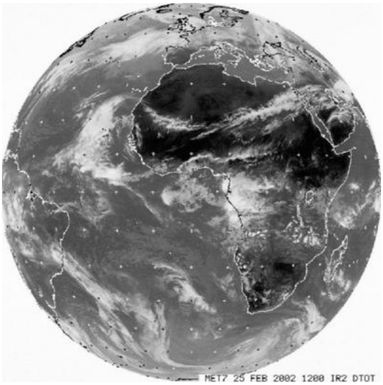


Figure 31 ***Earth View from Satellite***

In the next figure (see Figure 32), the orbit of the satellite is much closer to the Earth (about 800 kilometers); each square (black and red) represents an image. When you combine several images to form a bigger, scientists talk about a creation of a *mosaic* of images. Sometimes scientists wants to observe a particular feature in more detail zooming in small part of the Earth's surface.

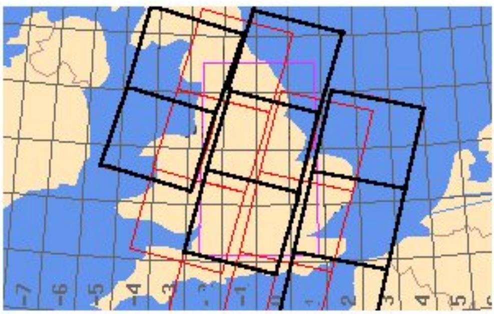


Figure 32 ***Picture Taken by a Satellite***

In

Figure **33** is shown a satellite image of *Zeytinburnu* Region, Istanbul – Turkey.

As another example in Figure 34 is reported a satellite image of London, while Figure 35 an aerial photograph of the same city. The choice of image we want to use depends on the goal we aim: for example, if we have to measure the growth of the city we can refer to the first figure, but if we want measure the traffic density of the Thames, the second one will be better.

Sometimes scientist need to observe some phenomena that occur on the Earth's surface; in these case they will want to zoom in and zoom in a particular feature than a specific region.

For example we decide to zoom into an hurricane, or a forest fire or flood-affected area.

Figure 33 ***Satellite Image of Zeytinburnu Region, Istanbul – Turkey (June, 2005)***

Figure 34 ***London Image by Satellite***

Figure 35 ***London Image by Aircraft***

3.4 Sensors

In remote sensing, the instrument used to acquire images is called *sensor*. We prefer use that term because it refers to a wider range possibilities of acquiring information comparing it to a camera. In fact, a camera refers to information that we can only see through eyes, while remote sensing also involves different types of radiation in the electromagnetic spectrum. When a camera takes a picture, using a radiation (light) in the visible, so when we shoot something, what appears in the image corresponds to what you see in the reality.

More in general, there are several types of sensors that are used in remote sensing satellites. These sensors vary according to the purposes for which they are used. Main type of sensors used in remote sensing are:

- ***The Multispectral Scanner***

The MSS is a mechanical scaning device that acquires data by scanning the earth's surface in strips normal to the satellite motion. Many lines are swept simultaneously by an scanning mirror and reflected solar radiation so detection is monitored in the detector. This allows us to monitor several spectral bands at the same time. Extensively used in Landsat series of satellites.

- ***The Thematic Mapper***

The Thematic mapper is also a mechanical scanning device as the MSS, but it has improved spectral, spatial, and radiometric characterstics. Whereas MSS of all Landsats scans and obtains data in one pass only(In the return pass it does not sense data) the thematic mapper can acquire data in both the scan directions. Also used in Landsat.

- ***The Microwave Radiometer***

These consist of a microwave antenna and amplifying and detection electronics. As the satellite moves over the earth the MR antenna picks up the microwaves radiated/Reflected by earth and the associated electronic payload detects & stores it. It is capable of detecting sea surface temperature, ocean winds, moisture content over the land and the sea etc. One such Radiometer (called SAMIR) was used in BHASKARA I and II.

- ***The Synthetic Aperture Radar***

The SAR is a radar which simulates a large antenna aperture using the fact that a satellite is in motion over the earth and the phenomena of Doppler Shift. As the resolution of a radar has a direct proportional relation to the Aperture area, The SAR is able to acquire data at quite high resolution. First satellite to carry a SAR on board was Seasat launched in June 1978.

- ***The Panchromatic Camera***

This a sensitive camera which is used quite frequently in recent times. Coupled with CCD devices, the camera can directly convert the images into digital format which are then beamed directly or after some on-board processing to earth. Sometimes it is also called the CCD Camera. An example of a CCD camera is the panchromatic camera carried in IRS 1-D.

3.4.1 Electromagnetic Spectrum

In remote sensing, sensors also allow to acquire information not capable by human eye. An example concerns the special viewers which allow you to see in the dark, like those worn in spy and action movies; that viewers increase the ability to display, using the infrared illumination. All objects reflect some of the light that reach them and it is precisely this amount of light that usually *gives* object their colors. For example, when you see that a plant is green, because it reflects part of the light that the eye sees like green. Several objects don't reflect the light that reaches them, but they also emit *radiation*. For example, a fire emits heat and light, and this causes that is visible even in the dark.

The light (and the heat) emitted and reflected by objects is called *radiation*. With the term radiation, we mean a set of particles with electric charge in motion, the movement, in this case, is a *wave*. Consequently, for measuring a radiation emitted or reflected from objects, it is necessary to measure their wavelength i.e. the wave length of radiation emanating from the object. Measuring the wavelength of different objects, we understood that some objects reflect wavelengths that are not visible to the human eye (e.g.: infrared). The set of all possible wavelengths is represented by the *electromagnetic spectrum* (see Figure 36).

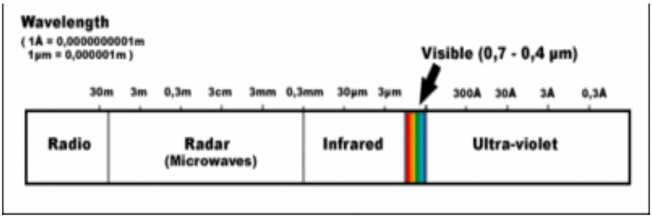


Figure 36 ***Wavelength Range***

3.4.2 Passive Sensors

In remote sensing as shown above we use a number of sensors with different sensitivities to different wavelengths in the electromagnetic spectrum. All sensors specialized in the reception of wavelengths reflected or emitted by objects observed are called *passive sensors*.

Some of these sensors are designed to receive all the wavelengths of the *green*, while others are more directed toward the infrared wavelengths. The infrared viewer, for example, is constructed specifically to see objects that emit radiation in the infrared (even in the dark).

The main disadvantage of passive sensors is that if the sky is covered by clouds or if it is dark, they cannot be used.

3.4.3 Active Sensors

To prevent the problem connected with the passive sensors, we must use a different type of sensor called *active sensor*, so called because they are able to emits a radiation, that once reflected by the objects, will come back to the sensor where, the energy stored inside this radiation, will be measured.

The active sensor most commonly used in remote sensing is the *RADAR*. Radar was secretly developed by several nations before and during World War II. The term RADAR was coined in 1940 by the United States Navy as anacronym for **RA**dio **D**etection **A**nd **R**anging. The term radar has since entered English and other languages as the common noun radar, losing all capitalization. Radar is an object detection system which uses radio waves to determine the range, altitude, direction, or speed of objects. It can be used to detect aircraft, ships, spacecraft, guided missiles, motor vehicles, weather formations, and terrain. The radar dish or antenna transmits pulses of radio waves or microwaves which bounce off any object in their path. The object returns a tiny part of the wave's energy to a dish or antenna which is usually located at the same site as the transmitter.

The modern uses of radar are highly diverse, including remote sensing, air traffic control, radar astronomy, air-defense systems, antimissile systems, marine radars to locate landmarks and other ships, aircraft anticollision systems, ocean surveillance systems, outer space surveillance and rendezvous systems, meteorological precipitation monitoring; altimetry and flight control systems, guided missile target locating systems, and ground-penetrating radar for geological observations. High tech radar systems are associated with digital signal processing and are capable of extracting useful information from very high noise levels.

Other systems similar to radar make use of other parts of the electromagnetic spectrum. One example is "lidar", which uses visible light from lasersrather than radio waves.In the following picture in shown the operating principle of radar (see Figure 37).

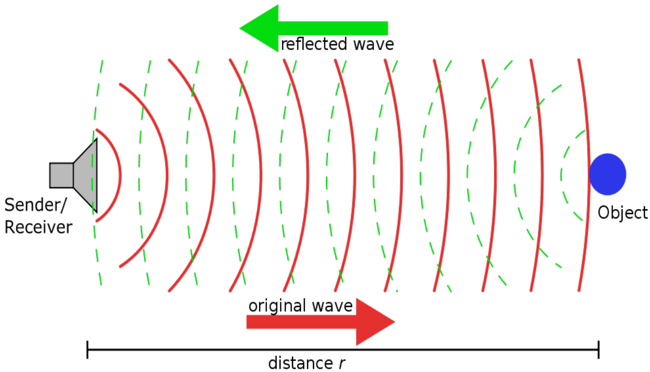


Figure 37 ***Operating Principle of RADAR***

3.5 Information into an Image

In remote sensing is very important to understand the differences in data provided by sensors in order to interpret them correctly, then, the first thing we have to understand what is a satellite image and how it differs from a photograph.

The main difference between a photograph and satellite image is that the photograph is in analog format and is usually printed on paper before being interpreted, the satellite image is a digital format and to analyze and interpret it we usually need a computer. The analog is also a format that saves all the data on an ongoing basis.

The digital format, rather, saves every piece of information separately; if you put enough zoom to a satellite image, you will distinguish many squares of different colors.

Figure 38 ***A Zoomed Satellite Image***

Figure 38 show us how it looks if a satellite image is very close zoom. You only see squares because the image is not continuous, but consists of a matrix of squares (also called *pixels*). This is a key feature of digital formats. The digital format is actually based on a mathematical procedure (called a *binary system*) which allows computers to record data and then render it, to calculate and store data, even a picture. The binary system is in fact the basis of all the world's computers. The only thing that a computer can *understand* are electric pulses: a pulse can be exists or not, can be 0 or 1. Mathematicians therefore thought to a system different from the one decimal for the computer, which is the one with which we usually count: 0 to 9, then part of a new series of tens from 10 to 19, from 20 to 29, etc…With computers it goes from 0 to 1, then starts a new series (0 when there is no electrical pulse and 1 when there is an impulse).

So in computers *language* we have:

0 = 00	5 = 101
1 = 01	6 = 110
2 = 10	7 = 111
3 = 11	8 = 1000
4 = 100	9 = 1001
10 = 1010	100 = 1100100

We have to mark the binary system:

- a 2 numbers group is called *bit*;
- a 8 numbers group is called *byte* (i.e. 256 in decimal system)

3.5.1 Pixels

A satellite image is made of many squares called *pixels*. Being the smallest unit on a satellite image, the pixel is very important: pixels taken together, provide, in fact, all the information that produces a complete picture.

3.5.2 Resolution

The first important thing to know about satellite images regarding their resolution. If we think of a satellite image of a city with a football stadium in the middle: the smallest square or pixel of that image could be the entire football stadium or the center spot of and, in the first case, the resolution the image would not be good, while in the second case, the image would be more detailed and therefore the resolution of the image excellent. The resolution of an image is the smallest distance that the sensor is able to detect.

3.5.3 Pixel Values

Each pixel in a picture has a value. The value corresponds to the intensity of radiation reflected from the object observed within the range of wavelength in which the sensor is sensitive.

For example, if the observed object is a plant (without flowers) and the sensor used is specially designed to detect green, the intensity will be very high. With the same sensor, if the observed object is a red car, the intensity will be very low.

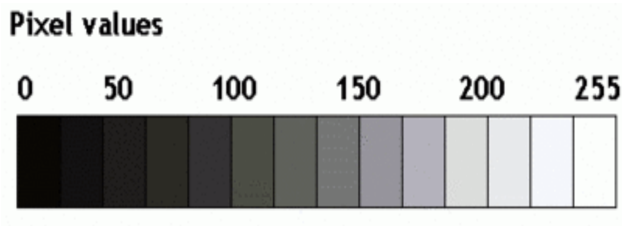


Figure 39 ***Pixel Values***

The range of pixel values is from 0 (i.e. black) to 255 (i.e.), so the possibilities are 256, i.e. a byte

3.5.4 RGB Images (red, green, blue)

In general, white light can be formed by mixing differently colored lights, the most common method is to use red, green, and blue or simply RGB (see).

A paradox of this system of image acquisition is that while satellite images post processing (finished images) appear in color, the starting values of the pixels are included only in gray scale (i.e. between 0-255). As a result during processing, to create a color image, is often combined multiple satellite images (from the same sensor, but in different bands or acquired on different days).

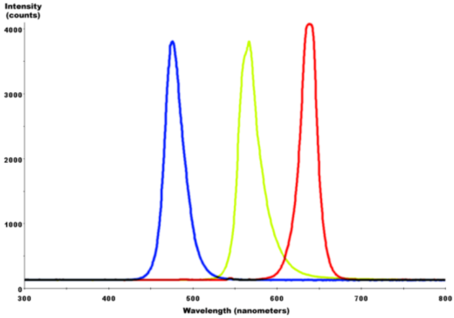


Figure 40 ***Combined Spectral Curves for Blue, Yellow-Green***

For example, it may take and the combination of three images in three different bands (i.e., three ranges of wavelength different) from the same sensor. To produce a color image, each of the three bands are assigned a color (red, green or blue).

Figure 41 is the result of this procedure.

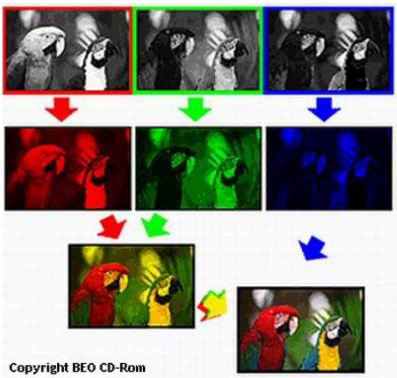


Figure 41 ***RGB Image***

Chapter 4.

Seismic Risk and Risk Mitigation.

An Overview

In this chapter, we would illustrate factors associated to seismic risk like hazard, vulnerability and exposure. From the combination of these factors or, more precisely, from their convolution, earthquake losses are originated and hence the seismic risk. We also pay attention to human losses, direct damage to buildings and structures.

Keywords: Seismic Risk, Risk Mitigation, Hazard, Vulnerability, Exposure.

4.1 Introduction to the Seismic Risk

Usually, risk is associated to the possible consequences following traumatic events which can impact on exposed goods in general.
Related with this concept are expression like “health risk due to smoke”, “environment risk due to pollution”, “real estate risk due to earthquakes occurring over a period of 50 years”.
All these statements have elements in common:

- *Cause*: the event causing traumatic consequences, which could be either the simple lighting of a cigarette, or the leakage of pollutant or the earthquake or in mentioned examples;
- *Exposition*: goods exposed to the event (e.g.: people, environment, buildings, etc.);
- *Primary vulnerability*: the way in which the good is injured by the event;
- *Secondary vulnerability*: the change in terms of availability of the good following the damage;
- *Losses*: The losses following the damage occurrence (lung disease, harvest polluted, economical expenditure for assuring either the safety and the usability of a damaged building);

The fuzziness of elements that we should examine requires their probabilistic treatment: this occurs in case of seismic events, the occurrence of which over a given period of time cannot be exactly predicted, as well as for the behaviour of a building under the action of an earthquake or for the number of people present in a given building over a given period of time. For each cases risk is commonly expressed as *the probability that a given level of loss* L *is exceeded, over a given period of time* T*, following a given type of event involving specific type of goods*.

The risk analysis has to process all the elements, at least the most important, contributing to the loss in correlation to the knowledge level of each of them.

We can define seismic risk by three factors:

- the primary cause of the damage, i.e. the start event (E: earthquake), that is expressed in terms of time frequency of soil displacements (e.g.: speed, acceleration) or in terms of severity. The first one is generally referred to as *Hazard*;
- the reaction of exposed goods (B) to the event (E), so the susceptibility of the goods to undergo changes of their status, following the event. This further factor is named *primary Vulnerability* (V_1), if the change of status is expressed by physical damage; *secondary Vulnerability* (V_2) if the status variation is a consequence of the physical damage (e.g. economic loss, functional loss and so on).
- the quality and quantity of goods undergoing losses, is defined as *Exposure*. It is more opportune define the inventory of exposed good

separately from their volumetric features, leaving to the Exposure the meaning of value.

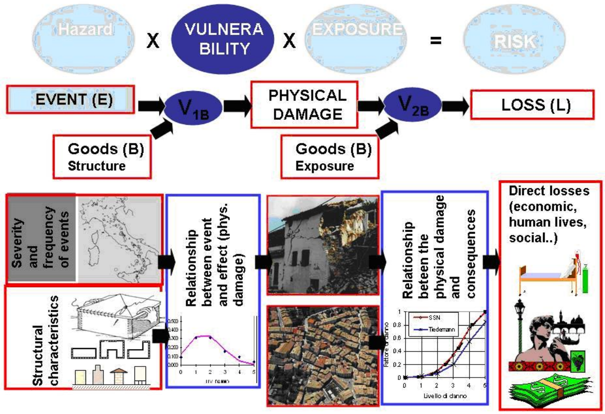


Figure 42 ***Process of Risk Assessment***

Under a theoretical point of view, the risk can be reduced by operating on all its factors. However, in the case of the seismic risk, very little can be done for what concerns the cause, as earthquakes cannot be impeded to occur. Nevertheless, it is possible to:

- choose quiet sites for new constructions when it is possible, where the event occurs with mitigated severity (local effects also contribute to the hazard);
- quantify the hazard and operate on the other risk factors so as to control the risk level.

Otherwise, the vulnerability is the factor on which is possible to operate: its mitigation involves the reduction of earthquake damage. The *Exposure* can be checked in particular cases, for instance by reducing either the number of occupants or the value of goods in vulnerable buildings.

The framework described above fits also for the usability checks of damaged buildings, obviously with limitations due to the low level of knowledge achievable in an emergency and massive operation.Essentially, the damage considered is the visible one (physical damage), the vulnerability is estimated on the basis of the recognition of the structural typology and of the knowledge of local constructive practices. Ideally the hazard should be given to the inspectors, so that they could refer to aftershocks of assigned severity. Very often this is not the case and the inspector consider the possibility of a new shaking equal to the previous one. The exposure (e.g.: people living in a building) is estimated on the basis of the data collected during the inspection. Finally, the time period over which the evaluation is carried out is generally the extension of the emergency phase.

4.2 Hazard

Seismic events always generate displacement of soil particles: often temporary (shaking), sometime permanent (Figure **43**). The first ones (temporary) are displacements variable with the time (Figure 44) and they are transmitted to building foundations causing the alteration of the static condition of the construction due to inertia forces which performs in addition to gravitational loads. Shaking gradually tends to vanish once the event is ended. Permanent displacements can be generated by co-seismic events:

- *Soil instability* (landslides);
- *Surface faulting*: surface fractures generated by the sudden release of energy in the hypocentral zone (Figure 45);
- *Liquefaction*: vanishing of the shear strength of sandy soil caused by the increase of pore pressure due to shaking (Figure 46);
- *Densification*: reduction of soil volume due to shaking.

Figure 43 ***Seismic Effects After an Earthquake in Lisbon in 1755***

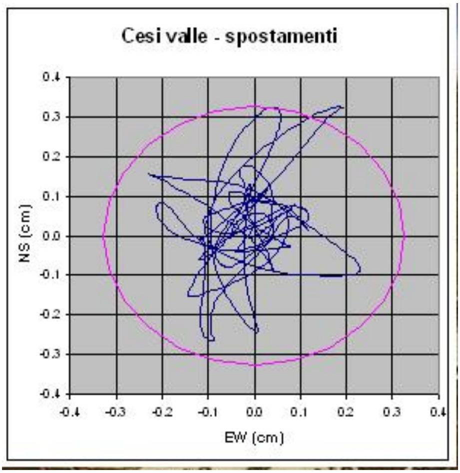


Figure 44 ***Motion of a Soil Particle in the Horizontal, from Registration in Cesi, 1997***

Figure 45 ***Surface Fault***

Very strong earthquakes, producing large displacements under the sea and they could give origin to tsunamis (i.e. large waves which height increase in proximity of the shore) causing very serious damage and injuries to people, buildings and infrastructures.

In the following the term hazard will be used only to define the shaking (temporary displacement), while permanent soil displacement will be neglected.

Since the beginning of the XX century in Italy an effort was done to define the seismic hazard in terms of frequency and severity of earthquakes, to individuate the zones in which these events were more able to produce serious effects.

Initially, the earthquake severity was described in terms of effects on construction and environment, hence by means of the macro-seismic intensity and including the vulnerability of the damaged objects. Macro-seismic scales proposed over the time, (from the MCS to MSK and EMS) have tried to filter the effect of vulnerability referring the effects to different construction typologies.

Nowadays, several approaches can be used: for example seismicity analysis based on historic records, or deterministic analysis simulating the source and the motion propagation, or probabilistic analyses at different levels of complexity, using both historical data, seismotectonic models, ground motion attenuation relationships.

A model largely used is the well-known probabilistic method developed by professor Cornell in 1968 and used by many others researchers until today, with different upgrading. The method can be shortly articulated as follows:

a) *Seismogenetic zoning*: all the points of each zone, whose borderline is defined on the basis of seismotectonic and historical studies, are assumed to originate events in the same way. In Figure 47 is shown an example of old classification/zoning of Italy;
b) Each zone has a characteristic *production of events*, obtained on the basis of the seismic catalogue (Figure 48). The recurrence law of the events is given by a relationship $N(M)$, with N number of events per year which have magnitude equal or bigger M (Figure 49);
c) The *shaking* of each event is propagated to each computation point through attenuation laws depending on M and distance from the epicenter (Figure 50);
d) For each computation point, the probability of exceeding a given severity of quake in a given period of time is determined. The final result is a *map* (Figure 51) where different shaking parameters (i.e. peak ground accelerations, spectral ordinate, etc.) are associated to each point, for different return periods, or probability of excess in a given number of years (Figure 52).

Figure 46 ***Permanent Displacement Caused by Liquefaction Following the Earthquake in 1964, Niigata, Japan***

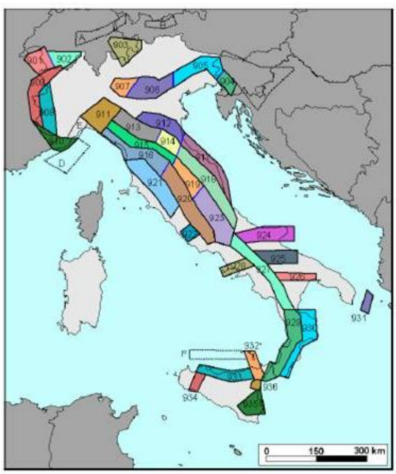


Figure 47 ***Example of old Seismogenetic Zoning ZS9 of Italy (before 2004)***

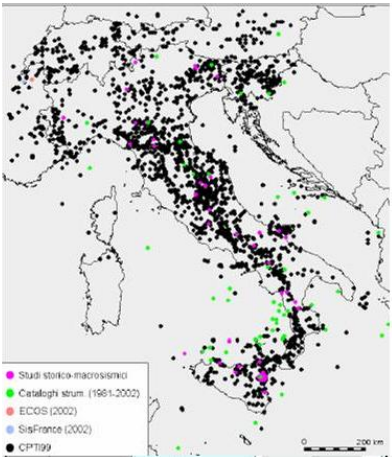


Figure 48 ***Earthquake Catalogue CPTI2 According to Data Origins***

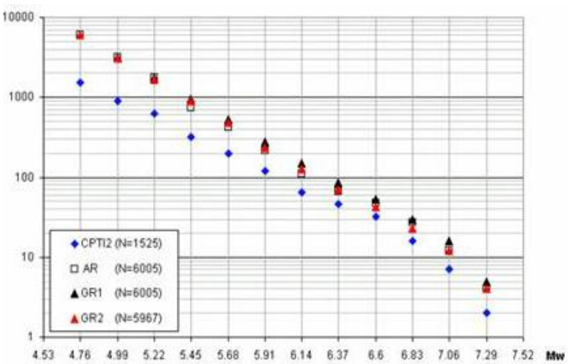


Figure 49 ***Cumulative of Occurrence for Events with Magnitude ≥Mw for all Italy (before 2004)***

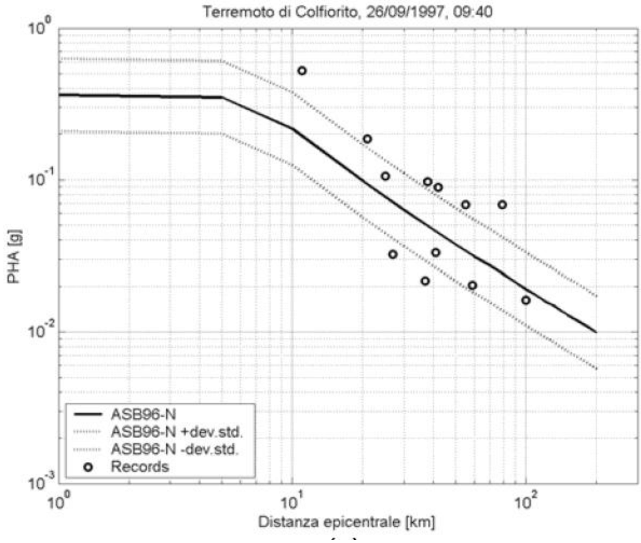


Figure 50 ***Example of PGA Attenuation Curves Compared with Recorded Data***

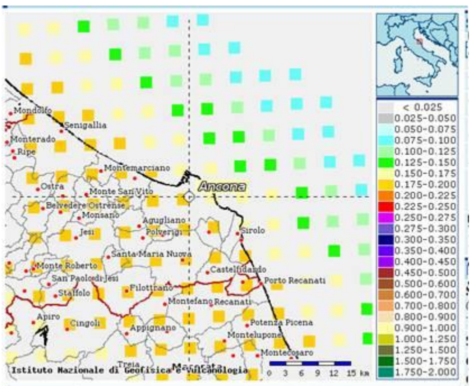


Figure 51 ***Hazard Computation Points for one Italian Territory Portion (Working group, 2004)***

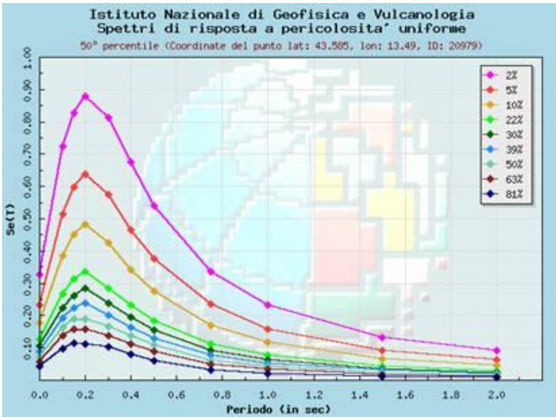


Figure 52 ***Elastic Spectrum Response of Horizontal Acceleration Related to Different Excess Probabilities in 50 Years (Working Group, 2004)***

Beyond the information given by the hazard maps, data useful for usability survey are obtained from historic data and particularly from seismic sequences occurred, which may predict the possible replication of shakes.

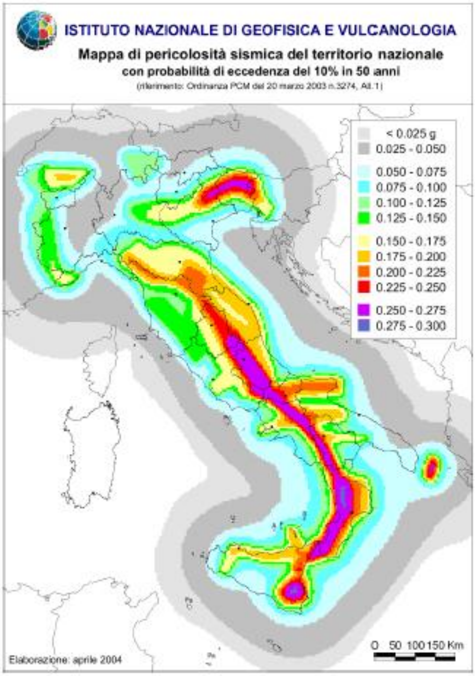


Figure 53 ***Basic Hazard Map (ITA - 2004)***

It is worth noticing that in several cases the seismic hazard of a given location can be highly influenced from the local state of the soil. The particular geological configuration, together with the associated variation of dynamic properties of soils, may produce notable amplification compared to the one that might occur in a perfectly stiff and flat soil. Similarly, topographic conditions and hidden morphology may be very influent.

Since the understanding of these situations cannot be easily achieved at large scale, hazard analysis at national scale are commonly related to ideal conditions of rigid soils (class A, according to the national seismic code), and are then corrected in different ways for being suitable for particular local situations. The new Italian seismic code (NTC 2008) provides the possibility, when detailed investigations are not available, to carry out modifications of the rigid soil spectrum according the *type of soil* and *topographic configuration*. One more detailed approach is represented by the specific studies of local seismic response.

4.3 Vulnerability

The vulnerability is that strictly connected to the characteristics of the exposed goods, among the connection to the risk.

We refer to seismic vulnerability of a building or of a different structure (e.g.: bridge, pipeline, dam, etc.) as a particular way of expressing their future performance in terms of damage following the shaking (and gravitational forces).

4.3.1 Vulnerability of Buildings

In Italy, the development of buildings has occurred faster than the consciousness of hazard: over a long period, the seismic classification has followed the occurrence of the earthquakes. Sometime happened that after a few year from the classification, de-classifications was decided for different reasons, for instance the consideration that seismic restraints might represent an obstacle to the development of territories. As a result many construction were built before the enforcement of the classification.

Furthermore, the development of the construction, started after the Second World War, highly accelerated in the following years, often with the use of questionable materials and techniques, contributing to accumulate a notable *seismic debt*.

The ISTAT 2001 census provides a picture of the real estate, which mostly dates back to the ’70, (Figure 54) with a high percentage of buildings realized in areas not seismically classified at that time. The number of towns seismically classified increased from 1600 before 1980, to 2965 in 1984, and finally to all the 8101 Italian towns in 2003 (3488 of which in the 4th zone, i.e. a very low seismicity).

The histogram in

Figure **55** shows the number (updated to 2001) of dwellings realized before and after the seismic classification, therefore more likely designed and constructed without complying with seismic codes.

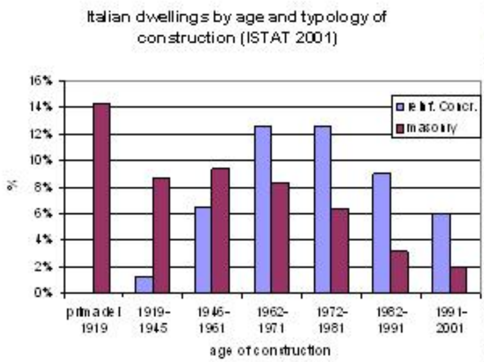


Figure 54 ***Distribution of Dwellings Per Construction Period (ISTAT 2001)***

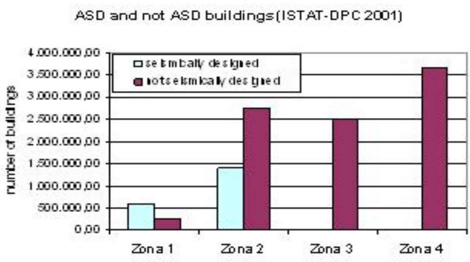


Figure 55 ***Seismically and not Seismically Designed Buildings by Seismic Zones (ISTAT-DPC 2001)***

We can observe from

Figure **55** that in *Zone 1*, almost one construction out of two is anti-seismic: the reason is that the classification of the territory has first involved the most seismic areas of the Country, where most of the constructions were realized following the classification. Otherwise, in towns included in the remaining Zones (2,3,4), the number of buildings realized before the classification is very high: averagely 2 against 3 in Zone 2, and almost the total in the other zones. To sum up, around the 82% of the existing buildings might be *unprotected* from the seismic event. As a matter of fact the seismic codes and the classification represent a powerful tool, immediately effective on new constructions, but also an instrument needing much longer times, to affect the existing heritage, in absence of specific policies.

4.3.2 Approaches for Vulnerability Assessment

The seismic vulnerability of a building can be described trough a *cause-effect law*, where the cause is the earthquake and the effect is the damage.

By resuming the general definition (Vulnerability is a relation between the event and the status alteration undergone by the exposed good, i.e. the damage) it is clear that different possible approaches for its practical definition are possible (Dolce et al., 1994). The following scheme shows how some of them are function of different ways of formulating the event as well as the status alteration.

To sum up, it is possible to move leftwards (Event) or rightwards (Damage) by using the different segments passing from for the central core representing the features (Vulnerability) of the exposed good.

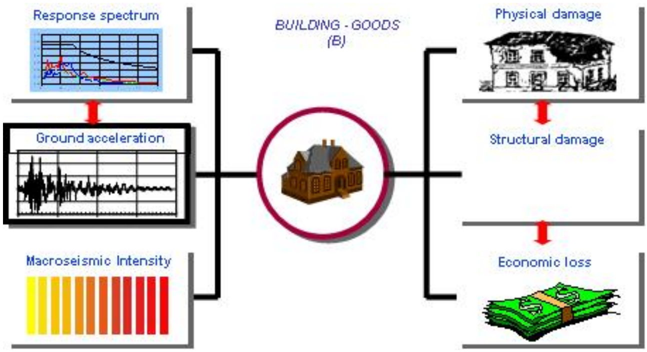


Figure 56 ***Different Approaches in the Seismic Vulnerability Through Possible Relationship Between Event and Damage***

From a practical point of view, the definition of vulnerability is strictly linked to the method according to which it is determined, which is on turn a function of time and resources available, as well as of final target to be achieved.

For the check of the safety of specific existing buildings, according to the seismic code, the event is generally represented by either a response spectrum, or by a time history, in case of non-linear analyses. Otherwise, the damage is expressed by one of the limit states (light damage, severe damage, collapse). In this case vulnerability can be formulated through a discrete criterion, by the three values of the peak ground acceleration corresponding to the reaching of the three limit states. In order this method to be applied, a notable computational effort is required together with a thorough knowledge of the input data.

For large scale assessments, the above approach is not generally applicable, as detailed data are not available and also because of the effort required to process data referred to millions objects. The assessment is carried out using a statistic method for vulnerability while hazard is computed in terms of macroseismic intensity.

In an intermediate position between the above two approaches is the "indirect" GNDT method, which is featured by some structural characteristics used for defining a Vulnerability index. This one is defined by a deterministic formulation between peak ground acceleration (PGA) and economical adimensional damage (Angeletti et al. 1988).

4.3.3 Framework of Methods for Vulnerability Assessment

Figure 57 shows a possible scheme outlining four different approaches, in a decreasing order proceeding leftwards, for what concerns economical effort and reliability for detailed analyses. Alternatively, on the opposite direction (rightwards) the number of analyses which can be carried out, with contained costs and reasonable time, increases. These approaches are shortly described in the following.

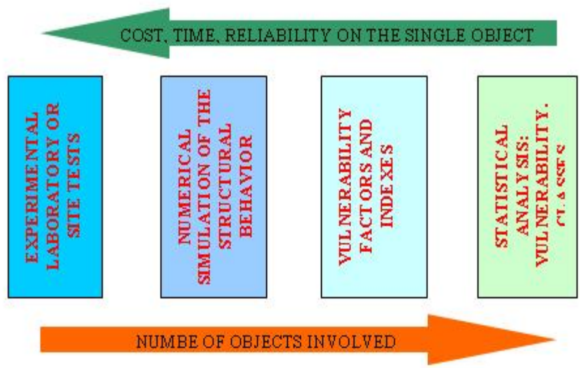


Figure 57 ***Scheme outlining different approaches for vulnerability assessment***

4.3.4 Detailed Vulnerability Through in Situ or Laboratory Tests

At this level of analyses, in situ or laboratory tests can be carried out (respectively on real structures and on models), depending on level of investigation allowed, hence as to measure the system response (damage) to control dynamic or pseudo-dynamic actions. The analysis of the actual response allows an high level of reliability of the vulnerability of the object tested.

If the object is a representative of a new constructive system, never realized in practice, the test campaign will play a central role and the cost will be justified. The method might be also justified in case of objects representative of a great number of similar structures. Figure 58 shows the experimental test on an arch masonry model, typical in some southern Italy zones, subjected to a dynamic excitation on a shaking table to find out the failure mechanisms and the safety margins of the arch.

Data collected consist generally in several registrations of the dynamic behaviour acquired through accelerometers and displacement transducers, as well as video registrations.

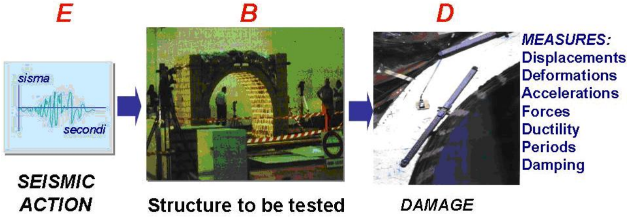


Figure 58 ***Laboratory tests on a masonry arch model***

4.3.5 Detailed Vulnerability Through Numeric Modelling of the Structural Performance

When we have to analyze a specific object, and enough information are available on its geometry, its constitutive materials and so on, is very common either in the structural design of new constructions and in the structural check of existing buildings, the use of numerical modeling. The example illustrated in the picture, shows the non-linear dynamic analysis of a bell tower damaged by the 1996 Italian earthquake, in Bagnolo in Piano close to Reggio Emilia.

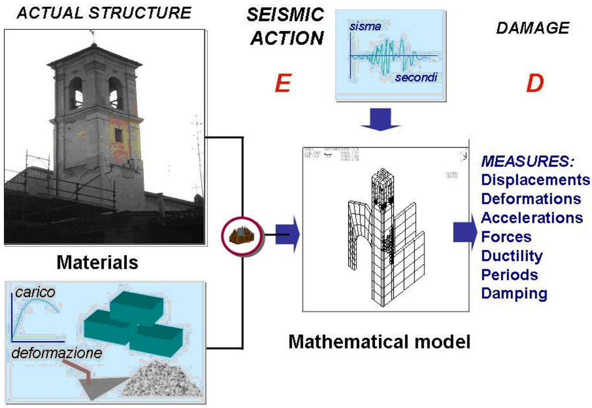


Figure 59 ***Dynamic Analysis on the Bell Tower of Bagnolo in Piano, Reggio Emilia***

4.3.6 Vulnerability at Medium Scale: Factors and Vulnerability Indices

When the target of analysis is the overall response of medium sized samples of buildings, and time and resources available are relatively limited, the operative tool commonly used in Italy is represented by 1st and 2nd level forms provided by GNDT (GNDT, 1993). This is a semeiotic method, which means that it is based on a qualitative assessment of vulnerability, rather than mechanical evaluations as in the previous approaches.

The 2nd level of analysis for masonry buildings consider 11 parameters, assumed representative of the structural performance during a seismic action, which are combined so as to attain a final index of vulnerability $\boldsymbol{I_v}$ (Figure 60). The parameters have all a qualitative nature except for the #3 (conventional resistance) based on the shear strength of the masonry. A score $\boldsymbol{W_i}$ is associated to each parameter *i* depending on the building features and each parameter is combined with the others through a weight p_i so as to balance its importance in relation to the one of the others (Figure 61).

The vulnerability index is formulated by the sum associated with the 11 (weighed) parameters, as follows: $\boldsymbol{I_v = \Sigma W_i p_i}$.

The index represents a straight measure of vulnerability: as the value is higher the vulnerability associated with the building is severe.

PARAMETRO	Wi per CLASSE			
	A	B	C	D
Organizzazione del sistema resistente	0	5	20	45
Qualità del sistema resistente	0	5	25	45
Resistenza convenzionale	0	5	25	45
Posizione edificio e fondazioni	0	2	25	45
Strutture orizzontali	0	5	15	45
Configurazione planimetrica	0	5	25	45
Configurazione in elevazione	0	5	25	45
Distanza massima tra le murature	0	5	25	45
Strutture di copertura	0	15	25	45
Elementi non strutturali	0	0	25	45
Stato di fatto	0	5	25	45

Figure 60 ***List of 11 Parameters Provided by 2nd Level GNDT Method***

PARAMETRO	PESO p_i
Tipo ed organizzazione del sistema resistente	1.00
Qualità del sistema resistente	0.25
Resistenza convenzionale	1.50
Posizione edificio e fondazioni	0.75
Strutture orizzontali	Variabile
Configurazione planimetrica	0.50
Configurazione in elevazione	Variabile
Distanza massima tra le murature	0.25
Strutture d copertura	Variabile
Elementi non strutturali	0.25
Stato di fatto	1.00

Figure 61 ***List of the Weights Associated to the 11 Parameters***

The absolute value of the index (I_v) has little significance by itself. It is more useful when associated with a fragility curve, correlating the peak ground acceleration to the expected damage. According to the GNDT method, the association between $\boldsymbol{I_v}$ and the **damage curve** (or fragility curve) is based on a statistic elaboration of data collected afterwards different earthquakes. The relation damage-acceleration, which should have a curvilinear pattern, is simplified by a tri-linear diagram, defined by two points associated with the damage on set and the collapse. According to this deterministic curve, the damage ranges between 0 and 1, where 1 represents the collapse. Similarly, along the *x* axis, $\mathbf{a_1}$ is the acceleration produced in the beginning of the damage and $\mathbf{a_c}$ is the one produced in the collapse.

