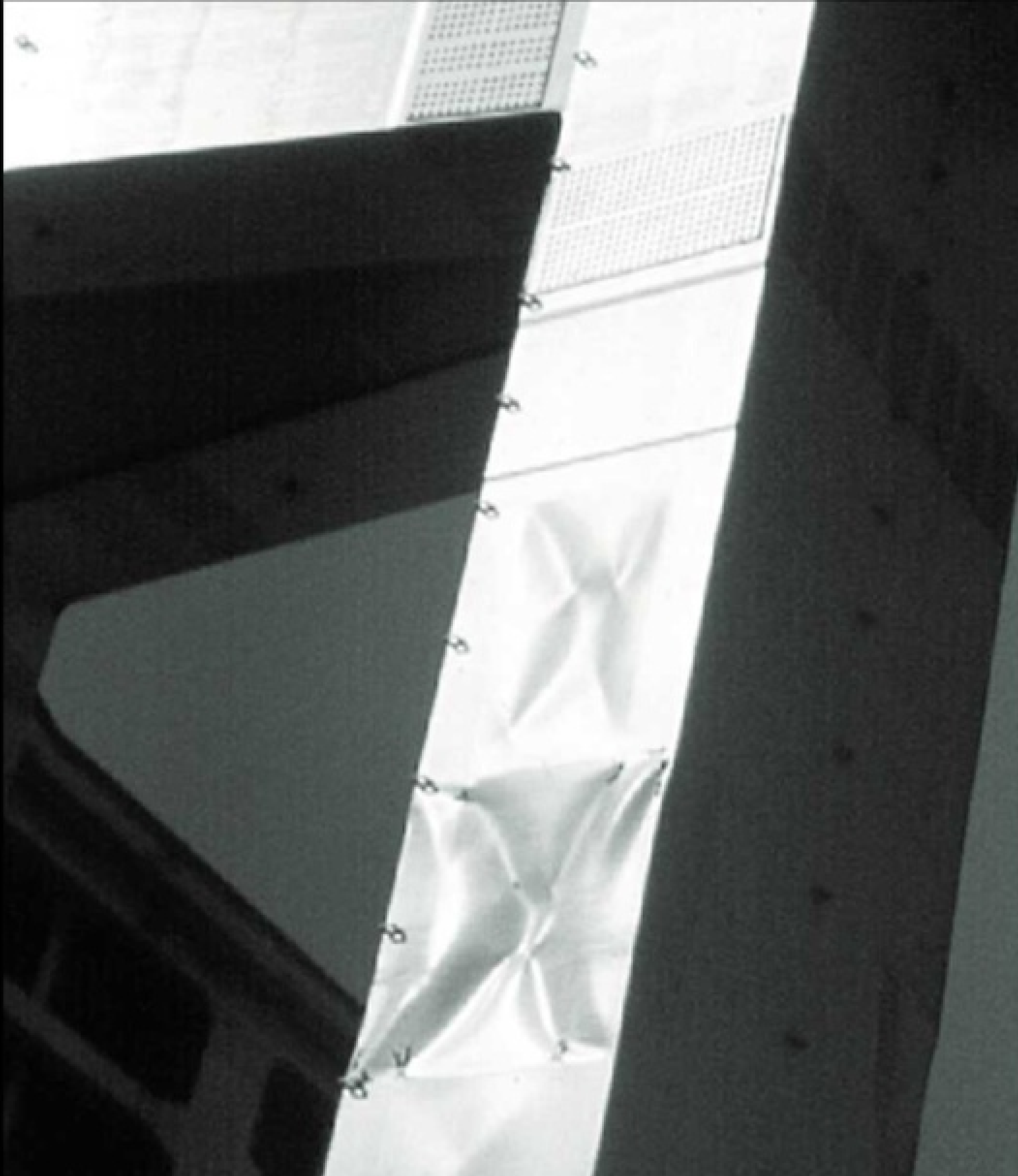

DUCTILITY OF SEISMIC RESISTANT STEEL STRUCTURES

Victor Gioncu and Federico M. Mazzolani

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To
Angela and Silvana

Contents

THE OAK AND THE REEDS
A violent storm uprooted an oak that grew on the bank of a river. The oak drifted across the stream and lodged among some reeds. Amazed that the reeds were still standing, the oak could not help asking them how they had escaped the fury of a storm that had torn him up by the roots. We bent our heads to the blast and it passed over us. You stood stiff and stubborn till you could stand no longer, they said. Moral: Sometimes it is better to bend with forces that are too strong to oppose.
Aesop's Fables

Preface

Earthquakes occur throughout the world. The earth is like a living body in constant motion. Every day small ground movements are registered in some parts of the world, every week a moderate earthquake is reported from some places. At least one significant earthquake causing damage and injury occurs every month, while every year two or three strong earthquakes fill the mass media with dramatic accounts of human and economic losses. Statistically, we can expect that somewhere each year there will be one earthquake of magnitude 8 or greater, 16 of magnitude 7 or greater, 150 of magnitude 6 or greater, and more than 1000 of magnitude 5. The globalisation of the mass media has not only meant that the whole world is informed of earthquake disasters but is also emotionally involved in efforts to save lives in the ruins of buildings. The problems of people in distant places have become everyones problems, and terms such as fault, epicentre, Richter and Mercalli scales, building structure, formerly used only by specialists, are nowadays part of everyday language.

Nowadays, earthquakes are capable of claiming more lives and doing more damage to the built environment than ever before. There are more people and buildings in earthquake-prone areas, which means more buildings, facilities, roads, bridges, dams, and so on are affected by earthquakes each year. Although seismic design has brought progress to engineering practice, there has been a marked increase in financial losses, because rapid and often uncontrolled urbanization and economic development in seismic areas has outpaced the gains from improvement in constructional methods. In any case, there are many old buildings that were erected before or during the early development of seismic design. Many of these were poorly built and are liable to collapse in moderate earthquakes, not just strong ones, killing more people than more recent buildings.

The collapse of both new and old buildings causes large loss of life. The real tragedy is that these human losses are often due not to the earthquakes themselves but to the failure of the construction of the builders. Builders thus become the makes of tools for killing people. Nowadays, for each life lost there is some person or organization that may be held liable for prosecution.

A building is the product of the activity of an interdisciplinary team. The architect is liable if the recently developed seismo-resistant architectural philosophy is ignored. The structural engineer, whether from ignorance or superficiality or weakness in resisting pressure from an architect more interested in the beauty of a building than its safety, may design or approve an unsafe structure. The constructional engineer, due to insufficient control of manufacture or use of poor structural materials, may erect a building that will fall victim to moderate or strong earthquakes. The owner, by poor monitoring and failure to maintain a building properly, can reduce its structural resistance. And last but not least, state or city authorities may be liable because they were slow to incorporate in building codes knowledge recently gained through theoretical and experimental research or through the examination of building behaviour in recent strong earthquakes. Further, lack of official interest in planning the strengthening of old buildings mark them as sure victims of the next earthquake. However, it should also be said that emphasising the deficiencies in the building process in this way holds out the hope that more effective organization of that complex process could substantially reduce the human and financial losses currently being incurred.