Figure 62 ***Fragility Curve Provided by GNDT***

Finally, it worth noting that buildings that have similar values of Iv, provide only an average measurement of vulnerability, which might be even incorrect if used as absolute assessment for one specific building. As a matter of fact this method is commonly used in order to obtain an early screening of the vulnerability conditions on extensive samples.

4.3.7 Data Statistical Elaboration of and Vulnerability Classes

Finally, we consider methods based on the recognition of wide typological classes. These methods enable very extensive samples of objects to be analyzed, the response of which may be predicted only through statistic elaborations. These are very helpful for risk analyses or territorial scenarios (e.g.: towns, regions, etc.), for which the results produced are always conceived in an aggregate way.

The direct assessment of vulnerability is carried out by processing in a statistical way data collected afterwards seismic events (Braga et al., 1983, Di Pasquale et al., 1995, Di Pasquale e Orsini, 1997). One important requirement for this method to be carried out is that the origin sample is *complete*, which means that all buildings included in the area hit by the seismic event are surveyed.

This technique has been widely used for setting up damage probability matrices for building classes defined in a rough way, i.e. through few indicators very easy to be on sight surveyed. The damage probability matrix of a class of buildings provides the probability of occurrence of each damage level as function of the macroseismic intensity.

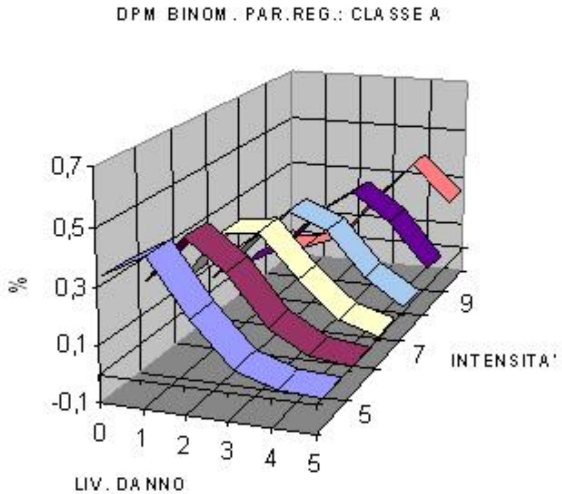


Figure 63 ***Example of Damage Probability Matrix***

One important aspect governing the applicability of this method is the criterion followed for vulnerability classes assignment. Classes are those provided by macroseismic scale MSK, and their attribution is made on the basis of typological features of the building. The table shown in (Figure 63) illustrates the assignment criterion as function of structural horizontal and vertical characteristics.

VULNERABILITY CLASSES ASSIGNMENT

Hor.str \ vert.str.	Rubble masnory	Roughly squared masonry	Brickwork or ashlars masonry	Reinforced concrete
Vaults	A	A	A	-
Timber	A	A	C	-
Ironbeams	B	B	C	-
slabs	C	C	C	C

Figure 64 ***Vulnerability Classes Assignment (Braga et al., 1983)***

4.3.8 Vulnerability of Strategic and Relevant Buildings

Buildings and infrastructures of strategic interest for civil protection are elements for which functionality plays a central role for the purpose of protection of population and goods after a seismic event: the major expectations in emergency situations rely on these building set. For this reason, while for other structures the damage, even severe, can be acceptable, for a strategic building it is required that it ensures functionality in order to contribute in helping the population. This means that, in principle, strategic buildings are expected to have a performance level higher than that required to ordinary buildings. This result is obtained designing these constructions for seismic actions characterised by lower excess probabilities, or using suitable *protection or importance coefficients* and hence increasing the design action.

In this field, similarly crucial are infrastructures which, if collapsed, involve a large number of human lives, large economic losses, high damage to environment or severe losses associated with cultural heritage.
The constructions commonly associated with these two categories are known as ***strategic*** and ***relevant***. Sometimes the prescriptions provided with the purpose of the vulnerability mitigation of the cultural heritage, might be in conflict with the requirement of preserving these buildings. Consequently, the operative tool used in this field are rather different from those followed for other structures, so as to accomplish the two basic requirements governing the cultural heritage: safety and preservation. Among strategic structures, a particular focus is made on hospitals.

4.4 Risk

As previously introduced, the risk can be defined as a convolution among *hazard*, *vulnerability* and *exposure*, where these three factors are in principle described through probabilistic functions and the convolution process is a mathematical operation of integration determining as final result the probability function of the loss, that is the risk.

In the following the risk is described in a simplified way, through an applicative example, used for setting up seismic risk maps of the building heritage in Italy, in 1996 (Bramerini et al.1995). The risk factors of this application are defined as follows.

4.4.1 Exposure

Exposure is provided by dwellings and population living in. For each town council these data are obtained by the ISTAT census, carried out every 10 years. In 1991 around 25 million dwellings and 57 millions of habitants were registered. Dwellings have been subdivided in decreasing vulnerability classes (A, B, C1, C2) following the statistical method introduced at §3.3.4. More precisely classes assignment has been carried out by correlating the building inventory described through *poor* indicators provided by ISTAT census (structural typology and construction date) with the layout referred to constructive characteristics of the same classes, obtained by specific census on some Italian regions by GNDT research groups. Three vulnerability classes have been considered for masonry by MSK (A,B,C1), increased by an additional one (C2) specific of the reinforced concrete.

The correlation has been set up considering the typology and damage surveys following the earthquakes of 1980 in Irpinia 1984 in Lazio and Abruzzo. More than

60.000 records have been processed, so as to establish a formulation between constructive characteristics and vulnerability classes (Braga et al.1982,1983).

The formulation is :

$V=M \bullet IS$

where

- V is a vector listing the percentage of dwellings of the town considered, by vulnerability classes;
- IS is a vector listing percentage of dwellings of the town by age of constructions, as provided by ISTAT census;
- M is a matrix correlating the two vectors V and IS.

In conclusion, the number of dwellings in each vulnerability class is determined from the number of dwellings by age of constructions by means of the numbers in the columns of matrix *M*. As it can be easily understood masonry dwellings are those mostly attributed to class A, while those more recently built mostly assigned with class *C1*.

4.4.2 Hazard

The hazard is described by the annual frequency of intensities occurred in the town council under observation. In other terms: the number of times per year (λ_I) in which a seismic event of given intensity MCS I occurs.

4.4.3 Vulnerability

The primary vulnerability is measured through the frequency of occurrence of a given damage level D, once the dwelling has been subjected to an earthquake of intensity I (DPM, damage probability matrix, see §3.3.4.

$DPM_T(I,D) = P(D=d|I)$ for the class *T*.

In this formulation:

- *D* : is the damage level observed, in the 6 levels provided by MSK;
- *I* : is the intensity of the event;
- *T* : is the vulnerability class (A, B, C1 o C2).

4.4.4 Risk

By means of hazard, vulnerability and exposure elements described at the previous points, an estimation of the possible consequences in terms of predictable losses has been carried out (Lucantoni et al, 2001, Di Pasquale et al.2005).

For a given town and given intensity *I*, the number of dwellings (and relative surfaces) undergoing a specific damage level is determined. This can be obtained by simply calculating the product between the probability of occurrence of the given damage level (relative either to the intensity felt and to the vulnerability class), for the number of dwellings (or areas of floors) of that class. Finally, the number of damaged dwellings is simply given by the sum of the damaged dwellings obtained per each vulnerability class.

$NAB(D) = S_T\ nab_T{*}\ DPM_T(D,I)\ S(D) = S_T\ Sup_T{*}\ DPM_T(D,I)$

In risk analyses, it is required to keep into account all events which, in the period of observation, can occur in the town and hence summing of the effects for each intensity level, each weighed with the number of occurrences.

For a period of one year, being λ_I the annual occurrences for intensity I in the town, the following expression is obtained:

$NAB(D) = S_T\ nab_T * (\ S_I\ l_I\ DPM_T(D,I))$

where :

- *NAB(D)* is the number of dwellings undergoing damage level D;
- nab_T is the number of dwellings of the town of vulnerability class T;

A similar expression can be written for the overall floor areas.

Once either the number of dwellings injured by a given damage level and the relative number of habitants is known, the economic and social consequences can be calculated, according the following relations:

- *Collapsed dwellings*;
- *Unusable dwellings*: those with damage 4 plus a fraction (40%) of those with damage 3;

- *Damaged but usable buildings*: those with damage 2, plus those with damage 3 not considered at previous point;
- *People potentially involved in total collapses*: people resident in collapsed dwellings (potential victims + injured in case of presence of habitant in the dwellings);
- *Homeless*: people whose dwellings are unusable.

For what concerns the number of victims and more in general the problem of casualties, a careful estimate would require parameters like the time of the event, the prevalent businesses in the area (residential, industrial, agricultural, and so on), and further variables on survival percentages. For a detailed exam of this problem the specific literature has been reviewed (Coburn et al., 1992).

By analysing expressions in the mentioned work together with statistics on last strong Italian events, it can be concluded (Bramerini et al., 1995) that the number of victims should be estimated between 25% and 50% of the involved population, on turn obtained by residents in the collapsed dwellings. Usually, it is assumed in the estimations that the number of victims is equal to 30% of the involved population. Moreover in order to obtain a quantitative measure of the total economic damage, a retrofitting cost is associated with each damage level. In the present work a criterion similar to the one provided by ATC (1975) has been followed, consisting in expressing the retrofitting cost as fraction of the total cost of reconstruction (damage factor).

The relation used between the annual average damage factor and the damage level of MSK, is shown below:

Damage Level	0	1	2	3	4	5
Damage Factor	0	0.01	0.1	0.35	0.75	1

It is sufficient to sum the products of the floor area having each damage level for the relative factors, in order to obtain an estimate of the damage in term of equivalent squared meters. The product between this equivalent surface and the reconstruction cost provides an estimate of the economic value of the damage.

Following the above criterion, a deterministic assessment of losses very close to the average value of the predictable loss is obtained (Di Pasquale e Orsini, 1998).

The loss is expressed according two methods:

a) economical loss expressed as cost for repairing the damaged dwellings;
b) loss of human lives, expressed by the population resident in dwellings for which a total collapse is predicted (this number is around 30% of the population in collapsed dwellings).

The results obtained may be expressed through absolute values or relative to the exposed heritage of each town.

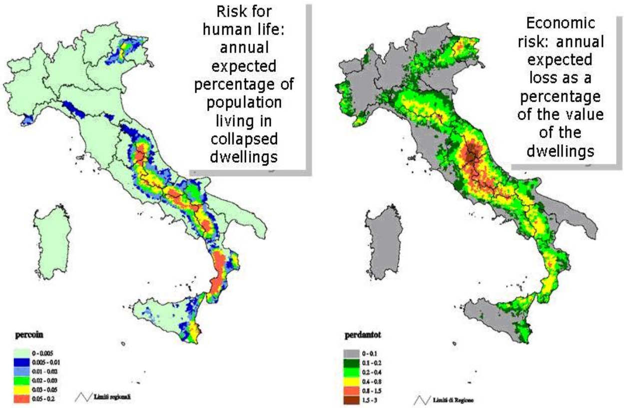


Figure 65 ***Italian Loss Maps (on the left: Human Life risk; on the right: Economic Risk)***

The absolute loss maps outline the dimension of the problem caused by the seismic event and can be related to the social risk. Conversely, percentage loss maps outline the intensity of the problem. The last ones, showed in Figure 65, are the most suitable for outlining the individual risk. One can clearly observe that the risk for human life is particularly high in regions where strongest, although not frequent, events are predicted (Calabria, Irpinia, eastern Sicily), while the economic risk is higher in the Apennine chain, featured by frequent though not catastrophic earthquakes.

4.5 Introduction to Risk Mitigation

Earthquakes are among the major natural hazards that some countries must cope with. Striking without warning, they can kill thousands of people and cause widespread damage disrupting regional or, in case of small countries, national economies. The sudden and normally without warning occurrence of earthquakes, combined with a deep-rooted feeling in people that they are helpless in protecting their lives and property from them, makes earthquakes one of the most fearsome natural disasters. On the other hand, the irregularity and long intervals between their occurrences are factors contributing to reduced awareness about earthquake risks among the public and its officials and hence to reduced allocation of resources for their mitigation, covering all aspects of the problem. Thus drawing from Greek experience, the various activities required for seismic risk mitigation, in order to place them into proper perspective. Research in seismology, earthquake resistant structures and earthquake prediction are among such activities whose contribution to the final goal - the reduction of seismic risk - is also examined. As a final point, current trends in earthquake resistant design are briefly reviewed and the seismological research requirements these trends generate are identified. Moreover, it is not uncommon to see misallocation of resources by non-knowledgeable decision makers and politicians, especially when they act under fear of criticism and public opinion pressures in the aftermath of a catastrophic event. It is therefore up to the scientists to help maintain an increased level of awareness about seismic hazards and also make policy makers understand that seismic risk reduction requires continuous, long-term efforts with a multitude of activities. The term seismic risk is used herein to mean expected effects of earthquakes to humans, their activities and works (i.e. the built environment), in accordance with the UNDRO (1979) definition. Quite often, however, seismic risk is mixed with seismic hazard, which, according to the same document, should be used to express the probability of occurrence, within a specific period of time in a given area, of a potentially damaging earthquake.

4.5.1 Quantification of Seismic Risk: Turkey as an Example

The seismic risk assessment applications have increased in Turkey after 1999 earthquakes occurred in Izmit and Duzce which have resulted in large number of damaged buildings, casualties and economic loss. As one of the most vulnerable city in Turkey seismic risk assessment studies have been focused on Istanbul. The methodologies developed by different research organization have been applied on a pilot region called Zeytinburnu, Istanbul. Based on the results obtained by each methodology most risk quantification in Istanbul have been determined and mitigation activities have been planned. A comprehensive effort has been performed in order to compile data from the field in order to get the relevant parameters of the buildings in Zeytinburnu for risk assessment studies. The pilot application for sismic risk quantification has been summarized below.

4.5.1.1 Seismic Vulnerability Assessment for Existing Buildings in Zeytinburnu Istanbul

The main purpose of this study is assessment of seismic vulnerability of existing RC and masonry building stock in Zeytinburnu, Istanbul, Turkey. The assessment procedure is composed of three levelsof analysis. The first level involves a procedure based on seismic resistance characteristics of building compiled through a site survey. The result of this procedure is used to prioritize seismic vulnerability of each building in the building stock. In other words this procedure acts as a filter to determine the building in the second stage to be used in more detailed level of vulnerability analysis. In Zeytinburnu case totally 16000 buildings are evaluated through this procedure

13000 and 3000 of which is RC and masonry respectively. Distribution of RC and masonry buildings are given in the figures below.

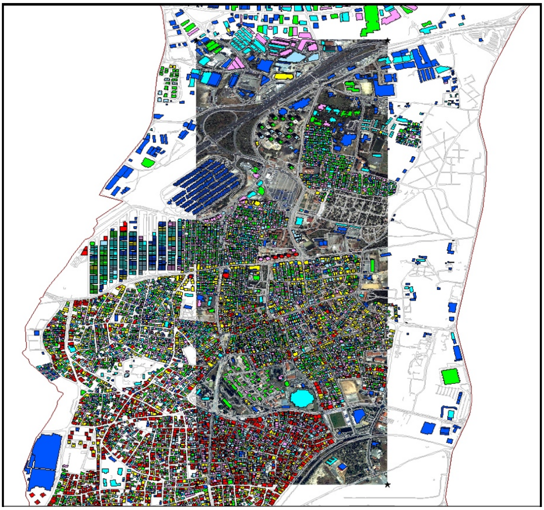


Figure 66(a). Distribution of RC Buildings in Zeytinburnu

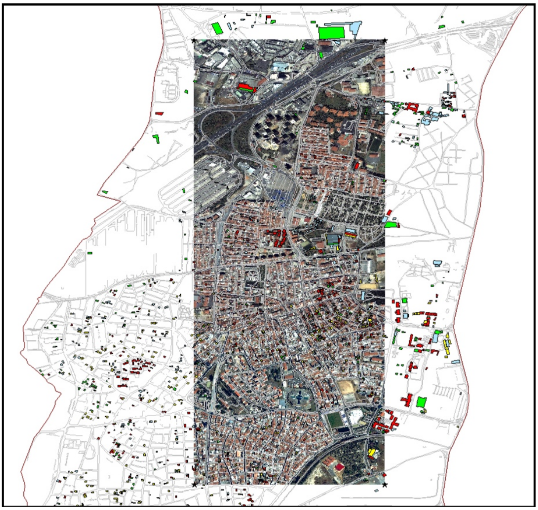


Figure 66 (b). Distribution of Masonry Buildings in Zeytinburnu

4.5.1.2 Description of the Methodology

The first level vulnerability assessment corresponds to pre evaluation of building inventory based on information compiled form street survey studies. The main purpose of the analysis performed in this level is evaluate the seismic vulnerability of the building stock with a limited information compiled in a short time in a rational fashion. This level of analysis is more effective when building stock involves thousand of high vulnerable buildings as in the case of Istanbul. The result of this level serves as tool to determined the most vulnerable areas in a region with high level of seismic risk.

In the results of the fist level analysis provides a prioritization scheme for buildings in the building stock. As such the next level analysis, which is more complex both in terms of input data and methodology, can be applied more effectively.

In the first level analysis the following properties of the buildings are complied from street survey. The properties are;

- GPS coordinate of the building
- Photo of the of the building from the front side
- Approximate base floor area of the building
- Total number of storeys(excluding the basements)
- Visual construction quality
- Adjacent building situation
- Story level situation with the adjacent building
- Type of the structural system
- Soft story (RC buildings)
- Weak story (RC buildings)
- Short column (RC buildings)
- Void ratio of the bearing walls (Masonry buildings)
- Irregularity of the void in the bearing walls (Masonry buildings)

4.5.1.3. Definition of Seismic Hazard and Performance Criteria

The seismic assessment of the existing buildings is determined based on the site- specific seismic hazard analysis rather than Code based definition. The seismic hazard analysis is performed both in probabilistic and deterministic manner. The probabilistic assessment is performed for 50%, 10% and 2 percent of exceeding probability in 50 years. The vulnerability assessment calculations

The performance criteria is defined as "life safety" for the seismic input determined for deterministic scenario and "collapse prevention" for probabilistic seismic hazard analysis for 10% probability of exceedance in 50 years.

The seismic input is defined by 5% damped site specific acceleration response spectrum defined in NEHRP through spectral acceleration values given for T=0.2 s an T=1.0 s computed for the specified location considering the local site conditions.

4.5.1.4 Determination of Seismic Demand

The seismic demand of the building is determined by using the accelaraiton response spectrum defined above. The procedure for determination of displacement demand is based on the procedure defined in FEMA356. The basic steps of this procedure is given as

1. Definition of 5% elastic acceleration response spectrum as defined in FEMA 356.

$S_{ae} = (1 + 1.5\, T / T_o)\, S_S / 2.5$ $\qquad (T < T_o)$

$S_{ae} = S_S$ $\qquad (T_o \leq T \leq T_S)$

$S_{ae} = S_1 / T$ $\qquad (T > T_S)$

$T_S = S_1 / S_S$ and $T_o = 0.2\, T_S$

2. The elastic displacement spectrum is defined as

$S_{de} = (T/2\pi)^2 S_{ae}$

3. Fundamental period of each building type is calculated as

$T = 0.15\, n$ (RC buildings)

$T = 0.075\, n$ (Masonry buildings)

n: number story excluding the basements

3. Inelastic spectral displacement is determined as

$S_{di} = C_d S_{de}$

C_d : Inelastic displacement modification factor defined in FEMA 356

C_d coefficient involves all the factors (C1, C2, C3) defined in FEMA356 and given based on number of story as

C_d coefficient

n	1	2	3	≥4
C_d	1.5	1.3	1.1	1.0

The inelastic displacement demand is correlated with the top displacement of the building. As such the “top displacement demand”, u_n, is determined. The “building drift demand”, D_d, is calculate as the ratio of the top displacement demand and building height, H_n, as

$D_d = u_n / H_n$

4.5.1.5 Determination of Displacement Capacity of the Buildings

The top displacement capacity of each building type for life safety and collapse prevention performance levels is compared with top displacement capacities defined as;

$D_c = D_{co} C_c$

D_{co}:Base top displacement capacity
C_c :capacity modification factor

Base top displacement capacity (D_{co})

Building Type	*Life Safety Performance Criteria*	*Collapse Prevention Performance Criteria*
Betonarme	0.008	0.014
Yığma	0.006	0.010

C_c factor is determined based on the seismic properties for RC and masonry buildings defined above.

4.5.1.6 Prioritization of Seismic Risk of Buildings

The purpose of the analysis in the first level is to provide information on "seismic vulnerability level of building among each other" rather than determination of seismic safety and deficiency of each building.

The final target in the first level analysis is to determine a "rational prioritization" for the building stock in Zeytinburnu and whole Istanbul based on "demand / capacity" ratios determined through the methodology explained above. As such the regions with high level of vulnerability can be determined for the next level analysis. The basic criteria for prioritization is the ratio of the "Capacity", D_c, and "Demand", D_d, in term do building drift for each building type. As the last point it should be emphasized that the results fo the first level analysis can act as guide for "urban renewal" applications. Results of the prioritization calculations are presented in the figures .

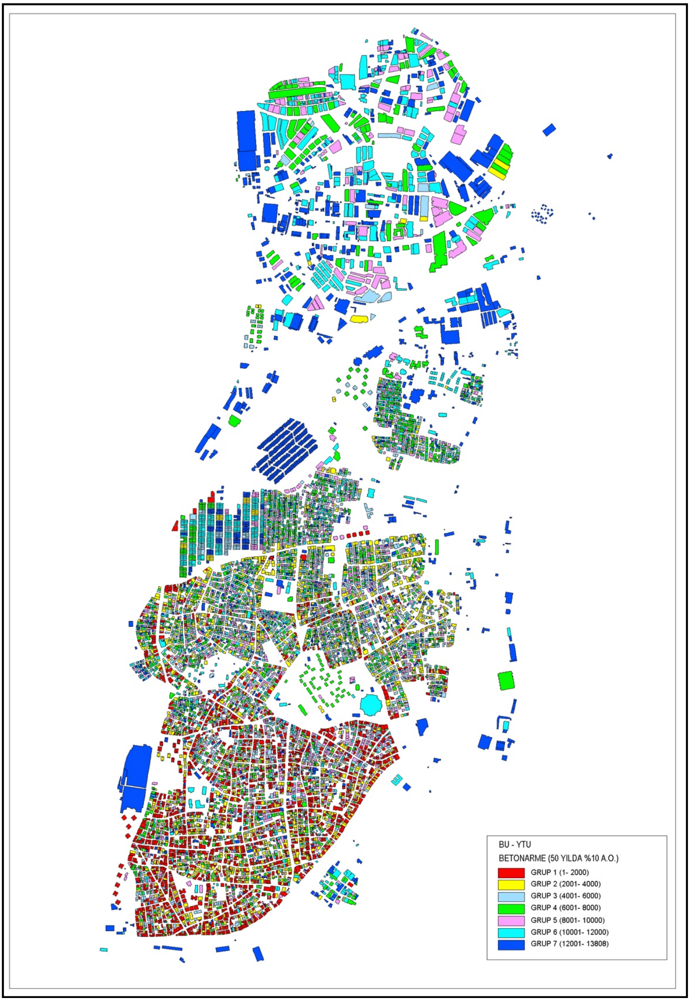


Figure 67. Prioritization Results for RC Buildings for 10% Probability in 50 Year of Seismic Input

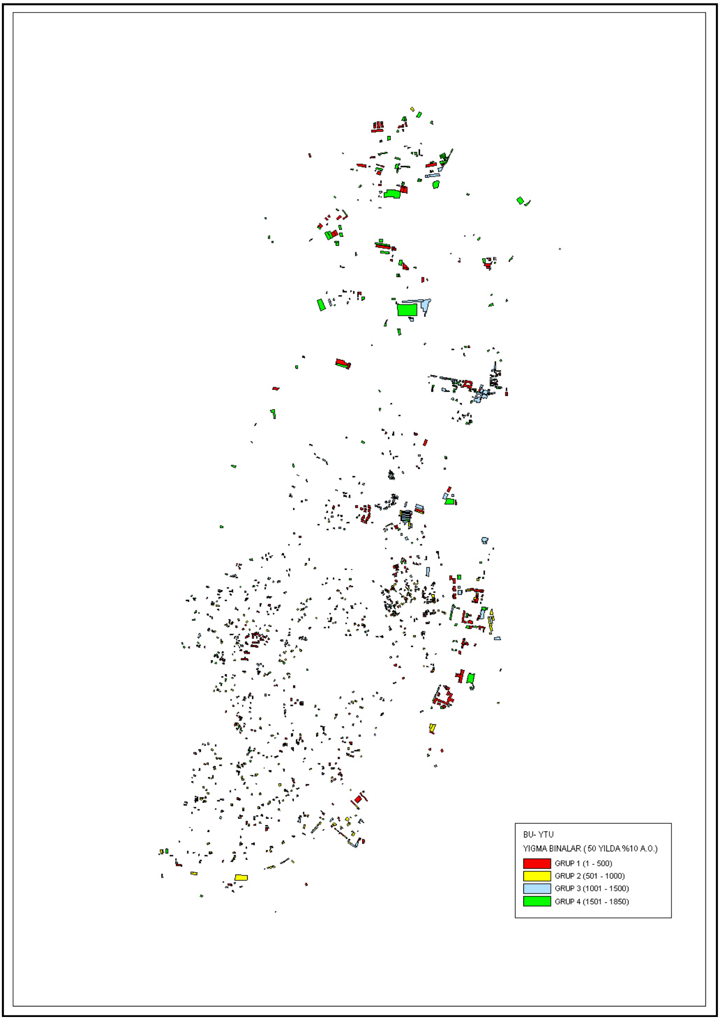


Figure 68. Prioritization Results for Masonry Buildings for 10% Probability in 50 Year of Seismic Input

4.5.2 Quantification of Seismic Risk: Austria as an Example

This summary provides a review on quantification of seismic risk in Austria. These activities are conducted on a larger scale within funded research projects, or they are based on individual research efforts. In recent years there was a particular focus on the seismic assessment of historical brick-masonry buildings located in Vienna. The Austrian seismic network and infrastructure is continually improving. Other research activities comprise the seismic assessment of modern buildings, further developments on passive and semi-active damping devices to mitigate earthquake induced vibrations, and historical earthquake research, amongst others. The Austrian Association for Earthquake Engineering and Structural Dynamics (OGE) represents individual members, academic and non-academic research institutions, and companies involved in earthquake engineering and seismic hazard assessment. Seismologists are organized in the Austrian Geophysical Society (AGS).

a) Seismic Hazard in Austria

Austria is partially located in a moderate seismic zone. The most active seismic regions are the Vienna

Basin, the Mürz Valley (*Mürztal*), and the Inn Valley (*Inntal*) in Tyrol. The southern part of Carinthia may be subjected to moderately large earthquakes, which have their origin in Italy and Slovenia. In average, every three years an earthquake of intensity 6 according to EMS-98 occurs, every 15 years an earthquake of intensity 7, and every 75 years an earthquake of intensity 8. According to Eurocode 8 (EC8 2004) Austria is divided into five seismic zones. Zone 1 with low seismic risk exhibits an effective horizontal acceleration smaller than 0.35 m/s^2, while in zone 5 the effective acceleration is larger than 1.00 m/s^2.

b) Assessment of Seismic Hazard in Austria

Seismic Network and Infrastructure in Austria

The Central Institute for Meteorology and Geodynamics (ZAMG) maintains the infrastructure for seismic monitoring in Austria. Today's network in and around Austria, whose data are routinely analysed, consists of more than 50 broadband and strong motion stations. Most data can be accessed via AutoDRM, or from the Data Center of the Observatories and Research Facilities for European Seismology (ORFEUS 2012). Recently, within the research project NERIES in the Conrad observatory (Austria) ZAMG conducted comparative measurement campaigns, and methods for investigation of the soil properties were developed and checked.

Historical Seismology in Austria and Vicinity

A few thousand written records, which date back almost 1000 years, were analysed during the past years to gain insight into the seismic history of several provinces of Austria. Two important outcomes should be mentioned: Firstly, besides numerous new seismic events, which could be found, a few seismic events have been proven to be fakes and secondly, experience has shown that previous earthquake magnitudes were rather over- than underestimated. Multiple entries, due to the combination of several earthquake catalogues, had also to be clarified. In Austria, amongst others ZAMG conducts ongoing research on historical earthquakes. As an example, Dr. Christa Hammerl headed a research project aiming at improving the assessment of seismic hazard in the province of Lower Austria. Recently, Dr. Georg Gangl and Dr. Kurt Decker from the University of Vienna provided a compilation of strong Austrian earthquakes with intensities higher than 7. Within the Interreg IVA project HAREIA, funded by the EC, six regions in the border area of Italy and Austria investigated in a collaborative effort historical earthquakes in this area.

c) Selected Activities on Earthquake Engineering in Austria

Seismic Assessment of Historical Viennese Brick-Masonry Buildings

The City Center of Vienna is dominated by historical residential brick-masonry buildings, built during a major urban expansion in the period of 1848 through 1918. At present, one third of the complete building stock of Vienna, that is 32,000 objects, consists of Viennese brick-masonry buildings. Recently, the behavior of these buildings under seismic loads has become of significant interest, because Eurocode 8 (2004) imposes additional seismic demands to these structures. As a consequence, their seismic resistance cannot be verified anymore with traditional methods of analysis available to the design engineer in practice. This has led to a drastic decline of rehabilitation and remodeling of this building type, and consequently to huge economic losses. Consequently, several collaborative and individual research initiatives were initiated in an effort to assess the seismic resistance of Viennese brick-masonry more realistically. Amongst others, examples are the national research projects SEISMID and *Wiener Baukultur*.

Seismic Behavior of Modern Clay Brick-Masonry

The increase of earthquake safety of buildings built with fired clay bricks and blocks is an important research topic of the Austrian based brick manufacturer *Wienerberger*. *Wienerberger* is currently involved in several projects to reduce the earthquake risk of modern brick-masonry.

Further Developments of the Tuned Liquid Column Damper

The main goal of research conducted under the guidance of Prof. Franz Ziegler at the Vienna University of Technology has been the improvement of the Tuned Liquid Column Damper (TLCD) by sealing the U- or V-shaped piping system and thus adding the resulting gas-spring effect.

Assessment of the Seismic Performance of Tuned Mass Dampers

At the University of Innsbruck Alexander Tributsch and Prof. Christoph Adam have assessed the seismic performance of TMDs based on sets of recorded ground motions. For the simplest configuration of a structure-TMD assembly, in a comprehensive study characteristic response quantities have been derived and statistically evaluated.

Seismic Collapse Capacity of Frame Structures Vulnerable to the P-Delta Effect

A topic of ongoing research at the University of Innsbruck is the prediction of the global collapse capacity of earthquake excited frame structures with simplified but yet reliable measures (Prof. C. Adam and Dr. C. Jäger). The proposed collapse capacity spectrum methodology is based on pushover analyses, equivalent single-degree-of-freedom systems, and a collapse capacity spectrum.

Field Testing for Seismic Vulnerability Assessment

An ongoing research activity of AIT Mobility (Prof. R. Flesch, H. Friedl, Dr. M. Ralbovsky) concerns the seismic assessment of important existing buildings such as hospitals. In order to assess an existing building by an elaborated structural model, a methodology was developed, where dynamic in-situ measurements on the structure and its subsoil are combined with finite element calculations.

4.5.3 Quantification of Seismic Risk: Italy as an Example

In order to show you an example, we will see very briefly in this part how the Italian code (named N.T.C. 2008 – 14 Gennaio 2008) take into account the seismic problem.

The above code said that all the structure and their structural components must be designed, executed, tested and subject to maintenance in such a way as to allow their intended use, in the form economically sustainable and the **level of safety** provided by these rules. The safety and performance of a structure or a part of it must be assessed in the *limit states* that may occur during the nominal life. In the more simple way, limit state is the condition after which the structure no longer meets the needs for which it was designed. In particular we can have these following requirements:

- Safety against ultimate limit states (ULS): ability to avoid crashes, loss of balance instability and serious, full or partial, that could compromise the safety of persons or result in the loss of property, or causing serious environmental and social damage, or put off-duty work. The main ULS, are listed on the next page:

a) loss of equilibrium of the structure or a part of its;

b) movements or excessive deformation;

c) achieving maximum capacity of resistance of parts of structures, connections,
foundations;

d) achieving maximum capacity for resistance of the structure as a whole;

e) achievement of failure mechanisms in the ground;

f) and so forth…

- Safety against serviceability limit states (SLS): the ability to guarantee the performance provided for the operating conditions. The main Limit States, referred to in § 2.1, are listed below:

  a) local damage (e.g. excessive cracking of concrete) that can reduce the durability of the structure, efficiency or appearance;

  b) displacements and deformations which could limit the use of the building, its efficiency and the appearance;

  c) displacements and deformations which could compromise the efficiency and the appearance of elements not structural, plant and equipment;

  d) vibration that may affect the use of the building;

  e) and so forth…

- Robustness against actions exceptional ability to avoid damage disproportionate magnitude of the triggering causes such as fire, explosions, impacts.

Exceeding either the ultimate limit state is irreversible and is called **collapse**, while exceeding either limit state may be of a **reversible** or **irreversible**.

For the evaluation of construction safety must be taken probabilistic criteria scientifically proven. The following are regulated semi-probabilistic method criteria limit states based on the use of partial safety factors, applicable in most cases; this method is said *first level*.

In the method semi-probabilistic limit states, structural safety must be verified by the comparison between the **resistance** (R_d) and the **effect** (E_d) of the actions, in others words:

$R_d \geq E_d$.

For structural safety, the resistance of the materials and **actions** (F_k) are represented by characteristic values, respectively, as the lower *fractile* of the resistors and the *fractile* (top or bottom) of actions that minimize safety.

About the *seismic design load*, the code recommended to evaluate this force for each limit state in agreement with the "basic seismic hazard" of the construction site. The seismic hazard is defined in terms of maximum expected horizontal acceleration a_g free field conditions on the reference site rigid horizontal surface topography as well as the ordinates of the **elastic response spectrum** in if acceleration corresponding to it $S_e(T)$, with reference to predetermined probability of exceedance P_{VR}, as defined in § 3.2.1, V_R during the reference period. Alternatively authorize the use of accelerograms, if properly commensurate with the seismic hazard of the site.

For the purpose of this code spectral shapes are defined, for each of the probability of exceeded during the reference period P_{VR}, from the values of the following parameters **on site** horizontal rigid reference:

- a_g: maximum horizontal acceleration at the site;
- F_o: maximum value of the amplification factor of the spectrum in the horizontal acceleration.
- T^*_C: period beginning tract at a constant speed in horizontal acceleration spectrum.

All these value are specified into this code. In particularly, the code provide an *excel* file in which you can find all this parameters in order to define in very simple way the seismic action (see Figure 669a).

It is important to underline that the code provide the value of the above parameters for each point the ground in field, defined within a grid about 5 x 5 km.

Figure 669a ***Main Page of Excel File – NTC 2008 – Seismic Parameters***

Against seismic limit states, both of them are identified referring to the performance of the construction as a whole, including the structural elements, non-structural and equipment.

In particular the *Serviceability Limit State*s are:

- Limit State of Operation (**LSO**): Following the earthquake, the building as a whole, including the structural elements, those non-structural, equipment relevant to its function, it must not be damaged and significant disruption of use;
- Damage Limit State (**LSD**): Following the earthquake, the building as a whole, including the structural elements, those non-structural, equipment

relevant to its function takes damage that would not put at risk users and do not compromise significantly the capacity of resistance and rigidity against vertical actions and horizontal, immediately usable while keeping the interruption of use of part of the equipment.

The *Ultimate Limit States* are:

- Limit State of preservation of life (**LSV**): Following the earthquake, the building undergoes failure and collapse of the non-structural components and installations and significant damage structural components which is associated with a significant loss of stiffness against horizontal actions; the construction preserves a part of the resistance and stiffness for vertical and a margin of safety against collapse due to seismic actions horizontal;
- Limit State of Collapse Prevention (**LSC**): following the earthquake the building suffered serious breakdown and collapse of non-structural components and installations and serious damage of structural components, the building still retains a margin of safety for actions vertical and a small margin of safety against collapse due to horizontal actions.

The probability of being exceeded during the reporting period P_{VR}, which refer to identify action
earthquake agent in each of the limit states considered are explained in the following table (see Figure 679b).

Stati Limite		P_{V_R} : Probabilità di superamento nel periodo di riferimento V_R
Stati limite di esercizio	SLO	81%
	SLD	63%
Stati limite ultimi	SLV	10%
	SLC	5%

Figure 679b ***Probability of Being Exceeded During the Reporting Period P_{VR} – NTC 2008***

Define the seismic load, or better the response spectrum, using the file provide by the Italian code is very simple. In order to show you how you can use it, on the other page we reported an example for the city of Pavia.
First of all, you have to localize your country using the command “Ricerca” (search) by name or coordinates. In our case we will search using name command, putting “Pavia” in the red box (see Figure 689c).

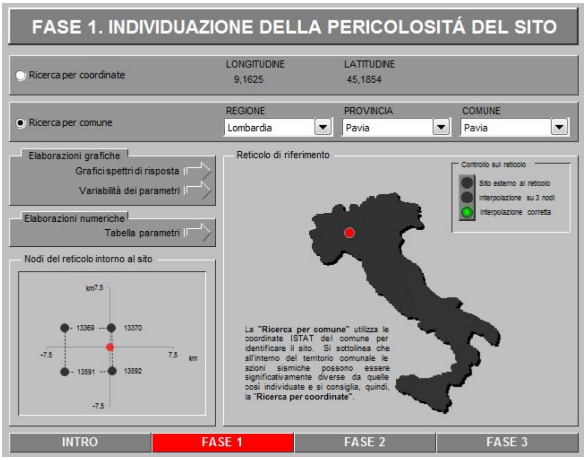


Figure 689c ***Phase 1 – Localize Your Site***

After this you have to define, in the red box below (see Figure 69d) some parameters, nominal life V_N and class of use C_U, related to your particular structure. These parameters are related to the different kind and importance of the structure under study. More information about this can be find within the code. Thus, using the default value for our case, we are ready to jump into the phase 3.

Figure 699d ***Phase 2 – Characterizes Your Site***

Now, the last step is to define the ground parameters and choosing the particular limit state under study (red box in Figure 70). For simplicity we use the default value. It is important to underline that this value must be define very carefully by the designer, because they can change significantly the final seismic load. The result of this process is shown in the following picture (see Figure 71).

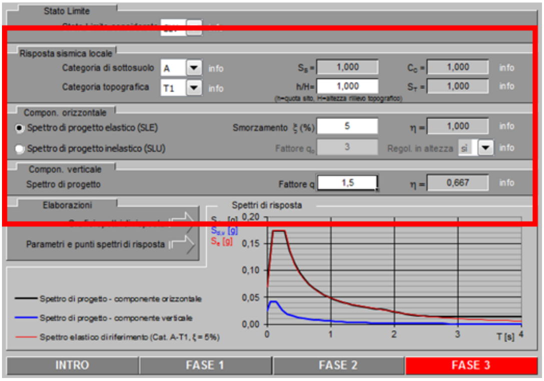


Figure 70 ***Phase 3 – Characterizes Your Analysis and Ground***

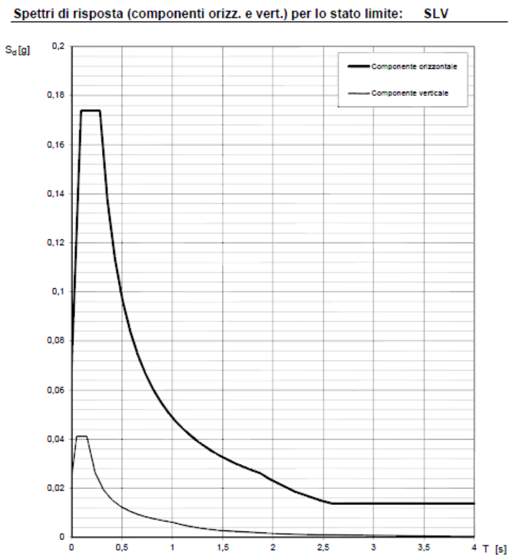


Figure 71 ***Horizontal and Vertical Response Spectrum – Using NTC 2008***

4.5.4 Quantification of Seismic risk: Greece as an Example

In the past three decades, several earthquakes have caused damage in various parts of Greece and a significant effort has been made by the authorities to assess and record their effects on buildings. Structural vulnerability assessment based on observed damage was first proposed in the form of damage probability matrices (DPMs) by Whitman et al. (1973) with the modified Mercalli intensity scale as its common denominator. DPMs express, in a discrete form, the conditional probability of obtaining a damage level (e.g., collapse or serious structural damage, etc.), due to a ground motion of certain intensity. Later on vulnerability (fragility) curves were derived from analysis of various observed damage data sets obtained after damaging earthquakes, expressing the probability of exceeding a given damage level at distinct levels of ground motion severity [described by the peak ground acceleration (PGA) or other ground motion parameters]. For buildings in Europe and elsewhere the studies of Spence et al. (1992), Rossetto and Elnashai (2003), Lagomarsino and Giovinazzi (2006), Colombi et al. (2008) and Rota et al. (2008) belong in this category. For Greek buildings, fragility curves have also been derived from hybrid methodologies (Kappos et al., 2006; Kappos and Panagopoulos, 2010), based on a combination of real observed damage data and analytical models. In addition, intensity-based DPM's were proposed for Greek buildings by Kappos et al. (2002) and Eleftheriadou and Karabinis (2008), the latter based on the post-earthquake damage surveys of the 1999 earthquake near Athens. Recently, Karababa and Pomonis (2011) have proposed a new set of empirical fragility curves resulting from the 2003 Lefkada Island earthquake observed damage data.

There are several advantages but also limitations when using observed damage data to study the vulnerability of the existing building stock in a region. Although post-earthquake damage surveys have been developed primarily for assessing the safety and usability of damaged buildings, they also provide a wealth of information that can be used towards understanding vulnerability. Furthermore, in most regions existing buildings are a mix of old non-engineered buildings, buildings that have been constructed prior to the introduction of earthquake codes as well as engineered buildings built during periods when different earthquake codes were in application. Analytical models for such a wealth of structural types are hard to construct and validate.Therefore, it is important that observed damage data are collected and further utilized to the benefit of society. Finally, a further useful aspect of utilizing and analyzing observed damage data sets is that they enable us to carry out checks and validations of vulnerability assessments based on purely analytical methods (Colombi *et al.*, 2008).

On the other hand, however, it is imperative that the observed damage data be extensive, covering a wide range of existing building types that have been subjected to a wide range of ground motion severities and that the damage surveys be complete (i.e., to cover all the buildings and all the damage states including the buildings that were not damaged within an affected study zone). This usually requires for a compilation to be made of observed damage data sets in a wide region and/or across several events so that the necessary breadth of information is captured. For Greece in particular it is also important that damage data sets contain sufficient information on the performance of unreinforced masonry (URM) buildings, as well as reinforced concrete (RC) buildings built during the period of application of the successive earthquake codes (in the case of Greece 1959, 1985, 1995 and 2004 are the benchmark years of introduction of successive earthquake codes).

In the present study whilst keeping all these limitations in mind we develop a Greek observed damage database based on 4 earthquakes that took place in the period 1986-2003 (the September 13,1986 Kalamata Earthquake; the March 26, 1993 Pyrgos Earthquake; the June 15, 1995

Aegion Earthquake and the August 14, 2003 Lefkada Island Earthquake). The four earthquakes had surface-wave magnitude in the range of 5.5 to 6.4. The first three of the events occurred within the space of 11 years in the region of Peloponnese and the damage survey procedures used were quite similar. Also all three affected towns were within zone II of the 1959, 1984 and 2004 earthquake codes of Greece and in zone III of the code that was in use in the period 1995-2003
(which unlike the other 3 codes contained four zones). Using this database we were able to derive EMS-intensity based vulnerability functions for several low-rise (1 to 3 floors) and medium-rise (4 to 7 floors) building types commonly found in Greece.

4.5.5 Earthquake Damage Characterization and Homogenization for Greek Building Type

Through the analysis of observed damage data sets, the heterogeneity both of the post-earthquake damage surveys and of the damage data from which vulnerability curves are derived (Rota *et al.*, 2008; Gaspari, 2009) became apparent. Intuitively, data should be as much as possible homogeneous both in time and space, since significant variations of construction techniques may occur during time and from place to place, and this may compromise the homogeneity of building classes. Furthermore, damage surveys across a long period of time are usually performed using different forms and procedures. To address this limitation, we tried to correlate the damage degrees of the varied damage scales used for each damage survey so that a uniform damage scale was formed for each structural type.

In Greece, earthquake disaster response and reconstruction was drastically reviewed after the experiences gained in the aftermath of the June 1978 Thessaloniki and the February-March 1981

Corinth Bay earthquakes. The Earthquake Planning and Protection Organization of Greece (EPPO), was established in 1983 with the task to deal with matters related to earthquake safety and to coordinate all private and public actions for earthquake protection. EPPO formally introduced the first post-earthquake building safety assessment (usability) procedures in Greece in 1984. Three categories of safety-usability and the corresponding posting (tagging) scheme ("green" for usable; "yellow" for temporarily unusable and "red" for unusable/dangerous) and six categories of damage, were formed. There were four grades of damage in structural elements (light, significant, serious and heavy damage) and an explicit description of the damage in each grade and structural element was issued, aiming to a uniform damage grading. A field manual was issued and distributed to engineers, administrators and agencies involved in post-earthquake inspection of buildings (EPPO, 1984).

In 1997, EPPO proposed a new post-earthquake building inspection procedure for the first level quick response damage and usability assessments (which largely maintained but simplified the 1984 inspection form) while keeping the three-colour building safety categorization unchanged. For the more detailed second degree inspections (taking place for buildings that have the "yellow" or "red" damage grade) the 1984 EPPO guidelines remain in use (EPPO, 1997). Details of these documents are also presented by Dandoulaki *et al.* (1998).

4.5.1.1 Damage Scale for RC Buildings

For the RC buildings, we had damage data sets from three of the four events (the 1993 Pyrgos earthquake caused only limited damage to RC buildings). As seen in Table 1, RC buildings in the 1986 Kalamata, 1995 Aegion and 2003 Lefkada earthquakes (Argyrakis *et al.*, 1987; Fardis *et al.*, 1997; Karababa, 2007; Karababa and Pomonis, 2011) were assigned the three EPPO usability classes, with the exception of Kalamata, where an additional fourth class "purple" was used for the buildings that collapsed or were heavily damaged and deemed irreparable.

In this analysis the “purple” buildings of Kalamata that did not collapse (39 out 44 RC buildings) were included in the “red” damage grade. In Greece the “red-tag” is given to buildings with the worst damage that are deemed unsafe for occupation and will need a second-degree damage and safety assessment.

Table 1 - - Correlation Between the Damage Scales Used in Three Post-Earthquake surveys of RC Buildings. The Tagging Colour is Used.

<table>
<tr><th colspan="3">Damage description</th></tr>
<tr><th>Kalamata (1986)</th><th>Aegion (1995)</th><th>Lefkada (2003)</th></tr>
<tr><td rowspan="2">Undamaged or slight non-structural damage</td><td rowspan="2">Undamaged or slight non-structural damage</td><td>No damage</td></tr>
<tr><td>Slight non-structural damage</td></tr>
<tr><td>Light (moderate) structural damage</td><td>Moderate to serious structural</td><td>Moderate to serious structural</td></tr>
<tr><td>Heavy structural damage</td><td rowspan="2">Very heavy structural damage - collapse</td><td rowspan="2">Very heavy structural damage - collapse</td></tr>
<tr><td>Very heavy structural damage (incl. partial and total collapse) that will be demolished</td></tr>
</table>

After the second inspection a building that continues to be classified as “red” is usually issued with a “protocol for demolition” as repair is deemed not feasible or uneconomical.

Furthermore, in order for the damage grades to be homogeneous across all 3 event-surveys, for the Kalamata data we assumed that the buildings with “red-tag” had actually suffered damage similar to those assigned the “yellow-tag” in Aegion and Lefkada Island. This is because in Kalamata due to strong aftershocks taking place in the days after the main shock a conservative approach was deemed necessary for the safety of the citizens which resulted in 893 RC buildings assigned the “red tag” (i.e., entry is absolutely prohibited) in the first-degree inspections. Later on when the more detailed second-degree damage and safety assessment took place the overwhelming majority of these buildings were re-assigned with “yellow tag” (personal communication with Costas Ioannides of the Greek Earthquake Rehabilitation Service). This is also corroborated by the fact that the number of RC buildings that were eventually issued with a “protocol for demolition” in Kalamata was not so great. It is possible that some under-estimation of the damage severity in Kalamata may be taking place due to this assumption (since in this way we assume that 52.4% of the RC buildings in Kalamata were yellow-tagged instead of 31.2% in the original 1986 first-degree inspections).

Table 2 - Proposed Damage Scale of RC Buildings (EPPO+).

Damage scale	Damage Description
White (D0)	No damage.
Green (D1)	Fine cracks to the infill walls and ceiling mortar.
	Hairline cracks in horizontal RC structural members.
Yellow (D2)	Large patches of mortar falling off walls and ceilings.
	Cracks in structural RC members (beams, columns, shear walls) but to an extent that does not constitute danger of collapse. Slight distortion
Red (D3)	Heavy damage and distortion of structural elements. Large number of crushed structural elements and
	Considerable dislocation of a storey and of the whole building
Black (D4)	Partial or complete collapse (loss of 50% of the building's volume has taken place at one or more floors).

One other problem lies in the fact that, buildings that may have not been damaged were not separately recorded in the case of Kalamata and Aegion but were grouped together with thebuildings that had light damage to the infill panels. Although, in the case of both towns it could be supposed (due to the strong shaking) that all buildings had at least slight non-structural damage, it was decided not to use the "green" buildings from the Kalamata and Aegion data sets in the vulnerability analysis.

Finally with the exception of the 1986 Kalamata data set, buildings that collapsed partially, extensively or totally were all included in the same red-tag usability class, which also contained buildings that were deemed unsafe for occupation but did not collapse. This problem was overcome because we were able to identify that only one RC building had collapsed in each of the study zones of the Aegion and Lefkada damage surveys, while five collapsed in Kalamata (see the respective sections about each event later on in this paper for more details). For this reason we introduced one more damage level ("black") to capture the proportion of buildings that suffered extensive or complete collapse, which is important information in human casualty estimation scenarios. In this study we use the definition of collapse proposed by the WHE-EERI PAGER (Prompt Assessment of Global Earthquake for Response) program (Pomonis *et al.*, 2009), whereby a building is considered to have collapsed when a 50% volume reduction or more has taken place at one or more floors.

In conclusion for the RC buildings we proposed a 4-level damage scale that we call EPPO+ as shown in Table 2. This damage scale is essentially the same as the one proposed in EPPO (1997) with the addition of the extra damage grade for the collapsed buildings.

4.5.5.2 Damage Scale for URM Buildings

In the case of URM buildings, it was necessary to modify the University of Patras damage scale (used in the 1993 Pyrgos and 1995 Aegion earthquake) and relate it to the

EMS-98 damage scale (Grünthal, 1998). The University of Patras 4-grade damage scale for masonry buildings is defined in Karantoni and Bouckovalas (1997). In addition to each damage grade, a damage degree from 1 to 4 respectively was assigned for each storey of each building (undamaged storeys and buildings were assigned damage degree zero). In the present study the mean damage degree of each building was derived as the mean of the damage degrees assigned to each of the two storeys. We excluded data on 3-storeyed masonry buildings as the small sample does not allow reasonable conclusions. In the case of Lefkada Island 2003 earthquake, Karababa and Pomonis (2011) classified the damage levels according to the EMS-98 damage scale after careful examination of the post-earthquake damage inspection forms in the archives of the local Earthquake Rehabilitation Office. The damage data to URM buildings in the city of Kalamata during the 1986 earthquake, although valuable were not used for reasons explained in the next section.

Table 3 - Correlation Between EMS-98 and University of Patras Damage Scales for URM Buildings.

Lefkada (2003)	Pyrgos (1993), Aegion (1995)	Damage characterization
Damage Grades (EMS-98)	Mean Damage Degree (Univ. Patras)	
D0	0	No damage
D1	0.5	Slight damage
D2	1.0 or 1.5	Moderate damage
D3	2.0 or 2.5	Heavy damage
D4	3.0 or 3.5	Very heavy damage
D5	4.0	Partial or total collapse

After careful examination the correlation between the two damage scales (University of Patras and EMS-98) shown in Table 3 was proposed and each of the masonry buildings in the University of Patras damage surveys was thus associated with a respective EMS-98 damage grade.

4.5.6 Compilation and Analysis of the Observed Damage Data Sets: Damage Data of the September 13 and 15, 1986 Kalamata Earthquakes

On September 13, 1986 at 20:24 local time an earthquake of M_s 6.2 (M_w=6.0) and focal depth of 8 km occurred 12 km north of Kalamata in south-western Peloponnese (Papazachos *et al.*, 1988). Damage was extensive in most parts of the town, as well as some nearby villages and 20 people lost their lives while 330 were injured (82 of which required hospitalization). Many aftershocks followed, the greatest of which occurred two days later and was centred within the town limits and had M_s 5.4 and focal depth 8 km. This aftershock caused an additional 37 injuries and further damage to the already weakened buildings. Kalamata, capital town of the Messinia prefecture, was at the time inhabited by around 43,000 people and spread over an area of around 10 km^2.

The damage distribution was not uniform across the town, with severe damage concentrated in the central and north-eastern parts of the city (Theodoulidis *et al.*, 2008), coinciding with the historic town center where old masonry buildings prevailed. Damage near the harbor and the coast was limited. This spatial variation in damage severity was attributed to soil conditions as well as source and directivity effects due to the causative fault's proximity (Gariel *et al.*, 1991).

Table 4 - Damage Distribution of RC Buildings After the September 13 and 15, 1986 Earthquakes Near Kalamata.

Structural type (age)	No. of storeys	Type of ground storey	Structural class code	No. of buildings	Percent of buildings Damage Grades (EPPO+)			
					D 0-1	D2	D3	D4
RC frame (pre-1987)	01-mar	soft-	RC2-	566	38.	59.	1	0
		regul ar	RC2-L	2,937	55. 1%	44. 1%	0 .9%	0 .0%
	04-lug	soft-storey	RC2-MP	392	15. 1%	83. 7%	0 .3%	1 .0%
		regul ar	RC2-M	309	20. 1%	1 .0%		0 .0%
Total				4,204	46.	52.	0	0

A strong motion accelerograph located in the city center recorded the main shock and aftershock. During the main-shock the horizontal *PGA* was 0.27 g and the peak horizontal velocity 32.3 cm/s, while the strong motion duration (acceleration over 0.1 g) was just 2.5 s (Anagnostopoulos *et al.*, 1987).

Damage inspection based on the 1984 EPPO guidelines followed immediately after the main shock (Andrikopoulou, 1987; Argyrakis *et al.*, 1987). The whole building stock of the town consisting of 10,171 buildings was inspected giving us a clear picture of the damage levels as a result of the combined effect of the main shock and the aftershock that followed about 36 hours later. We must also point out that the Kalamata damage data are not available on building by building basis, but only as overall damage distributions in the town by: structural type (masonry, mixed, RC); number of floors (1, 2, 3, 4, 5, 6 or 7 floors) and the existence or not of pilotis (soft-storey) or shops at ground floor level.

In 1986 in Kalamata, 41% of the buildings were RC frames with unreinforced clay brick infill walls constructed mostly in the period 1959-1985. These were buildings of 1 to 7 floors, 22.8% of which had soft-storey at ground floor level (open ground floor for car parking or other use, as well as buildings with shops or other commercial use in the ground floor). There were 44 buildings classed as "purple" (damaged beyond repair or collapsed partially or totally) and 893 classed as "red".

Anagnostopoulos *et al.* (1987) give us information and description of the RC buildings that collapsed during the main shock and the aftershocks that followed (4 buildings collapsed during the main shock and 3 during the aftershocks). Based on our proposed definition of collapse (see Table 2) five of these buildings were assigned to damage grade D4 ("black"). Table 4 shows the damage distribution by height (low and mid-rise) and by the existence or not of a soft-storey of the RC buildings in Kalamata. We note that it is not possible to differentiate undamaged from lightly damaged buildings. We also note that the majority of the mid-rise buildings had a soft storey and that 1% collapsed (as did 0.2% of their low-rise counterparts). In addition, we note that among the RC buildings with regular ground floor there was no case of collapse.

In 1986 in Kalamata, 44% of the buildings were old load-bearing URM (mostly rubble or hewn stone or mixed rubble and hewn stone masonry, but also some adobe buildings). In terms of height 98.8% of these were of one or two storeys. In addition 15% of the buildings were of mixed structural type or with mixed masonry materials. In this category are included URM buildings of mixed materials (e.g., rubble and hewn stone, stone and adobe, etc.) as well as buildings that contain both RC and URM sections (e.g., extensions of old masonry buildings with RC, either horizontally or vertically or both). In total 1,931 URM and 289 mixed structure or mixed masonry material buildings were classed as "purple" (43.2% and 19.3% respectively).

One mixed structure building collapsed during the main-shock (Anagnostopoulos *et al.*, 1987). The number of URM and mixed masonry material buildings that collapsed is unfortunately not known. Because the URM and mixed buildings data are incomplete (e.g., we do not know the proportion of various types of masonry, unknown number of buildings that collapsed, unknown types of mixed masonry materials) it was decided not to use these data in the vulnerability analysis. However, we did use the URM data for the estimation of the seismic intensity in Kalamata.

4.5.7 Damage Data of the March 26, 1993 Pyrgos Earthquake

On March 26, 1993 at 13:58 local time a moderate magnitude earthquake (M_s=5.5; M_w=5.4) with focal depth of 10-15 km, occurred at a distance of about 3 km north of the town of Pyrgos in north-western Peloponnese (Stavrakakis, 1996). As a result of the earthquake, one old woman lost her life whilst trying to escape a building, 16 people were injured and two wings of the Pyrgos hospital were seriously damaged and had to be evacuated. The population of Pyrgos town (capital of the Elia prefecture) was at the time around 22,000 spread over an area of approximately 4 km^2. In Pyrgos, 66% of the existing buildings were single-storey and 46.5% were old load-bearing masonry structures. More than half of the buildings in Pyrgos (52.2%) were of RC frames with unreinforced clay brick infill panels. Many of the load-bearing masonry buildings are quite old (20.6% of the existing building stock was built before 1946). In addition, at the time of the earthquake the town of Pyrgos was undergoing a construction boom and there were around 1,800

RC buildings constructed following the revision of the Greek earthquake code in 1984.

Although horizontal *PGA* of 0.45 g was recorded in the town center, damage was not as serious as in the 1986 Kalamata earthquake. Damage to RC buildings was generally slight to moderate with most of the buildings exhibiting non-structural damage to the hollow clay brick infill masonry walls. However, 22 RC buildings of 2-7 storeys had some structural damage, 9 of which were damaged more seriously but were repairable (a very small proportion of the RC buildings in the town). Further details for the RC building damage distributions are not available as the University of Patras damage survey focused on the effects to masonry structures where damage was more serious (Karantoni and Bouckovalas, 1997). The distribution of damage to masonry buildings was not uniform due to local soil conditions and the direction of the fault rupture (Bouckovalas *et al.*, 1996; Stavrakakis, 1996).

In total 1,023 load-bearing masonry buildings in the town center (approximately a quarter of those existing in Pyrgos) were included in the University of Patras damage survey.

Table 5 - Damage Distribution of URM Buildings by Type of Masonry and Number of Floors in the March 26, 1993 Earthquakes Near Pyrgos.

Masonry material	No. of storeys	Structural class code	No. of buildings	Percent of buildings by Damage Grades (EMS-98)				
				D1	D2	D3	D4	D5
Adobe	01-feb	LBAM-L	150	00.	10.	9.3	23.	40.
Simple stone	01-feb	LBSM-L	672	3.9 %	22. 0%	13. 5%	8.6 %	3.0 %
Clay bricks	01-feb	LBBM-L	110	0.0 %	9.1 %	10. 9%	9.1 %	6.4 %
Mixed masonry materials	01-feb	MIXM-L	72	5.6 %	16. 7%	20. 8%	30. 6%	6.9 %
Total			1,004	3.1	18.	13.	12.	9.2

All the masonry buildings in the defined study zone were assessed. Analysis showed that 43% were not damaged, while 22% suffered heavy damage. The type of load-bearing masonry proved to be the main factor influencing the relative performance of the masonry buildings, as it was found that the load-bearing adobe masonry (LBAM) suffered more than stone masonry (LBSM) and brick masonry (LBBM). In addition, there were 72 buildings with mixed load-bearing masonry materials such as any combinations of adobe, stone, brick and concrete blocks. Table 5 shows the damage distribution by type of masonry building (19 three-storeyed masonry buildings are not included in the table as the sample was deemed too small for definite conclusions).

4.5.8 Damage Data of the June 15, 1995 Aegion Earthquake

On June 15, 1995 at 03:15 a.m. local time a strong M_w=6.4 earthquake with focal depth of 14 km struck the town of Aegion in northern Peloponnese and the surrounding villages (Lekidis *et al.,* 1999). The earthquake was centred approximately 18 km NE of the town in the Gulf of Corinth. A strong aftershock (M_w=5.6) followed 15 minutes later causing further damage including the partial or total collapse of a few buildings. As a result, sections of two RC buildings (a 4-storey coastal hotel and a 5-storey apartment block in the town center) collapsed causing the loss of 26 lives. In addition, 4 other RC buildings collapsed outside the town limits of Aegion but did not cause loss of life (these were an industrial building and three low-rise houses with soft-storey at ground level). The population of Aegion in 1995 was approximately 23,000 people.

In 1995 in Aegion town, 53% of the existing buildings were single-storey and just 6.3% had three to seven floors, while 41.6% of the buildings were load-bearing masonry structures with the proportion of adobe buildings being quite significant (26.2%). More than half of the buildings in Aegion (57.7%) was from RC frames with unreinforced clay brick infill panels. Many of the load-bearing masonry buildings were quite old (26.6% of the existing building stock was built before 1946). In addition, there were around 800 RC buildings that were constructed following the 1984 Greek earthquake code revision making this one the first events to test the performance of these buildings under significantly strong ground motion, as the event in Pyrgos town two years earlier was less severe (see also section 4.5.10 for the characteristics of the strong ground motion recorded in each of the events).

The one strong motion instrument in operation at the time of the main shock in the center of Aegion (in the Telecom building) recorded horizontal *PGA* of 0.54 g (the highest ever recorded in Greece) and was situated in the immediate vicinity of the collapsed apartment building (Bouckovalas *et al.*, 1999; Lekidis *et al.*, 1999). This recording was also characterized by long- period pulses which resulted in high horizontal velocity (51.8 cm/s). There is evidence in the literature that such long-period pulses are related to directivity phenomena (Bouckovalas *et al.*, 1999). The northern side of Aegion city is essentially bounded by a normal fault running in E-W direction, parallel to the coast. This fault produces a cliff with an almost vertical drop of about 90 m.

Table 6 - Damage Distribution for Each Building Class of RC Buildings in Aegion After the June 15, 1995 Earthquake.

Construction period	No. of storeys	Type of ground storey	Structural class code	No. of buildings	Percent of buildings by damage grade			
					Damage Grades (EPPO+)			
					D 0-1	D2	D3	D4
Prior to 1959	01-mar	soft-storey	RC1-LP	13	76.9%	15.4%	7.7%	0.0%
		regular	RC1-L	89	61.8%	30.3%	7.9%	0.0%
	04-lug	soft-storey	RC1-MP	0	-	-	-	-
		regular	RC1-M	4	25.0%	50.0%	25.0%	0.0%
1959-1984	01-mar	soft-storey	RC2-LP	164	82.9%	15.2%	1.8%	0.0%
		regular	RC2-L	537	78.6%	19.9%	1.5%	0.0%
	04-lug	soft-storey	RC2-MP	47	55.3%	42.6%	2.1%	0.0%
		regular	RC2-M	95	57.9%	34.7%	6.3%	1.1%
1985-1995	01-mar	soft-storey	RC3-LP	42	97.6%	2.4%	0.0%	0.0%
		regular	RC3-L	70	95.7%	4.3%	0.0%	0.0%
	04-lug	soft-storey	RC3-MP	50	84.0%	16.0%	0.0%	0.0%
		regular	RC3-M	36	94.4%	5.6%	0.0%	0.0%
	8+	soft-storey	RC3-HP	1	100.0%	0.0%	0.0%	0.0%
		regular	RC3-H	1	0.0%	100.0%	0.0%	0.0%
Total				1,14	77.5	20.1	2.3	0.

The residential part of Aegion lies almost entirely on the up-throw region of the fault, while the harbor is built on the down-throw region. Like in the harbor area of Kalamata, damage in the harbor area of Aegion was limited as a result of dampened ground motions in the soft and deep clayey deposits. The absence of damage in the waterfront area of the town becomes more impressive when it is noted that in this area the buildings are very old (some already ruined) and without any seismic resistance provisions (Athanasopoulos *et al*., 1998).

The University of Patras damage survey (Fardis *et al.*, 1997; Karantoni and Fardis, 2004; Karantoni and Fardis, 2005) was carried-out in the Aegion town center (included the harbor area) and assessed the damage to all the buildings within the chosen study zone (it contains 2,108 buildings i.e., around 26% of the town's existing building stock at the time of the earthquake). There were 1,149 RC and 859 masonry and mixed structure buildings in the study zone. Most of the damage occurred in the area of the historic town center in a zone of about 0.2 km^2 (Fardis *et al.*, 1997; Lekkas, 2002). Within the study zone one single RC building collapsed causing the loss of 16 lives (Lekkas *et al.*, 1997; Papazachos and Papazachou, 2003; Karantoni and Fardis, 2004). Analysis of the damage to RC buildings shows that the ratios of "yellow", "red" and "black" were 20.1%, 2.3% and 0.1% respectively. In particular, 28 of the 1,149 inspected RC buildings suffered heavy and in some cases irreparable damage and had to be evacuated. In Table 6 the damage distribution of the RC buildings is presented, where the type of ground floor, the number of floors and the period of construction are the key variables. We note that it is not possible to differentiate

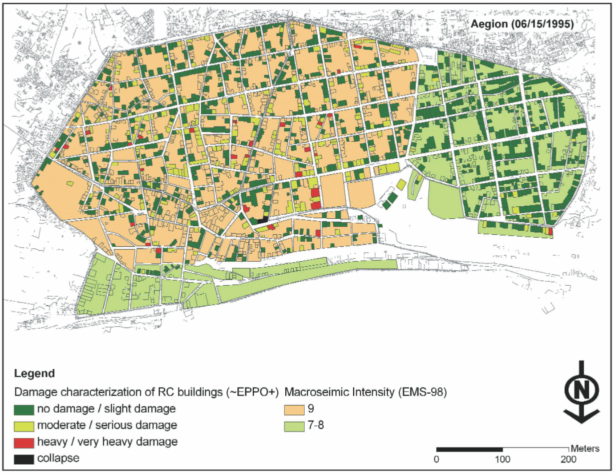


Fig. 6.9 - Damage Distribution of RC Buildings in the 1995 Aegion Earthquake, According to the EPPO+ Damage Scale Proposed in This Study (Original Data Source: Fardis *et al.*, 1997). In the present study the damage survey area has been separated into two zones of greatly different damage severity, seen in beige and green colour respectively.

undamaged from lightly damaged buildings. In total, the RC buildings are split into 14 classes (combinations of age, height and ground floor type), however when the sample of buildings in a class is smaller than 20 this class was not considered in the vulnerability analysis. The spatial distribution of the damage is shown in Fig. 6.9

where it is noted that most of the “yellow” and “red” buildings are in the central and eastern part of the study zone (area with beige background colour). It is also noted that there were very few “yellow” (and just one “red”) buildings in the western part of the study zone.

Comparing the damage distributions of RC buildings in Kalamata and Aegion, we note that in Kalamata there were a lot more buildings in the “yellow” damage grade (52.4% vs. 22.8%) possibly due to the added effect of the strong aftershock. Unlike in Kalamata, in Aegion the existence of soft-storey did not play significant role in the performance of the pre-1985 buildings. In Aegion the buildings built after 1984 revision suffered less damage with none suffering damage degree ≥D3.

The damage inspection of load-bearing masonry buildings in the Aegion study zone was based on the assessment of the damage to the external walls of the buildings. The damage degrees were assigned according to the 5-level damage scale of the University of Patras (also used in the 1993 Pyrgos survey) which differs only slightly from the EMS-98 damage scale (see Table 3). The analysis of the data showed that 38.1% suffered serious and heavy damage (≥D3). By comparison in Kalamata 64.3% suffered serious and heavy damage (≥D3). In Table 7 the damage distribution of the load-bearing masonry buildings in the Aegion study zone is presented where the type of masonry material and number of floors are the key variables.

Table 7 - Damage Distribution of Masonry Buildings by Type of Masonry and Number of Floors in Aegion, After the June15, 1995 Earthquake.

Masonry material	No. of storeys	Structural class code	No. of buildings	Percent of buildings by damage grade				
				Damage Grades (EMS-98)				
				D1	D2	D3	D4	D5
Si"mple	01-feb	LBSM	253	5.1	24.5%	11.1%	9.5	8.3
Adobe	01-feb	LBAM-L	426	2.3	14.6%	16.0%	17.1%	13.8%
Mixed masonry	01-feb	MIXM-L	52	3.8 %	25.0%	13.5%	13.5%	5.8 %
Mixed (stone masonry & RC fr.)	01-feb	MIXS-L	47	2.1 %	4.3 %	2.1 %	31.9%	0.0 %
Brick	01-feb	LBBM-L	69	2.9	7.2	14.5%	4.3	2.9
Concrete block	01-feb	LBCB-L	8	0.0 %	0.0 %	12.5%	0.0 %	0.0 %
Total			855	3.3	17.0%	13.5%	12.8%	11.8%

4.5.9 Damage Data of the August 14, 2003 Lefkada Island Earthquake

On August 14, 2003 at 08:15 a.m. local time an earthquake of magnitude M_w=6.3 (M_s=6.4) with focal depth of 9 km, occurred near Lefkada Island in the Ionian Sea (Papadimitriou *et al.*, 2006). Lefkada is third in size among the Ionian Islands with a land area of 302.5 km^2. According to the 2001 population census, there are 22,506 permanent residents on the island. The capital of the island is the town of Lefkada on the northern tip of the island. The earthquake's epicentre was 10 km west of the Lefkada town. As a result of the earthquake at least 50 people were injured, while in Lefkada town one 3-storey RC building on pilotis with a timber frame attic extension, gradually constructed in the 1959-1984 period, partially collapsed. Extensive landslides and rock falls took place on the western part of the island seriously injuring some people. The highest damage rates occurred in Lefkada town and villages on the western part of the island, but generally damage was not extensive despite the magnitude and proximity of the earthquake. Lefkada Island belongs to the highest zone of the Greek earthquake code and its buildings (like those of Cephalonia, Zakynthos and Ithaca islands) are most likely to be stronger and less vulnerable.

Lefkada town contains 23% of the island's buildings and its historic centre is founded on soft- loose ground conditions, while the neighbourhoods of Bei and Neapoli are founded on better ground and contain the newer buildings (ITSAK, 2004). In Lefkada town, 57% of the buildings are RC, 6% stone masonry, 9% mixed RC and masonry structures and 28% are either wooden or buildings of LBSM with timber frame elements (Karababa, 2007).

Table 8 - Damage Distribution of Low-Rise RC Buildings (regular ground storey, 1-3 storeys) in the 2003 Lefkada Island Earthquake.

Construction Period	No. of storeys	Structural class code	No. of buildings	Percent of buildings by damage grade			
				Damage Grades (EPPO+)			
				D1	D2	D3	D4
Pre-1959	01-mar	RC1-L	350	12. 6%	7. 7%	0. 0%	0. 0%
1959-1984	01-mar	RC2-L	3,0 79	16. 6%	4. 6%	0. 1%	0. 0%
1985-1994	01-mar	RC3-L	1,4 96	22. 3%	3. 4%	0. 0%	0. 0%
1995-2003	01-mar	RC4-L	1,7 62	9 .1%	0. 7%	0. 0%	0. 0%
Total			6,6	15.	3.	0.	0.

Karababa (2007) collected all the damage data related to this earthquake from the local Earthquake Rehabilitation Office (included the first and second degree building damage inspection forms) and created a damage database containing 4,211 inspected buildings (which is around 26% of the island's total building stock). In Greece, building assessments are undertaken once the building owner has filed an application requesting an inspection. Evidently, this is dependent on the owner's judgement and consequently, the assessed buildings are in general likely to be found damaged to some degree. The assumption was made that buildings not inspected were undamaged. Although it is acknowledged that other reasons may have led to their exclusion from the inspection process, given that building owners are responsible for the integrity of their building, under Greek law, this assumption is likely to be largely valid. Based on this assumption, the number of undamaged buildings was calculated by subtracting the number of the damaged buildings within each building type from the total number of buildings for the respective type as this was determined through the census data of 2001 (Karababa, 2007).

It is seen that the performance of low-rise RC buildings with regular ground floor in Lefkada Island as a whole was related to the period of construction as the newer buildings suffered much less damage.

RC buildings on Lefkada Island are up to 4-storeyed and in 2003 formed about 43% of the building stock, 5% of which have been built before the 1959 earthquake code and 46% built in the period 1959-1984. There are very few RC buildings with soft-storey (around 0.5% of the RC stock) as this practice is avoided in this seismically active region and very few 4-storeyed structures. From the total of 6,687 RC buildings on the island 3.5% (233 buildings) were in the "yellow" class and just 0.1% in the "red" class (7 buildings).

Fig. 6.10 shows the distribution of the damage by municipal department and by EMS-98 intensity. Table 8 shows the damage distribution of low-rise RC buildings (1 to 3 storeys) without soft-storey by period of construction.

This is the first time in Greece where the full suite of RC structures (when sub-divided by their period of construction, i.e., the earthquake code in use at the time of construction) has been tested and for which detailed and reliable damage data sets are available (though only for low-rise structures without soft-storeys, situated in the highest zone of the Greek earthquake code).

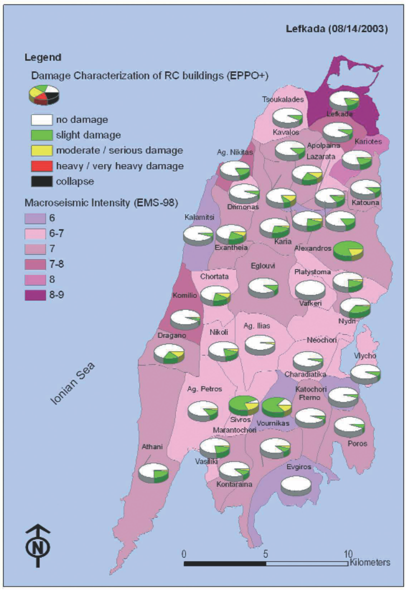


Fig. 6.10 - RC damage Distribution by Municipal Department and Municipality-Level EMS-98 Intensity in Lefkada Island

It is also seen that, damage was much less than in Kalamata and Aegion as these buildings have been designed with higher demands because Lefkada Island is situated in Greece's highest earthquake code zone.

On Lefkada Island there are 4 types of vernacular buildings (ITSAK, 2004; Karababa, 2007):

- 1-2 storey LBSM (31% of the total building stock, assigned to LBSM class);
- 1-2 storey load-bearing stone or brick masonry confined in vertical and (or) horizontal RC structural elements which are not designed to perform as moment-resisting frames (14%, assigned to the mixed structure class);
- 1-3 storey timber frame structures (4%) made initially with locally supplied oak, cypress, pine, fir, olive elements and in recent decades from imported timber;
- 1-3 storey mixed LBSM with timber frame (10%).

In Table 9 we show the damage distributions of the first two classes that their vulnerability is examined in this paper.

4.5.10 Summary of the Recorded Strong Motions During the Four Events of the Damage Database

Table 10 summarises the commonly used ground motion parameters of the five strong motion recordings registered within the damage survey zones (peak acceleration, velocity, displacement and bracket duration). These recordings are discussed in detail by Anagnostopoulos *et al*. (1987),

Table 9 - Damage Distribution of LBSM and Load-Bearing Masonry Mixed with RC Frame (with 1-2 storeys and regular ground storey) in the 2003 Earthquake in Lefkada Island.

	No. of storeys	Structural class code	No. of buildings	Percent of buildings by damage Damage Grades (EMS-98)				
				D1	D2	D3	D4	D5
Simple	01-feb	LBSM-L	4,819	8.7	10.	10.	3.9	0.0
Mixed (masonry & RC frame)	01-feb	MIXS-L	1,946	6.7 %	9.6 %	5.3 %	0.2 %	0.0 %
Total			6,765	8.1	9.9	8.9	2.8	0.0

Stavrakakis (1996), Lekidis *et al.* (1999) and ITSAK (2004). It is must be noted that these recordings are representative of the ground motion near the recording stations while the damage surveys took place over wider areas with varied ground conditions.

In Fig. 6.11, we see the acceleration response spectra (transversal component) for each of the four main shock recordings. For low and medium-rise RC buildings examined in this study the fundamental periods are expected to be in the range of 0.1-0.6 s. It is seen that ductility demand on these buildings was quite strong in all four events at this period range. The demand is highest in the case of the record in Lefkada town, but also Aegion town. This ductility demand is much higher than the design code coefficients used by Greek engineers which after allowance for safety factor (1.75), increase in allowable stresses for seismic design (20%) and a multi-degree of freedom effect (0.85) result in the case of the pre-1995 buildings (built on average ground conditions) at base shear coefficients equal to 0.12 for Kalamata and Aegion (Lekidis *et al.,* 1999), 0.18 for settlements on Lefkada Island and 0.23 for Lefkada town which lies mostly on unconsolidated ground deposits (ITSAK, 2004).

4.5.11 Assessment of EMS-98 Intensity in the Damage Survey Areas

The quantification and prediction of damage due to seismic actions to various structural types is an important problem. A growing number of theoretical and experimental investigations as well as field observations after damaging earthquakes, indicate that the *PGA* does not correlate well with the observed structural damage as it does not contain information on the duration or energy content of the ground motion across the frequency range that relates to building structures and influence the damage potential of ground motion (Pomonis *et al.,* 1992; Koliopoulos *et al.*, 1998). Moreover, it is often the case that observed damage data are not the result of one single event or recorded ground motion, but the cumulative effect of a main shock, strong aftershocks and (or) foreshocks that may follow or precede, while in some cases damage from previous earthquakes may also play a role (particularly in regions of high seismicity and on older buildings).

Seismic intensity scales provide an alternative to this problem as they categorize the strength of the ground motion through the careful study of the macroseismic effects of an earthquake in a place. They provide, therefore, a first insight into the strength or damage potential of the experienced ground motion and as a result they continue to have extensive use. Blong (2003) makes a detailed analysis of the various seismic intensity scales that are in use at present (such as the Modified Mercalli, the MSK, the MCS, the JMA and the EMS-98 scales) and suggests that an intensity scale needs to be dynamic and adaptable to the ever changing conditions, since the ageing of buildings and the development of new earthquake code regulations constantly change the vulnerability of existing building stocks. Such an example is the European Macroseismic Scale (EMS-98, Grünthal, 1998) which was adopted after a 10-year trial and consultation period by the European Seismological Commission and replaces the previously used MSK intensity scale. The EMS-98 intensity scale more than any of the previous scales gives emphasis to the performance of existing buildings to accurately assess the intensity and incorporates new types of buildings, especially those including earthquake-resistant design features. Building structures of various types (stone masonry, reinforced concrete frames, etc.) are broadly classified into 6 classes (A to F) according to their vulnerability to ground shaking (the level of earthquake design is also taken into account). In addition, clear definitions are given for the various levels of damage (damage grades) for masonry and RC structures respectively [five damage grades – from DG1 for negligible to slight damage to DG5 for destruction (very heavy structural damage)]. In the range of intensities that by definition are capable of causing damage to buildings (V and above) the likely ranges in the proportions of the buildings in each vulnerability class to suffer a certain damage grade are clearly defined. Most importantly, it is probabilistic in its approach to damage; as for any type (strength) of building at a particular level of intensity, damage can be considered as a distribution of damage grades (Musson, 2000). Clear guidelines and explanation are included in the official EMS-98 intensity scale document (Grünthal, 1998), including photos of damage which clearly show the types of damage for each damage grade. These improvements made the EMS-98 scale more robust by reducing the uncertainties associated with intensity assignments using previous macroseismic scales and is thus considered adequate for use in seismic risk assessment (Musson, 2000). An example of EMS-98 intensity estimation is discussed hereafter related to the Aegion data set (Fig. 6.9). Following the guidelines of EMS-98 the typologies described in Tables 6 and 7 were assigned to vulnerability classes.

Table 10 - Peak Ground Motion Parameters and Strong Motion Duration as Recorded During the Main Shocks (and main aftershock in the case of the 1986 Kalamata Earthquake) in the Four Damage Survey Zones (obtained from ITSAK, 2003).