Another way of reducing the risk of such losses is improving earthquakeresistant design. In recent times, the defeatist attitude that an earthquake is a fatal force that it is not possible to resist is being transformed by tremendous progress. In the last thirty years, understanding of the nature of earthquakes, the effect of the site soil on the characteristic seismic properties, and of structural response of building subject to seismic waves, has made real advances. The installation of comprehensive instrumentation in high earthquake risk areas has provided a great amount of information about the main characteristics of ground motions. Examination of these records led to the recognition of the differences between near-field and far-field earthquakes, one of the most important recent contributions to advances in seismic design.

The great and costly damage caused by recent strong earthquakes has shown the need to develop a design methodology based on multi-level performance. Different aspects must be considered when a structure is designed for serviceability, susceptibility to damage or ultimate limit states. In this methodological framework, special attention must be paid to ductility as a key factor in resisting partial or global structural failure during very strong earthquakes. Here, ductility is understood as the ability of a structure to sustain large deformations in the plastic range without significant loss of resistance. As in Aesops fable, it is better for a structure to yield under large seismic forces that are too strong to be resisted. Unfortunately, in present design practice, such performance can be attained only through general construction rules. However, real progress in this area is offered by the possibility of ascertaining structural ductility at the same level as for displacement and strength: available ductility, determined at the level of a structural member, must be greater than required ductility, imposed at the level of the structure by an actual near-field or far-field earthquake.

In modern design practice is it generally accepted that steel is an excellent material for seismic-resistant structures because of its performance in terms of strength and ductility: it is capable of withstanding substantial inelastic deformation. In general this is true: the percentage of failure of steel structures has always been very small compared to other constructional materials. But in the last few decades, specialists have recognized that the so-called good ductility of steel structures under particular conditions may be an uncritical dogma that is denied by reality. In the decade 1985 1995, strong earthquakes in Mexico City (1985), Loma Prieta (1989), Northridge (1994), and Kobe (1995) have seriously compromised this ideal image of steel as the perfect material for seismic areas. The performance of steel structures in some cases was very bad, and the same type of damage was caused by different seismic events, clearly showing that there were significant shortcomings in current practice. So now seems the right moment for a critical analysis of progress recently made in conception, design and construction of buildings in seismic areas, to consider the lessons to be learned from these recent dramatic events. Of these lessons, improving the ductility of structures under unfavourable conditions takes a leading place.

The best way to look into the future is to understand the past. However, the international scientific community is also aware of the urgent need to investigate new topics and consequently to improve the current range of provisions for seismic design. The whole framework of modern seismic codes needs a complete review in order to determine and revise the design rules that failed in these recent earthquakes. The challenge for the immediate future is to transfer research achievements into practice, to bridge the gap that has opened between accumulated knowledge and design codes.

Accordingly, this book provides a state-of-the art review of the most advanced issues in the analysis of seismic-resistant steel structures, with the accent on the assessment of structure ductility as the most efficient method of preparing the structure to resist unexpected strong seismic events. At the same time it presents the most recent research results obtained by the authors, which in the near future can be used to improve existing building codes.

In organizing the book, the main idea has been to present the simplest possible formulations, even though these may be no more than approxi-mations of just one phenomenon only, rather than try to elaborate exact specifications. A high degree of exactness is not possible in seismic design because of uncertainties in input data on earthquake characteristics. The most refined and accurate method is useless if the values used in the specifications are not correctly determined.

Chapter 1 begins with a definition of ductility and a description of its place in structural design. Progress in design methods and challenges to building codes since the last strong earthquakes are then presented. Chapter 2 deals with the main lessons to learn from the Michoacan, Northridge and Kobe earthquakes, in which steel structures showed much unexpected damage. Chapter 3 discusses basic elements of design philosophy, such as multi-level design criteria, the modelling of ground motion, and structure conformation and design. Chapter 4 analyses ductility problems at the level of elements and materials, while Chapter 5 deals with the ductility of sections and stubs. Member ductility in a structure is the concern of Chapter 6, which also considers the effect of joints on structural behaviour. Chapter 7 reports the results of state-of-the-art theoretical and experimental research on the main section types used in structures. Finally, Chapter 8 sets out a comprehensive methodology for ductility design, and compares the required ductility for moment-resisting frames with the available ductility determined as the local level, both ductilities being determined using the plastic mechanism method. The Appendix presents the DUCTROT M computer program used to evaluate the rotation capacity of members working in a structure. A CD-ROM containing this program is attached to the back cover of this book.

The authors hope this book will serve as a guide for structural designers seeking to design more economical but safer steel structures, and to open new doors to future developments in the seismic design for all people interested in research and codification. However, they are aware that because the book is the first attempt to analyse the ductility of steel structures from a single, coherent point of view, it is likely to have many shortcomings and make assertions that are open to dispute. They refer readers who encounter these to the words of the wise Chinese author, who said This book would never have appeared if perfection had been awaited.

The present book concentrates on local ductility at the level of members. The authors intend to take their approach further by extending the analysis of ductility to the level of structure, in a book provisionally entitled Global Analysis of Seismic Resistant Steel Structures. They would be grateful for any comments and suggestions about the content of this first book that might help in preparing the second.

The authors are grateful to all colleagues who contributed to the research reported in this book. They would also like to thank those who helped prepare the book for publication, in particular to Emil Danetiu for the illustrations, and Dr Dana Petcu for elaborating the DUCTROT M computer program and the computerized setting of the text.