Location (Date)	Peak Horizontal Ground Acceleration, *PHGA* (g)	Peak Horizontal Ground Velocity, *PHGV* (cm/s)	Peak Horizontal Ground Displacement, *PHGD* (cm)	Bracket Duration, *BD* (ag>0.1g) (s)
Kalamata city centre - Prefecture building - main shock (13.9.1986)	0.27	30.4	5.4	2.3
Kalamata Prefecture Building - main after- shock (15.9.1986)	0.23	22.8	3.3	0.7
Pyrgos city centre (26.3.1993)	0.45	20.8	2.3	1.3
Aegion city centre - Telecom building (15.6.1995)	0.54	51.8	6.2	2.1
Lefkada Town (14.8.2003)	0.42	31.7	4.6	10.7

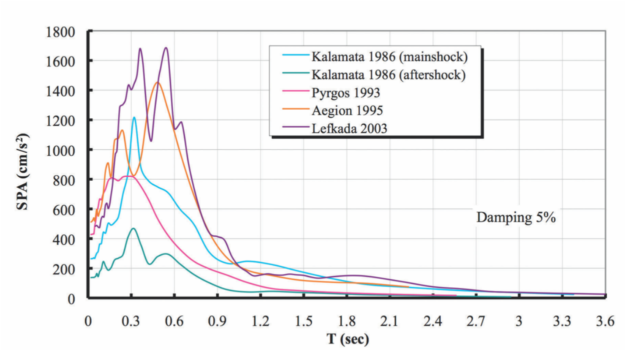


Fig. 6.11 - Response Spectra (transversal component) of Examined Earthquakes (source: ITSAK, 2003).

RC buildings constructed before 1959 (RC1) were assigned to class C, those constructed with the 1959 code (RC2) to class D and those constructed after 1984 (RC3) to class E. With regard to masonry buildings, adobe and stone masonry (LBAM, LBSM) were assigned to class A. For the case of RC buildings due, to the nature of the Greek post- earthquake building usability assessment data, it was also necessary to correlate the 4-grade EPPO+ damage scale with the 5-grade EMS-98 damage scale (further discussion on this is included in section 5.3 of this paper). Having defined the damage distribution according to the EMS-98 damage scale for each vulnerability class, it was possible to assign an intensity degree for each class and then by taking into account the total number of buildings a weighted mean intensity was estimated. For the case of the 1995 Aegion data set, we estimated intensity VIII-IX for the whole survey zone, IX for the eastern and central part and VII-VIII for the western part (shown in Table. 11). We note that within the small zone of the Aegion damage survey area (less than 0.5 km^2), depending on structural class quite different intensities were obtained (in this case from intensity VII up to X). Similarly, Tertulliani *et al.* (2011), following the same standard procedure, assigned EMS-98 intensity to downtown L'Aquila (a zone of 2 km^2) affected by the April 6, 2009 earthquake and found that the intensity was VIII or IX.

The EMS-98 intensities estimated for each event's data set are shown in Table 11 (second column from the left) while those assigned by other authors are also shown. General agreement can be observed, despite different methods used in the derivation of intensity by the other authors [in Kalamata (Leventakis *et al.,* 1992) assessed Modified Mercalli intensity distribution throughout the city by combining the results of field and a questionnaire surveys; Lekkas (1996) and Lekkas *et al.* (2000) used geological observations to derive the EMS-92 intensity in Pyrgos during the 1993 earthquake; Stavrakakis (1996) generated a synthetic isoseismal for the Pyrgos 1993 earthquake; Papathanassiou and Pavlides (2005) tested the INQUA scale on the island of Lefkada; Karababa and

Pomonis (2011) used the parameterless scale of intensity (PSI) proposed by Spence *et al.* (1992), on the island of Lefkada and then derived EMS-98 values based on a correlation between PSI and MSK scales].

Table 11 - Estimated Macroseismic Intensity (I_{EMS}) for each Event.

Seismic event	EMS-98 Intensity assigned by this study	Assigned Intensity by Other Authors	References
Kalamata (13/09/1986)	9 (for the city as a whole)	VI to IX	Leventakis *et al.* (1992)
		IX	Papazachos and Papazachou (2003)
		IX	Pomonis *et al.* (2009)
Pyrgos (26/03/1993)	7 (based on damage to masonry buildings)	VI+	Stavrakakis (1996)
		VIII	Lekkas (1996), Lekkas *et al.* (2000)
		VIII	Penelis *et al.* (2002)
Aegion (15/06/1995)	08-set (for the whole study zone)	VIII	Papazachos and Papazachou (2003)
		VIII	Penelis *et al.* (2002)
		VIII	Pomonis *et al.* (2009)
Lefkada Island (14/08/2003)	6 to 8 (8-9 in Lefkada town)	V to VIII	Papadopoulos *et al.* (2003)
		V to VIII	Papathanassiou and Pavlides (2005)
		VI to VIII	Karababa and Pomonis (2011)

It has been shown that peak horizontal ground velocity (*PHGV*) is better correlated to macroseismic intensity than *PGA* in locations where the intensity is greater or equal to VII (Wald *et al.,* 1999), as is the case in all four damage survey zones in this study. Using the proposed *PHGV* to Modified Mercalli intensity conversion equation for Greece by Koliopoulos *et al.* (1998) and the recorded *PHGV* values shown in Table 10, we note that the correlation between the EMS-98 intensities assessed in this study (assumed to be equivalent to the Modified Mercalli intensity) and those derived by the conversion equation is very good for the case of Pyrgos (I_K=7.46), Aegion (I_K=8.54) and Lefkada town (I_K=7.96), but not in the case of Kalamata
(I_K=7.91). In Kalamata we estimated EMS-98 intensity IX for the city as a whole as we do not
have the damage data at neighbourhood level that would allow us to estimate intensity at a finer resolution. However, the recording station's location (in the Messina prefecture building) was in a zone that has been assessed by Leventakis *et al.* (1992) as experiencing intensity VIII which correlates very well with the value of intensity predicted by the aforementioned conversion equation.
In terms of the local site effects (topography or soil), EMS-98 indicates that "absolutely no attempt should be made to discard or reduce intensity assignments on the

grounds that they were influenced by soil conditions" and advises that "it is also desirable to assign values to locations which are reasonably homogeneous, especially with regard to soil types" and "in the case where a town has areas in which the geotechnical conditions are very different then different intensity values should be assessed for the two parts of the town independently" (Grünthal, 1998). Complying with these guidelines, we proposed different intensity values for the Aegion study zone and across Lefkada Island including Lefkada town whose historic center is founded on alluvial sediments of loose sand and soft marine clays. Unfortunately, it was not possible to do this in Kalamata, where local site conditions are considered to have played a role in the significant variations in damage severity observed across the town (Anagnostopoulos *et al.,* 1987; Theodoulidis *et al.,* 2008) as the available data are aggregates for the whole town. In Kalamata neighbourhood-level damage data do exist for a subset of 7,101 out of the 10,171 buildings for 26 neighbourhoods but neighbourhood boundaries are approximate and do not correlate well with differing soil conditions across the town (Pomonis *et al.,* 2009).

4.5.12 Vulnerability Analysis

Observed damage-based seismic fragility curves can be described by (cumulative) normal, lognormal, beta, binomial, or other distributions, provided that sufficient data sets are available for constructing them. The ground motion severity can be described either by intensity scales (which are discrete values) or by instrumental ground motion parameters such as *PGA* or spectral displacement (which are continuous variables) or by the use of the parameterless scale of intensity (PSI) proposed by Spence *et al.* (1992). However because instrumental data in areas where detailed and reliable damage data sets are available are usually not sufficient, analytical methods are often used to derive the fragility curves (Kircher *et al.*, 1997). Hybrid methods combining both observed damage data sets and analytical methods have also been proposed (e.g., Kappos *et al.*, 1998, 2006).

In the present study the severity of the ground motion is described in terms of the EMS-98 macroseismic intensity scale. Assuming that the macroseismic intensity is a continuous variable, cumulative normal distribution curves have been fitted to the damage distribution data sets at each intensity degree for each type of structure. The normal (Gaussian) distribution in its cumulative form has been used in previous seismic vulnerability studies (e.g., Spence *et al.,* 1992; Orsini, 1999; Karababa and Pomonis, 2011). The main hypothesis for the distribution model of the damage grades is that for a generic structure belonging to a specific structural vulnerability class, the intensity at which the structure overcomes a determined threshold of damage is distributed according to a Gaussian model. Though belonging to the same structural vulnerability class, the behaviour of the structures in a class is not identical of course; the results are scattered around the mean and are normally distributed (Orsini, 1999). There is though a fundamental problem with the normal distribution in that it gives positive values of damage probability even for zero values of intensity, however as stated before we consider the present analysis as being valid only for the EMS-98 intensity range of VI to IX.

4.5.12.1 Structural Types Used in the Vulnerability Analysis

The combination of damage data sets from earthquakes that occurred in different parts of Greece and at different times showed that different methods have been used during the post-earthquake damage assessments and that the damage grades recorded in each case are not identical. This is neither unusual nor unexpected considering that the damage surveys took place across a time span of 17 years. The data set from the 1986 Kalamata earthquake is such a case, where the otherwise valuable information (more than 10,000 buildings subjected to varying degrees of seismic intensity in the range of V to X) are limited in terms of the vulnerability parameters captured (e.g., period of construction, type of load-bearing masonry, typologies of the mixed structures).

An additional problem with the Greek data sets is that in the “green” damage grade it is often not clear if undamaged buildings are included or not. For example in the Kalamata (1986) and Aegion (1995) damage data sets for RC buildings it is stated that the “green” class includes the buildings that were not damaged. Therefore, in the vulnerability analysis for the D1 (“green- tag”) damage grade the information from these two data sets was not taken into account. Furthermore, for the derivation of reasonable conclusions we selected structural classes that have been subjected to three or more distinct levels of intensity if the buildings’ sample was greater than 20. As a result from the total of 28,747 buildings in the database, only approximately 62.5% took part in the analysis. With this selection process it was possible to assess the vulnerability of ten structural classes. The list of these 10 classes including codif ication for ease of reference, vertical load-bearing structural system, the period of construction, number of storeys, type of ground floor and the number of buildings of each class used in the analysis are shown in Table 12.

Furthermore, these ten structural classes were sought out in the last building census of Greece (EL. STAT., 2000) in order to estimate the percentage of the Greek building stock that belongs to these classes. According to the available data sets, nine out of ten building classes can be assigned to the building classes in the census (the mixed structures are not clearly specified in the census). It was found that the 9 classes (excluded the mixed structures) cover approximately 66.7% of the building stock in Zone II of the 2004 Greek earthquake code, where approximately 54.5% of the country's building stock is situated. URM buildings (LBSM-L, LBAM-L) are considered in this analysis to have similar vulnerability regardless of earthquake code zone, as construction of load- bearing masonry buildings in Greece has been quite limited from 1960 onwards. In general though, a more thorough analysis would need to consider possible differences in masonry construction across time (Karantoni and Bouckovalas, 1997) and in different parts of the country, but the available data sets did not permit us to consider these factors in this analysis (e.g., lack of information about the period of construction in the data sets of the first 3 earthquakes in this analysis). We thus estimate that including the mixed structures (which could account for approximately 5 to 10% of the existing building stock) the analysed classes cover approximately

50% of the country's existing building stock (which amounted to 4.35 million buildings at the end of 2009).

In Fig. 6.12 we see the total number of buildings in each earthquake survey by structural type (left) as well as the number of buildings that was used in the vulnerability analysis (right). Unfortunately, only a small part of the Kalamata damage data set could be used because of the aforementioned limitations.

4.5.12.2 Derivation of Fragility Curves for Greek Buildings

In the cumulative normal distribution, the probability that under a given macroseismic intensity (I) a building suffers damage as described by damage grade D_i or greater, is given. Therefore, each fragility curve depends on only two parameters, the mean and standard deviation, which are derived by fitting the curve to the cumulative damage distribution data corresponding to each damage grade by minimizing the fit errors. The results of this analysis are shown in Tables 13 to 15 for low-rise load-bearing masonry and mixed structures (1 to 2 floors), low-rise RC (1 to 3 floors), and mid-rise RC (4 to 7 floors) structures, respectively. In addition to mean (μ) and standard deviation (σ) of the normal distribution the coefficients of variation (σ/μ) and correlation (R) are also shown. We note that there is no curve fit for damage grades $\geq$D3 ("red") and D4 ("black") for some RC classes as no buildings reached these damage levels in any of the surveyed areas. The standard deviation has been kept constant across the damage grade curves of each structural typology but was increased from 1.15 to 2.75 (lower σ was assigned to the URM buildings and higher σ to the RC buildings). In this way we obtained flattened curves for the newer RC buildings that better fitted the data scatter. Fig. 6.13 shows the fragility curves and data scatter for the LBSM-L and RC2-L structural classes.

Table 12 - Final List of Structural Classes for which Observed Damage-Based Vulnerability Analysis was Possible and the Population of Buildings in Each Structural Class.

Structural	Type of vertical load-bearing structure	Construction period	No. of storeys	Type of ground floor	No. of buildings
RC1-L		Prior to 1959			368
RC2-L		1959-1984	01-mar	regular	4,514
RC2-LP		1959-1984		soft-storey	730
RC2-M		1959- 1984		regular	404
RC2-MP	Reinforced Concrete Frames with Infill Masonry Walls	1959-1984	04-lug	soft-storey	439
RC3-L		1985-1995	01-mar	regular	1,499
RC4-L		1995-2003			1,762
LBSM-L	Stone Masonry	Mostly prior to 1960		Regular or shops at ground level	5,727
LBAM-L	Adobe	Prior to 1960	01-feb		576
MIXS-L	Mixed (masonry & frame)	-			1,909

It must be pointed out that in this methodology a circularity of reasoning is taking place, as the EMS-98 intensity for each location has been derived from the observed damage distributions and the fitted fragility curves have intensity as the common denominator. However, we believe this approach is still valid as we derive relative curves for 10 different types of structures, including some typologies that are quite important in Greece but not described specifically in EMS-98 (e.g., RC frames with infill walls built in 4 different time periods according to height (low and medium-rise) and according to the existence or not of soft-storey at ground level) whilst also quantifying the uncertainties.

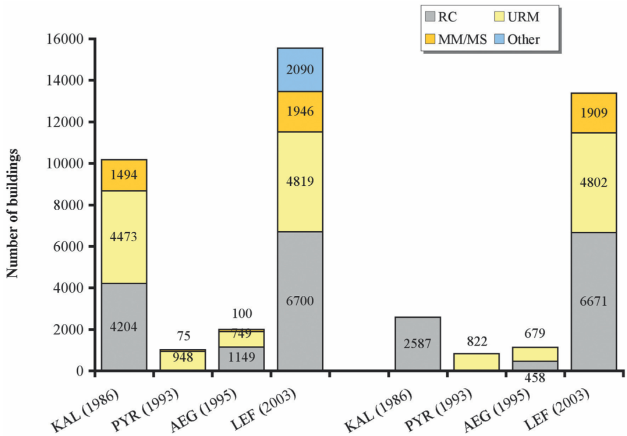


Fig. 6.12 - Left: Number of Buildings by Type of Load-Bearing System and Event (RC: reinforced concrete, URM: unreinforced masonry, MM/MS: mixed masonry/mixed structure). Right: Number of Buildings from each Event Used in the Vulnerability Analysis.

In addition, for RC buildings built after 1959 (RC2, RC3, and RC4) and in order to use the Lefkada Island data set, it was necessary to assume that under the same seismotectonic- geological-local site conditions the RC buildings of Lefkada Island are expected to suffer less damage due to their higher resistance in consequence of the fact that in the island a higher base shear coefficient (ε) is used. According to the 1959 Greek earthquake code (and its 1984 revision), the island of Lefkada was in zone III where depending on the ground conditions the base shear coefficient was $\varepsilon = 0.08 \div 0.16$, whereas the buildings in Kalamata, Aegion (and Pyrgos) were in zone II with base shear coefficient $\varepsilon = 0.06 \div 0.12$. A similar difference takes place in the 1995 Greek earthquake code (which has four zones, with Lefkada Island being in zone IV) and its 2004 modification (which reverts to three zones, with Lefkada being in zone III). This difference in base shear coefficient and resistance was assumed to be equivalent to half-degree in the EMS-98 intensity scale (i.e., all other things being equal it is assumed that RC buildings in zone II will suffer the same levels of damage as their counterparts in zone III at an intensity that is lower by half-degree).

Table 13 - Parameters of Cumulative Normal Distribution for Low-Rise (1 to 2 floors) with Regular Ground Floor LBSM and Mixed Masonry with RC Building Classes in Greece.

Structural Class Code	Parameter	Damage Grades (EMS-98)				
		≥D1	≥D2	≥D3	≥D4	D5
LBAM-L	ΣEMS	1.15	1.15	1.15	1.15	1.15
	MEMS	7.53	7.59	8.25	8.95	9.86
	σ/μ	0.153	0.152	0.139	0.128	117
	R	0.265	0.239	0.217	0.032	-0.200
	No. of data sets	4	4	4	4	4
LBSM-L	ΣEMS	1.25	1.25	1.25	1.25	1.25
	MEMS	7.53	7.89	8.95	9.67	10.62
	σ/μ	0.166	0.158	0.140	0.129	0.118
	R	0.794	0.850	0.760	0.894	0.658
	No. of data sets	10	10	10	10	10
MIXS-L	ΣEMS	1.75	1.75	1.75	1.75	-
	MEMS	8.66	9.37	10.91	12.80	-
	σ/μ	0.202	0.187	0.160	0.137	-
	R	0.179	0.032	-0.130	0.978	-
	No. of data sets	5	5	5	5	5

Papaioannou and Papazachos (2000) assessed the earthquake hazard of 144 broad sites across Greece (cities, towns and villages) in terms of the expected macroseismic intensity and *PGA* with a mean return period of 475 years. They reported that in zone IV (of the 1995 code) the average values for these two parameters are 8.2 and 0.37 g respectively, while in zone III they are 7.6 and 0.25 g respectively. The base shear coefficient adopted for soil type B (stiff) in the 1995 Greek earthquake code was 0.36 and 0.24 in zones IV and III respectively, i.e., almost identical to the values derived by Papaioannou and Papazachos (2000). The difference in the average macroseismic intensity with 475 years return period between the two zones is 0.6, i.e., almost the same to our proposed hypothesis.

4.5.12.3 Discussion and Comparisons with other Vulnerability Studies

The analysis described in the previous section led to the development of fragility curves for 10 types of structural classes commonly found in Greece. These curves are considered valid for buildings in zone II of the earthquake code in practice during the period 1959-1994, zone III of the earthquake code valid in the period 1995-2003 and for the EMS-98 intensity range VI to IX. The coefficient of correlation (*R*) seen in Tables 13 to 15, is that derived from linear regression between the observed and predicted values respectively and in some cases it is negative. The best fit was obtained for classes LBSM-L, RC2-L, RC2-LP, RC2-M and RC2-MP for most damage grades. The goodness of fit for the remaining 5 types of structural classes (LBAM-L, MIXS-L, RC1-L, RC3-L and RC4-L) is poor due to the scatter of the data and the limited number of data sets.

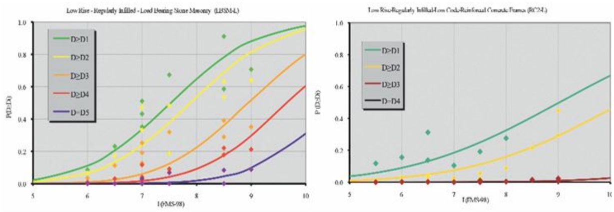


Fig. 6.13 - Observed Damage-Based Fragility Curves for Two Building Classes: 1-2 Storey Unreinforced Stone Masonry (left) and 1-3 Storey RC Frame with Infill Masonry Walls Without Soft-Storey, Built in Zone II of the Greek Earthquake Code Valid in the 1959-1984 Period (right).

The data scatter is also related to inconsistencies between the damage survey and data collection methodologies used in the 4 examined earthquakes as described in sections 4.5.6 to 4.5.9 , uncertainties in the accuracy of the damage assessments and their correlation with the hereby adopted damage grades for RC and masonry buildings (discussed in sections 4.5.5.1 and 4.5.5.2). For the case of LBAM-L the data from Pyrgos earthquake, suggesting 40% of the buildings had collapsed, need to be re-examined as we believe that actually most were severely damaged, not collapsed. For the classes MIXS-L, RC1-L, RC3-L and RC4-L we have limited data sets almost exclusively from the 2003 earthquake in Lefkada Island. Therefore, the proposed curves for the latter 5 structural classes should be viewed with caution and as preliminary.

We have compared the curves obtained for the 5 structural classes with good fit to fragility curves proposed by Rossetto and Elnashai (2003), Kappos *et al.* (2006) and Rota *et al.* (2008). The main obstacle in the comparison with these studies has to do with the descriptor of ground motion severity as *PGA*, spectral acceleration or spectral displacement is used. Converting *PGA* or spectral parameters to intensity adds to the uncertainty of the comparisons. On the other hand, the study of Lagomarsino and Giovinazzi (2006), although not based on observed damage data, offers itself for the most direct comparison because it is related to the EMS-98 intensity scale and contains sub-classes of masonry (by type of load-bearing wall material and type of diaphragm) and RC, by number of floors (using height bands identical to those in this study) and ductility class. Comparing macroseismic and mechanical models, Lagomarsino and Giovinazzi (2006) redefined the parameters of vulnerability and capacity curves after cross-validation and calibration providing the potential for advancing each other. Taking their proposed vulnerability and ductility indices (V and Q) for each of the examined structural classes, EMS-98 based fragility curves were obtained using the binomial distribution and were compared with their counterparts in this study. It must be noted though, that because both studies are based on the EMS-98 scale, a certain degree of similarity is expected despite the use of different smoothing functions and models. The results of this comparison are presented and discussed herein.

In Fig. 6.14 we see the comparison for the LBSM-L and RC2-L classes with: low rise (1-2 floors) simple stone with wooden diaphragms (class code: M3.w_L) and low-rise (1 to 3 floors) medium ductility class RC frames in zone II of the Italian earthquake code (class code: RC1 DCM-II_L) respectively.

Table 14 - Parameters of cumulative normal distribution for low-rise (1 to 3 floors) RC building classes in Greece according to the period of construction and the existence or not of a soft-storey.

Structural Class Code	Parameter	Damage Grades (EPPO+)			
		≥D1	≥D2	≥D3	≥D4
RC1-L (pre-1959)	σEMS	2.00	2.00	2.00	-
	μEMS	9.31	9.96	11.99	-
	σ/μ	0.215	0.201	0.167	-
	R	-0.470	0.600	0.847	-

	No. of data sets	5	8	8	8
RC2-L (1959-1984)	σEMS	2.25	2.25	2.25	-
	μEMS	9.02	10.24	14.37	-
	σ/μ	0.249	0.220	0.157	-
	R	0.448	0.915	0.793	-
	No. of data sets	7	11	11	11
RC2-LP (1959-1984)	σEMS	2.10	2.10	2.10	2.10
	μEMS	8.00	9.85	13.17	15.49
	σ/μ	0.263	0.213	0.159	0.136
	R	0.946	0.740	0.914	0.546
	No. of data sets	4	4	4	4
RC3-L (1985-1994)	σEMS	2.50	2.50	-	-
	μEMS	9.15	12.43	-	
	σ/μ	0.273	0.201	-	-
	R	0.000	0.424	-	-
	No. of data sets	7	10	10	10
RC4-L (post-1994)	σEMS	2.75	2.75	-	-
	μEMS	10.27	14.34	-	-
	σ/μ	0.268	0.192	-	-
	R	0.210	0.045	-	-
	No. of data sets	7	7	7	7

We consider these two structural classes in Italy quite similar to the examined Greek classes, as in Greece LBSM-L buildings have almost exclusively wooden diaphragms, while RC2-L buildings in Greece can be considered as having medium ductility as they have been designed according to the 1959 earthquake code. However, it is also noted that the Italian classes do not differentiate between RC buildings with or without soft-storey (pilotis). In addition, we compared this study's class RC2-LP with class RC1 DCM-II_L, as well as this study's classes RC2-M and RC2-MP with class RC1 DCM-II_M of the Italian study.

In the case of LBSM-L, we note that the damage scale used in this study is identical to the

EMS-98 damage scale also used in the Italian study, thus direct comparisons can be made for all five damage grades (D1 to D5). We note that in the present study somewhat higher vulnerability is estimated for the Greek stone masonry buildings for all damage grades except grade D1 (slight damage) where we estimated lower probabilities for the intensity range VI to IX.

Table 15 - Parameters of Cumulative Normal Distribution for Mid-Rise (4 to 7 floors) RC Building Classes Constructed in the Period 1959-1985 in Greece for Regular Structures and Structures with a Soft-Storey.

Structural Class Code	Parameter	(EPPO+) Damage Grades			
		≥D1	≥D2	≥D3	≥D4
RC2-M (1959-1984)	σEMS	2.10	2.10	2.10	
	μEMS	7.44	8.69	12.54	4.31
	σ/μ	0.282	0.242	0.167	0.147
	R	0.857	0.929	0.456	0.349
	No. of data sets	4	4	4	4
RC2 MP (1959-1984)	σEMS	2.15	2.15	2.15	2.15
	μEMS	7.20	8.25	12.50	14.05
	σ/μ	0.299	0.261	0.172	0.153
	R	0.742	0.656	-0.803	0.500
	No. of data sets	3	3	3	3

In the case of RC buildings, the EPPO+ damage scale consists of four instead of five damage grades (D1 to D4) and is based on the EPPO instructions (EPPO, 1997), as described in section 4.5.5.1. In addition, the EPPO descriptions for the three-colour tag scheme for RC buildings, do not match very well with the descriptions given for the five damage grades of the EMS-98 scale

(Grünthal, 1998). However, after careful examination of the two damage scales we consider that EPPO+ damage grades D1 ("green-tag"), D3 ("red-tag") and D4 ("collapsed") can be considered to be nearly equivalent to EMS-98 damage grades D1, D4 and D5 respectively, while EPPO+ damage grade D2 ("yellow-tag") can be considered to cover the range of damage grades D2 and D3 in EMS-98. Furthermore, it is necessary to point-out the following differences: a) in EPPO-D1 ("green-tag") there is the description "hairline cracks in horizontal RC structural members" that does not exist in EMS-98-D1 that assumes no structural damage at this level (in the following section we shall see that the cost of repair of "green-tag" buildings in Greece is not negligible and it usually ranges between 4% and 7% of a building's replacement value); b) in EMS-98-D4 there is the description "collapse of a single upper floor" which we would assign to EPPO+ damage grade D4 ("black"). These issues need to be kept in mind when interpreting the comparisons between the f indings of this study versus those in Lagomarsino and Giovinazzi (2006).

In the case of the RC2-L, RC2-LP vs. RC1 DCM-II_L comparison, we note that in the Italian study the mean value of intensity for damage grades D1 and D2 is I=8.47 and 9.87 respectively, i.e., between the values proposed in this study for RC buildings with and without soft-storey respectively. For damage grade D3 (“red-tag”) we estimate that 0.85% and 2.25% of the Greek buildings would suffer this level of damage at intensity IX (non-cumulative), as opposed to 0.48% (non-cumulative) in the Italian study (EMS-98 damage grade D4). For the probability of collapse (D4 in EPPO+) there have been no cases of collapse in any of the four examined events for the RC2-L class, while we estimate that 0.10% of the buildings with soft-storey would be expected to collapse at intensity IX.

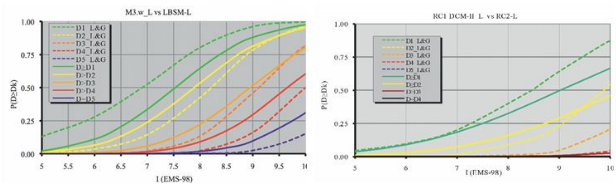


Fig. 6.14 – Comparison of the LBSM-L (left) and RC2-L (right) Fragility Curves Proposed in This Study with Those Proposed by Lagomarsino and Giovinazzi (2006) for Similar Structural Types.

The Italian study estimates that 0.02% of the buildings would collapse (EMS-98 damage grade D5) at intensity IX (although some cases of collapse would also be included in damage grade D4 of the EMS-98 scale, which contains buildings where one upper storey collapsed). We conclude that the findings of the two studies are similar for the case of low- rise RC frame buildings.

In the case of the RC2-M, RC2-MP vs. RC1 DCM-II_M comparison, we note that in the Italian study mean value of intensity for damage grades D1 and D2 is I=8.06 and 9.63 respectively, i.e., significantly higher (less vulnerable) than the Greek classes. For damage grade D3 ("red-tag"), we estimate that 4% and 4.25% of the Greek buildings would suffer this level of damage at intensity IX (non-cumulative), as opposed to 1% (non-cumulative) in the Italian study (EMS-98 damage D4). For the probability of collapse (D4 in EPPO+) we estimate that 0.57% and 0.94% of the Greek buildings would be expected to collapse at intensity IX, as opposed to 0.06% in the Italian study (although some more cases of collapse would also be included in damage grade D4 of the EMS-98 scale, which contains buildings where one upper storey collapsed). We conclude that the findings of this study suggest that mid-rise RC frame buildings constructed in the period 1959-1984, situated in zone II of the Greek earthquake code are more vulnerable than suggested by Lagomarsino and Giovinazzi (2006) for their Italian counterparts.

We have also compared our findings with the Modified Mercalli intensity-based DPMs proposed by Eleftheriadou and Karabinis (2008) developed on the basis of observed damage statistics at municipality level from the September 7, 1999 earthquake near Athens. For the masonry buildings we found very good correlation in the intensity range V to VIII even though it was possible to derive only four damage grades from the 1999 Athens building safety assessment data, while the data for intensity IX were not sufficient to draw definite conclusions. Also the Athens 1999 damage data for masonry buildings are unfortunately not differentiated by masonry material (they include adobe, simple stone, brick, mixed masonry, as well as monumental masonry buildings).

We also found quite good correlation with the proposed DPM for RC buildings built in the period 1959-1984 where for damage grades D2 (“yellow-tag”) and D3 (“red-tag”) we propose lower probabilities in the intensity range VII to IX for classes RC2-L and RC2-LP. This is considered reasonable as the areas affected by the 1999 earthquake near Athens were in that time period in zone I of the Greek earthquake code (the worst affected western suburbs of Athens were subsequently moved to zone II of the 2004 Greek earthquake code). We also note that 33.3% of the inspected buildings included in the DPM, was of mixed structure (RC frames mixed with URM) and that the data did not allow differentiation of the RC buildings by the number of floors and the existence or not of soft-storey. Also the number of undamaged buildings was estimated from the December 2000 Greek buildings census by subtracting the inspected buildings from the municipality-level totals. These shortcomings did not allow more definite comparative conclusions to be drawn against the findings of our study that is based on more detailed damage data.

Furthermore, in terms of RC collapse probability, in the 1999 Athens earthquake, unlike the four events examined in this study, there was an unprecedented number of collapsed buildings. According to Karabinis *et al.* (2003), 69 RC buildings collapsed, in 28 of which there was loss of lives. Of these 54 were low-rise (1-3 floors) and 15 were mid-rise (4-7 floors), while 14 of the 69 had a soft-storey. Only 2 RC buildings built after the 1984 earthquake code revision collapsed. Almost all of the collapsed RC buildings were situated in the zones of intensity VIII and IX shown in Pomonis (2002). The total number of damaged (inspected) RC buildings ("green", "yellow" and "red-tag") in the municipalities assigned intensity VIII and IX by Eleftheriadou and Karabinis (2008) was 12,797 while another circa 30,000 RC buildings were undamaged, i.e., a collapse rate circa 0.20% can be estimated for pre-1985 RC buildings. In our study, probability of collapse from 0 to 0.94% is proposed for the range of intensity VIII and IX for classes RC2- L, RC2-LP, RC2-M and RC2-MP (built in 1959-1984). We consider that the difference in earthquake code zone could be one of the main reasons for the increased number of collapsed RC buildings in Athens, although definite comparisons with our study cannot be drawn due to the incomplete nature of the Athens 1999 damage data.

4.5.13 Economic Damage Factors Applicable to Greek URM and RC Buildings

For economic loss estimation, we correlated the structural damage grades (D_i) with the respective expected loss of each damage grade. The economic damage index depends on the extent of the damage and can vary for the same building type and damage grade. Usually, for each of the damage grades a central damage factor (CDF_i) and a coefficient of variance are estimated. The *CDF* is expressed as the ratio of the cost of repair (and in some cases strengthening) to the replacement cost of the building (Kappos *et al.*, 1998; Coburn and Spence, 2002). A better assessment of the economic loss factors is useful not only for loss estimation applications, but also for the better assessment of the cost-benefit potential of various vulnerability mitigation measures.

For load-bearing masonry and mixed structures we used the damage factors proposed by Dolce *et al.*

(2006), as actual cost of repair data from Greece are very limited. These are 3.5%; 14.5%; 30.5%; 80.0% and 95.0% for damage grades 1 to 5 respectively (described in Table 3). For each damage grade they also give the parameters of a standard beta probability density function to account for the uncertainty in these factors.

For the damage factors that apply to the damage grades of Greek RC buildings (described in Table 2) we collected data on the cost of repair by damage grade, location, number of storeys and construction period that resulted following the 1999 Athens earthquake, including insurance claims (Karabinis and Baltzopoulou, 2006; Kappos et al., 2007; Vlachos and Vlachos, 2008).

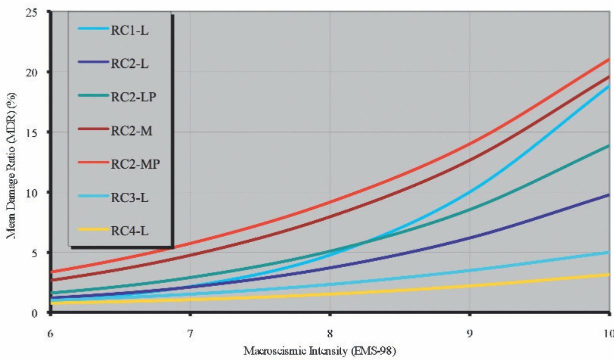


Fig. 7 - Mean Damage Ratio (MDR) as a function of macroseismic intensity (I_{EMS}) for seven classes of RC buildings for which damage data sets were sufficient to develop the vulnerability analysis

The cost of repair of the "yellow-tagged" RC buildings in particular exhibits a significant degree of scatter as buildings with a variety of damage types (structural and non-structural) are included. Analysis of the cost of repair of 874 government financed "yellow-tag" RC buildings in the municipalities of Aharnes and Ano Liosia (the 2 worst affected by the 1999 Athens earthquake) showed that the *CDF* depends on factors such as (Karabinis and Baltzopoulou, 2006): the intensity of the ground motion, the size of the property, the number of storeys and period of construction, e.g., in Aharnes the *CDF* was equal to 12.7% and in Ano Liosia 19.5%, while buildings built in 1959-84 had *CDF* of 14.4%, those built in 1985-94 had 12.7% and those built in 1995-98 had 11.9% damage factor respectively. We believe that another factor is the existence or not of soft-storey, which was not taken into account in the above studies but contributes to the scatter.

For the "red" RC buildings the damage factor is also varied, despite the fact that in the overwhelming majority of cases a protocol for demolition is issued (100% loss). In case of repairable "red-tag" RC buildings the economic cost factor can range between 40 and 70% of the replacement value and may include strengthening measures. In the case of "green-tag" RC buildings a cost factor of 7.5% was reported.

Kappos *et al.* (2007), using a different data set from the same two municipalities, reported *CDF* of 6.7% for “green”; 17.5% for “yellow” and 68.4% for repaired “red” RC buildings respectively. For the “black” RC buildings the economic loss factor is 100% as these buildings collapsed to such a degree that repair is impossible.

We have also analysed the *CDF’s* derived from insurance claims (Vlachos and Vlachos, 2008). This contains 1,848 claims by damage grade. We found that in the case of “green” and “yellow” buildings the paid insurance claims were broadly on the same level as the government compensation. We found that the *CDF* for “green” buildings was 4.4%, for “yellow” buildings 15.5% and for a few repaired “red” buildings 43.7%. More work is needed with the original data sets of these sources in order to derive the coefficients of variation of the *CDF* per damage grade.

Next the mean damage ratio (*MDR*) was calculated, which is defined as the sum of the products of the probability of occurrence of a certain damage grade (D_i) with the corresponding central damage factor (CDF_i). In Fig. 7 we show the *MDR* curves for seven classes of RC buildings for which damage data sets were sufficient to develop the vulnerability analysis. In this analysis we used the following *CDF*: 5% for “green” (D1) damage grade, 15%

for "yellow" (D2) damage grade, 80% for "red" (D3) damage grade and 100% for "black" (D4) damage grade.

4.6 Activities for Seismic Risk Mitigation

4.6.1 Activities for Increasing Seismic Safety of Construction

The most effective means for mitigating earthquake risks is by building earthquake resistant structures that will withstand the strongest anticipated earthquakes for the region. In engineering terminology these are called design earthquakes, they are specified in the codes and their characteristics are determined from seismological research that utilizes historical, instrumental and geotectonic data. Success in the direction of producing safer structures requires:

(a) Existence of adequate and up-to-date regulations for earthquake resistant design of structures, for which it is important to have:
  - activity in earthquake engineering and seismological research;
  - a mechanism in the governmental bureaucracy for code revision and implementation

(b) Well trained engineers and construction technicians who will be aware of the seismic risk and of the consequences of poor design and construction practices. This requires:

- academic programs in engineering schools, where earthquake engineering and earthquake resistant design are taught;
- continuing education programs for older engineers and construction technicians, especially after new codes are introduced.

(c) Good quality control procedures that will include design review, as well as inspections at the construction site to observe compliance with the construction drawings and specifications.

Since in every populated area threatened by earthquakes many buildings have been built either before earthquake resistant design codes were introduced or in periods when the applicable regulations were inadequate in comparison to current standards, effective reduction of seismic risk requires intervention into the existing building stock. This is a far more difficult and expensive task that has rarely been applied. When it was tried, it was done selectively for:

- structures of high importance (e.g. hospitals, buildings where large numbers of people are assembled, important bridges, monumental structures etc.);
- specific types of building construction exhibiting the highest seismic risk (e.g.

the most vulnerable buildings in areas of highest seismicity).

Given that structures of high importance belong typically to the public sector, the pertinent cost of intervention will be covered with public funds. On the other hand, buildings in the second category are in most cases privately owned and thus the burden for their seismic upgrading will fall on individuals. Considering, however, that many of the owners of such buildings will usually be low income people from the poorest classes of society, seismic upgrading of their houses will rank quite low in their priorities for life improvement. Obviously, any large scale seismic upgrading program of buildings in 2^{nd} category, before an earthquake strikes, will require a great deal of incentives and even then the chances of successful application are quite low. Thus, it is not surprising that such programs have rarely, if ever, been applied and completed anywhere in the world.

4.6.2 Activities for Preparedness and Pre-Disaster Planning

The activities listed in the preceding section will have no effect on the existing building stock, except for those few cases of structures specifically targeted for rehabilitation and strengthening. Therefore, earthquakes will continue to be catastrophic in the foreseeable future and thus the basic means for mitigating their consequences will be through effective preparedness and pre-disaster planning in order to optimize emergency response, relief operations and subsequent rehabilitation. This will require the creation at national and local levels of pre and post-earthquake planning and response units charged with:

(a) assessing the consequences of catastrophic earthquakes in high-risk areas, on the basis of risk assessment techniques and scenario type studies (e.g Anagnostopoulos et al., 2008);
(b) preparing detailed response plans, keeping them up to date and testing them with occasional drill exercises at proper time intervals;
(c) establishing formal procedures for dealing with possible earthquake predictions;
(d) planning in detail large scale operations for post-earthquake emergency assessment of building safety, including the development of well-defined criteria for such assessments and the creation of an appropriate legal framework covering all emergency operations;
(e) educating the general public about the necessary pre and post-earthquake actions expected from them and maintaining an acceptable level of awareness about earthquake risks;
(f) Planning the rehabilitation phase which includes:
    - developing repair and strengthening procedures for damaged buildings, as

well as cost assessment guidelines for the pertinent works and criteria by which the amount of financial assistance to be provided by the State for such works in determined;

- developing a financial plan for securing the funds necessary for repair, strengthening or reconstruction and more generally for financial assistance to those who suffer losses in the earthquake and need help to bring their lives back to normal.

4.7 Risk Mitigation Policies

It becomes obvious now that mitigation of seismic risk requires policies for supporting the broad range of activities listed above. These policies should be formulated to address the problem in two time horizons: short term and long term.

4.7.1 Short Term Measures

In the short term activities should be focused on:

(a) emergency response, preparedness and rehabilitation planning. These plans must be tailored to local needs in high risk areas, ideally on the basis of Risk assessment techniques using scenario type studies. Regional (prefecture) and local (city) authorities should be the basic players in the implementation of such activities;

(b) capacity and operational capability assessment of critical facilities (hospitals, fire stations, communication centers, etc.) under a worst case of earthquake scenario and seismic upgrading where needed;

(c) establishing a dense National Strong Motion data recording Network that will cover adequately the whole country and all major cities. This is basic infrastructure for most risk mitigation activities;

(d) requiring seismic upgrading of high risk buildings, i.e. vulnerable buildings, housing many people in areas of high seismic hazard, by recommending practical, easy and relatively inexpensive solutions, while providing strong economic incentives to the owners. The time horizon for such interventions could be anywhere from 2 to 5 years. The best timing for implementing such measures is immediately after a catastrophic earthquake when people are shocked by TV pictures of earthquake catastrophes;

(e) passing the necessary legislation for private earthquake insurance, make it compulsory for all new construction and create a State fund for reinsurance purposes. The latter could grow by allocating a portion of building permit fees and of other building related taxes, at no additional cost to the tax-payer. The Californian experience with private and State earthquake insurance could prove quite useful.