Victor Gioncu
Federico M.Mazzolani

Notation

Latin Small Letters

a—	plate length
a—	weld thickness
a—	geometrical dimension of buckled shape
a—	distance of vertical web stiffner from column face
a—	acceleration
a_0—	acceleration amplitude
a_g—	ground motion acceleration
a_s—	acceleration corresponding to SLS
a_d—	acceleration corresponding to DLS
a_u—	acceleration corresponding to ULS
b—	plate width
b—	flange width
b_e—	total width of material removed from the flange
b_{eff}—	effective width of concrete slab
c—	flange semi-width
c_a—	coefficient for plastic rotation accumulation
c_r—	reduced semi-width of weakened flange
C_T—	strain-rate coefficient considering the temperature influence
c_w—	strain-rate coefficient considering the welding influence
d—	epicentral distance
d—	web depth
d_c—	effective web depth in compression
d_{fc}—	distance from plastic neutral axis to compression flange
d_s—	distance from compression flange to web horizontal stiffener
e_0—	nondimensional eccentricity
f_y—	nominal yield stress
f_{yc}—	corner yield stress
f_{ya}—	actual yield stress
f_{yr}—	random yield stress

f_{yu}—	upper yield stress
f_{yl}—	lower yield stress
f_{yf}—	flange yield stress
f_{yw}—	web yield stress
f_{ysr}—	increased yield stress due strain-rate effect
f_{yt}—	through-thickness yield stress
f_{ymax}—	maximum random value of yield stress
f_{ymin}—	minimum random value of yield stress
f_u—	ultimate stress
f_{uf}—	flange ultimate stress
f_{uw}—	web ultimate stress
f_{usr}—	increased ultimate stress due strain-rate effect
f_{ut}—	through-thickness ultimate stress
f_{umax}—	maximum random value of ultimate stress
f_{umin}—	minimum random value of ultimate stress
g—	gravity acceleration
h—	focal depth
h—	section depth
h_c—	total depth of composite section
h_s—	slab thickness of composite section
i—	index
j—	index
k—	index
k—	plate buckling coefficient
k—	elastic rigidity
l—	index
m—	mass
m_b—	multiplier for plastic buckling moment
m_h—	multiplier for strain-hardening moment
m_p—	actual moment level related to full plastic moment
m_y—	multiplier considering the actual yield stress
m_M—	multiplier considering the maximum moment
n—	ductility criterium for stubs
n—	number of pulses until section fracture initiation
n_b—	number of pulses until flange buckling
n_r—	number of pulses after buckling until fracture initiation
n_p—	actual axial force level related to full plastic axial force

$n(>M)$—	number of events in one year having magnitude greater than magnitude M
p_r—	return period
q—	distributed beam load
q—	behaviour factor, reduction factor
q_μ—	strength reduction factor
q_s—	overstrength factor
r—	radius of flange-web junction
s—	multiplier for maximum moment
s—	coefficient for strain-hardening effect
t—	time
t—	thickness
t_f—	flange thickness
t_w—	web thickness
u—	horizontal displacement
v—	vertical displacement
v—	velocity
v_g—	ground motion velocity
v_o—	velocity at the outset of plastic deformations
v_p—	velocity of P wave
v_s—	velocity of S wave
w—	transverse plate displacement
w_a—	accumulate transverse plate displacement
w_i—	initial plate displacement
w_k—	weight of storey k
x—	axis
x—	distance from the neutral axis to the top of a composite section
x_i—	distance of beam inflection point from left end
x_m—	distance of beam maximum moment from the left end
z—	vertical displacement
z_p—	plastic vertical displacement

Latin Capital Letters

A—	total section area
A—	numerical coefficient for rigid-plastic mechanism behaviour
A_a—	reinforcement area for composite section

A_c— corner area

A_f— flange area

A_s— steel profile area for composite section

A_w— web area

B— normalized width-thickness ratio

B— numerical coefficient for rigid-plastic mechanism behaviour

C— numerical coefficient for rigid-plastic mechanism behaviour

C_i— initial cost of building

C_d— damage cost

C_t— total cost

D— damage index under Miner's assumption

D— ground motion duration

E— elastic modulus

E_h— strain-hardening modulus

E_r— reduced elastic modulus

E_s— secant modulus

E_y— energy corresponding to yield strain

E_u— energy corresponding to ultimate strain

F— plate axial loading

F_b— base shear force

F_k— horizontal force acting at storey k

F_{bd}— base shear force for DLS

F_{bs}— base shear force for SLS

F_{bu}— base shear force for ULS

F_d— design plate strength

F_e— elastic plate strength

F_p— plastic plate strength

F_u— ultimate plate strength

G— shear elastic modulus

G_p— shear plastic modulus

H— storey height

H— horizontal component of seismic load

H_k— height of k-storey from base

H_m— height of mass center

H_p— length of column plastic zone

H_s— structure height

I—	earthquake intensity
I—	moment of inertia
I_d—	damage index
I_w—	warping section constant
I_{dg}—	global damage index
I_{dm}—	member damage index
I_{ds}—	storey damage index
K—	non-dimensional coefficient of connection stiffness
K—	torsional of web stiffness
L—	stub length
L—	beam span
L_l—	left span for standard beam
L_r—	right span for standard beam
L_b—	length of buckled shape
L_p—	length of plastic zone
L_p—	loading potential
L_w—	length of web buckling shape
M—	magnitude
M—	bending moment
M_b—	bending moment for flange buckling
M_p—	full plastic moment
M_l—	left end moment of beam
M_r—	right end moment of beam
M_u—	ultimate moment
M_w—	bending moment due to vertical loads
M_y—	bending moment for the first yielding
M_{cu}—	upper column moment
M_{cl}—	lower column moment
M_{pb}—	plastic moment of beam
M_{pc}—	plastic moment of column
M_{pj}—	plastic moment of joint
M_{ph}—	moment in strain-hardening range
M_{pf}—	moment corresponding to flange plasticization
M_{pl}—	left end plastic moment
M_{psr}—	right end plastic moment
M_{pN}—	reduced plastic moment due to interaction with axial force
M_{pred}—	reduced plastic moment by weakening of flanges

M_{max}—	maximum moment
M_{uN}—	reduced ultimate moment due to interaction with axial force
N—	axial force
N_b—	buckling axial force
N_p—	full plastic axial force
N_f—	face axial component
N_c—	corner axial component
N_w—	axial force from structure weight
N_{ct}—	corner torsional axial component
N_{cr}—	critical axial load
P—	beam concentrate transverse load
S—	soil parameter
S_a—	acceleration spectral value
Sv—	velocity spectral value
S_d—	spectral value for DLS
S_s—	spectral value for SLS
S_u—	spectral value for ULS
S_{el}—	elastic spectral value
T—	period of vibration
T^{o}	temperature (Celsius grade)
T_c—	corner period
T_d—	period of damaged structure
T_g—	natural period of ground motion
T_{cd}—	corner period of DLS spectrum
$T(>M)$—	recurrence interval of an earthquake greater than magnitude M
T_{cs}—	corner period of SLS spectrum
U—	strain energy
U_z—	strain energy of plastic zone
U_l—	strain energy of yield line
V—	total potential energy
V—	vertical component of seismic load
V—	shear force
V_p—	plastic shear force
W—	total structure weight
Z—	elastic section modulus
Z_p—	plastic section modulus