4.7.2 Long term measures

In the long term, activities should be focused on:

(a) establishing a mechanism for code revisions at appropriate intervals;
(b) introducing earthquake engineering courses in academic programs of engineering schools;
(c) supporting special training programs for engineers and construction technicians when new codes are introduced;
(d) funding earthquake engineering and seismological research;
(e) establishing quality control procedures in the design and construction of buildings, including the production of building materials (e.g. steel, concrete, etc.);
(f) identifying and mapping the active faults of the country;
(g) encouraging the strengthening or removal of old buildings by providing economic incentives and technical know-how.

4.8 Contribution of Research on Seismic Mitigation

Earthquake related research may be classified in three broad categories:

(a) seismological research, which deals with the earthquake as a natural phenomenon, looking into the causes of earthquakes, their expected locations (fault mapping), generation mechanisms, transmission through the earth, expected motions they will generate at any given site, etc.;

(b) earthquake engineering research aimed at improved structural performance under earthquake excitations. Its ultimate goal is to generate the know-how for earthquake resistant structures whose performance in the event of earthquakes ensures that:
   - human lives are protected
   - damage is limited
   - structures important for civil protection remain operational

(c) Earthquake prediction research aimed at predicting location, magnitude and time of occurrence of earthquakes with error margins that make the predictions useful for practical applications.

It is not difficult now to see where and how each category of earthquake related research contributes to seismic risk mitigation.

Seismological research will lead to better seismic hazard mapping and improved predictions of future ground motions. This is translated into better predictions of earthquake loadings for structures i.e. better input for structural engineers. In addition, the level and extent of pre-disaster planning and preparedness will depend on the seismic hazard mapping and its temporal variations. We note here that recent experience in Greece has shown once more that earthquakes may happen along unknown faults, near cities and in areas indicated as low seismic hazard areas. Thus, fault mapping and identification of active faults must remain high in the research priorities for seismic risk reduction.

Earthquake engineering research contributes obviously to safer structures and to limit damage to economically acceptable levels. It contributes also to pre-disaster planning and preparedness, as it is necessary for reliable assessment of the safety of damaged buildings and successful completion of the post-earthquake inspection operation, after which usage of the safe buildings will be allowed and the number of homeless people will be reduced. Finally, it contributes to the rehabilitation phase through improved repair, strengthening or reconstruction methods.

Earthquake prediction research has not yet reached a level of maturity that could lead to practical applications. This means that predictions of earthquakes cannot be made yet with that minimum level of reliability necessary to take action, e.g. in the form of issuing an earthquake watch or warning to the public as it is done for other natural phenomena (e.g. hurricanes, tornadoes, floods, etc.). If this is achieved, it will be utilized for pre-disaster planning and preparedness to save primarily human lives. It must be noted here that results from this type of research must be handled with the highest possible level of responsibility and professionalism, because earthquake prediction, whether successful or not, is quite a sensitive issue due to the many and far reaching consequences it can.

Chapter 5. *Building Response Under Earthquake Event*

In this chapter, we will see the different kinds of structural typology used by the civil engineers to design a building able to bear not only the gravity loads, but also the horizontal loads like, as precisely, the seismic loads.

Keywords: Building, Response of Building under earthquake event, Seismic Resistant Design.

5.1 Introductions

After an earthquake, building inspectors will be asked to survey the damaged building stock and classify the buildings according to a classification system that is applicable to the country where the buildings are located. In the majority of the cases such a system has as a target to separate the buildings into safe and unsafe ones, so as to restrict the use of the buildings by the occupants and avoid any further casualties during the post-earthquake period.

The building inspectors should, therefore, be able, by just looking at various failures that took place in the building, to decide on their severity and classify the building accordingly. In order to be able to do this they should know the failure mechanisms of the structural system they are examining and the severity of these failures to its structural integrity.

5.1.1 Basic Principles

We will start with some initials consideration very simple but necessary to define the point of the situation. There are many typology of building constructions, but within the purpose of this courses, we can subdivide it in these four classes:

- Masonry Buildings;
- Reinforced Concrete Buildings;
- Steel Buildings;
- Wooden Buildings.

Figure 72 ***Example of a Masonry Structure***

Figure 73 ***Example of a Concrete Structure***

Figure 74 ***Example of a Steel Structure***

Figure 75 ***Example of a Wooden Structure***

It should be noted that, within the scope of this e-book, and due to the traditional typology of building constructions in Turkey, all the next information will referrer only to the first two types of structures written above.

Talking about the rules for design and to construct structures/buildings with seismic resistant, there are some aspect to taking account regardless the type of the building or material used in construction. The structural elements which constitute the load bearing system of the building basically have to respond to three so called *golden rules* that are: **continuity**, **evenly distributed** and **well connected**. It is important to follow these rules in order to obtain in general the best structural respond both for gravity and seismic loads.

More in details:

- **Continuity**: load bearing elements (i.e columns and/or bearing walls) must be continuous through the building height. In other words this rule can be defined as the formation of the structural system, or the load bearing system of a building, without interruption from the ground level to the roof. Recent earthquakes revealed that building with this feature have better performance and in the regions of the buildings where discontinuity of structural members exists subject to great damage. Many building code impose to follow this condition.
- **Evenly distributed**: rule can be defined as the distribution of structural members or load bearing system as symmetric as possible in order to have a better seismic response. Most of the time this rule could not be obeyed due to architectural aspects. It must be emphasized that the word “symmetry” in the definition does not mean a perfect symmetry in both orthogonal directions in plan. The degree of asymmetry for each building type is defined in Seismic Codes in order to avoid as negative effect of this feature during an earthquake. In fact recent study shown as buildings with irregular structural elements in its plan, suffered great damage post-earthquake event.
- **Well connected**: rule can be defined as the proper connection of horizontal and vertical structural elements between each other in two orthogonal directions in plan. This feature is very important

for the integrity of the load bearing system of the building. Since the “joints”, where horizontal and vertical structural elements meets, are subjected to very high forces during an earthquake damage are accumulated in these regions. On the other hand connection between horizontal and vertical structural members should be satisfied in two orthogonal directions in plan.

The above rules are only the principal aspect that taking part of the seismic response of a structure during an earthquake. It is important to remember that there are many other items that effect the seismic repose of building.

In a simple way we can assume that the earthquake resistant design can be illustrated as a chain of rules that should be satisfied. In this model each ring represents the different rules that should be well treated in order to avoid that the chain loses its function, and so in other words the design process fails. Thus, each rings should be designed and constructed by experts of the related field in order to have a proper seismically resistant buildings.

5.2 Masonry Building Response Under Earthquake Event

In this chapter we will see the behavior of the masonry building under a seismic load. In this part we will see also some problem connect to the masonry's technologic, and how is possible to classify the damage in this structures due to an earthquake event..

Keywords: Masonry Building, Response of Building under earthquake event, Seismic Resistant Design.

5.2.1 Response of Masonry Buildings

Probably the masonry was one of the first system employed by the humans to build its structures. The material used was stone and/or clay.

With the reference to the Turkey, one of the most widely used structural types, is the masonry buildings. In this structures the main load bearing elements in these buildings are the wall made of very different materials. Mostly used materials in construction of masonry buildings are "brick" (see Figure 76) and "stone", depending on the materials available in the region. The bricks and stones are connected to each other by mortar applied in vertical and horizontal direction.

Figure 76 ***Brick's Wall***

Unfortunately the seismic performance of masonry buildings in recent earthquakes are very poor. One of the main reasons of this result is that they are constructed without engineering service both in design and construction process. They are just constructed by the building owners by using low quality construction materials available in the vicinity by unskilled workmanship. Additionally most of them are constructed without considering the principles of seismic resistant design concept (see Figure 77 and Figure 78).

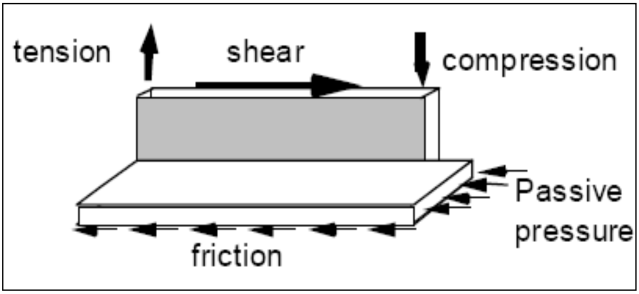


Figure 77 ***Basic Response Quantities in Masonry Buildings***

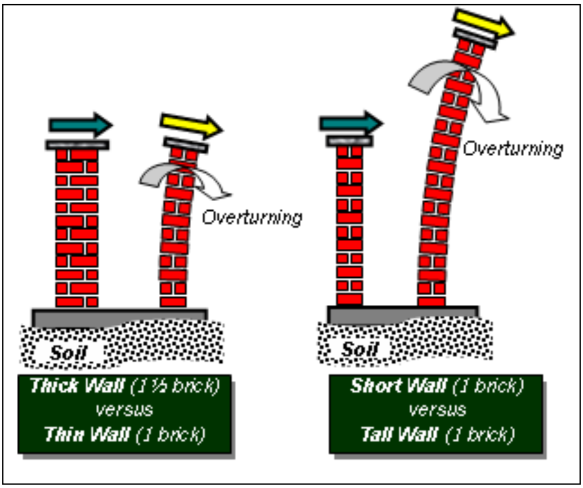
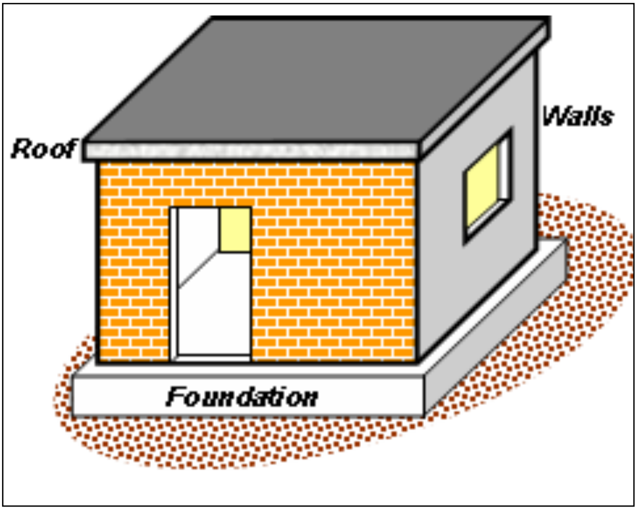


Figure 78 ***Basic Actions Acting on Masonry Walls in case of an Earthquake***

In some cases where bearing walls have a specific geometrical features additional elements called *lintels* are used both in horizontal and vertical direction. Lintels are reinforced concrete members but they do not have the same properties of columns and beams as in reinforced concrete buildings. They do not have load bearing property, they are just used for improving load carrying capacity of the bearing walls and provide integrity between the other parts of the bearing walls. Normally horizontal lintels are applied above the openings (windows and doors – see Figure 79) where some weeks points occur in the bearing wall, and also on the levels where bearing walls connect with foundation and roof.

Figure 79 ***Horizontal Lintels over a Window***

Vertical lintels instead are used where the length of the bearing walls exceed a certain level which is defined in the seismic codes. The other members of masonry buildings are foundations where all the vertical and horizontal loads to the ground. The foundations are formed along the load bearing walls. The other members of masonry buildings are the roofs whose primary functions is to prevent people from outside weather conditions are also help to provide integrity of the load bearing walls with connections (see Figure 80).



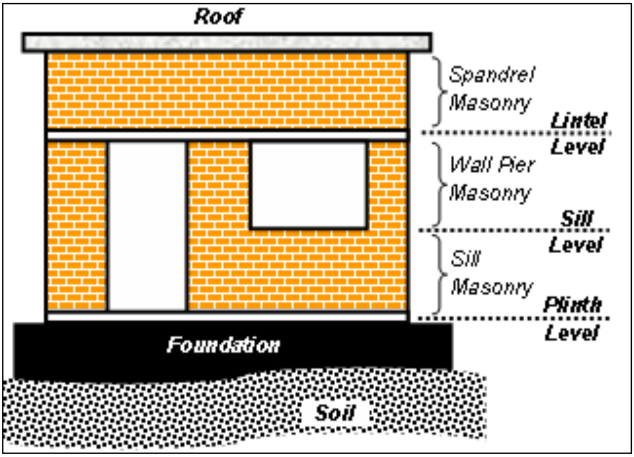


Figure 80 ***Components of a Typical Masonry Building***

In addition to the rules to be seismically resistant listed above for masonry buildings such as “continuity”, “evenly distributed” and “well connected”, additional rules should be satisfied. As in other structural types, some applications which originates from architectural and functional purposes or from lack of knowledge, may have negative effect on seismic response of the building. Factors which have a significant effect on seismic response of masonry buildings are:

- Plan configuration and connection of load bearing walls;
- Existence of lintels;
- Number of story.

In the following section we will describe this point one-by-one.

5.2.2 Plan Configuration and Connection of Load Bearing Walls

As said in the previous paragraph, the symmetric configuration of load bearing elements in a structure has a positive effect related to the seismic response of the whole building.

In masonry buildings location of load bearing walls has to be as symmetric as possible in order to have a better response in both orthogonal directions of the building (see Figure 81). Moreover the connection of load bearing walls is very important to ensure the integrity of the structure during an earthquake.

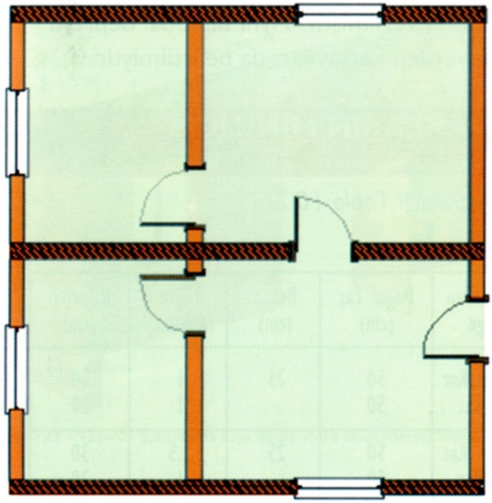


Figure 81 ***Symmetrically Located Load Bearing Walls in Two Orthogonal Direction***

In the following picture is shown in schematic, but exhaustive way, how the masonry’s brick response during a seismic action.

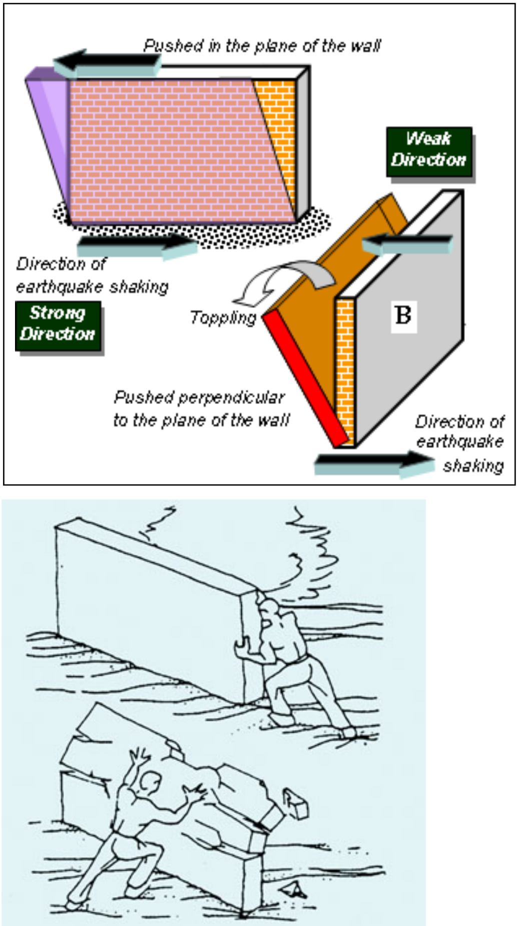


Figure 82 ***Response of Load Bearing Walls in Two Orthogonal Directions during Seismic Action***

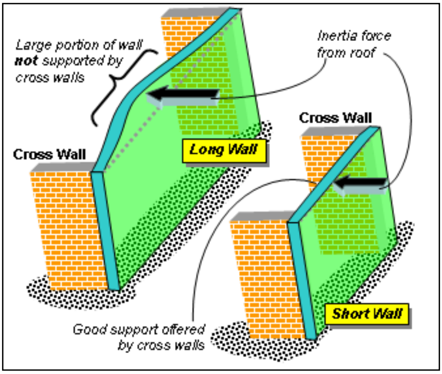


Figure 83 ***Response of Long and Short Load Bearing Walls during Seismic Action***

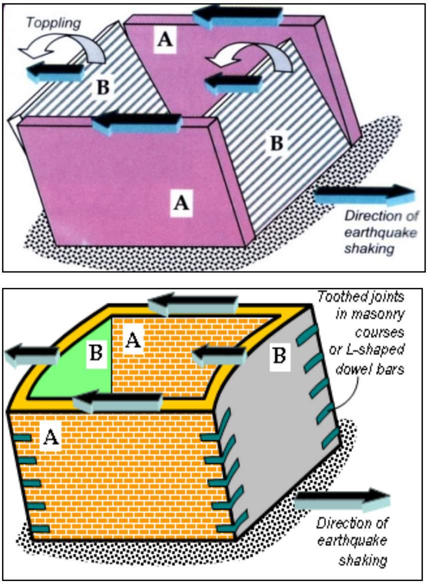


Figure 84 ***Different Response of Well and Weak Connected Load Bearing Walls***

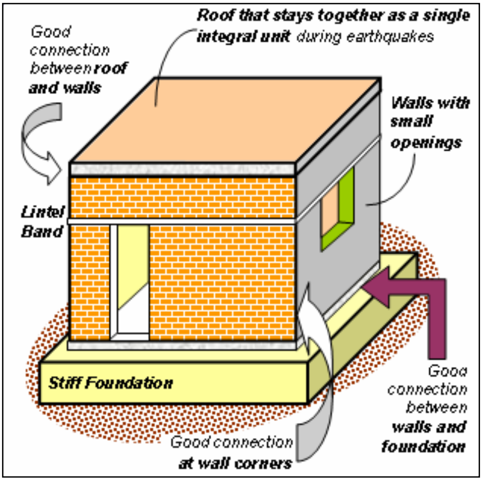


Figure 85 ***Properly Connected Load Bearing Walls of a Masonry Building***

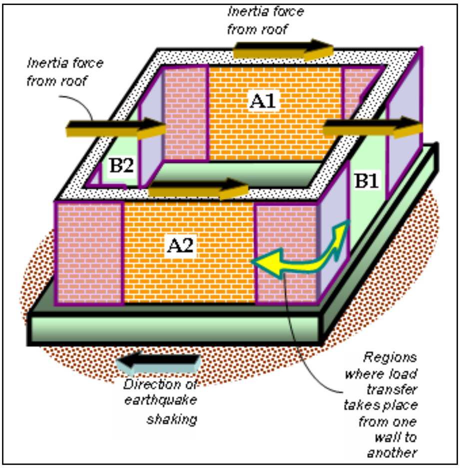


Figure 86 ***Load Transfer Path in a Masonry Building***

Vertical lintels should be formed in the connection region of two orthogonal load bearing walls (see Figure 87). Recent study on field post a seismic event, shown as these regions are one of the most vulnerable parts in masonry buildings.

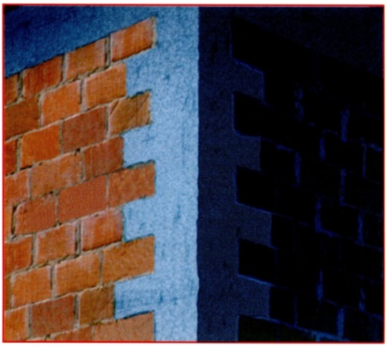
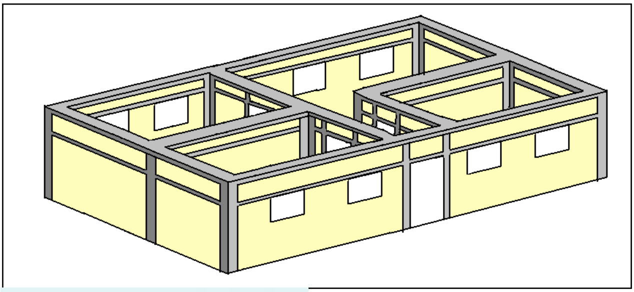


Figure 87 ***Connection of Two Load Bearing Walls at the Corner***

As mention above lintels are the horizontal and/or vertical members which is formed at the connection regions of two load bearing walls or at the top of the openings in the load bearing walls, as in the case of doors and windows. They are not primarily load bearing members, so they do not act as a column or beam as in the case of reinforced concrete frame system, but that have an important role to tie the entire load bearing wall system to each other as well as to the foundation and to the roof. In other words they realize a sort of chain that link all the structural part of the masonries, realizing, within a rigid slab made by concrete, wood, steel and concrete and so on…,a sort of “closed box”. In fact is usual call this behavior of masonry structure as “box-behavior” (see Figure 88). This request is fundamental for a correct response of a masonry building. In fact, thinking about that said above, in this way the structure is able to transfer the horizontal loads (earthquake) to the walls with “strong directions” as you can see in Figure 82, Figure 86 and Figure 89.



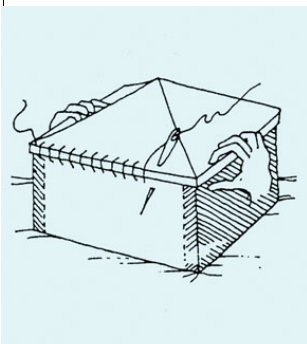


Figure 88 ***Box-Behavior. Horizontal and Vertical Lintels Locations in a Masonry Building***

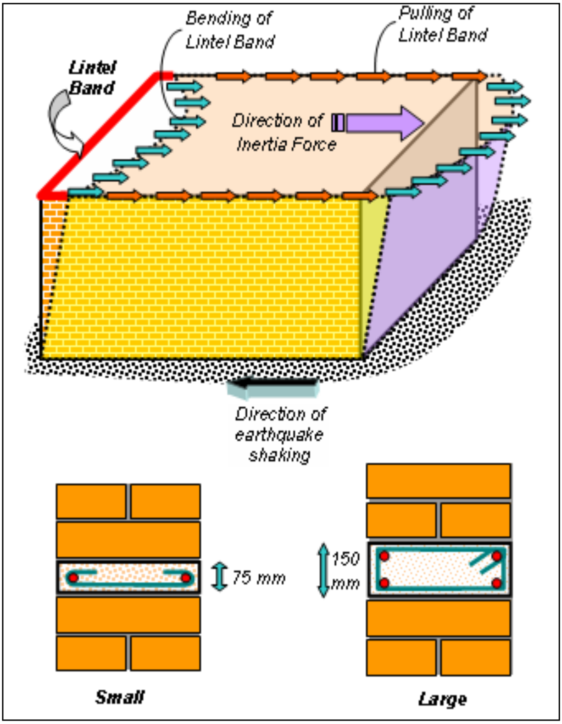


Figure 89 ***Effect of Lintels on Lateral Response of Load Bearing Walls***

Vertical lintels are formed especially in the connection regions or inside a load bearing wall whose length exceeds the limit defined in the Code. Horizontal lintels are formed on top of door and window openings where load bearing wall has discontinuity. Horizontal lintels are also formed at certain levels of load bearing walls whose height is greater the limit defined in the Code. On the other hand horizontal lintels are formed at foundation and roof level in order to have a proper connection between building, roof and foundation. Lintels at these levels have an positive effect on integrity of the load bearing wall system.

In the followings picture are represented some particulars characteristic about the lintels.

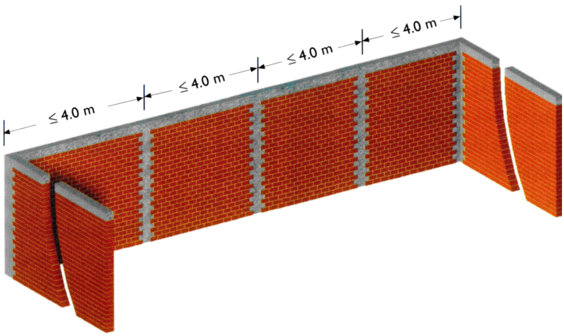


Figure 90 ***Application Distance of Vertical Lintels***

Figure 91 ***Vertical Lintel Application***

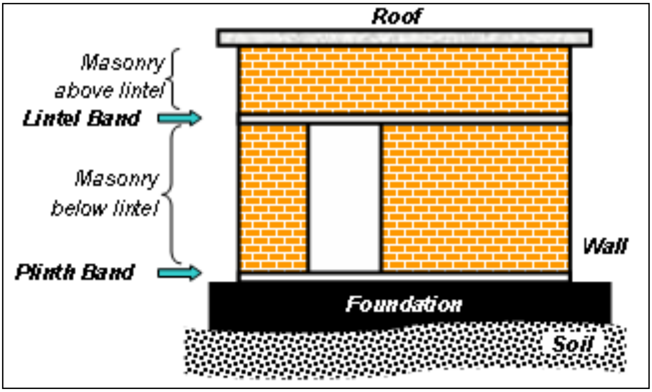


Figure 92 *Location of Horizontal Lintels*

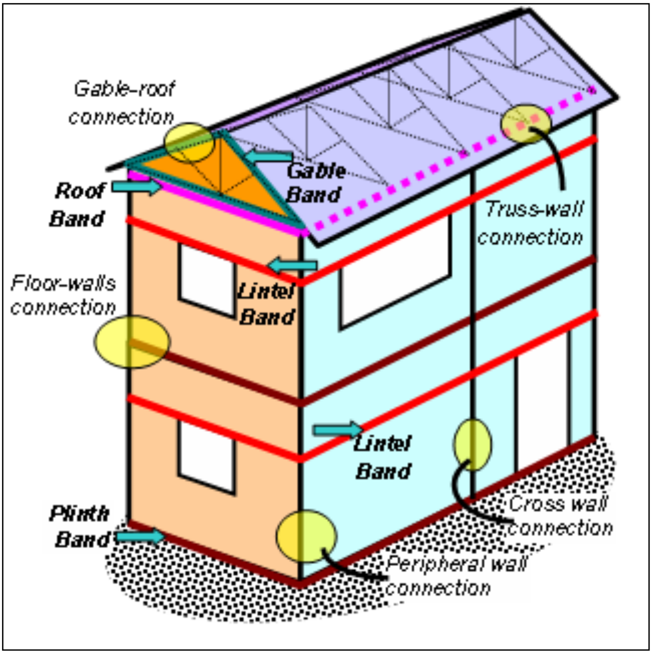


Figure 93 *Location of Horizontal and Vertical Lintels*

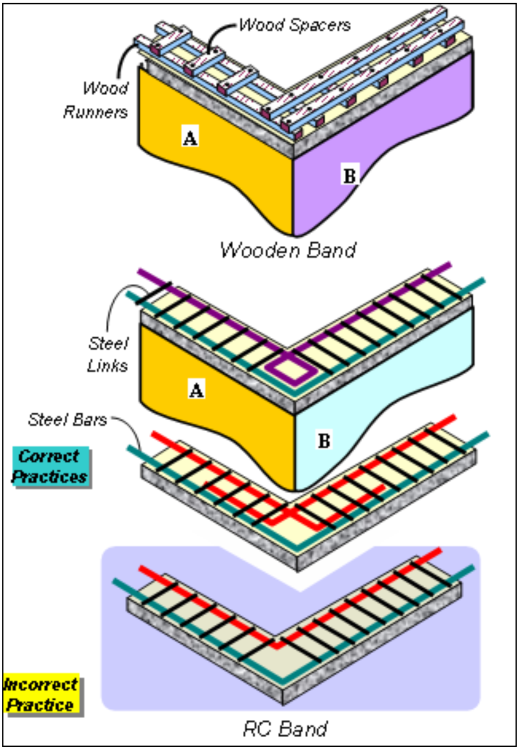


Figure 94 ***Connection Lintels with Load Bearing Walls***

5.2.3 Openings in a Load Bearing Walls

Depending on the function of the structure, in load bearing walls of masonry buildings some opening, such as doors and windows, can or better have to be are formed, with different numbers and dimensions. These opening may result in negative effects on seismic resistance of building if the dimensions exceed certain values defined in Seismic Codes, because they lower the bearing capacity of the walls. For this reason the dimensions, distance of openings in a load bearing wall is limited by the codes. An example of the effects of large openings on response of masonry building is shown below (see Figure 95, Figure 96 and Figure 97).

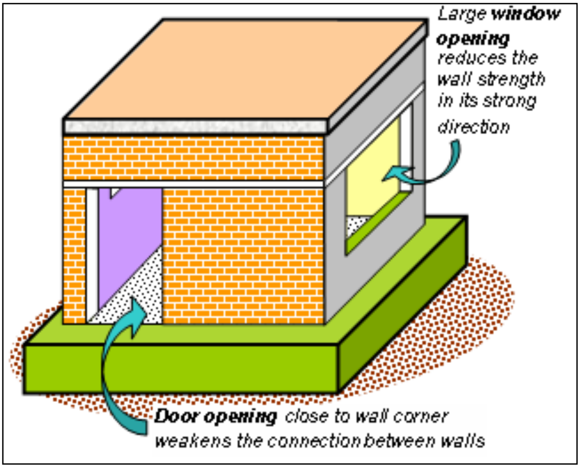


Figure 95 ***Effect of Openings on Response of Masonry Buildings***

Figure 96 ***Openings of Load Bearing Walls Which Exceeds Limit***

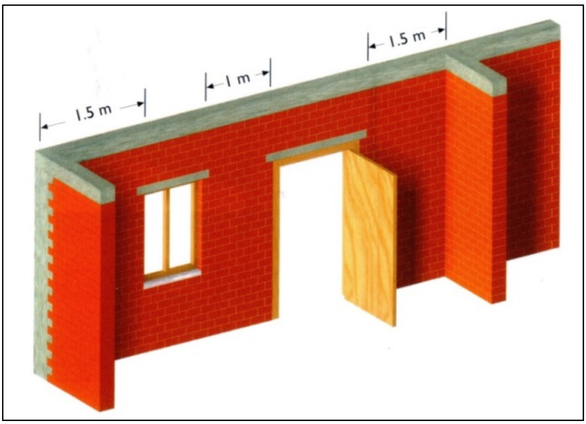


Figure 97 ***Example of Typical Geometric Limits in a Load Bearing Wall***

5.2.4 Number of Floor in a Masonry Buildings

It is a well-known that the seismic force action on a building during an earthquake is directly proportional to the total weight of the structure. In other words more the structure weighs more force will act on the structure. Since the material used in construction of masonry is limited capacity and have brittle failure modes, number of story for masonry building is limited by the Codes. In Turkish Seismic Code maximum number of story allowed in seismic zone 1 is 2 stories, in zone 2 and 3 is 3 stories.

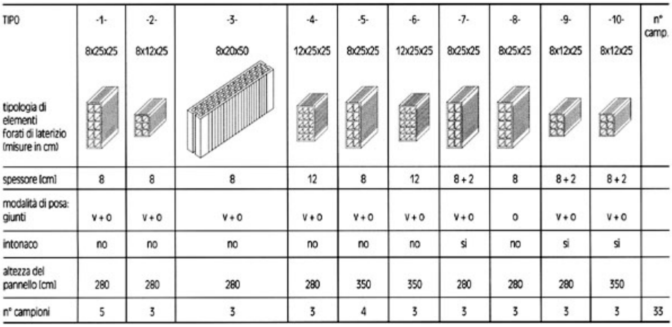
Unfortunately, this rule is not always complied specially in the past and so in the old buildings. Thus, a lot of buildings were realized with 3 or worst up to 4 or even 5 stories. This negative situation combines with the other effect and result in heavy damages due to excessive forces during the seismic action.

5.2.5 Construction Material and Construction Practice

Even if all the principles of seismic resistant design, briefly discussed above, are observed in a building, the structure may not resist to the seismic loads unless it is constructed using proper materials and construction practices. As already said, seismic resistant design is a chain of rules, so the construction of masonry buildings with high quality material and skilled workmanship is one of the important rings in this chain. One of the main issues in masonry building construction is the quality and property of the material to be used. As said probably the most widely used material for construction of masonry buildings are the bricks (see Figure 98).

Figure 98 ***Example of Bricks Materials***

Bricks should be manufactured through an approved process and it should comply the rules given in the standard in terms of dimensions, geometry and material quality. In some cases additional test should be performed in order to determine the strength, quality of the material. In application geometry of the bricks and the dimensions of the holes of the bricks are substandard (see Figure 99 and Figure 100).



TIPO	-1-	-2-	-3-	-4-	-5-	-6-	-7-	-8-	-9-	-10-	n° camp.
	8x25x25	8x12x25	8x20x50	12x25x25	8x25x25	12x25x25	8x25x25	8x25x25	8x12x25	8x12x25	
tipologia di elementi forati di laterizio (misure in cm)											
spessore (cm)	8	8	8	12	8	12	8+2	8	8+2	8+2	
modalità di posa: giunti	V+O	V+O	V+O	V+O	V+O	V+O	V+O	O	V+O	V+O	
intonaco	no	no	no	no	no	no	si	no	si	si	
altezza del pannello (cm)	280	280	280	280	350	350	280	280	280	350	
n° campioni	5	3	3	3	4	3	3	3	3	3	33

Figure 99 ***Example of Italian's Bricks Datasheet***

Figure 100 ***Brick with Proper Dimension and Geometry***

Another important point is the use of proper **plaster** in order to bond the bricks together in a correct way, it mean link them in a "safety" way. The components of the plaster should have the proper amount of ingredients and water content to have an adequate bond strength.

Additionally, the strength of bricks are lower that it should be due to uncontrolled production with low quality ingredients. On the other hand another problem is the low quality plaster with low water content which results in low bond strength and in application process plasters are applied just in horizontal direction between the bricks while they should be applied in both horizontal and vertical directions. Another wrong application in brick wall construction is that bricks are placed as holes are horizontal instead of vertical direction.

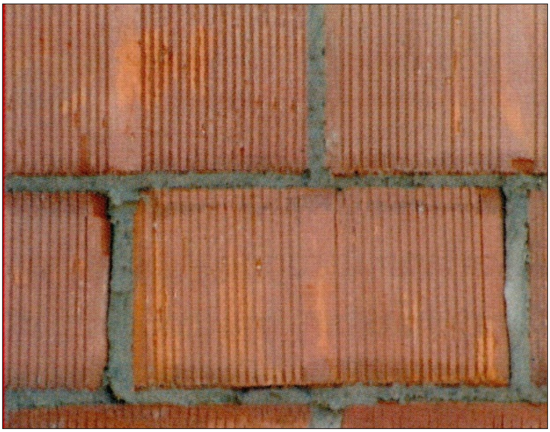


Figure 101 ***Proper Application of Plaster in Both Directions***

Figure 102 ***Improper Application of Plaster and Low Quality Brick***

Figure 103 ***Proper Construction of Load Bearing Walls***

As showed, another important point in construction of masonry buildings, is the possibility to realize a good connection between load bearing walls, roof and foundation, in order to realize the "box-behavior" (see Figure 88). The connection could be done by using rods between the load bearing wall and the horizontal lintels on foundation and roof level (see Figure 104).

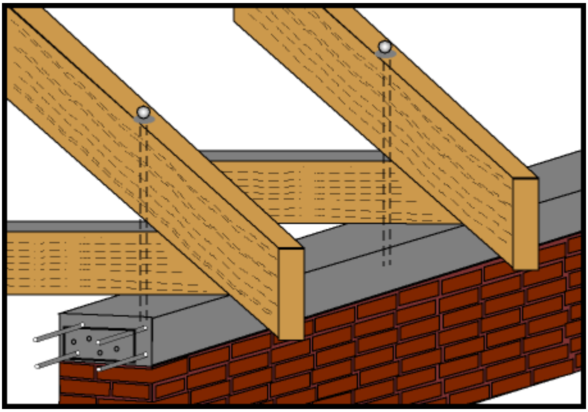


Figure 104 ***Wall- Roof Connection Detail***

5.2.6 Protection and Maintenance of Masonry Buildings

Masonry building should be protected against environmental effect during construction, for example protecting the bricks against a long exposition of humidity and/or water. Another point very important to preserve the behavior of the masonry against the "aging" is guarantee an adequate maintenance along its life. Especially within the seismic resistance, the maintenance process is of great importance for the buildings to keep its all features as long as the building is on service.

Some of the basic precautions to be taken, and rules to be applied for a seismically resistant masonry building, are listed below:

- Construct a well water proof system since water and moisture is one of the main reason of deterioration of load bearing walls during service life. Since masonry buildings are very sensitive to water or moisture, bricks and mortar loses their strength rapidly which can cause damage in case of an earthquake (see Figure 105). At this point maintenance plays an important role on for sustainable durability of the masonry building for seismic resistance.
- Basically to keep a properly constructed building in all aspects as it is built is the main application for a safe building along its service life
- To keep the as built geometry of the structure and avoid adding or demolishing load bearing walls as they are the main structural elements to resist seismic effects.

Figure 105 ***Deterioration of Load Bearing Walls due to Water***

5.2.7 Vulnerability of Masonry Buildings

Past earthquake revealed that the seismic resistance of masonry buildings is not adequate. The main reason of that huge amount of damage is the low quality material and bad workmanship. Since the masonry buildings are constructed in rural regions, people construct their own home without considering material quality and basic rules of seismic resistant design. Additionally vulnerability of these buildings increase due to lack of maintenance and improper application on the structural system of the building as shown in previous chapter. Especially, it is seen that the bricks used as material of load bearing system, has very low strength and was very sensitive to the external weather conditions which weakens the load carrying capacity of the walls.

5.2.8 Typical Damage Scenario in Masonry Buildings

The basic damage observed in masonry buildings after an seismic event, can be classified in terms of behavior as listed below:

- **Shear Failure**

The major damage type observed in masonry building is the “shear” type damage which is the most critical. As you know the mechanical properties of the bricks used to construct the load bearing wall is very good only in compression, while very poor in tension (see Figure 106). The seismic loads acting on the building causes both compressive and tension forces and when the tension forces exceed the capacity of the wall the so-called shear failure occurs. The typical crack pattern of a shear failure is "*X*" shaped cracks occurred in load bearing walls of the building (see Figure 107 and Figure 108).

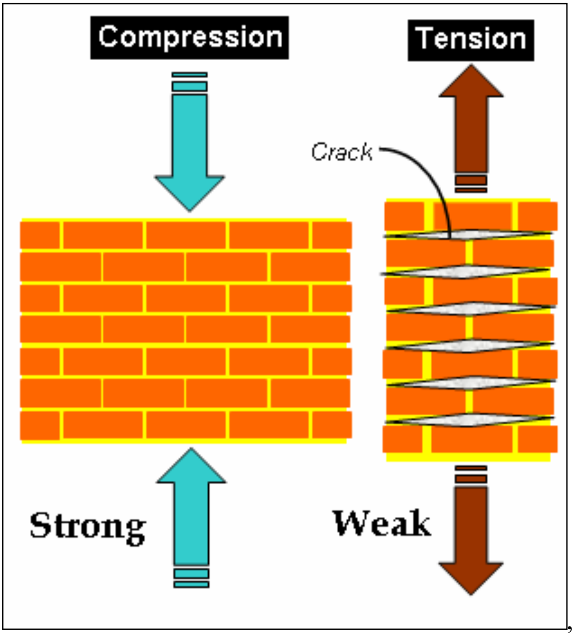


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Figure 106 ***Strength of Load Bearing Masonry Walls in Different Loading Conditions***

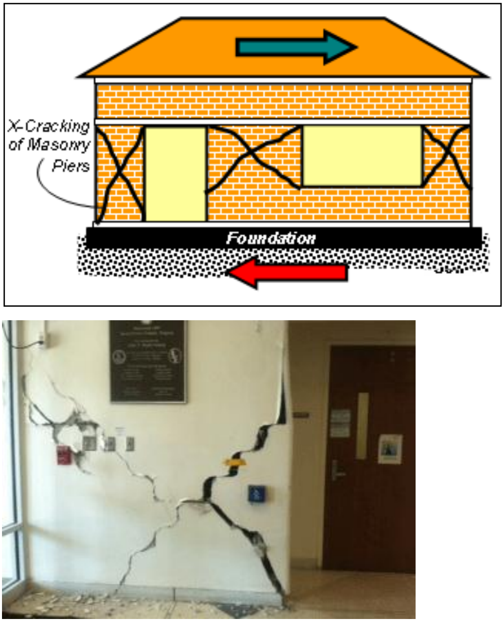


Figure 107 *Typical Shear Failure in Load Bearing Walls*

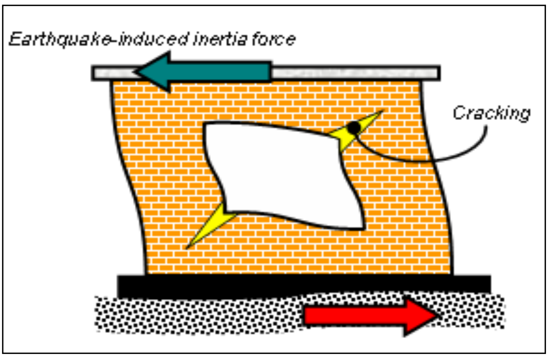


Figure 108 ***Shear Failure Mechanism at the edge of the Opening of the Wall***

- **Sliding Failure**

The sliding failure in a masonry building is observed in case of weak mortar application in foundation and horizontal lintels level. Normally this type of failure is one of the frequent failures since the quality and application of mortar is substandard most of the times. This kind of damage has a typical shape of horizontal cracks along a portion of the load bearing walls of the masonry building (see Figure 109).