Z_{pr}— reduced plastic section modulus

Greek Letters

α—	angle
α—	normalized slenderness
α—	multiplier of horizontal forces
α—	coefficient of pulse asymmetry
α_c—	collapse multiplier
α_d—	multiplier corresponding to DLS
α_f—	fracture rotation of buckled flange
α_y—	multiplier for first yield
α_N—	numerical coefficient for rigid-plastic stub behaviour
α_M—	numerical coefficient for rigid-plastic beam behaviour
β—	angle of inclined yield line
β—	parameter of buckled shape length
$\beta(T)$—	spectral amplification factor
β_d—	spectral amplification factor for DLS
β_s—	spectral amplification factor for SLS
γ—	parameter of plastic zone length
γ_s—	partial safety factor for seismic action
δ—	axial shortening of plate and stub
δ—	top sway displacement of structure
δ—	parameter of web buckled shape
δ_i—	initial axial shortening of stub
δ_p—	plastic axial shortening of stub
δ_u—	ultimate displacement
δ_y—	first yield displacement
ϵ—	normal strain
$\dot{\epsilon}$—	strain-rate
ϵ_h—	strain at the outset of strain-hardening
ϵ_t—	total strain
ϵ_u—	uniform strain
ϵ_y—	yield strain
ϵ_{sh}—	strain in hardening range
ϵ_{cu}—	ultimate strain of concrete

ζ—	numerical coefficient for plate fracture
η—	parameter for asymmetry of buckled web shape
η—	damper correction factor with reference value 1.0 for 5% viscous damping
θ—	rotation
θ_a—	accumulated rotation
θ_f—	fracture rotation
θ_i—	initial rotation
θ_m—	rotation corresponding to maximum moment
θ_p—	rotation corresponding to formation of plastic hinge
θ_r—	ultimate plastic rotation
θ_u—	ultimate rotation
θ_y—	yield rotation
θ_{rh}—	hysteretic plastic rotation
θ_{rk}—	kinematic plastic rotation
θ_{rr}—	required plastic rotation
θ_{rs}—	reduced rotation due to strain-rate effect
θ_{uc}—	ultimate rotation for cyclic action
λ_v—	lateral beam slenderness
$\bar{\lambda}$—	column normalized slenderness ratio
$\bar{\lambda}_f$—	flange normalized slenderness ratio
$\bar{\lambda}_w$—	web normalized slenderness ratio
μ—	ductility
μ_ε—	material ductility
μ_x—	curvature ductility
μ_θ—	rotation ductility
μ_θ—	rotation capacity
μ_δ—	displacement ductility
μ_E—	energy ductility
μ_d—	global ductility
μ_r—	local ductility
$\mu_{\theta c}$—	rotation capacity for cyclic action
$\mu_{\theta 0.9}$—	rotation capacity for 0.9 M_p ductility criterium
v—	reduction factor for SLS
v—	Poisson's ratio
v—	damping coefficient
v_m—	left to right plastic moments ratio

ξ— viscous damping ratio expressed in percent
ρ_{vsr}— yield ratio
σ— normal stress
σ_{vsr}— yield ratio for strain rate
σ— standard deviation
σ_b— buckling stress
σ_{cr}— critical stress
τ— shear stress
τ_y— shear yield stress
τ_o— rupture duration
χ— curvature
χ_b— buckling curvature
χ_h— curvature at the outset of strain-hardening
χ_u— ultimate curvature
χ_y— yielding curvature
χ_{max}— curvature at the maximum moment
ϕ— circular frequency of ground motion
ω— natural circular frequency
Δ— interstorey drift
Δ— plate displacement
Δ_i— plate initial displacement
Δ_c— displacement of compression flange
Δ_t— displacement of tension flange

Abbreviations

al— aluminum
hs— high strength steel
ms— mild steel
cd— constant displacement
id— increasing displacement
rd— random displacement
pd— pulse displacement
ip— inflection point
ph— plastic hinge
FO— full operational

O—	operational
LS—	life save
LB—	local buckling
LT—	lateral-torsional buckling
NC—	near collapse
SLS—	serviceability limit state
DLS—	damageability limit state
ULS—	ultimate limit state
MRF—	moment resisting frame
S-MRF—	special moment resisting frame
S-MRFd—	special moment resisting frame with limited interstorey drift
G-MRF—	global moment resisting frame
CBF—	concentrically braced frame
EBF—	eccentrically braced frame
DF—	dual frame
SSL—	space structure layout
PSL—	perimetral structure layout
SDOF—	single-degree of freedom system
MDOF—	multi-degree of freedom system
NDT—	nil-ductility temperature
FTP—	fracture transition plastic temperature
HD—	high ductility class
MD—	medium ductility class
LD—	low ductility class
SB 1—	standard beam type 1 for moment gradient
SB 2—	standard beam type 2 for quasi-constant moment
GSB—	generalized standard beam, including the joint effect
SSB—	strengthened standard beam
WSB—	weakened standard beam
RS—	rolled section
CRS—	cold rolled section
CFS—	cold formed section
WS—	welded section
CoFS—	concrete fillet section
MG—	moment gradient
CM—	constant moment

MT—	monotonic test
CT—	cyclic test
EC—	experimental calibration
PS—	parametrical study
BM—	beam member
BCM—	beam-column member
CP—	control parameter
COV—	coefficient of variation
PEEQ—	effective plastic strain index

1
Why Ductility Control?

1.1
Nature of the Problem

1.1.1
Main Purpose of Seismic Design

From a structural viewpoint, the design term refers to a synthesis of various disciplines of construction science, aiming to create a building that is of such a size and design that the demands for functionality, aesthetics, and resistance are satisfied at the same time in the same measure.

S*tructural engineering is the science and art of designing and making, with elegance and economy, buildings, bridges, frameworks, and other similar structures so that they can safely resist the forces to which they may be subjected.* (Petroski 1985).

The main purpose of structural design is to produce a suitable structure, in which we must consider not only the initial cost, but also the cost of maintenance, damage and failure, together with the benefits derived from the structure function. Thus, the optimum design of a structure requires a clear understanding of the role of each of above aspects and, therefore, requires a general view on the total process (Bertero 1996).

This objective can be achieved by the engineer, involved in designing a specific building, without great difficulty for conventional actions such as dead, live, wind, and snow loads, but with difficulty for exceptional loads produced by natural disasters such as hurricanes, tornadoes, floods, earthquakes, etc. Among these natural disasters, earthquakes are responsible for almost 60 per cent of deaths (Fig. 1.1, IAEE, 1992). In contrast to other natural disasters, which occur above the surface, earthquakes are the consequence of the release, in a very short period of time, of massive energy stored in the interior of the Earth, and, therefore, are very difficult to predict and model for analysis.