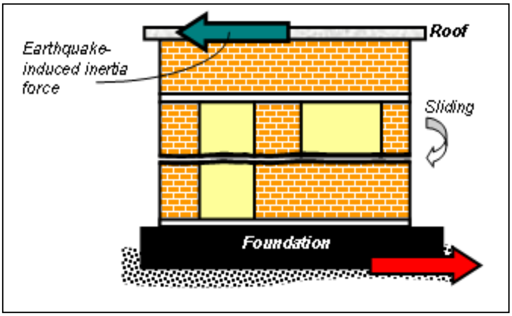


Figure 109 ***Sliding Failure Mechanism in Masonry Building***

- **Connection Failure**

As said connection plays one of the more important roles within the global behavior of the masonry building under a seismic load. One of the most critical section is the connection of load bearing walls with each other, foundation and roof. Once a failure occurred in connection region, whole or some part of the building, loses its integrity and out of plane failure of walls and partial collapse of the building occurs (see Figure 110).

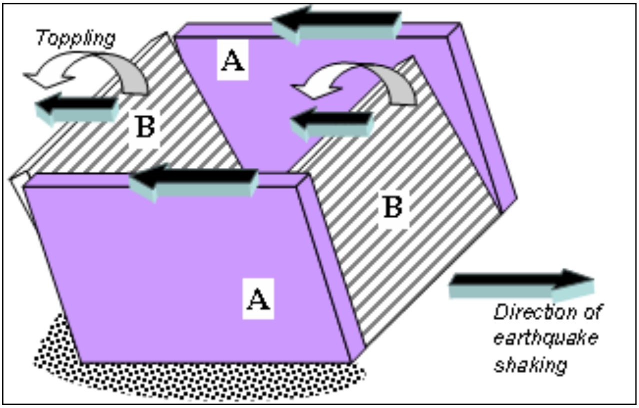


Figure 110 ***Behavior of Poor Connected Walls***

- **Compressive Failure**

As mentioned above, during a seismic action the load bearing walls are exposed to compression and tension forces which causes rocking and overturning of the walls. Depending on the material quality, connection or the geometry of the wall, these forces may result in high compressive forces exceeding the capacity of the wall. The observed damage (see Figure 111) pattern in this case is the crush of the wall material and horizontal cracks along the damaged region of the wall.

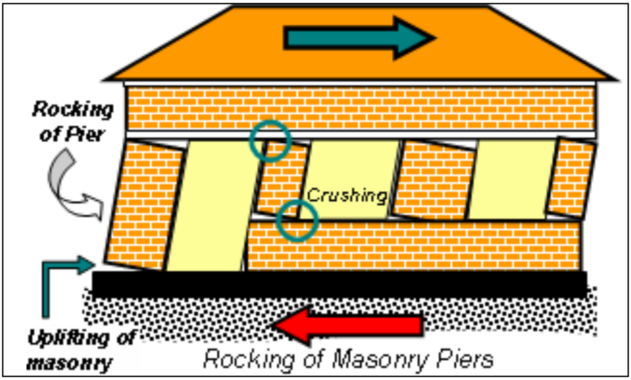


Figure 111 ***Rocking Motion Resulting in Crushing of the Wall***

About the damage observed in field about the components of masonry buildings, we can said as follow:

- **Load Bearing Walls**:

Since load bearing walls are the major components of lateral load resisting system of the masonry buildings, damage observed in these elements are of great importance in terms of safety. Recent earthquakes revealed that the most common type of damage is the shear failure resulting in "*X*" shape cracks (see Figure 112). Most of the shear failure in masonry buildings occur in regions near window and door openings. In some cases whole wall could have shear failure due to un even distribution of lateral loads.

Figure 112 ***Shear Failure - "X" Shape Cracks***

The common reason of this kind of damage is due to the poor quality of the material employed to build the load bearing wall. As already mentioned before most of the masonry buildings are constructed by using the material available in the vicinity which is produced control. In addition to the bad quality material, the construction process is often done without any control and engineering service.

By the combination of low shear strength of masonry walls with inadequate materials and construction practice, masonry buildings suffer high levels of shear failure in earthquakes.

- **Connections**

The other vital point in seismic resistance of masonry buildings are the connection of load bearing walls with the other walls, roof and foundation. Since they play an important role on the integrity of the load bearing wall system of masonry buildings any kind of dislocation in connection regions result in heavy damage or partial collapse of masonry buildings. Damage in connection regions of the load bearing walls could be due to lack of vertical lintels the excessive length of walls without any support and improper practice of mortar application in these regions. This kind of damage is also an important in terms of building safety which may result in partial collapse in case of an aftershock. It is important to examine the width of the separation and dimensions of separated walls in order to evaluate the level of safety.

- **Openings**

The weakest regions of masonry buildings are the window and door openings. Since most of the applications exceed the limits defined in codes, most of the damage is accumulated in these regions (see Figure 113). The typical damage type is the shear cracks of masonry walls due to their low shear strength and bad construction practice with improper mortar application. Most of the time shear failure in large openings result in partial collapse of the walls. Damage detected in regions of the openings should be evaluated carefully for the safety assessment.

Figure 113 ***Crack Near a Window***

- ## Roof and Foundation

The connection of roof and foundation with the load bearing walls is of great importance for the overall safety of the masonry building. Since the roofs are heavy elements of the buildings, any failure in connection regions with the walls could result in collapse of the roof and heavy damage to the masonry building (see Figure 114).

Figure 114 ***Damage in Roof Zone***

On the other hand, other connection failure with the foundation could effects the integrity of the masonry walls against lateral loads. The quality of connection of masonry walls with roof could be checked with the existence of horizontal lintels at roof level. Observation of a separation between roof and wall where horizontal lintel do not exist is a sign of heavy damage and should be evaluated as high risk in safety assessment. On the other hand damage observed in foundation level is also has the same level of risk in case of an aftershock.

In the next three figures are represented some picture that can be very useful in the field inspection of a damaged masonry building, because they summarize as said above.

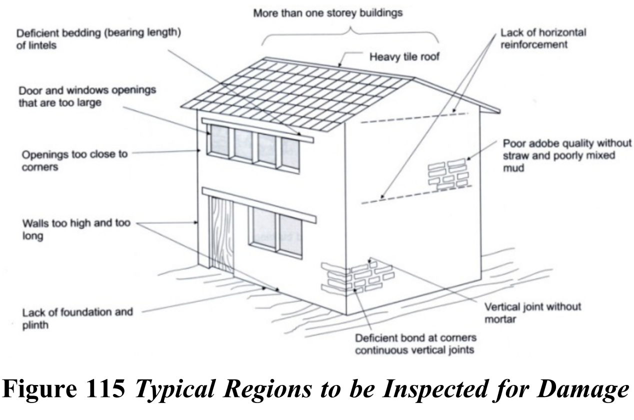
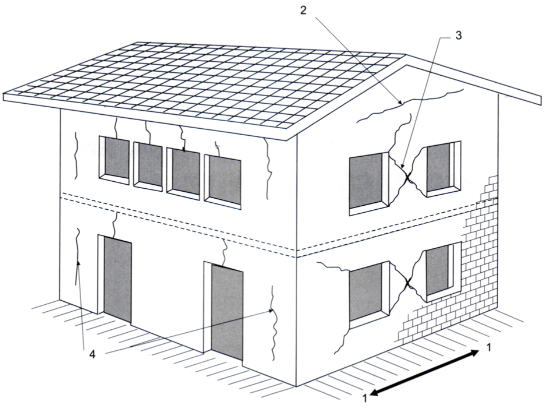


Figure 115 ***Typical Regions to be Inspected for Damage in Masonry Buildings***



(1-1 : Earthquake motion, 2: Horizontal crack, 3: Shear crack, 4: Bending crack)

Figure 116 ***Typical Crack Patterns in Masonry Buildings***

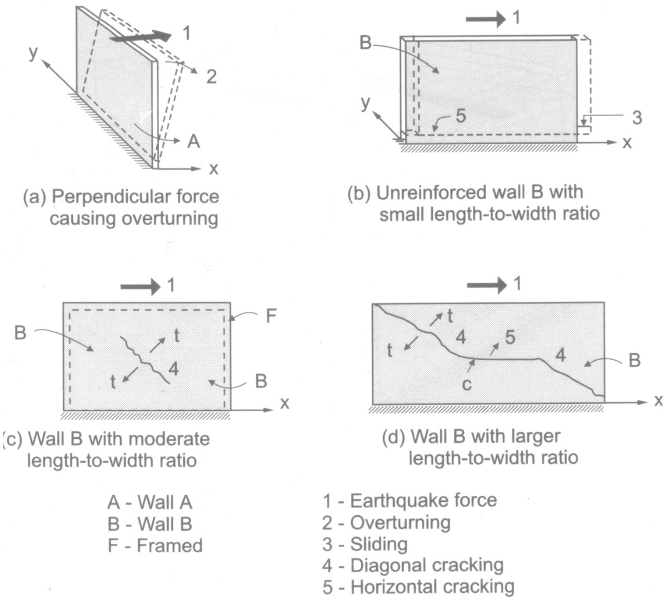


Figure 117 ***Examples of Damage Types in Masonry Bearing Walls***

5.2.9 Damage Level Definitions

The damage levels and definitions of damage observed in masonry buildings could be different in various documents/codes related to the issue. But there are certain documents which have common definitions for damage levels. One of them is the European Microseismic Scale (EMS) which is widely used for pre and post-earthquake damage assessment of buildings. Damage grades and their definition is given in Table 1 and Table 2.

Table 1. EMS Scale -Classification of Damage to Masonry Buildings

Classification of Damage to Masonry Buildings	
Grade 1	Negligible to slight damage (no structural damage). Hairline cracks in very few walls; fall of small pieces of plaster only. Fall of loose stones from upper parts of buildings in very few cases only.
Grade 2	Moderate damage (slight structural damage, moderate non-structural damage) Cracks in many walls; fall of fairly large pieces of plaster; parts of chimney fall down.
Grade 3	Substantial to heavy damage (moderate structural damage, heavy non-structural damage) Large and extensive cracks in most walls ; pan tiles or slates slip off. Chimneys are broken at the roof line; failure of individual non-structural elements.
Grade 4	Very heavy damage (heavy structural damage, very heavy non-structural damage) Serious failure of walls; partial structural failure.
Grade 5	Destruction (very heavy structural damage) Total or near total collapse.

Table 2. Definition of Damage Level of Masonry Walls(*)

Damage Level	Definition
I	Hair crack in finishing
II	Hair line crack which does not cross the wall section, in masonry walls
III	Moderate crack, which crosses the wall section, in masonry walls
IV	Partial collapse and serious damage of masonry walls
V	Collapse of masonry walls

(*)Building Damage around Bam Seismological Observatory Following the Bam, Iran Earthquake of Dec. 26, 2003 (Yasushi Sanada, Ali Niousha, Masaki Maeda, Toshimi Kabeyasawa and Mohammad Reza Ghayamghamian)

Observed damage for different damage levels are shown in the figures on the other page.

Figure 118 ***Damage Grade-1 (Masonry Building)***

Figure 119 ***Damage Grade-2 (Masonry Building)***

Figure 120 ***Damage Grade-3 (Masonry Building)***

Figure 121 ***Damage Grade-4 (Masonry Building)***

On the other hand, damage levels can be defined in terms of crack widths. The limits used for quantification of damage is basically based on laboratory test which are verified with the post- earthquake damage surveys. But it must be kept in mind that it hard to identify the damage by measuring crack widths in case of safety assessment procedure. On the other hand to have an idea of the limits for the relationship between damage levels and cracks widths will be useful before giving a final decision during assessment process. There have been numerous studies concerning the quantification of damage in masonry bearing walls by introducing crack width limits in order to serve as a guide for structural assessment studies.

Damage classification in masonry bearing walls (GNDT form)(*)

Level	**Severity**	**Description**
A	None	No visible damage
B	Slight	Any crack up to 1 mm
C	Medium	Cracks up to 4 mm (type 1,5,6) Cracks up to 2 mm (type 2,3,7) Cracks up to 1 mm (type 4,8,9)
D	Heavy	Cracks up to 10 mm (type 1,5,6) Cracks up to 5 mm (type 2,3,7) Cracks up to 1 mm (type 4,8,9)
E	Very Heavy	Cracks and damages higher “D”
F	Destruction	

(*)"An overview of post-earthquake damage assessment in Italy", A, Goretti, G. Di Pasquale

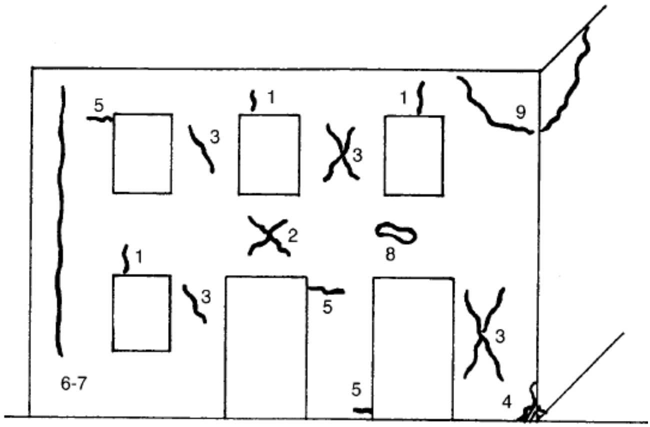


Figure 122 ***Damage Types in masonry bearing walls. 1 : Vertical cracks in openings, 2 : Diagonal cracks on parapets, doors and window lintels, 3 : Diagonal cracks on vertical walls between openings, 4 : Local masonry crushing with or without "spalling", 5 : Horizontal flexural cracks on top or bottom of vertical walls between openings, 6 : Vertical cracks at wall intersections, 7 : Passing through vertical cracks at wall intersections, 8: Spalling of material due to beam or floor pounding, 9: Separation and expulsion of two corner walls***

Damage classification in masonry bearing walls (*)

DAMAGE SEVERITY	DAMAGE DESCRIPTION
1 = None	1. No signs of any distress 2. Hairline cracks in partition walls visible from one side only
2 = Slight	1. Small cracks in partition walls visible from both sides (width $d \leq 3.0$ mm) 2. Small cracks in bearing walls, starting mostly at the corners of a few openings ($d \leq\sim 3$ mm). 3. Patches of mortar falling from ceilings or walls 4. Disturbance, partial sliding and falling down of some roof tiles
3=Moderate - Heavy	1. Substantial cracking of partition walls ($d > \sim 3.0$ mm) 2. Diagonal cracking in bearing walls ($d < \sim 5.0$ mm), but not so extensive as to constitute failure 3. Movement, separation or local failure of roof and floor framing supports 4. Dislocation and/or partial collapse of chimneys, parapets or roofs 5. Local heavy damage in some part of the building
4 =Severe - Total	1. Bearing walls with large cracks ($d > \sim 5.0$ mm), visible from both sides 2. Partial or total failure of bearing walls, floors and/or roof 3. Walls out of plumb 4. Failure of floor and roof support areas and dislocation of their framing 5. Any type of damage indicating considerable danger for collapse

Notation: d: width of cracks

(*)Post earthquake emergency of Building Damage, Safety and Usability – Part 1: Technical Issues , S. Anagnostopoulos, M. Moretti

5.2.10 Seismic Features and Performance of Masonry Building Elements: Summary

Structural Element	Seismic Deficiency	Earthquake Resistance Features	Earthquake Damage Patterns
Walls	The disposition of walls sometimes does not respect rules concerning uniform distribution of mass and stiffness. Brickwork can be extensively worn out (poor maintenance, decay)	Good quality lime mortar. Because of the wall-roof connection, which do not assure the spatial cooperation of the structures, the appeared dissymmetry causes significant general torsion effects under	Some cracks in the plaster, Vulnerability to pounding. In some buildings: diagonal cracks on the facades and on the wall.

	No reinforced concrete vertical lintels. Use of mortars with moderate strength.	the action of seismic forces.	
Roof and Floors	No stiff floors no co-operation of load bearing walls and floors, so eventual capacity deficiencies of walls cannot be compensated by a uniform distribution of loads through the floors to walls with	Timber floors with joists can assure an uniform distribution of rigidities in the plane avoiding torsional effects. Timber joists are sustained by the longitudinal walls. Roof support on these girders	In some buildings the timber floors were damaged to collapse. Especially in some cases the edge of the floor above the ground floor was separated from the wall, but the building was not damaged significantly. The movement

	higher capacity. Linear load bearing elements with one direction load transmission, not anchored to the walls. No tie beams.	leads to the fact that horizontal forces from earthquakes are absorbed without causing significant damages.	and collapse of the roof is also characteristic for affected buildings.
Openings	Not always respecting the actual prescriptions regarding the dimensions and the areas of openings in walls. Piers (between windows) of reduced sections compared to		In some buildings: X shaped cracks above the openings; Z shaped cracks on the "parapet" (under the window); cracks in the lintels over the entry door; cracks in the

	the loads to be supported. Lintels are usually brick vaults, timber or metal joists.		piers of the facade.

5.3 Reinforced Concrete Building Response Under Earthquake Event

In this chapter, we will see the behavior of the reinforced concrete building, or more simply RC buildings, under a seismic load. Moreover the pathology of reinforced concrete buildings subjected to earthquake loads will be presented. The approach that will be followed is to examine the various structural members of these structures, and describe the possible failure mechanisms of each component, showing their severity with relation with the structural integrity of "whole structural system".

Keywords: RC Building, Response of Building under earthquake event, Seismic Resistant Design.

5.3.1 Classifications of RC Buildings

There is no only one way to classify the RC structures. Speaking about the vulnerabilities of structural systems due to earthquake loads, the following classification could be the best one. According to the method of construction we can have: cast-in-place (see) or precast reinforced concrete structures (see Figure 124). More in details:

- **Cast-in-place reinforced concrete structures**

In the case of the cast-in-place RC structures, all the structural elements are constructed on site. The quality of construction is of paramount importance in the behavior of the structure under earthquake loading. Bad concrete quality, deviations in the detailing of reinforcement between drawings and construction, gross analytical errors etc…, are some of the factors that may contribute to bad behavior of a structure under earthquake loading that may lead to collapse.

In such a case, the inspector should start looking at structural component failures and try to establish whether those failures pose a threat to the structural stability of the structure and hence to the safety of the occupants.

Figure 123 ***A Reinforced Concrete Buildings***

- **Precast reinforce concrete structures**

In the case of the precast structures we have elements that are cast in a factory under controlled conditions and they are then assembled at the site. The assembly process and the quality of the joints are of paramount importance in the behavior of the structure under seismic loads. If the joints are such that continuity of the members is warranted then the system will exhibit similar behavior to a cast-in-place structure. Otherwise, the inspector should look for structural elements, like bracing, which are designed to withstand the horizontal loads induced from the earthquake loads.

Figure 124 ***A precast Concrete Walled House***

5.3.2 Structural System for RC Buildings

In order to understand the behavior of RC constructions is important to know, the different structural system can be employed within this kinds of building. Essentially there are two main systems that one can encounter, with another one, the third, that become from the combination of the first two. Thus we can taking

account about the **frame system** and the **wall system** and their combination called **dual system**.
More in details:

- **Frame system**

In the frame system the structural elements resisting both the gravity and the earthquake loads, are regular columns and of course, beam (see Figure 125). According to the definition of EC8 for a structural system to be classified as a frame one, at least 65% of the shear at the base of the structure should be carried by column elements.

Figure 125 ***Example of Frame System***

- **Wall system**

In the wall system the structural elements resisting both the gravity and the earthquake loads are walls (defined as having an aspect ratio of 4:1 – see Figure 126). According to the definition of EC8 for a structural system to be classified as a wall one, at least 65% of the shear at the base of the structure should be carried by wall elements.

Figure 126 ***Example of Wall System***

- **Dual system**

In this structural system support to the vertical loads is mainly provided by a spatial frame and resistance to lateral loads is contributed to in part by the frame system and in part by structural walls. Therefore, if more than 50% of the horizontal loads at the base of the structure is carried by frame elements, then the system is called frame-equivalent dual system, and if it is carried by walls, then it is called wall-equivalent dual system.

In the light of what, now it is more clear why it is important to know the structural system adopted to build the structure, not only for a simple classification of the structures, but because it is of paramount importance during post-earthquake inspection. For example if in a structural system in which the main earthquake resisting elements are walls, a failure of several column after a seismic event may not be severe at all to realize a structural failure (unless of course, these column's collapse cause instability of all structure).

5.3.3 Non Structural Elements

The functions of a building are facilitated by the presence of non-structural elements which provide the external skin and internal partitions. These non-structural elements contribute to the earthquake loads by increasing the mass of the structure and if they have significant horizontal stiffness they can resist part of the earthquake load. They therefore contribute to both sides of the equation, that is: as *resistant elements* and as *loads*.

The most frequently found non-structural element in RC structures that has the above qualities is the masonry infill walls (see Figure 127).

Figure 127 ***External Masonry Infill Walls***

These walls have significant contribution to the lateral stiffness of a structure and can have both beneficial (most of the cases) and detrimental effects (in some cases) in the structural behavior, if the designer does not taking them into account in the right way. For example these masonry can change extremely the stiffness of the columns of a frame system. So it is task of designer taking this in its mind during the design process (i.e. FEM modeling). Just to give you an idea, it possible taking account the stiffness of them, substituting the masonry with a “truss” put along the its diagonal. An example is shown in the following picture (see Figure 128).

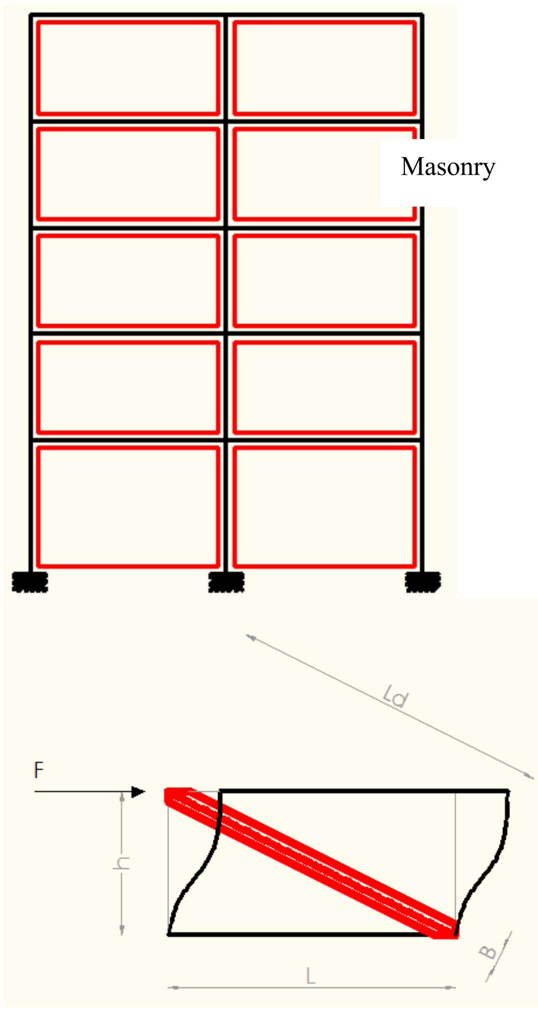


Figure 128 ***Schematization of Masonry with Truss. More details can be found in literature***

If well designed these masonry can be useful to the structure response as already said. In fact it has been shown as numerous times that the survival of a structure to an earthquake, can be attributed to the presence of infill walls.

It is important to remember that the position of such elements in the plan may create **torsional imbalances** that have not been taken into account in the analysis. In addition, they may cause irregularities in elevation if their presence is interrupted at a certain floor.

A very well know case is the *soft storey*. In this particular kind of structures infill walls are not realized in several stories. Normally it happen in the ground floor in order to create for example a parking spaces for the cars. This absence of the infilling masonry created a considerable difference in stiffness between the different stories. Thus the behavior of the whole structure change completely above all during an earthquake. The effect of this behavior is shown in the follow picture (see Figure 129). It show clearly as the "soft storey" scenario after a strong seismic event, consist of the same structure that lose one storey, the soft storey precisely.

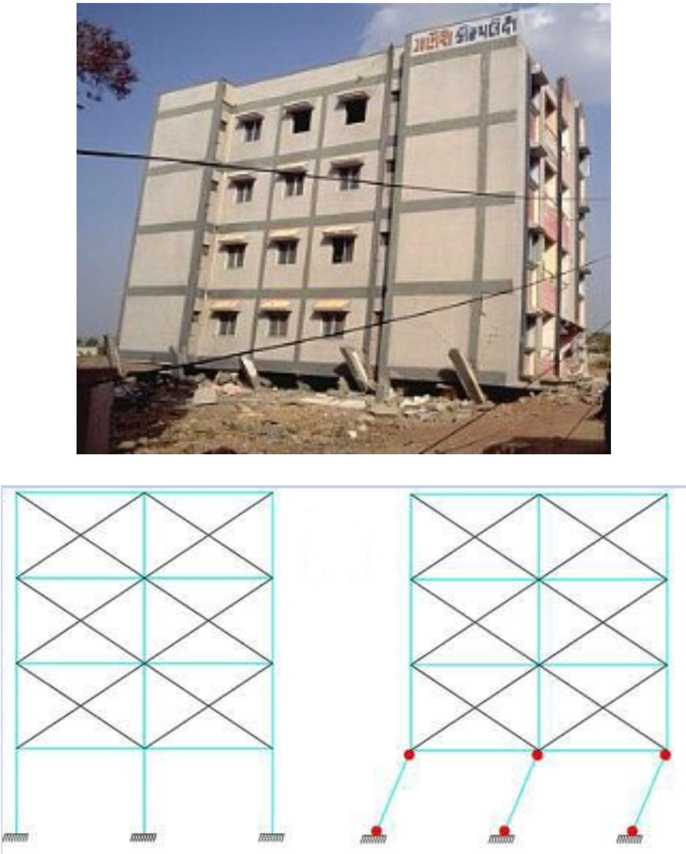


Figure 129 ***Soft Storey Collapse. Below the Model's Collapse***

So to conclude, once again it is very important to taking into account the real presence of these infilling walls not only into the design process, but also during the inspections after an earthquake event. In fact building inspectors should be aware of the contributions of non-structural elements in the whole structural seismic response, and they should look for evidence of detrimental effects caused by them in order to obtain the best result from the survey operations.

5.3.4 Component Failures

One of the difficulties in the evaluation of the structural's condition after an earthquake is how to decide on its global condition ("structural's health") based on the local evaluation of the condition of structural components. It is therefore imperative that at least at the local level the inspector is able to judge the condition of the structural element in the best possible way, and then from there, use the guidelines of the inspection forms and his engineering judgment in order to arrive at a correct classification.

The most frequent failures of structural components showed in field, are related principally to:

- Columns,
- Walls,
- Beams,
- Joints,
- Slabs.

Let see these point more in details.

5.3.4.1 Column damage

Columns are vital elements in the structural integrity (expecially in *frame structures*) of a building and special attention should be paid in assessing their capacity after an earthquake. It should be examined whether both their vertical load carrying capacity as well as their lateral stiffness and capability of carrying horizontal loads is compromised or not.

In most of the cases, when heavy column damage is found in a structure, then temporary support measures of the structure should be specified. If many columns of the structure are found to have heavy damage then this is a reason to restrict access to the building.

Columns can fail due to two main reasons (see Figure 130):

- High flexure and low shear under high axial compression: manifested by horizontal cracks near the top and bottom of the column, while the second,
- High shear and low flexure under high axial compression: manifested by inclined cracks.

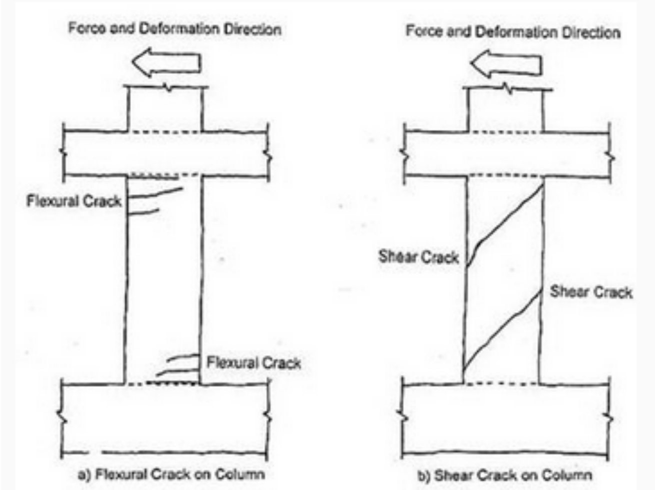


Figure 130 ***Example of Flexural (left) and Shear (Right) Failures in Columns***

The above failure depend on the ratio of the moment to shear capacity of the member, in particular this relation could be useful:

$$\frac{M}{Vd} \approx \frac{VL}{2Vd} = \frac{L}{2d}$$

where if the above ratio:

- is larger than 3.5, we can assume that the column is of moderate to high slenderness, then the flexural failure takes place.
- is less than 3.5, we can assume that the column is of moderate to low slenderness, then the brittle shear failure takes place.

More in details, for the *Flexural Failure*, the width of the cracks and the horizontal or vertical movement of one part of the column relative to another, is usually the means of judging the severity of the failure. If hairline cracks appear this means that the reinforcement has not been severely strained and therefore there load carrying capacity of the member has not been severely affected. On the other hand, if the width of the cracks is considerable and there is evidence of horizontal or vertical movement then the column has most probably lost its load carrying capacity and there is a need for taking temporary support measures. In the case that there are high vertical loads, then bulging of the column takes place with buckling of the reinforcement and simultaneous downwards vertical movement (Figure 131). This shows full disintegration of the top or bottom part of the column near the supports with limited load carrying capacity (both vertically and horizontally).

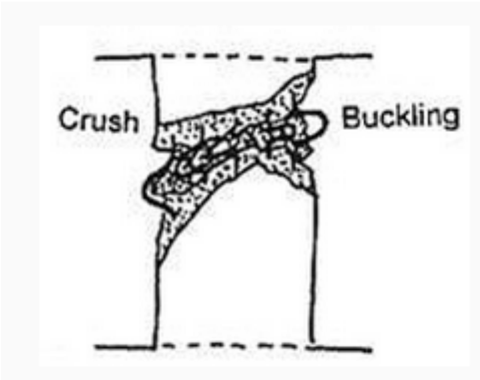


Figure 131 ***Column Failure***

The *Shear Failure* is more dangerous for the structure because it is a **brittle failure**, that takes usually place in short stocky columns. In such columns their flexural is larger than their shear capacity and failure is manifested in the form of a shear failure. A sudden cleavage shear failure takes place that in many times has as a result the complete loss of load carrying capacity and it may contribute to the collapse of a building. The severity of this type of failure can, as in the case of flexural failure, be judged based on the width of the cracks and the relative movement of one part of the column relative to the other in both the horizontal and the vertical directions. The cracks are diagonal and they form an *X*, as shown in Figure 132.

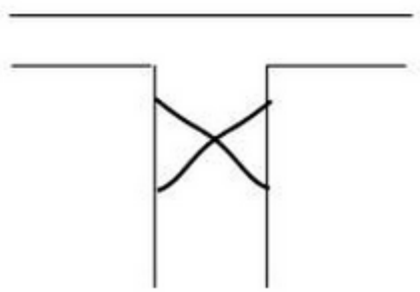


Figure 132 ***Shear Failure of a Column***

Finally, it is shown that Shear Failure can be also caused by masonry infill walls. This can happen when the infill is stronger than the column, or where there is an empty panel next to an infilled one (i.e. see Figure 133).

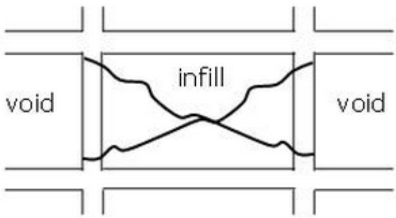


Figure 133 ***Failure of Column due to Interaction with Infill Wall***

5.3.4.2 Wall damage

As for the case of the columns, walls are vital elements in the structural integrity of a building, and special attention should be paid in assessing their capacity after an earthquake. It should be examined whether both their vertical load carrying capacity as well as their lateral stiffness and capability of carrying horizontal loads is compromised or not.

In most of the cases, when heavy wall damage is found in a structure, then temporary support measures of the structure should be specified. If many walls of the structure are found to have heavy damage then this is a reason to restrict access to the building.

Column failures can be classified into three categories:

- Shear failures,
- Flexural failures,
- Failure of the construction join

Shear Failure is the most critical failure for a wall since it is a brittle failure. As in the case of columns, due to the aspect ratio of the element, shear failure becomes more critical than flexural failure and when the shear capacity is exceeded diagonal cracks appear on the wall forming *X*-shaped cracks as shown in Figure 134. In this failure there is a tendency of outward movement of the triangular wedges that are formed between the cracks, contributing to the vertical shortening of the wall and buckling of the vertical reinforcement, with considerable reduction in the load carrying capacity of the wall.

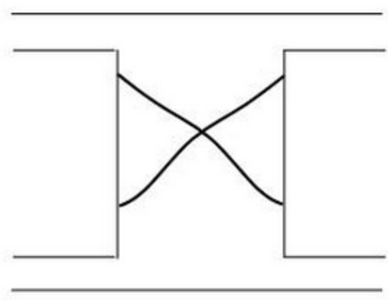


Figure 134 ***Wall's Shear Failure***

About *Flexural Failure* we can say that it is a rather rare case for RC walls. This failure manifests itself with tensile horizontal cracks on one side of the wall and compressive failure with concrete crashing on the other side (see Figure 135).

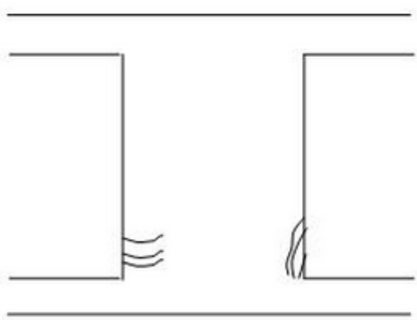


Figure 135 ***Flexural Failure of Wall***

Finally, about the *Failure Of The Construction Joint,* we can observe that it is a frequently occurring failure with very little effect on the load carrying capacity of the member unless there is considerable horizontal movement between the two faces of the construction joint (which is rather rare), causing instability problems (see Figure 136).

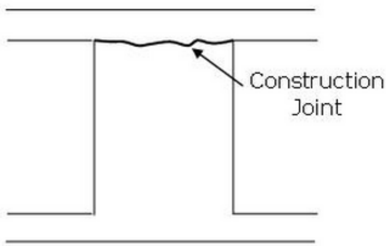


Figure 136 ***Failure at the Construction Joint***

5.3.4.3 Beam damage

Damage to beam elements is the most frequently occurring damage on a structure due to earthquake loads. Fortunately they do not contribute, in most of the cases, to the collapse of buildings.

The possible damage types that can be encountered are:

- Flexural cracks in the tension zone of the span,
- Flexural cracks at the top and bottom near the supports,
- Shear cracks near the supports,
- Shear or flexural cracks at the points where beams are supporting either columns or other beams.

Flexural Cracks In The Tension Zone Of The Span are cracks that most probably existed before the earthquake, and they have become evident due to the vertical vibration of the beam (see Figure 137). They do not pose a threat to the structural integrity of the building unless they become very wide.

Figure 137 ***Flexural Cracks in Beam Span***

Flexural Cracks At The Top And Bottom Near The Supports are cracks that are formed due to the moments caused by the horizontal earthquake forces (see Figure 138).

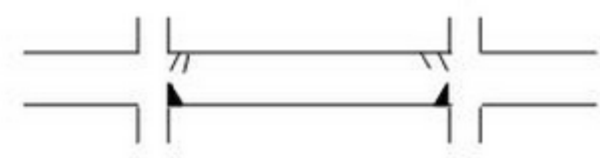


Figure 138 ***Flexural Cracks at the Top and Bottom Near the Supports***

The ones at the bottom are in most cases attributed to bad anchorage of the bottom reinforcement into the supports and in some cases can cause local failure of the beam, which may affect the overall stability of the structure due to load redistribution. If such a failure is found in a structure (pull-out of bottom reinforcement) then temporary support measures should be specified.

About *Shear Cracks Near The Supports* it is shown that is a frequent type of failure and it is considered to be serious due to its brittle nature (see Figure 139). Nevertheless, in only a few instances can jeopardize the overall stability of the structural system.

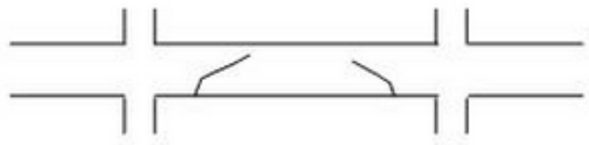


Figure 139 ***Shear Cracks Near The Supports***

The *Shear Or Flexural Cracks At Intermediate Support Points* happen when beams serve as supports for other beams or columns (see Figure 140). Thus, flexural or shear cracks appear due to the vertical vibrations induced by the earthquake (see Such failures, especially in the case of supporting columns can affect the overall stability of the structure due to progressive collapse.

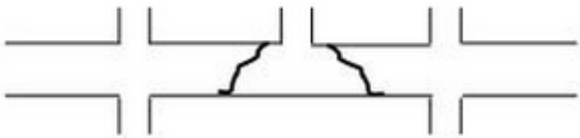


Figure 140 ***Shear Or Flexural Cracks At Intermediate Support Points***

5.3.4.4 Joint damage

Joint damage is one of the most serious failures that can occur in a structure and they should always be treated as such. Their severity is equally important to those for the columns and the walls. Both external and internal joints are affected with the external ones being more serious since they can cause instability to the structure (see). The main reasons for having failures in the joints are:

- Pour detailing,
- Pour casting or concrete due to congestion of reinforcement,

- High shear forces at the joints.

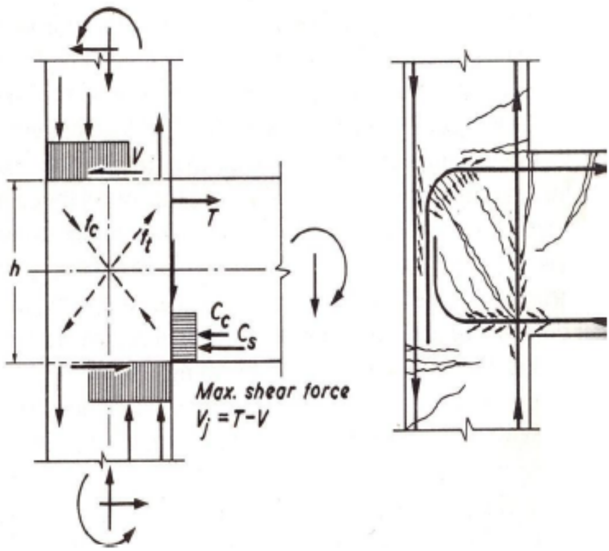


Figure 141 ***Joint Damage – Model***

5.3.4.5 Slab damage

Slabs have a great ability of load redistribution and they do not pose a threat to the overall stability of a structure, except in the case of flat slabs where the slabs are support directly on columns and there is the possibility of punching shear which can cause progressive collapse. If such a building type is to be inspected, all the column to slab connections should be examined carefully to make sure that no excessive damage has taken place. Of importance are also cracks on cantilever slabs which appear in the direction perpendicular to the main reinforcement and should be treated with care since this is a statically determinate system with no means of redistribution of forces. The cracks on cantilevering slabs cannot be easily seen since they appear on the top of the slab which is covered with tiles. Therefore evidence of these cracks should be looked for, such as tilting of tiles and so on.

5.4 Mixed Building Response Under Earthquake Event

In this chapter, we will see the behavior of the "mixed" building under a seismic load. These kind of structure are widely used by designer.

Keywords: Mixed Building, Response of Building under earthquake event, Seismic Resistant Design.

5.4.1 Introduction of "Mixed" Buildings

Mixed buildings are realized by ordinary or reinforced masonry structures, with no interaction between ground and framing system.

Vertical structural elements, inserted in mixed buildings, support a part of external vertical loads. They can be realized using different materials from the principal structure, as reinforced concrete, steel, wood, etc…

5.4.2 Structural Response of "Mixed" Buildings

In order to understand the behavior of these structures, you can imagine, that their skeleton can be idealized as a series or parallel system jointly with the masonry walls.

Typical cases are:

- the higher floors of masonry buildings are realized in reinforced concrete;
- reinforced concrete buildings present framing risings in masonry structure;
- buildings with vertical structures realized both in masonry and in reinforced concrete columns or walls at the same level. The most usual case concerns a mixed building with structural framing in reinforced concrete with external walls in masonry.

For the cases listed above, refined modeling of structures must be done in order to assure an exactly identification of the structural typology and to consider the interaction between members with different material and stiffness. In particular cases, it would be advisable using well-tested nonlinear analysis methods.

Keep in your mind, that the mixed character of buildings does not necessarily increases the vulnerability index of the structure. However, structural response of mixed buildings appears often not homogeneous and it can happen that strength concentration creates damages at local level.

Limited to the third cases listed above, the design of buildings does not satisfy the statements of the national seismic regulation. For example, in those zones, which have been declared seismic area after the building of masonry structures, mixed buildings are realized with normal or lightened bricks, with a percentage of cavities not less than 55% and a type-M2 cement mortar. Usually, masonry walls are located along the perimeter of buildings, whereas structural framings are built using reinforced columns and slab beams. Floors are realized in a traditional way, using brick panels and lighted "*predalles*" slabs with expanded polystyrene. Normally, the last ones are used in basement and underground parking.

On the contrary, for mixed buildings located in seismic area, the building consists in reinforced masonry, such that structures are realized with thermal bricks with a percentage of cavities lesser than 45%, and vertical and horizontal reinforcements with improved grip. Vertical cavity of masonry structures are filled to the type-M2 and type-M1 cement mortar. An example is shown in the following picture (see Figure 142), where it is possible to see the dispositions of the bricks, of the vertical reinforcement bars, and the stirrup bars.

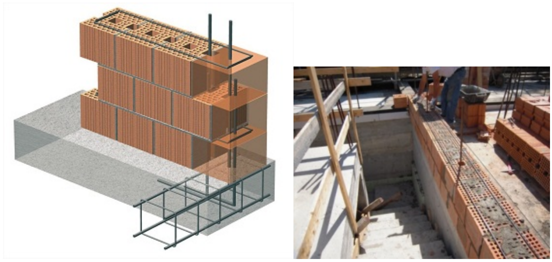


Figure 142 ***Example of Reinforced Masonry Structure. Above: Typical/ideal disposition of vertical reinforcements and stirrup bars; Below: realization on field***

Reinforced concrete structures are always built using reinforced columns and slab beams. In this case, it is clear that any seismic forces are totally absorbed by reinforced masonry, whereas columns absorb only static loads.

Chapter 6.

Safety Evaluation Methodology

In this chapter, we will show the safety assessment methodology.

Keywords: Safety evaluation, Safety assessment methodology, Building Survey.

6.1 Introduction

After an earthquake thousands buildings can be damaged and further shocks can occur. As citizens are not usually able to evaluate the residual building strength and as the number of buildings to be inspected is very huge, safety inspections are usually managed by proper institutions. Inspectors have to balance the safety of the citizenship, so when the building is judged safe, people would keep living there.

Often associated to the safety assessment is the evaluation of the short term countermeasures able to guarantee private and public safety, as propping, supports, barriers, protected crossings, etc. Concerning the time schedule of the activities within the emergency cycle (before, during and after the earthquake), safety evaluation and short term countermeasures are managed during the earthquake.

Although all above items have an undoubted importance, it is only recently that, in most of the seismic prone countries in the World, post-earthquake safety and damage assessment is being approached in a comprehensive way.

In Japan, the first research program started in 1981, after the Miyagiken-oki earthquake (1978), leading to the *"Guidelines for Post-earthquake Inspection and Restoring Techniques"*. The last version has been revised in September 2001.

In Italy, after destructive earthquakes technical activities were mainly devoted to short term countermeasures, temporary shelters and damage assessment to support the reconstruction funding. However, a research program aimed at introducing a first level usability and damage inspection form started in 1995, but the form was in a preliminary version when 1997 Umbria-Marche earthquake struck. The final version with its manual, was published in 2000 and used after Molise 2002 earthquake, when 23.000 buildings were inspected.

Also in California, the form for usability assessment contained the damage grade to several building components together with an overall measure of damage. Forms and procedures have been continuously updated as earthquakes occurred. The actual methodology originates in 1989 just before Loma Prieta earthquake and it has been revised in 1995.

From the above overview it appears that:

(a) methodologies are relatively recent and date back to almost the same period, the end of the 1970s;

(b) forms and methodologies have been often revised, due to lessons learnt after each destructive earthquake.

6.2 Safety Evaluation

The post-earthquake building survey consists in safety evaluation or damage assessment. Depending on the aim of the survey, both items can be performed. From a conceptual point of view damage and usability are strictly correlated and, hence, any usability classification implies damage classification. However, damage assessment can take more time than the safety evaluation. This is the reason because in many countries, as in Japan and Turkey, damage assessment is performed after safety evaluation.

Post-earthquake usability assessment is commonly aimed to evaluate the possible short term use of the damaged buildings. Inspectors individuate buildings that can be safely used in case of aftershocks, together with the emergency countermeasures to be taken in order to reduce the citizens' risk and, if necessary, to preserve the building. Safety inspections are directed towards reducing human discomfort, promoting the return to the pre-event social situation and limiting the number of temporary shelters, guaranteeing, at the same time, the safety of the citizens. Although the purpose is rather clear, seismic codes do not usually address the topic. It has been noted that technical activities are included in codes and technicians know what is to be done in the building design in terms of drawings, calculations, qualifications, responsibilities, and the framework for safety inspections is often lacking, especially in terms of codes and responsibilities. This is the reason because no clear and unambiguous definition of usability can be found in the literature.

Sometimes the usable building is defined as the one that is not unsafe (Japan), sometimes as the one with minor or null damage (Turkey). In other cases (California) the term usable is not used, and possibly safe buildings are classified as inspected. In Italy, post-earthquake usability evaluation is a quick and temporarily limited assessment, based on expert judgment, on visual screening and on data easily collected, aimed to detect if, during the actual seismic crisis, buildings damaged by earthquake can be used, being reasonably safeguarded the human life. Further damage is also accepted during aftershocks, but, in order for the building to be usable, life must remain reasonably safe. This definition does not include functionality of plant, although the Italian form considers damage to plant. In other forms, as in the Japanese one, damage to plant is not included. The concerns regarding this concept center on those structures where the utilities have been damaged and turned off. There is no sanitation, electricity, or gas. This means the structure does not comply with the minimum requirements of other Codes (e.g.: Health and Safety Code in California).

6.3 Overview of Past and Current Worldwide Methodologies

6.3.1 Organization of Damage Assessment in Turkey

In Turkey, the main responsibility for post-earthquake damage assessment and evaluation of damage is given to the ministry of Public Works and Settlement. The damage assessment tasks are coordinated under *General Directorate of Disaster Affairs* by *Earthquake Research Department*. All the activities related to post earthquake damage assessment is controlled by other departments under this department. This department has divisions almost every city in Turkey and all the technical personnel take part in activities in case of an earthquake. The number of responsible technical stuff can be increased depending on the size of the disaster or efficiency of the division in terms of number of stuff in the region under consideration.

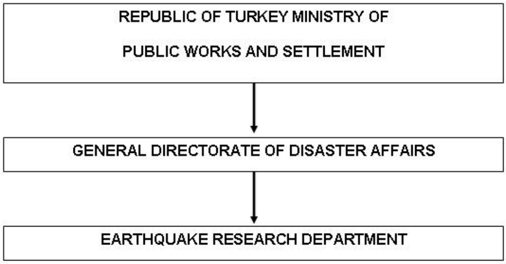


Figure 143 ***General Organizational Scheme of Damage Assessment in Turkey***

6.3.2 Damage Assessment Process in Turkey

In Turkey, according to the regulations the damage assessment procedure after an earthquake is mainly composed of three stages. These stages are:

(1) preliminary damage assessment;
(2) definite damage assessment;
(3) damage de-assessment upon objection.

But recent earthquake experience has revealed that this process could not applied as it is defined due to many reasons. Unfortunately only the second stage could be applied to some extent with many deficiencies.

The ministry has issued official forms for both first and the second stage. But the forms for the first stage has never been used properly, on the other hand the usage of the official form was very limited and the quality of the information collected was not good since the inspectors did not have enough information about the issue. But nevertheless there have been studies both by ministry officials and universities in order to prepare a document for post-earthquake damage assessment and a methodology in order to fill it.

However, forms, which were created by Middle East Technical University in the past, are revised in cooperation with *General Directorate of Disaster Affairs*; in order to take a part on a new project, started in the end of 2006, called *Disaster Information System*. Revised version of the forms (Figure 144 and Figure 145) does not distinguish the buildings whether it is built with engineering knowledge or traditional methods, instead, revised forms provides a general and quick input for the software which composes a database for loss assessment. Main purpose of the forms is to determine disaster stricken, and additionally to provide statistical data such as local distribution of earthquake damages or contribution of various parameters on losses. In scope of *Disaster Information System* project, data is collected with optical forms and then, they are converted into electronic format. The software, used to process the data, is capable to make logical decisions; therefore, effects of possible mistakes in filling the forms are minimized. In order to obtain adequate identification of the building, forms have parts for GPS coordinates or utility service numbers and pictures or small scale sketches. Also, with mobile stations, collected data on site can be transferred to the main system. This project had been applied successfully after Bala earthquake on 20th December 2007, and the system is still in use.

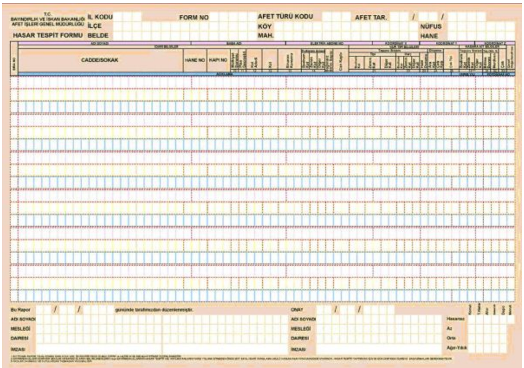


Figure 144 ***Optical Form Back Page***

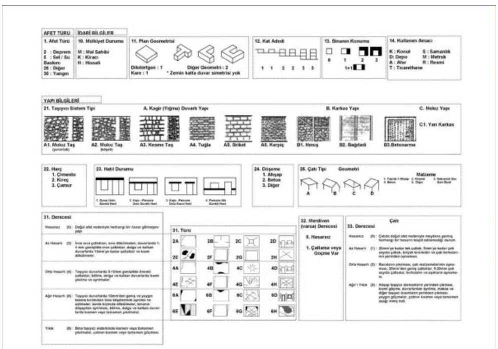


Figure 145 ***Complete Damage Information Form***

6.3.3 Introduction to Current Worldwide Methodologies

Usability and damage can be assessed for dwellings or buildings. However from the structural point of view, it is necessary to inspect the whole building, as damage can be highly localised and, at the same time, detrimental to the whole building safety.

In all the procedures where inspections are requested by the dwelling's owners or users, there is a difference between the object of the request, the dwelling, and the object of the inspection, the building. It can then happen frequently to have erroneous repeated requests on the same building. Inspection of buildings requires also defining and selecting the building. This is particularly difficult in countries, such as Italy, where most of the villages show historical areas where buildings have been aggregated during the centuries, often sharing the same walls. This problem is not so important in countries such as Japan, where buildings are recent and can be considered as isolated buildings.

6.3.4 Basis of Procedures

The process of usability assessment is highlighted in Figure 146. In $T = T_0$ a shock occurred and possibly damaged a building. An inspection is then requested and inspectors should judge about the future behaviour of the damaged building when stricken by another shock in $T = T_1$. Sources of uncertainty are the intensity of the next shock and the behaviour of the damaged building. They motivate the difficulty of a reliable quantitative assessment.

The time evolution of the seismic sequence in relation to the safety inspection process is reported in Figure 5.

From a methodological point of view, usability evaluation should be based on the three following items:

- the reference earthquake, to which the building should withstand;
- the damage suffered by the building;
- the building vulnerability.

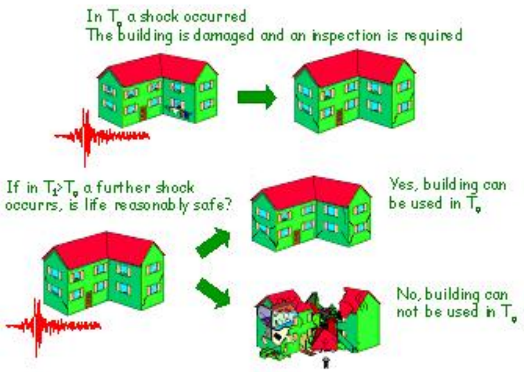


Figure 146 ***Safety Inspection Process***

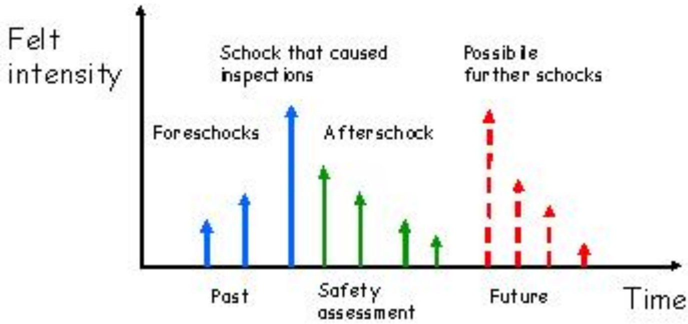


Figure 147 ***Seismic Sequence and Safety Inspection***

All the available methodologies consider the building damage as the main item to decide on building safety. Few of them address the reference earthquake and the building vulnerability, but they should be included in the methodology.

From the safety inspection process it appears that the same damaged building can be safe or unsafe depending on the intensity of the possible further shock that is supposed to occur. It is also important to point out that, at least implicitly, everyone has in his mind a given reference earthquake when judging about the post-earthquake building safety, just as in the design of a new building designed to resist to seismic actions. For a homogeneous action the reference earthquake, being the external action which buildings should withstand, should be the same for all the inspectors in the same location and it should be clearly stated before the inspections. It can be different from location to location, depending on the probability of occurrence of a further shock of a given intensity. In this the moving of the epicentre during the seismic sequence should be considered and, hence, some non- epicentre areas can be became epicentre areas in a successive shock. In principle the following logical tree (like shown in Table 3) should be used to evaluate the reference earthquake.

Table 3 ***Logical Tree for the Selection of the Reference Earthquake***

Information	Seismic crisis	Areas		Reference earthquake
No		All		Maximum felt intensity at site during the sequence
Yes	Ended	All		-
	In progress	Epicentre area		Maximum felt intensity at site during the sequence
		No epicentre area	Future non epicentre area	Maximum felt intensity at site during the sequence
			Possible future epicentre area	Intensity (MCS) 1 or 2 degrees greater than the felt one

If it is for sure that, the seismic sequence is ended, short term safety assessment can be performed in relation only to gravity loads. In practice this never happens. Generally, due to the inability to predict the evolution of the seismic activity, the reference earthquake is hardly ever stated before the inspections and is selected, in a given location, as the maximum felt intensity during the sequence in the same location.

The damage is the main item to be considered in usability assessment. The analysis should be extended to:

(a) structural elements;
(b) non-structural elements;
(c) soil and foundation and;
(d) external elements.

Structural damage is associated to structural elements as columns and walls. It can lead to the building partial or total collapse. Non-structural damage is associated to non-structural components as partitions, infill walls, signs, banners, plants. Damage to non-structural components does not involve necessarily the collapse of the whole building, but, in case of heavy damage, life safety cannot be assured, due to local partial collapses. Damage to soil can be due to landslide, liquefaction, lateral spreading, etc…They can produce settlement and tilting of the building as a rigid body, as well as cracks in the foundation. Damage to external elements can increase the life risk if objects can fall on the building under inspection. Typical is the case of adjacent bell towers, tall buildings or even rocks.

The general methodology for damage analysis is reported in the following scheme:

Analysis of the observed damage

↓

Change in the structural/non-structural conditions

↓

Importance for expected performances (life-safety)

Risk assessment in relation to life-safety requires the analysis of the observed damage under the building material, constructional type and possible failure modes. The same crack width can be more or less dangerous if occurred in a masonry building or in a RC building. A 3 mm crack width can be serious in a RC column, but can be negligible in a masonry wall. At the same time the failure mode can give more or less relevance to the crack pattern. Vertical cracks at the corner of masonry walls indicate a possible dangerous overturning of the wall, while the same crack type but at the edge of a refill of an existing opening, not connected to the wall, is not so dangerous. The damage analysis is made explicit in two following examples, specific to masonry buildings shown in Figure 148 and Figure 149.

Observed damage	Separation cracks between walls and walls and floor
Change in structural conditions	Walls are no more connected each other
Consequences on building performance	Possible out of plane overturning and fall of the floor

Figure 148 ***Masonry Building: Structural Damage***

Observed damage	Fallen tiles
Change in structural conditions	Tiles are not anchored to the roof
Consequences on building performance	Possible further fall of tiles

Figure 149 ***Masonry Building: Non-Structural Damage***

Damage grade is often established according to the classification included in macroseismic scales as Medvedev-Sponheuer-Karnik scale (MSK, 1981) or European Macroseismic Scale (EMS, 1998). As the damage is classified taking into account the building material, the structural system and the failure mode, they should be determined in the building under inspection.

This can be accomplished through visual inspection. However, when plaster or wallpaper hides the features of the building components, inspectors have to resort to interviews with the owner, tenants or with local technicians. As the damage cannot be immediately visible, it is also important to know where to look for the physical damage. Inspectors can take advantage of the fact that failure modes are characterized by specific cracks patterns.

The third item in post-earthquake safety assessment is vulnerability. In principle, vulnerability should be taken into account only when the intensity of the reference earthquake is one or two degrees higher than the maximum felt intensity during the sequence and at the same time the building is not seriously damaged. In this case the absence of damage is not evidence of good seismic performance because the shock has been of small intensity.

Hence to extrapolate the building seismic behavior to higher intensities, one has to resort to vulnerability indicators. Referring to the specific literature for a complete treatment of building vulnerability, here we recall a (non-exhaustive) list of major vulnerability indicators.

In case of masonry buildings, main vulnerability factors are: bad quality materials (HCT, round stones, pebbles, etc.) or irregular layout; lack of connections between the two wythe of the same wall, in the so called *a sacco* walls; lack of vertical connection among walls at the same level; lack of horizontal connection among walls and floors;

heavy and thrusting floors or roofs; excessive distance between parallel walls; excessive, irregular or near corner openings; excessive leaning; poor maintenance. Other contributing factors include modifications made over time, such as buttresses and ties (which improve performance) and additional stories (which tend to compromise performance); the characteristics, quality, and condition of the mortar; the thickness of the bearing and non-bearing walls; the method and configuration of the attachment of the floors and roof to the walls; the presence of chimneys inside the walls.

In case of RC buildings, main vulnerability factors are: poor concrete resistance; irregularity in plan or height (soft story); weak columns and strong beams; short columns due to shear walls or partial infill walls; frames in only one direction; age of construction in relation to seismic code and seismic zonation; infill walls not inserted in the structural frame; poor maintenance; lack of detailing.

According to the presented methodology, in case of significant structural, non- structural or geotechnical deficiencies, not worsened by the earthquake, as pre-existent cracks or settlements, the buildings should be judged safe if the reference earthquake is equal or less than the maximum felt intensity in the sequence, unsafe if it is significantly greater. Hence, as the reference earthquake is usually assumed as the maximum felt intensity, building vulnerability should not be considered in safety assessment. In practice, however, at least in the Italian experience, the percentage of buildings judged unsafe by inspectors increases as the building vulnerability increases, given the same suffered damage (see following statistics).

After observation of damage to building and soil, the safety evaluation is made with a synthesis of all the information acquired, interpreting damage and, if necessary, vulnerability, in relation to the possible further shocks to be felt by the building. The safety is evaluated associating the life-safety risk to the observed physical damage. Hence inspectors have to deal with structural risk, non-structural risk, geotechnical risk and external risk.

If the risk is judged low for each of the above items, the buildings will be safe; if it is high at least for an item the buildings will be judged unsafe. Restrictions in the building use can be imposed when the risk is highly localised in an area of the building and the possible partial collapse of that area does not involve risk in other areas. When the risk is high, but can be lowered with emergency countermeasures, the use of the building can be accepted only with the execution of the proper countermeasures. When buildings and soils are undamaged and hence the structural, non- structural and geotechnical risk can be considered low, but the external risk is high due, for instance, to the possible collapse of a bell tower, the building should be judged unsafe. When the situation is not clear and/or inspectors are not enough experts to decide on building safety, inspection should be repeated by an expert team. However inspectors should be informed that this case should be avoided as much as possible in order to not increase the number of inspections to be done. In the Figures 8 to 10 examples are reported for unusable, restricted use buildings and usable buildings. The relationship between damage and usability classification is not strictly imposed in any methodology. It will be examined in the following.

Figure 150 ***Unusable Building***

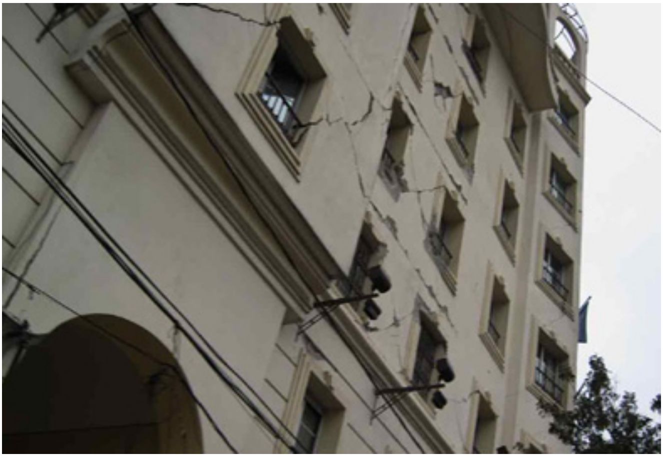


Figure 151 ***Restricted Used Building***

Figure 152 ***Usable Building***

6.4 Methodology in Turkey - Damage Assessment Methodology Used in Turkey

The damage assessment of buildings are performed by using the above mentioned form which is based on the methodology details of which is given below.

1. *Administrative Information*: in this section detailed address information of the building is obtained in addition to the information about the owner, inhabitants. The usage purpose of the building is also important for statistical aspect so as to formation of building stock database. Also an information about the presence of technical drawings of the building under consideration in order to compare the differences between actual and design situation of the building. This information is also useful for evaluation to what extent that the code regulations are satisfied in actual life.
2. *General Information*: the general purpose of this section is to determine any kind of deficiencies that could cause damage and determine damage score of each member for determination of overall damage level. The first action to be done by the inspector is to draw a sketch of the plan and elevation of the building to the related pages of the form. Additionally plan geometry and total number of stories (indicating number of basement) will be determined. The plan and elevation irregularity of the building will be

determined since they have a negative effect on vulnerability of the building. Also vertical continuity of masonry infill walls will be indicated for RC structures since they are the main source of “soft story” phenomena. Any kind of irregularity will be taken in to account as a score for over all damage determination again. Position of the building with respect to adjacent building is very important for the seismic response of the building under consideration. The information of building position whether it is at the corner or in the middle and the story level is same or different in comparison with the neighboring building is determined in order to assign a score for this feature for the global damage of the building.

6.5 The Reference Earthquakes

6.5.1 Introduction

One of the main problems in post-earthquake damage assessment is comparison of level of seismic action with the one that is expected to occur in the region. In other words comparison of seismic event in terms of design level earthquake that is defined in the seismic code. This issues is also important in order to evaluate the event as an pre-shock or a main event and on the other hand to evaluate it in terms of aftershocks.

On the other hand, from the inspection point of view, the building with the same level of damage can be safer or unsafe according to the intensity of the possible further shock that is supposed to happen. The judgment of the damage is so closely related to the level of the seismic action as in the case of new design.

The reference earthquake should be defined as the external action which buildings should withstand and it should be clearly stated for the all the inspectors in the same location in order to perform a homogenous damage assessment. This issue should be defined clearly since it can be different form location to location depending on the probability of occurrence of further shock of a given intensity.

6.5.2 Definition of Seismic Input

In earthquake prone countries the design of all types of structures are regulated by a series of rules named as *Code*. In general all of the seismic codes have common properties regarding to analysis procedures, regulation about dimensions, detailing and definition of seismic input. The term *seismic input* can be defined as the definition of probable earthquake risk in an area under consideration.

The general definition of seismic input in Codes is in the form of *Response Spectrum* which is a plot of maximum responses (e.g. spectral acceleration, spectral velocity or spectral displacement) versus period which represents different structural systems.

Since earthquakes differ from each other with respect to different properties, the maximum responses or in other words response spectrum of each ground motion also differs. In order to overcome this issue Codes define mean values for response spectra of numerous ground motions that could occur in that region. This kind of definition of seismic input is very powerful since it represents the values of all possible responses to strong ground motions in a specific region and it also covers most of the structural types with different properties. On the other hand use of response spectrum for design purpose is effective to take into account all possible maximum responses. It must be emphasized that parameters used to define the shape and values of a response spectrum is highly dependent on seismicity, soil conditions of the region under consideration. Almost all of the response spectra defined in Codes in all around the world have the same shape. (Figure 153). It can be seen from the graph below that the code spectrum is somehow an average of all possible values.

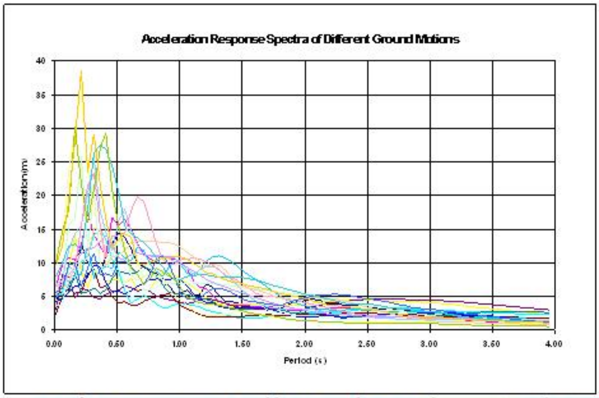


Figure 153 ***Acceleration Response Spectrum of Different Ground Motion***

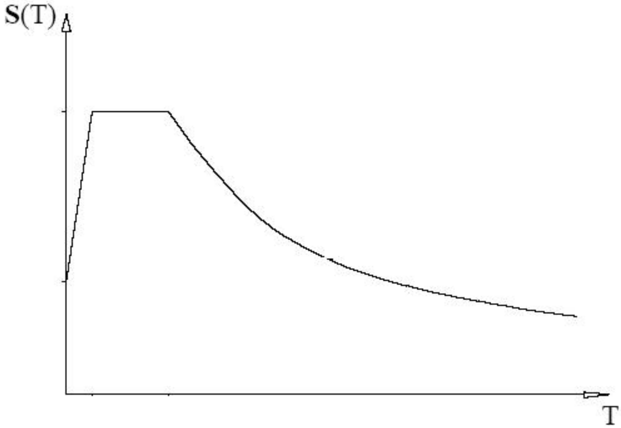


Figure 154 ***Acceleration Response Spectrum Shape Defined by Code***

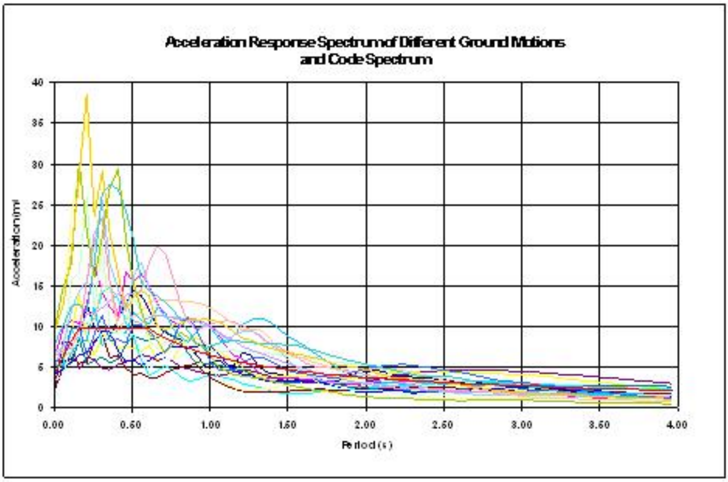


Figure 155 ***Comparison of Acceleration Response Spectrum of Different Ground Motion and Code Spectrum***

The other representation of seismic input is using acceleration time history that has been recorded in the region under consideration or simulation of ground motion by considering seismic parameters and soil properties of the region. But use of use of ground motion time history for design purposes is rather a complicated and advanced process.

The seismic input is defined under a certain probability for a certain return period. In general Code spectra are defined for 10% excess in 50 years as a rule of thumb. On the other hand the performance of the structure under this seismic input is defined as *Life Safety* which means that structure will experience a certain amount of damage but still have a global stability and occupants will survive. This means that the seismic input is defined by considering two aspects as; probability of excess in a certain time period and performance of structure under certain seismic input.

It should be kept in mind that although response spectrum defined in Codes are determined by consideration of seismicity and soil conditions. Representation these parameters are rather rough and differences in local site conditions are not taken into account. Additional investigation on seismic source (fault source) and site conditions should be performed and as a result site specific parameters for construction of response spectrum or simulation of specific ground motion for the region under consideration could be determined. Additionally other performance levels could be defined for special purposes.

6.5.3 Evaluation of Seismic Action in a Damage Assessment

In general, during post-earthquake damage assessment process the seismic action occurred in the region should be compared with the design basis seismic action in order to evaluate the damage level and comment on the aftershocks. But in most cases the level of ground motion is not evaluated in terms of ground motion parameters that could be experienced in the region. This application result in misleading damage survey results since the ground motion level may be below the expected or defined level in codes.

In the case of Turkey, the ground motion resulted in damage is not evaluated or compared with any predefined seismic input. The damage survey is just done structural response base and neither the ground motion level nor the expected structural performance is not taken into account. This application may lead to miss any probable aftershock which may result in worse consequences.

In Turkey, the damage survey is performed basically for the loan to be taken from the government for different damage levels. In some cases just the level of damage determined by the inspector could be re-evaluated in case of any objection. But as mention before the damage level is not compared with any codified performance objective.

6.6 Building Damage Classification in Turkey

Based on recent studies that have been performed on damage definition and classification of damage of building a homogenized damage scale has been introduced. Proposed homogenized damage scale for reinforced concrete structures (HRC) is shown in below table. The scale is subdivided into seven damage states, in terms of typical structural and non-structural damage expected in the four main types of reinforced concrete structures found in Europe.

DAMAGE STATE	DUCTILE MRF	NON-DUCTILE MRF	INFILLED MRF	SHEAR-WALL
None	No damage	No damage	No damage	No damage
Slight	Fine cracks in plaster partitions/infills	Fine cracks in plaster partitions/infills	Fine cracks in plaster partitions/infills	Fine cracks in plaster partitions/infills
Light	Start of structural damage	Start of structural damage	Cracking at wall-frame interfaces	Start of structural damage
	Hairline cracking in beams and columns near joints (<1mm)	Hairline cracking in beams and columns near joints (<1mm)	Cracking initiates from corners of openings	Hairline cracking on shear-wall surfaces & coupling beams
			Diagonal cracking of walls. Limited crushing of bricks at b/c connections	Onset of concrete spalling at a few locations
Moderate	Cracking in most beams & columns	Flexural & shear cracking in most beams & columns	Increased birck crushing at b/c connections	Most shear walls exhibit cracks
	Yielding in a limited number of beams & columns	Yielding in a limited number of beams & columns	Start of structural damage	Some walls reach yield capacity
	Larger flexural cracks & start of concrete spalling	Shear cracking & spalling limited	Some diagonal shear cracksin members especially for exterior frames	Increased diagonal cracking & spalling at wall corners
Extensive	Ultimate capacity reached in some elements-large flexural cracking, concrete spalling & re-bar buckling	Loss of bond at lap-splices, bar pull-out, broken ties	Extensive cracking of infills, heavier damage in frame members, some fail in shear	Most shear walls have exceeded yield, some reach ultimate capacity, boundary element distress seen
	Short column failure	Main re-bar may buckle or elements fail in shear	Partial failure of many infills, heavier damage in frame members, some fail in shear	Re-bar buckling, extensive cracking & through-wall cracks. Shear failureof some members
Partial Collapse	Collapse of a few columns, a building wing or single upper floor	Shear failure of many columns or impending soft-storey failure	Beams &/or columns fail in shear causing partial collapse. Near total infill failure	Coupling beams shattered and some shear walls fail
Collapse	Complete or impending building collapse	Complete or soft-storey failure at ground floor	Complete or impending building collapse	Complete or impending building collapse

6.7 Building Type Classification in Turkey

For general building stock the following parameters affect the damage and loss characteristics: Structural (system, height, and building practices), nonstructural elements and Occupancy (residential, commercial, and governmental).

The building classification systems used in vulnerability and loss assessments should not be country-, even region-specific to have uniform applicability in all major urban centers. The inter-regional difference in building architecture and construction practices should be reflected in building classifications for the development of inventories and vulnerability information.

A comprehensive building type classification for Europe, that incorporated the characteristic features of the European building typologies, was developed in the European Commission funded RISK-UE project (RISK-UE, 2001-2004) entitled "An Advanced Approach to Earthquake Risk Scenarios with Application to Different European Towns".

The European building typology encompasses the main building classes given in Table 1. Table 2 presents the classification of the building.

Detailed typology matrix for the R/C buildings is presented in Table 3. The proposed classification system, essentially corresponds to that adopted by EMS-98 (Grunthal 1998), apart from the inclusion of reinforced concrete dual system typology RC3 and for the introduction of sub-typologies. For all building typologies three classes of height (Low-Rise, Mid-Rise and High-Rise) have been considered. For buildings designed to a seismic code, the level of seismic action depending on seismic hazard zone has been differentiated. Four ductility classes (No, Low, Medium and High) were considered.

Table 4*: *The European Building Typology

Code	Definition
M1	Rubble stone
M2	Adobe (earth bricks)
M3	Simple stone
M4	Massive stone
M5	U Masonry (old bricks)
M6	U Masonry – r.c. floors
M7	Reinforced/confined masonry
RC1	Concrete Moment Frame
RC2	Concrete Shear Walls
RC3	Dual System

Table 5: Classification of Buildings Occupancy

Code	Occupancy category	Importance & exposure category		
		1	2	3
B	General Building stock			
B1	*Residential*			
1.1	Single family dwelling (house)			x
1.2	Multi family dwelling (apartment bldg.)			
1.3	Low-rise (1-2)			x
1.4	Mid-rise (3-7)			x
1.5	High-rise (8+)		x 1)	x
1.6	Institutional dormitory		x 1)	x
B2	*Commercial*			
2.1	Supermarkets, Malls		x 2)	x
2.2	Offices		x 2)	x
2.3	Services			x
2.4	Hotels, Motels		x 2)	x
2.5	Restaurants, Bars			x
2.6	Parking			x
2.7	Warehouse			x
B3	*Cultural*			
3.1	Museums		x 3)	x
3.2	Theatres, Cinemas		x 2)	x
3.3	Public event buildings		x 2)	x
3.4	Stadiums		x 2)	x

1) Buildings with capacity greater than 150 people

2) Buildings with capacity greater than 300 people or where more than 300 people congregate in one area

Table 6: *R/C Building Typology Matrix*

Label	Building type description	Height description			Code level*			
		Name	No. of stories	Height h, m	N	L	M	H
RC	*Reinforced concrete structures*							
RC1	Concrete moment frames	Low-rise Mid-rise High-rise	1 - 3 4 - 7 8+	$h \leq 9$ $9 < h \leq 21$ $h > 21$				
RC2	Concrete shear walls	Low-rise Mid-rise High-rise	1 - 3 4 - 7 8+	$h \leq 9$ $9 < h \leq 21$ $h > 21$				
RC3	Concrete frames with unreinforced masonry infill walls							
3.1	Regularly infilled frames	Low-rise Mid-rise High-rise	1 - 3 4 - 7 8+	$h \leq 9$ $9 < h \leq 21$ $h > 21$				
3.2	Irregularly frames (i.e., irregular structural system, irregular infills, soft/weak story)	Low-rise Mid-rise High-rise	1 - 3 4 - 7 8+	$h \leq 9$ $9 < h \leq 21$ $h > 21$				
RC4	RC Dual systems (RC frames and walls)	Low-rise Mid-rise High-rise	1 - 3 4 - 7 8+	$h \leq 9$ $9 < h \leq 21$ $h > 21$				
RC5	Precast Concrete Tilt-Up Walls	Low-rise Mid-rise High-rise	1 - 3 4 - 7 8+	$h \leq 9$ $9 < h \leq 21$ $h > 21$				
RC6	Precast Concrete Frames with Concrete shear walls	Low-rise Mid-rise High-rise	1 - 3 4 - 7 8+	$h \leq 9$ $9 < h \leq 21$ $h > 21$				

*Code level N - no code;

L - low-code (designed with unique arbitrary base shear seismic coefficient);

M - moderate-code;

H - high-code (code comparable with Eurocode 8)

<u>*References:*</u>

- *Grunthal, G. (1998), European Macroseismic Scale European Centre of Geodynamic & Seismology, Luxemburg, 15*
- *RISK-UE (2001-2004), "An Advanced Approach to Earthquake Risk Scenarios with Applications to Different European Towns"*

Research Project, European Commission, DG

XII2001-2004, CEC Contract Number: EVK4-CT-2000-00014.

6.8 Utility Lines and Hazardous Materials

6.8.1 Source of Hazardous Materials

Before determination of release of earthquake induced hazardous materials, it important to evaluate the source of hazardous material release. The main source of hazardous materials is the industrial facilities, laboratories of hospitals and residential buildings. Additionally the lifeline systems concerning the waste water pipes, natural gas and pipelines are the other major source of hazardous materials.

In residential buildings although kitchen, bathroom are used to store chemicals used for different purposes it is not a big danger for any kind of release in case of an earthquake. But on the other hand the piping systems used

for natural gas, waste water transmission may cause more dangerous results in an earthquake.
In industrial facilities and hospitals the hazardous material are kept in one place and most of the time they are indicated by different types of signs in order to avoid people.

6.8.2 Observe Damage in Earthquakes

The source of damage in hazardous material sources are both structural and nonstructural. In case of a heavy damage in structural members of a building ,all the members in the building experience a deformation in different level resulting in falling, breaking or cracking. Even the structural system has no significant damage the contents of the building may have been damaged due to high level of shaking which again results in same consequences.

At this point a new type of damage called *non-structural damage* should be explained. The nonstructural elements are the members of the buildings other than the members constitute the load bearing system. Recent earthquakes revealed that the nonstructural elements has an important effect on seismic damage in terms of money and hazardous release.

The level of shaking may cause the nonstructural element such as cabinets to fall down, pipe systems to break which results release of hazardous materials to the region. This type of damage is the most common damage observed in recent earthquakes.

Especially in case of ground movement or leaning of building the waste water, natural gas system is directly effected in terms of breaks in connection regions.

Some of the hazardous materials are flammable and they are one of the major source of fires after earthquakes. This feature of the hazardous material should be kept in mind for the safety of the assessment.

6.8.3 Actions in Assessment Process for Hazardous Materials

First of all, the inspectors assessing the safety of a building damaged by an earthquake must be careful not to expose themselves to hazardous conditions. It has been indicated that generally a building that has suffered serious, but not always readily observable, structural damage from the main shock may be in danger of collapse in the event of an aftershock. Therefore, the basic safety rule that inspectors must always observe is not to enter a building, unless they feel it is safe to do so. This requires a careful survey of the building exterior for a gross assessment of the building's condition. Inspectors should work in pairs, not only for reaching more reliable assessments by exchanging views when in doubt, but also for having one team member provide assistance or call for help in case the other member is injured. When entering a damaged building, inspectors must be well equipped, wearing their hard hats and in constant alert for aftershocks, falling hazards (parapets, glazing, appendages, ornamentation etc.) or leaks of hazardous substances.

In case they enter a factory, laboratory or storage area, they should be quite careful to recognize any suspected hazardous material release either from sighting damaged containers and spills or by odor, eye irritation, breathing problems etc., all of which constitute warnings for potential chemical hazard. In such cases the inspectors should exit from the suspected area, take note and inform the field office immediately. Damaged utility installations, such as electrical or gas, may also constitute serious hazards, e.g. in cases of exposed electrical wiring or when there are gas leaks. Gas can usually be detected by odor and if this situation is encountered an effort should be made to locate and shut off the main valve. Obviously the inspectors should refrain from smoking when entering such premises and it is prudent to do so also when entering any damaged building.

In summary, the main precautions that inspectors should take when carrying out their field work are:

(1) travel and inspect in pairs;
(2) wear always a hard hat while in a damaged building or in its surroundings.
(3) survey carefully the building exterior before entering and enter only if deemed safe to do so;
(4) be alert for aftershocks and prepared to take cover when one comes;
(5) inspect carefully for falling hazards (e.g. parapets, glazing, ornamentation etc.) and do not take risks attempting to remove them;
(6) be alert for damaged utility installations, avoid exposed electrical wiring or downed power lines and try to shut off the electricity and gas supplies;
(7) be alert for leaks of hazardous (toxic or other) substances and avoid all areas where the presence of such substances is suspected;
(8) do not use elevators;
(9) play it safe whenever in doubt about something that may entail risk.