Powerful earthquakes are responsible for large losses of life and property. In the past the number of deaths were always very high, and continue to be so up to the present day. From analysis of data regarding losses from previous earthquakes, it is clear that the majority of lives lost was as a consequence of relatively few powerful earthquakes. In fact, during the

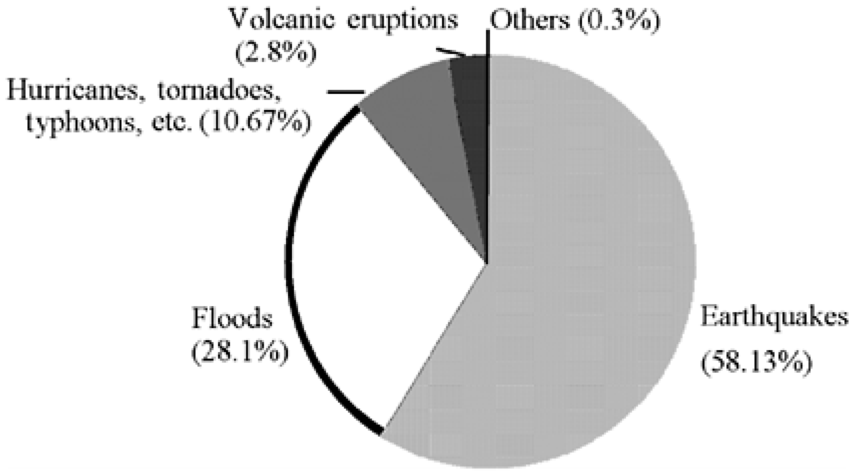


Figure 1.1: Percentage of deaths due to natural disasters

twentieth century, despite 100 events throughout the world of magnitude 7, only 20 earthquakes caused losses of life greater than 10,000 and only 2 greater than 200,000 (Dolce et al. 1995). In 1976, one of the most disastrous earthquakes claimed 250,000 lives in Tangshan, China.

Analysis of these events shows that it is not just ground motion severity that is responsible for this loss of life. Indeed, the main feature of earthquakes is that most human and economic losses are due to failure of human constructions—buildings, bridges, transport systems, dams, etc.—designed and built for ease of travel and comfort of humans. Major losses of life are always concentrated in poor regions with old buildings or with very poorly built constructions. In contrast, major economic losses are localized in rich regions with modern buildings, where, despite few building collapses, the resulting damage is significant and the cost of repairs very high.

We can see that buildings which are poorly designed and constructed suffer much more damage in moderate earthquakes than well-designed and constructed buildings in strong earthquakes. Moreover, these latter buildings should easily be able to withstand severe earthquakes with no loss of life and without severe damage. So, in many cases of building collapse, it is the builders themselves who are responsible for buildings that can kill. This is depressing, but at the same time encouraging, because it seems that earthquake problems are solvable (Bertero 1992), at least theoretically. For a well-designed and erected building, the risk of collapse during a strong earthquake is substantially reduced. It is the duty of building professionals to detect errors which contribute to building collapse and to improve the conceptual design and quality of buildings.

Generally, the engineering approach to design is quantitative and the structural members must be sized to have resistance greater than the actions caused by these events. But the design for the largest credibly imagined loads, resulting from the strongest expected earthquake on the structure site, is unreasonable and economically unacceptable. The design require-ments are set at a given level smaller than that associated with the largest possible loads. So, the structures occasionally fail to perform their intended function under these requirements which exceed the design values and, consequently, they may suffer local damage, by the loss of resistance in a single member or in a small portion of the structure. But a properly designed structure must preserve the general integrity, which is the quality of being able to sustain local damage, the structure as a whole remaining stable. This purpose can be achieved by an arrangement of structural elements that gives stability of the entire structural system. The local damaged portions must be able to dissipate a great part of seismic energy, being able to support important deformation in plastic range; the other parts remain in elastic range. The ability of a structure to undergo plastic deformations without any significant reduction of strength, represents the structural ductility, being a measure of the suitable structure behaviour during a severe earthquake. It is easy to understand that, in function of earthquake severity, there are different levels of ductility demands, and the ability to design a good structure is to supply it with sufficient available ductility.

But these excursions in plastic range cause damage, which must be repaired after the events and, in this perspective, the design process against earthquakes becomes a balance between the initial investment and the repair costs after the earthquake. This very difficult design philosophy was the subject of many research works (Waszawski et al, 1996) with very disputable results. So, in spite of the great efforts paid in recent years in solving satisfactorily this problem, recent earthquakes are capable of doing more damage today than ever before. Instead of observing a reduction of damage produced by earthquakes, a marked increase of financial losses results (Fig. 1.2). The years 94 (Northridge earthquake) and 95 (Kobe earthquake) reach the maximum of losses. The reason for this remarkable increase of losses in recent years is due to the fact that many buildings were constructed when Earthquake Engineering had not started, or was in its early stage. But, at the same time, many new buildings were damaged due to the concentration of population and industrialization in high seismic risk regions, the high vulnerability of modern technologies, and even because seismic codes are not infallible.

The last big events of Northridge and Kobe produced enormous damage, but they can be considered minor in comparison with potential losses in big cities like Mexico City, Los Angeles, San Francisco, Tokyo, etc. A recent calculation model predicted losses of US$100–150 billion for “The Big One” earthquake in California and over US$1000 billion in the Tokyo earthquake, if the 1923 earthquake happened today with the same magnitude.

So, the main purpose of modern seismic design is to reduce economic losses, and at the same time to save human life. The control of structure ductility is the key to solve this problem.

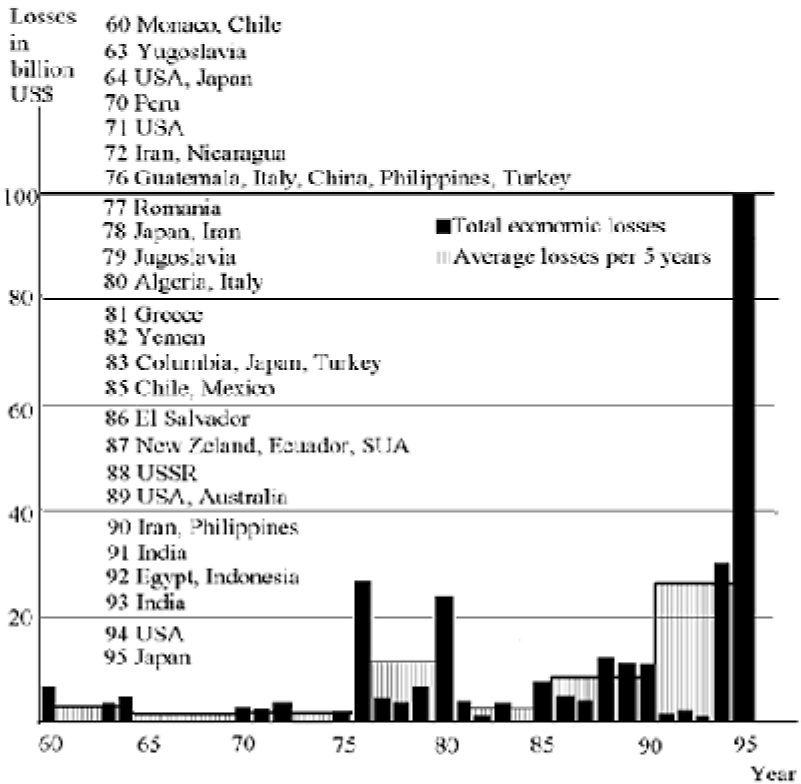


Figure 1.2: Cost of loses due to earthquakes from 1960 to 1995

1.1.2.
Main Lessons after the Last Strong Earthquakes

It is very important that some general features of the earthquakes that occurred recently should be seriously analysed:

(i) It is well known that all the attention is concentrated on the regions were earthquakes occurred in the past. At the same time, it can be observed that the major devastating earthquakes occurred in areas where no previous events have been recorded and where the current knowledge would suggest the existence of a quiescent area. The Tangshan-China and KillaryIndia are examples of such areas. So, there are few, if any, areas of world which are immune to earthquake effects and to which minimum protection can be considered in design. But in any case, all structures must be provided with some ductility properties, as a precaution against an unexpected earthquake.

(ii) Each event is unique, offering new surprises in the vulnerability of buildings affected by earthquakes and showing the great complexity of the phenomenon. Referring only to steel structures, as no serious damage during some major earthquakes was recorded for a long time, so the persuasion that steel structures are a very safe solution for seismic areas have been consolidated among the structural designers. But the 1985 Michoacan earthquake in Mexico City produced the first collapse of a high-rise steel building, due to the difference between required and available ductilities. The 1994 Northridge and 1995 Kobe earthquakes produced many failures in steel moment-resisting frames, which were not expected by the engineering community. These failures were mainly due to the near source position of the structures and also due to very high velocities recorded near the epicenters. These failures dealt a great blow to the use of steel structures in seismic areas, but at the same time there was the start of large activity in research works, in which the control of ductility for severe conditions plays a leading role.

(iii) For a long time the main purpose of seismic design was the protection of the public from loss of life or serious injuries and the prevention of building collapse under the maximum intensity earthquake. The second goal was the reduction of property damage. After each earthquake, attention is concerned only on the performance of structure and, therefore, very little is known about the vulnerability of non-structural elements, secondary structures, contents, installations, equipments, etc., which can produce more economic losses than the structural damage itself, even for a moderate earthquake. Only recently specialists have been informed that to only fulfil the condition of live protection is economically unacceptable. They realized that it is necessary to pay more attention to the reduction of damage of all building elements for all ranges of earthquake intensities. Consequently, a multi-level design approach has been developed and different levels of ductility demand are now used in design process, as a function of the considered seismic intensity.

1.1.3.
Required Steps for Control of Ductility Demand

The control of ductility demand requires attention to be given to some important aspects (Fig. 1.3):

(i) *Seismic macro-zonation,* which is an official zoning map to Country scale, based on a hazard analysis elaborated by geologists and seismologists. This map divides the national territory into different categories and provides each area with values of earthquake intensities, on the basis of design spectra. At the same time, this macro-zonation must characterize the possible ground motion types, as a surface or a deep source, an interplate or intraplate fault, etc. The ductility demands are very different for each ground motion type.

(ii) *Seismic micro-zonation,* which considers the possible earthquake sources at the level of region or town, on the basis of common local investigation of seismologists and geologists. The result of these studies is a local map, indicating the positions and the characteristics of the sources, general informations about the soil conditions and design spectra. It is very useful to accompany the time-history accelerograms with very precise

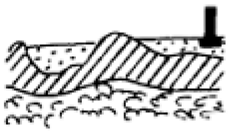
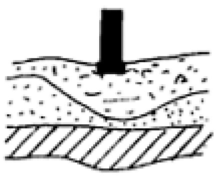
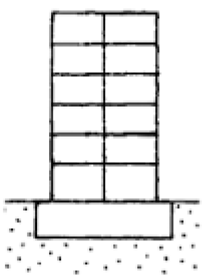
ACTIVITY	SCHEME	AUTHORS	INFORMATION
MACRO-ZONATION		• geologists • seismologists	• type of possible earthquakes • magnitude • intensities
MICRO-ZONATION		• geologists • seismologists	• source position • magnitude • intensities • attenuation • duration
SITE CONDITION		• geologists • geotechnical engineers	• soil stratification • framing in soil type • duration • time-history records • design spectrum
STRUCTURE CHARACTE-RISTICS		• geotechnical engineers • structural engineers • mechanical engineers • architects • builders • owner	• level of protection • general configuration • materials • foundation type • structure type

Figure 1.3: Steps in control of ductility demand

indications about the place where they have been recorded (directions, distance from epicenters, soil condition, etc.). Recordings such as magnitude, distance from source, attenuation, duration, etc., are directly involved in ductility demand.

(iii) *Site conditions,* established by geologists and geotechnical engineers, from the examination of the stratification under the proposed structure site. This is a very important step, because dramatic changes of earthquake characteristics within a few hundred of meters distance is not an unusual observation during an earthquake. These differences are mainly caused by the different soil conditions. The geotechnical engineers must provide exact information about the framing of local conditions in the soil type, in function of code demands. For soft soil, the ductility demand is more important than for rigid soil. The geotechnical engineers must propose the most suitable design spectrum or time-history records, taking into account of soil conditions.

(iv) *Structure characteristics,* result from the collaboration of geotechnical, structural and mechanical engineers, architects, builders and owners. At this step, the levels of protection are established and the ductility demand is fixed as a function of these levels. This is the main step in the design process; the good or bad behaviour of the structure during a strong earthquake depends on the decisions taken during these discussions. General configuration, structural material, foundation and elevation structural types, technology of erection, etc., result from this activity.

1.2 Evolution Process of Ductility Concepts

In comparison with the other branches of structural engineering, the design of seismic resistant structures is a relatively new branch, with first attempts being developed only at the beginning of this century, and the most important concepts being achieved during the last 40 years. The evolution of design concepts is characterized by a continuous flow of information between the architects, structural engineers, geotechnists and seismologists.

The history of development of ductility concepts is divided into three principal periods (Fig. 1.4).