6.9 Unsafe Areas

6.9.1 Introduction

In most of the cases after an earthquake there is disruption of the free movement of people and vehicles, which results in difficulties in fulfilling the objectives of the earthquake response operations.

It is vital that unsafe areas that obstruct movement be located as soon as possible and alternative routes to people and vehicles be established, so as to allow the smooth running of the operations.

6.9.2 Possible Causes

The possible causes of the creation of unsafe areas are the following:

(1) *collapse of buildings or falling debris from buildings into streets*: in most cases debris from collapsed buildings falls into the streets and form an obstacle in the free movement of emergency crews. It is vital that such cases be located so as to avoid any delays in the response operations

(2) *movement of the ground along a fault line or soil settlement or up heave*: lateral or normal movement of the ground along a fault line can create significant movement problems. Horizontal movements can disrupt train lines and streets due to the creation of a horizontal discontinuity. In a same manner normal movements can create steps on the ground that cannot be easily overcome. Movements of the order of 2-3 m have been reported in past earthquakes. Similar problems can be caused by the settlement or up-heave of soil.

(3) *Break-down of a life line, such as a water mains, a gas line, collapse of electricity or telephone poles*: another serious cause of disruption is the break-down of life lines. The breakdown of water

mains can cause flooding, while the breakdown of a gas line poses a threat of explosion and the cause of fire. The fall of telephone and more importantly electricity poles, apart from forming an obstacle, they can be a threat due to electrocution in case that the electricity lines are still alive.

(4) *Eruption of fires due to gas leakage or other causes*: a serious threat to people after an earthquake is fire. Nowadays with many countries having gas pipelines running their cities, the threat of the eruption of after an earthquake is very high. It is therefore very important that such leakages be located and the necessary measures be taken. A famous fire that destroyed what remained standing after the earthquake is that of San Francisco in 1906.

6.9.3 Steps to Be Taken

The following steps should be taken in order to minimize the effects of unsafe areas:

Reconnaissance groups should report the location of unsafe areas to the command center

(1) The command center should:

   a. make sure that the unsafe areas are isolated by preventing access to those areas so as to protect the public and the emergency crews;
   b. communicate the information to the relevant authorities so as to take the necessary measures (e.g.: in case of a gas leakage then the gas company should be notified to make sure that supply is interrupted);
   c. in case that the route is vital for the response operation, and there is no alternative route, then arrange for the clearing of the route;
   d. establish alternative routes, where possible, and communicate the information to all the emergency crews.

Chapter 7.*Building Information Database Form (Annex A)*

BUILDING INFORMATION DATABASE FORM

Proposed by:

Prof. Dr. Can BALKAYA

July, 2012

BUPIM International Project & Consulting Co.
Ankara, Turkey

I- BUILDING AND INVESTIGATION INFORMATION

BUILDING NO : (Ada ve Parcel No)
INVESTIGATION DATE :
BUILDING NAME :
ADDRESS :

AUTHORITY NAME :
PHONE/FAX :
E-MAIL :

SURVEY TEAM :

II-CONSTRUCTION & PROJECT INFORMATION

CONSTRUCTION YEAR :

ARCHITECTURAL DRAWINGS Exists Yes ( ) No ( )
STRUCTURAL DRAWINGS Exists Yes ( ) No ( )
FOUNDATION DRAWINGS Exists Yes ( ) No ( )

PROJECT DESIGNED EARTHQUAKE CODE YEAR:

BASEMENT FLOOR NUMBERS : HEIGHTS :
AREAS : AVERAGE WEIGHTS :
NORMAL FLOOR NUMBERS : HEIGHTS :
AREAS : AVERAGE WEIGHTS :
(including roof floor)

III-EARTHQUAKE CODE (TO BE USED CURRENT ONE) 2007 TURKISCH EARTHQUAKE CODE

EARTHQUAKE REGION :

ACCELERATION COEFFICIENT Ao :

BUILDING IMPORTANCE COEFF. I :

STRUCTURAL BEHAVIOR COEFF. R :
(considering stirrup spacings, etc. for the existing structural system)

IV-STRUCTURAL INFORMATION RC STRUCTURES

STRUCTURAL SYSTEM

REINFORCED CONCERETE FRAME ( )
REINFORCED CONCRETE FRAME AND SHEAR WALL SYSTEM ( )

FLOOR/SLAB SYSTEM

SLAP PLATE WITH BEAMS ( )
RIBBED SLAB WITH HOLLOW BLOCKS ( )
RIBBED SLAB ( )
SLAP PLATE WITHOUT BEAMS ( )
JOIST SLAB ( )
PREFABRICATED SLAB ( )
OTHERS :

FOUNDATION SYSTEM

SINGULAR FOOTING ( )
CONTINUOUS FOOTING ( )
MAT FOUNDATION ( )
PILE FOUNDATION ( )

STRUCTURAL SYSTEM IRREGULARITIES (Acc. to 2007 Turkisch Earthquake Code)

A- PLAN IRREGULARITY YES ( ) NO ( )

A1: TORSIONAL IRREGULARITY ( )
A2: FLOOR SLAB DISCONTINUITY ( )
A3: PLAN IRREGULARITIES ( )
A4: IF STRUCTURAL MEMBERS' AXES ARE NOT PARALEL ( )

B- ELEVATION IRREGULARITY YES ( ) NO ( )

B1: STRENGTH IRREGULARITY (WEAK FLOOR) ( )
B2: RIGIDITY IRREGULARIES (SOFT FLOOR) ( )
B3: VERTICAL MEMBER DISCONTINUITY ( )

STRUCTURAL FRAME DISCONTINUITY YES ( ) NO ( )

IN X- DIRECTION ( )
IN Y- DIRECTION ( )
IN BOTH DIRECTION ( )

V- MATERIAL PROPERTIES

A- CONCRETE
AVERAGE CONCRETE STRENGTH: kg/cm2 (in cylinder)
STANDARD DEVIATION : kg/cm2 (in cylinder)
EXISTING CONCRETE STRENGTH:
EXISTING CONCRETE TYPE:

B- STEEL
IS THERE ANY CORROSION YES( ) NO( )
EXISTING STEEL TYPE :
(Considering Corrosion)
STIRRUP SPACING IN BEAMS:
STIRRUP SPACING IN COLUMNS:

PROJECT VALUE
CONCRETE TYPE:
STEEL TYPE:

C- WALL MATERIAL TYPE

FACTORY BRICKS (HOLED) ( )
SOLID BRICKS ( )
CONCRETE BRICKS ( )
BLENDED BRICK ( )
CONCRETE BLOKS OR PANELS ( )
OTHER:

VI- STRUCTURAL SYSTEM STATISTICAL INFORMATION

(1) Total Cross Sectional Area of the Ground Floor **Columns**: m^2
(2) Ground Floor Plan Area : m^2
(3) Total Building Floor Area (Except Basement Floor): m^2
(4) Total Shear Wall Area in x- direction at Ground Floor: m^2
(5) Total Brick Wall Area in x-direction at Ground Floor: m^2
(6) Total Shear Wall Area in y-direction at Ground Floor: m^2
(7) Total Brick Wall Area in y-direction at Ground Level: m^2

VULNERABILITY INDEX
X-DIRECTION INDEX: (0.5(1)+((4)+0.1(5)))/3 =
Y-DIRECTION INDEX: (0.5(1)+((6)+0.1(7)))/3 =

VII- SOIL AND GEOLOGICAL INFORMATION

SOIL TYPE IN GENERAL UNDER FOUNDATION
ROCK ( ) SAND ( ) CLAY ( ) UNKOWN ( )

SOIL TYPE Z1( ) Z2( ) Z3( ) Z4( )
(According to 2007 Turkisch Earthquake Code)

SOIL BEARING CAPACITY :
GROUND WATER LEVEL :
SOIL LIQUEFACTION POTENTIAL :
SETTLEMENT PROBLEM :

VIII- DAMAGE INFORMATION (IF ANY)

DAMAGE DUE TO PREVIOUS EARTHQUAKES YES ( ) NO ( )

A-STRUCTURAL DAMAGE

DAMAGE IN SHEARWALL YES ( ) NO ( )
DAMAGE IN COLUMNS YES ( ) NO ( )
DAMAGE IN BEAMS YES ( ) NO ( )
DAMAGE IN FOUNDATION) YES ( ) NO ( )

B-NON STRUCTURAL DAMAGE

DAMAGE IN WALLS YES ( ) NO ( )
MINOR DAMAGE DUE TO SETTLEMENT YES ( ) NO ( )

IX- OTHERS

FOUNDATON ELEVATION:
GROUND ELEVATION:

EXPANSION JOINTS IN ADJACENT BLOCKS YES ( ) NO ( )
FLOOR NUMBER AT ADJACENT BLOCKS SAME( ) NO ( )

DID AN ADITIONAL FLOOR CONSTRUCTED YES( ) NO ( )

FOUNDATION SETTLEMENT PROBLEM YES( ) NO ( )
DUE TO ENVIRONMENTAL WATER
OR RAINWATER SYSTEM YES( ) NO ( )

Chapter 8. *Form For Seismic Performance Evaluation (Annex B)*

INFORMATION SYSTEMS FOR

INITIAL SEISMIC PERFORMANCE EVALUATION

Proposed by:

Prof. Dr. Can BALKAYA

July, 2012

BUPIM International Project & Consulting Co.
Ankara, Turkey

INFORMATION SYSTEMS FOR INITIAL SEISMIC PERFORMANCE EVALUATION

Seismic performance of the existing buildings can be initially estimated based on the obtained previous investigated database by considering wall indices

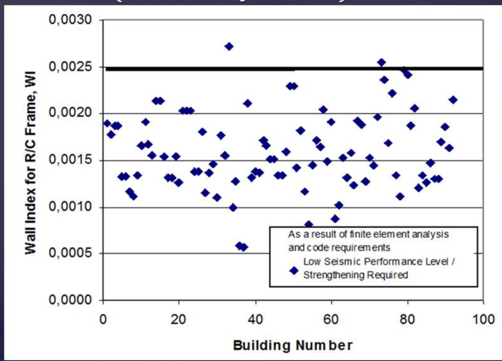
Ratio of total wall area to typical floor area in both x and y directions result in wall indices, which can be effectively used for damaged and non-damaged existing RC buildings.

Wall indices are calculated in x and y directions as follows:

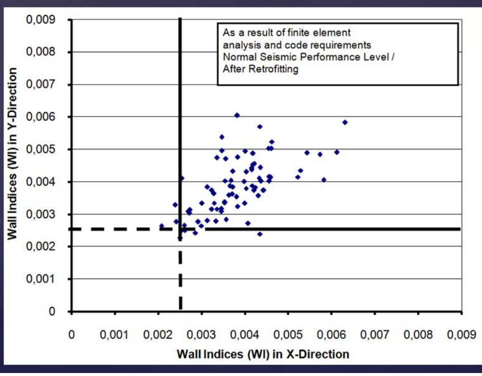
$$WI_x = \sum (0.5\ A_c + A_{wx} + 0.10\ A_{bwx}) / A_t$$

Where,
WI_x : Wall index in x-direction
A_c : Cross-sectional area of columns in x-direction in the first floor
A_{wx} : Cross-sectional area of shear-walls in the first floor
A_{bwx} : Cross-sectional area of brick-walls in x-direction in the first floor
A_t : Total floor area

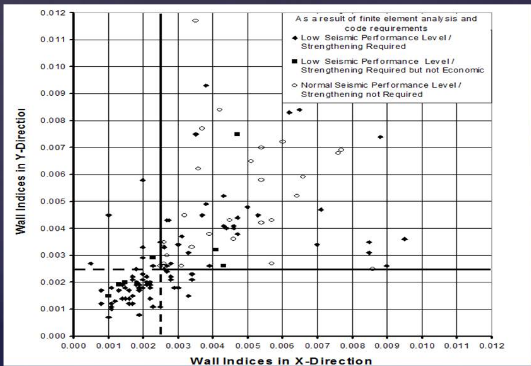
CASE STUDY I:
INITIAL SEISMIC PERFORMANCE OF EXISTING DAMAGED BUILDINGS (POST-EARTHQUAKE CASE)

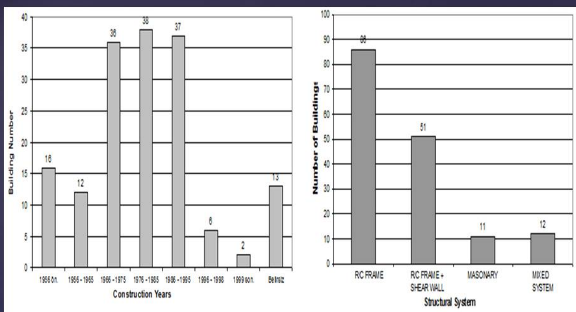


CASE STUDY I:
INITIAL SEISMIC PERFORMANCE OF EXISTING DAMAGED BUILDINGS AFTER RETROFITTING

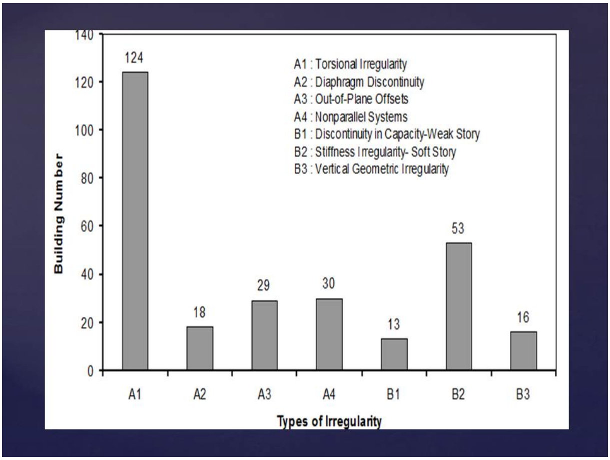


CASE STUDY II:
INITIAL SEISMIC PERFORMANCE OF EXISTING BUILDINGS (PRE-EARTHQUAKE CASE)





Investigated Non-damaged Building Information
a: Construction Years b: Structural Types



140
120
100
80
60
40
20
0
Building Number
124
18
29
30
13
53
16
A1 : Torsional Irregularity
A2 : Diaphragm Discontinuity
A3 : Out-of-Plane Offsets
A4 : Nonparallel Systems
B1 : Discontinuity in Capacity-Weak Story
B2 : Stiffness Irregularity- Soft Story
B3 : Vertical Geometric Irregularity
A1
A2
A3
A4
B1
B2
B3
Types of Irregularity

Chapter 9. *Using QuantumGIS Software (Annex C)*

In this chapter we will show the principal command used to realize the *QGIS* file for our purpose.

Keywords: QGIS, DataBase, Image

*9.1 Quantum*GIS Software

Gary Sherman began development of Quantum GIS[2] (Figure 156) in early 2002, and it became an incubator project of the Open Source Geospatial Foundation in 2007. Version 1.0 was released in January 2009. Written in C++, Quantum GIS makes extensive use of the Qt library. Quantum GIS allows integration of plugins developed using either C++ or Python. In addition to Qt, required dependencies of Quantum GIS include GEOS and SQLite. GDAL, GRASS GIS, PostGIS, and PostgreSQL are also recommended, as they provide access to additional data formats. Quantum GIS runs on multiple operating systems including Mac OS X, Linux, UNIX, and Microsoft Windows. For Mac users, the advantage of Quantum GIS over GRASS GIS is that it does not require the X11 windowing system in order to run, and the interface is much cleaner and faster. Quantum GIS can also be used as a graphical user interface to GRASS. Quantum GIS has a small file size compared to commercial GIS's and requires less RAM and processing

[2] For more information and/or teaching materials, please visit: http://www.qgis.org/en.html

power; hence it can be used on older hardware or running simultaneously with other applications where CPU power may be limited. Quantum GIS is maintained by an active group of volunteer developers who regularly release updates and bug fixes. As of 2012 developers have translated Quantum GIS into 48 languages and the application is used internationally in academic and professional environments.

As a free software application under the GNU GPL, Quantum GIS can be freely modified to perform different or more specialized tasks. Two examples are the QGIS Browser and QGIS Server applications, which use the same code for data access and rendering, but present different front-end interfaces. There are also numerous plug-ins available which expand the software's core functionality.

Quantum GIS allows use of shapefiles, coverages, and personal geodatabases. MapInfo, PostGIS, and a number of other formats are supported in Quantum GIS. Web services, including Web Map Service and Web Feature Service, are also supported to allow use of data from external sources.

Quantum GIS provides integration with other open source GIS packages, including PostGIS, GRASS, and MapServer to give users extensive functionality.

Plugins, written in Python, extend the capabilities of QGIS. There are plugins to geocode using the Google Geocoding API, perform geoprocessing (fTools) similar to the standard tools found in ArcGIS, interface with PostgreSQL and MySQL databases, and use Mapnik as a map renderer.

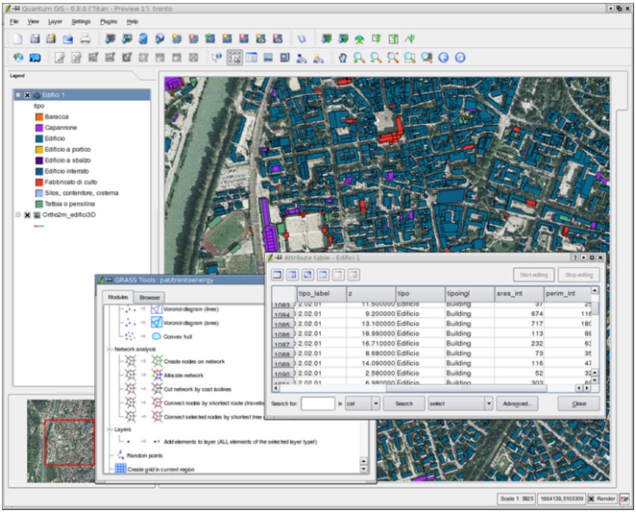


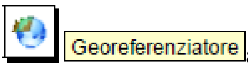
Figure 156 ***About QGIS***

9.2 Introduction

The following work is based to use the software Quantum GIS (QGIS) that is a Geographic Information System open source (Open Source).

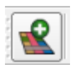
The aim of the work is to create by this software a dynamic database that describes in detail some of the buildings, located in the region of Zeytinburnu, west of Istanbul (Turkey), and therefore allows us to obtain all the information needed to implement a preventive analysis and an analysis of the image pre-and post-event on the affected area through a difference in intensity of the pixels of the two images.

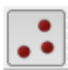
9.3 The Georeferencer

This 

allows you to generate files for georeferencing raster.

You must select points on the raster, add their coordinates, and the plugin will compute the parameters for the file georeferencing; if you provide it more coordinates, the result will better.

As a first step you need to start QGIS, load the plug-in “geotagging” and upload the file. Jpg format with the command "Add raster". 

With the button "Add  Point" you can begin to add points on the raster and input their coordinates; the plugin will compute the parameters of the georeferencing file (see Figure 157).

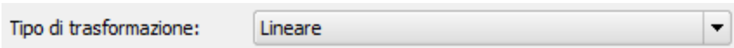
For the procedure you have two options:

- you can click on the raster manually to enter the X and Y coordinates of the point;
- you can click on the raster and press the button “to map” to add the X and Y coordinates with the help of an already georeferenced map and loaded in QGIS.

In our case we used, we use the second option and CGPS included the following:

	X	*Y*
Left Upper	*28.90336*	*41.01962*
Bottom Left	*28.90336*	*40.99082*
Right Upper	*28.91573*	*41.01962*
Right Bottom	*28.91573*	*40.99082*

After adding these 4 points to the image, you must select the type of transformation for the process of georeferencing and save the resulting file along with the file. Tif.
In our case, we chose a "Transform type" linear type:



The linear transformation (affine) is a transformation of the first order and it is used to scale, translate and rotate geometrically correct images. You use the coordinates of the points to georeference the raster image and finally load the new geo-referenced map .tif and superimpose the original map.

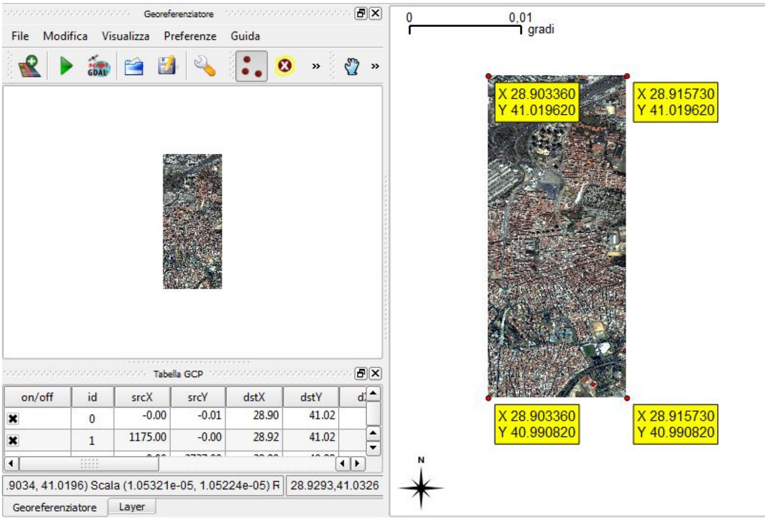
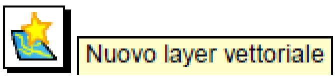


Figure 157. ***Georeferencing Raster: Zeytinburnu Region***

9.3.1 Create Layers

To create a new layer, you must use the menu



by "Layer" which also allows you to choose the type of geometry you want to include in it (point, line or polygon).

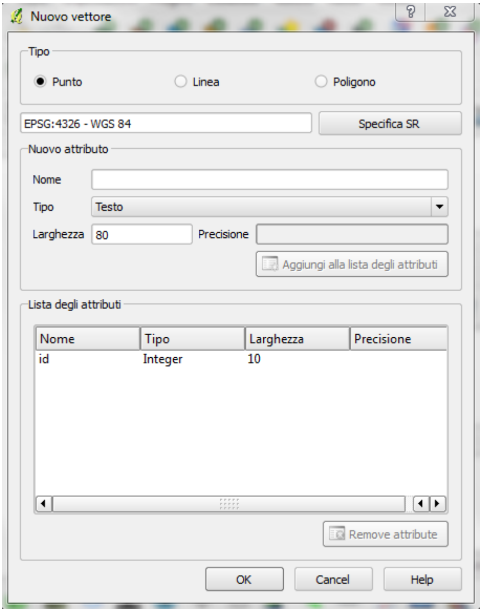


Figure 158. ***Window: New layer***

In order to complete the creation of the new layer you can define the attribute schema, specifying the name and type of the column to be inserted in the table and clicking on the "Add Attribute". Once defined the schema and assign a name to the Shapefile, it will be added to the map and can be modified.

For the quarter that I study, I have been created five different layers (see Figure 159 and Figure 160):

- *Highway* layer lists the main roads surrounding the area in question;
- *Major Road* layer lists the secondary roads that are tied to those listed above;

- *Residential Street* layer describes the minor roads that serve to access to the houses that will be analyzed;
- *Buildings* layer shows the structures to be analyzed and contains a link (with the command "Actions") to a dynamic database, which describes the structural analysis of each building, and the "Attribute table" has the geographic location, the indices of structural vulnerability in the x and y the maximum displacement capacity always in the two main directions;
- *Photo* layer, finally, offers the possibility to get an overview of the area through some photo taken at the points of interest in the area and taken from Google Maps.

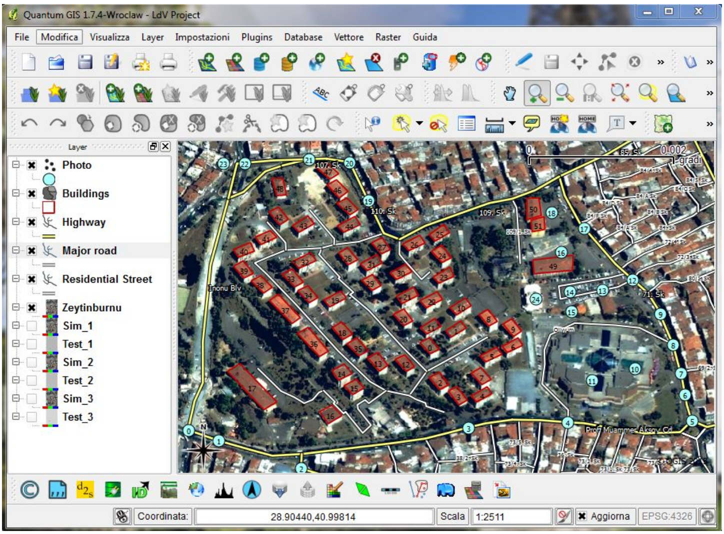
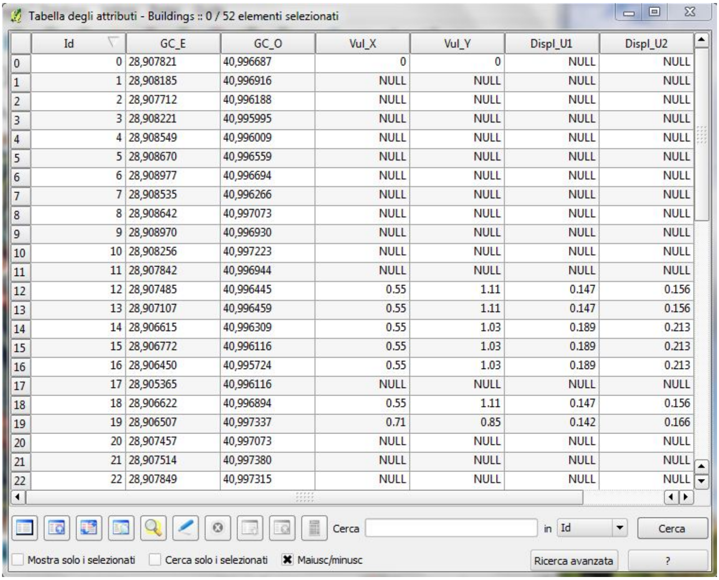


Figure 159. ***Layers Made in QGIS***



	Id	GC_E	GC_O	Vul_X	Vul_Y	Displ_U1	Displ_U2
0	0	28,907821	40,996687	0	0	NULL	NULL
1	1	28,908185	40,996916	NULL	NULL	NULL	NULL
2	2	28,907712	40,996188	NULL	NULL	NULL	NULL
3	3	28,908221	40,995995	NULL	NULL	NULL	NULL
4	4	28,908549	40,996009	NULL	NULL	NULL	NULL
5	5	28,908670	40,996559	NULL	NULL	NULL	NULL
6	6	28,908977	40,996694	NULL	NULL	NULL	NULL
7	7	28,908535	40,996266	NULL	NULL	NULL	NULL
8	8	28,908642	40,997073	NULL	NULL	NULL	NULL
9	9	28,908970	40,996930	NULL	NULL	NULL	NULL
10	10	28,908256	40,997223	NULL	NULL	NULL	NULL
11	11	28,907842	40,996944	NULL	NULL	NULL	NULL
12	12	28,907485	40,996445	0.55	1.11	0.147	0.156
13	13	28,907107	40,996459	0.55	1.11	0.147	0.156
14	14	28,906615	40,996309	0.55	1.03	0.189	0.213
15	15	28,906772	40,996116	0.55	1.03	0.189	0.213
16	16	28,906450	40,995724	0.55	1.03	0.189	0.213
17	17	28,905365	40,996116	NULL	NULL	NULL	NULL
18	18	28,906622	40,996894	0.55	1.11	0.147	0.156
19	19	28,906507	40,997337	0.71	0.85	0.142	0.166
20	20	28,907457	40,997073	NULL	NULL	NULL	NULL
21	21	28,907514	40,997380	NULL	NULL	NULL	NULL
22	22	28,907849	40,997315	NULL	NULL	NULL	NULL

Figure 160. ***Attribute Table by Layer Buildings***

9.3.2 Define Actions

QGIS provides to perform actions by the attributes associated with the individual elements of the vector layer. This you allows to perform a number of actions, for example, run a program with arguments built from the attributes of a feature or passing parameters to a web reporting tool.

Define “Actions” is useful to launch an external application or viewing a web page based on one or more values associated with the vector layer. To set an action, open the "Properties" dialog, click on the “Actions” and

provide a description of the action in the “Name” field. The “Action” must contain the name or path of an application that will be executed when the action is invoked.
The action can be made dependent on one or more fields in the attribute table and when it is invoked, each text string that starts with % followed by the name of a field in the attribute table will be replaced by the value of that field (Figure 161).

NOTE: The quotation marks (ex. "% Id") MUST be used to group text in a single argument to the program or command you want to execute. The dialog window "About results" that appears when you use the tool "Information Elements" has a voice (Actions) that contains information that is dependent on the type of layer questioned.

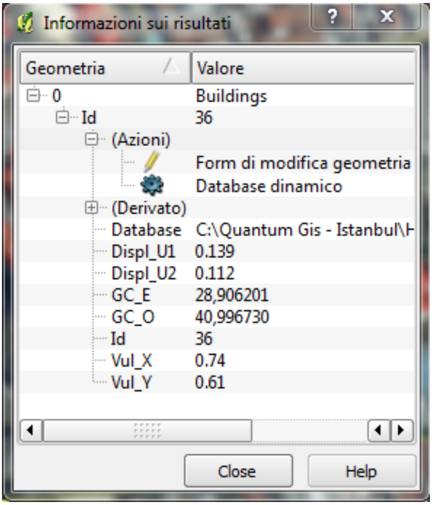


Figure 161. ***Window: About results***

9.3.3 Pre and Post-Event

When you conveyed all the information related to buildings in a single database, the system has been subjected to a series of simulations of seismic type which could cause the collapse of some of the elements analyzed. To do this you need to "dirty" the pixels of the original image relative to those structures that would collapse by its ability to move. For this purpose we used the software “Photo Shop”, whose graphic works were first georeferenced and then imported as raster GIS in the project shown above. If you want analyze the image pre- and post-event, you must use a plugin called "Raster Calculator" that is activated by the command "Raster".

The main function of this plugin is to produce a DIFFERENCE of pixels intensity by the two images and it allow us to plot on the screen only the areas of the buildings that have reached the collapse or instead is damaged. Then we got a unique process that allows us to associate a plugin like "visual" to a "structural parameter" and it allows us to speed up the buildings search damaged by a natural disaster or a seismic phenomenon.

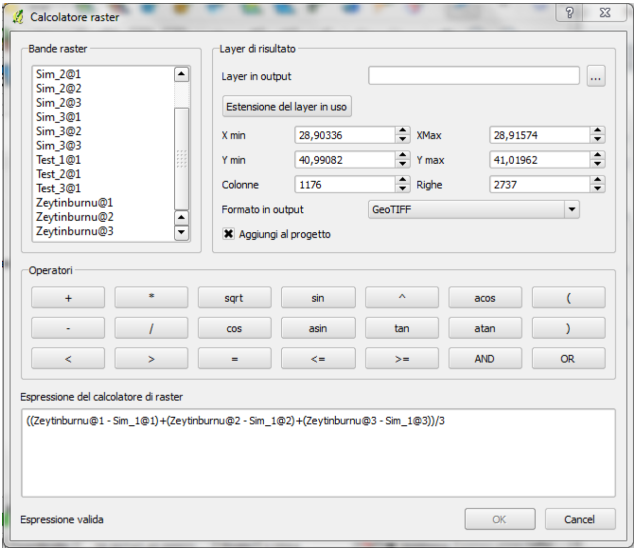


Figure 162. ***Raster Calculator***

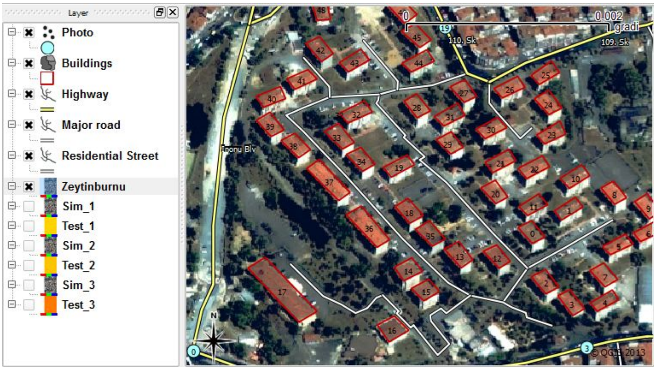


Figure 163. ***Zeytinburnu Region: pre-event***

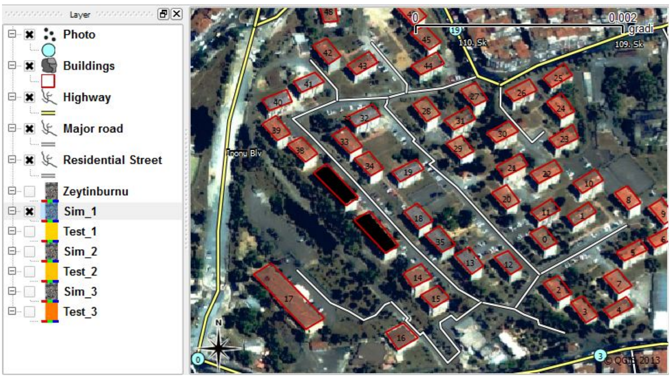


Figure 164. ***Simulation 1: post-event***

Figure 165. ***Simulation 1: pre and post-event***

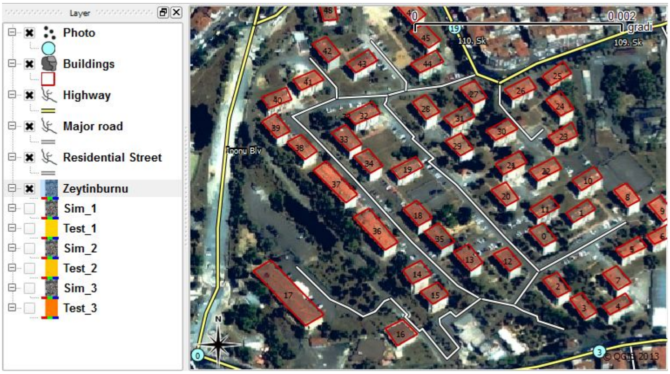


Figure 166. ***Zeytinburnu Region: pre-event***

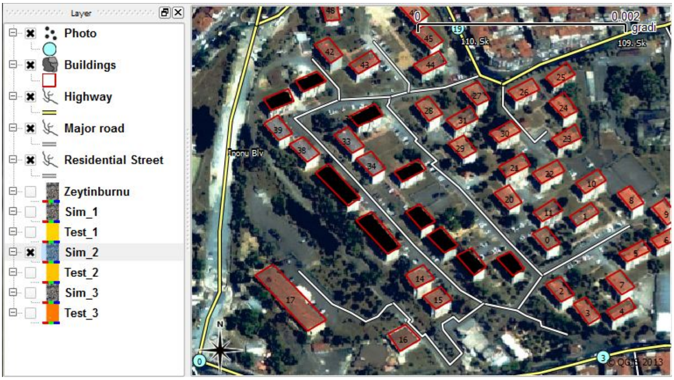


Figure 167. *Simulation 2: post-event*

Figure 168. *Simulation 2: pre and post-event*

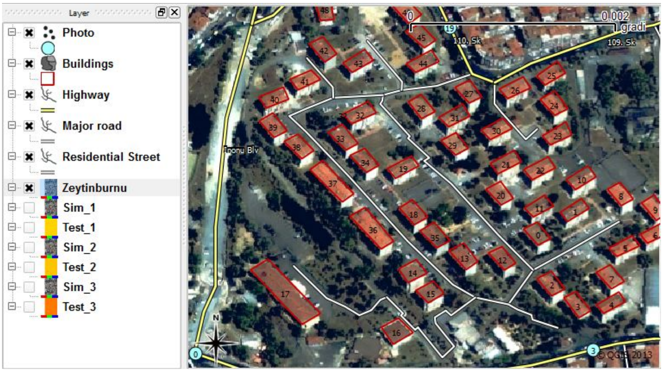


Figure 169. *Zeytinburnu Region: pre-event*

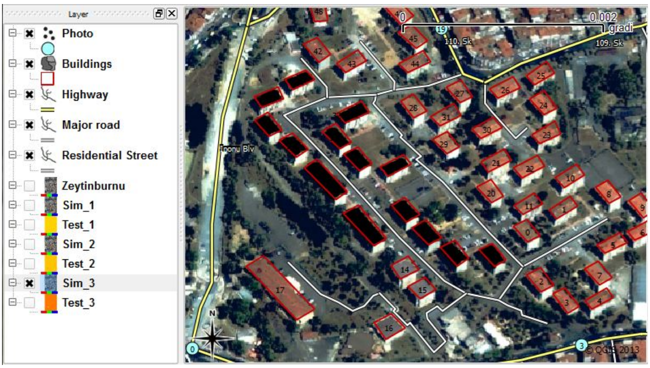


Figure 170. *Simulation 3: post-event*

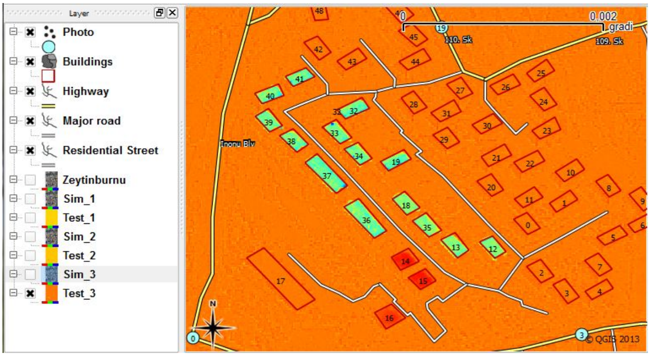


Figure 171. ***Simulation 3: pre and post-event***

Chapter 10. *Using OpenSees Software (Annex D)*

In this chapter we, will show the principal command used to realize the *ArcGIS* file for our purpose.

Keywords: OpenSees, Structural, Earthquake Engineering

10.1 OpenSees Software

OpenSees (i.e.: Open System for Earthquake Engineering Simulation)[3] is an open source software framework used to model structural and geotechnical systems, simulating their earthquake response.

This framework has been under development by the Pacific Earthquake Engineering Research Center (PEER) since 1997. Furthermore this software has been recently adopted as a NEESgrid simulation component.

10.2 Object-Oriented Software

OperSees is based on some concepts of Object-Oriented Programming (OOP). In fact an object is a bundle of variables and related methods, that are defined as an operation which can modify an object behavior. In other words, a method changes an object by manipulating its variables and so only an object method should modify its variables. A class is a blueprint for an object (i.e., it is a data structure, but not the memory allocated for the object). There is a process of allocating the memory to

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[3] For more information and/or teaching materials, please visit: http://opensees.berkeley.edu/

implement a class and it's named instantiation; from this one derives the inheritance which define a subclass functionally from another class definition.

Furthermore OpenSees is a C++ based FE software and it permits you to analyze a structural model by using Direct Integration Analysis.

10.3 OpenSees Features

In OpenSees one can find the library of materials, elements and analysis powerful tool for numerical simulation of non-linear systems; the library is very useful for the numerical-simulation models. The interface is based on a command-line which enables the user to create more-versatile input and it permits one to create the own material, element or other properties directly into OpenSees. Finally through the models one could do linear and non-linear structural and geotechnical analyses.

In addition, with this software one can do lot of simulations as follows:

- static push-over analyses
- static reversed-cyclic analyses
- dynamic time-series analyses
- uniform-support excitation
- multi-support excitation

10.4 OpenSees and Tcl

Tcl (Tool Command Language) is a string-based scripting language and interpreter, first developed by John Ousterhout. Tcl was also designed for easy learning, but it provides all the powerful functions the expert programmers want and OpenSees.exe is an extension of the Tcl interpreter for finite element analysis using OpenSees.

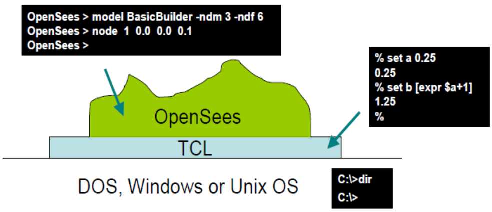

Figure 172. ***OpenSees and Tcl***

In addition, Tcl scripts are made up of commands separated by newlines or command syntax (e.g., **command arg1 arg2…**). The perfect cohesion between Tcl and OpenSees is demonstrated by lot of commands added for the finite element analysis: *modeling*, which creates nodes, elements, load and constraints, *analysis* which specifies the analysis procedure and *output specification* which specifies what it's you want to monitor during the analysis.

10.5 Defining a Problem in OpenSees

For an approach to OpenSees, it is very important defining a problem starting from the choosing of the kind of structure. First of all, one needs to define the geometry of the problem (that consists in nodes of the element, cross-section of the elements,…), then one could choose the materials of the structural elements. After that, operation is very important to choose the loads which act upon the elements and the analysis. Finally, the user is ready to do all the analyses needed.

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