1.2.1. Early Development

The preliminary design concepts commenced after the severe earthquakes at the beginning of the 20th century. The great builder Gustave Eiffel had the intuition to model the earthquake forces by means of an equivalent wind load. The San Francisco city was rebuilt after the 1906 great earthquake using a 1.4kPa equivalent wind load. It was not until after the Santa Barbara earthquake in 1925 and the 1933 Long Beach earthquake, that the concept of lateral forces proportional to mass was introduced into practice. The buildings have been designed to withstand lateral forces of about 7.5 percent for rigid soil and 10 percent for soft soil of their dead load. This rule constituted due to the observation that the great majority of well designed and constructed buildings survived strong ground motions, even if they were designed only for a fraction of the forces that would develop if the structure behaved entirely linearly elastic (Fajfar, 1995). In 1943, the Los Angeles

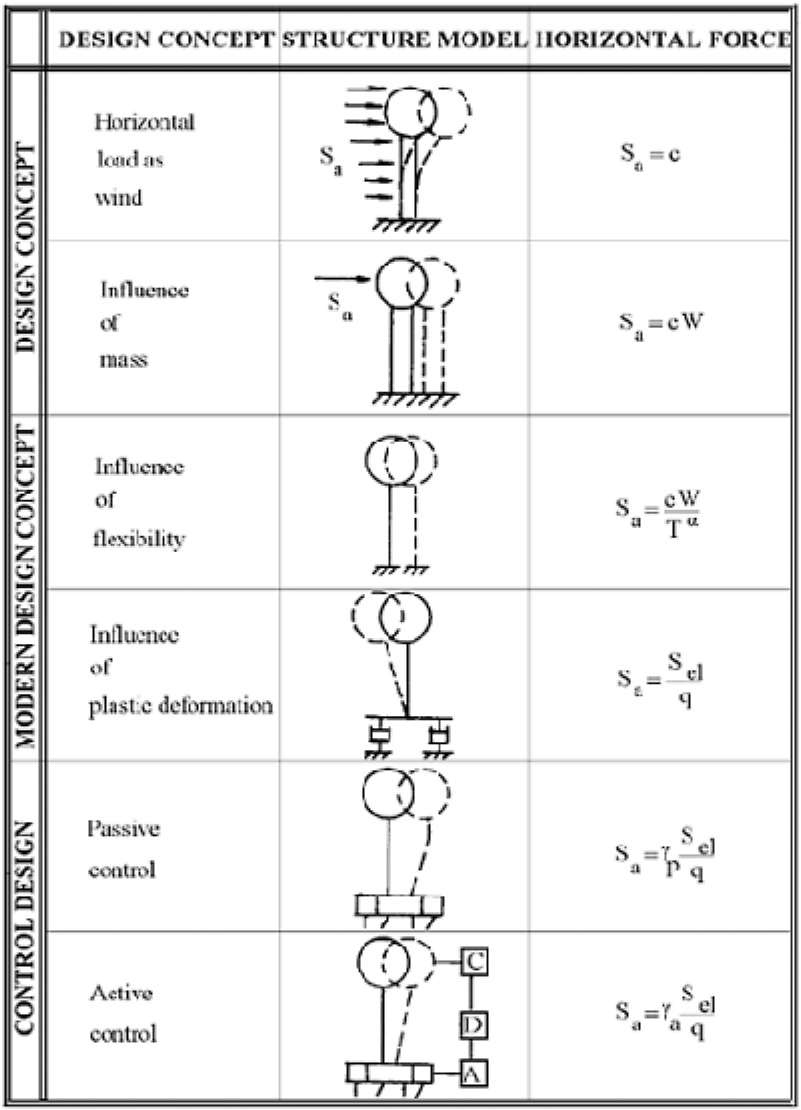


Figure 1.4: Evolution of design concepts

city code recognized the influence of flexibility of structures, and considered the number of structure levels in the design forces. The first provisions where the influence of the fundamental period of structure were introduced, were the San Francisco recommendations, introducing a relation stating that seismic forces are inversely proportional to this period (Bertero, 1992, Popov, 1994).

These preliminary concepts are based on grossly simplified physical models, engineering judgment and a number of empirical coefficients. Influenced by the conventional design concepts, the earthquake actions are considered as static loads and the structures as elastic systems. This simple concept has been the standard design methodology for several decades. There are good reasons for the success of this design approach. This methodology has been well understood by structural engineers because it is relatively easy to be implemented. In most cases this approach helps professional activity, but in some cases, it may lead to inadequate protection (Krawinkler, 1995). Because of these limits new concepts have been developed.

1.2.2. Modern Design Concepts

The beginning of the modern design concepts may be fixed in the 1930s, when the concepts of response spectrum and plastic deformation were introduced to earthquake engineering. The first concept considering the elastic response spectrum was used by Benioff in 1934 and Biot in 1941 (Miranda, 1993). Linear elastic response spectra provide a reliable tool to estimate the level of forces and deformations developed in structures. In 1935 Tanabashi proposed an advanced theory, which suggested that the earthquake resistance capacity of a structure should be measured by the amount of energy that the structure can absorb before collapse. In terms used nowadays, this energy can be interpreted as the dissipated energy through the ductility of structure (Takanashi and Nakashima, 1994).

The first attempts to combine these two aspects, the response spectrum and the dissipation of seismic energy through plastic deformations, was made by Housner (1956, 1959), who made a quantitative evaluation of the total amount of energy input that contributes to the building response, using the velocity response spectra in the elastic system, and, assuming that the energy input, responsible for the damage in the elastic-plastic system, is identical to that in the elastic system (Akiyama, 1985). Housner verified his hypothesis by examining several examples of damage. So, his method proposed a limit design type analysis to ensure that there is sufficient energyabsorbing capacity to give an adequate factor of safety against collapse in the event of extremely strong ground motion. The first study on the inelastic spectrum was conducted in 1960 by Velestos and Newmark. They obtained the maximum response deformation for the elastic-perfectly plastic structure. Since its first application in seismic design, the response spectrum has become a standard measure of the demand of ground motion. Although it is based on a simple single-degree-of-freedom linear system, the concept of the response spectrum has been extended to multi-degree-of-freedom systems, nonlinear elastic systems and inelastic hysteretic systems. The utility of the response spectrum lies on the fact that it gives a simple and direct indication of the overall displacement and acceleration demands of earthquake ground motion, for structures having different period and damping characteristics, without needing to perform detailed numerical analysis.

A new concept was proposed in 1969 by Newmark and Hall, by constructing spectra based on accelerations, velocities, and displacements, in short, medium and long period ranges, respectively. This concept remained a proposal until after the Northridge and Kobe earthquakes, when the importance of velocity and displacement spectra was recognized.

More recently, for structures situated in near-field region of an earthquake, another methodology has been elaborated (Iwan, 1997), based on the drift spectrum of a continuous medium, in opposition with the concept of discrete medium. This concept is based on the observation that the ground motions in near-field regions are qualitatively different from that of the commonly used far-field earthquake ground motions. For near-field earthquakes, the use of the equivalence of multi-degree-of-freedom systems with only one degree-of-freedom gives inaccurate results, because the importance of the superior vibration modes is ignored. So, a new direction of research works for ductility of structures in near-field regions began to be explored.

Since the early 70s a crucial change in seismic design concept has taken place, thanks to the availability of personal computers and the implementation of a great number of programs for structural engineering, which very easly perform static and dynamic analyses in elastic and elasto-plastic ranges. This technological advance allow us to obtain more refined results, and gives to the researchers the perspective to improve the methodology of using the design spectra in current practice, with a more correct calibration of design values. At the same time for important structures, a time-history methodology, using real recorded accelerograms, can be applied and the behaviour of structures under seismic actions can be evaluated in a more precise way, according to the spectrum methodology.

But this concept has been criticized in recent years due to the fact that large deformations, such as those necessary for the building components to provide the required ductility, are associated for strong earthquakes with local buckling, cracking and other damage in structural and non-structural elements, with a very high cost of repairing after each event. In order to minimize this damage, a new approach in seismic design has been developed, mainly based on the idea of controlling the response of the structure, by reducing the dynamic interaction between the ground motion and the structure itself.

1.2.3.
Response Control Concept

A significant progress has been recently made in the development and application of innovative systems for seismic protection. The aim of these systems is the modification of the dynamic interaction between structure and earthquake ground motion, in order to minimize the structure damage and to control the structure response. So, this concept is very different from the conventional one, according to which the structure is unable to behave successfully when subjected to load conditions different from the ones it has been designed.

The control of the structural response produced by earthquakes can be done by various means, such as modifying rigidities, masses, damping and providing passive or counteractive forces (Housner et al, 1997). This control is based on two different approaches, either the modification of the dynamic characteristics or the modification of the energy absorption capacity of the structure. In the first case, the structural period is shifted away from the predominant periods of the seismic input, thus avoiding the risk of resonance occurrence. In the second case, the capacity of the structure to absorb energy is enhanced through appropriate devices which reduce damage to the structure (Mele and

De Luca, 1995). Both these approaches can be implemented in passive, active or hybrid systems.

(i) *Passive systems* are systems which do not require an external power source. The properties of the structure (period and/or damping capacity) do not vary depending on the seismic ground motion. The base isolation or damping devices serve as the first line of defence against seismic forces, leaving the structure itself and its inelastic reserve of strength as a second defence line. So the structures receive only a part of seismic forces, the rest being dissipated by the behaviour of the devices (Romero, 1995).

(ii) *Active systems* are systems in which an external source of power controls the actuators. Thus, in these systems, the structure's characteristics are modified just as a function of the seismic input. The modifications are obtained by integrating within the structural system a control system consisting in three main components: sensors, interpretation and decision systems/actuators. Thus, the structures are able to determine the present state, to decide in a rational manner on a set of actions which would change its state to a more desirable state and to carry out these actions in a controlled manner and in a short period of time. The goals of active systems are to keep forces, displacements and accelerations of structure below specific bounds, in order to reduce the damage in case of strong earthquakes.

(iii) *Hybrid systems* are systems implying the combined use of passive and active control systems. For example, a base isolated structure is equiped with actuators which actively control the enhancement of its performance.

Recently response control systems, including seismic isolation, have been gradually applied to various structures, e.g. buildings, highway bridges and power plants. The response control systems are utilized not only for the new structures but also for existing structures to retrofit them (Mazzolani et al, 1994a,b).

The response control systems are classified as shown in Fig. 1.5 (ISO 3010, 1998) and illustred in Fig. 1.6. All systems, except active and hybrid control systems, can be classified into passive control systems. The seismic isolation is to reduce the response of the structure by the isolators which are usually installed between the foundation and the structure. In the case of suspended structures, the isolators can be located on the top of the building (Mazzolani, 1986, Mazzolani and Serino, 1997a). Since the isolators elongate the natural period of the structure and dampers increase damping, the acceleration response is reduced, as shown in Fig. 1.7, but the large relative displacement occurs at the isolator installed story. Energy ab-

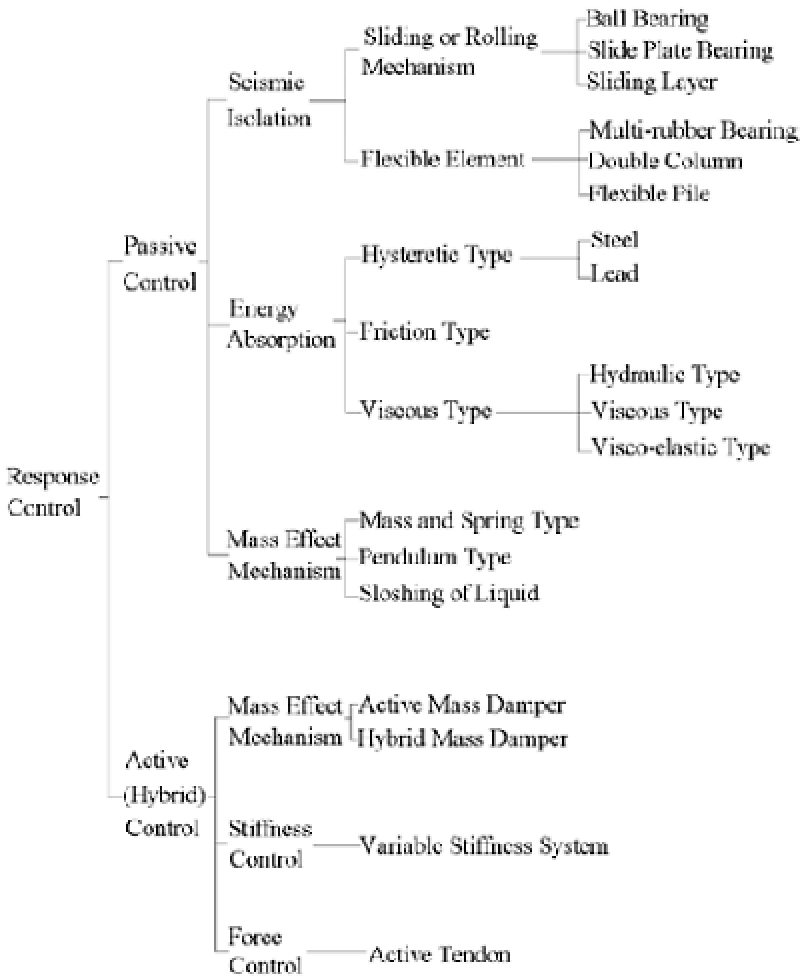


Figure 1.5: Classification of response control systems

sorption devices and additional masses to structure are also used to control the response. The energy absorption devices increase the damping within the structure by plastic deformation or viscous resistance of the devices. In some cases the use of oleodynamic devices can protect the structure from the formation of plastic hinges (Mazzolani and Serino, 1997b). The response of a structure is also reduced by vibration of additional masses and liquid materials. The active response control systems reduce the response of a structure caused by earthquakes and winds using computer controlled additional masses or tendons.

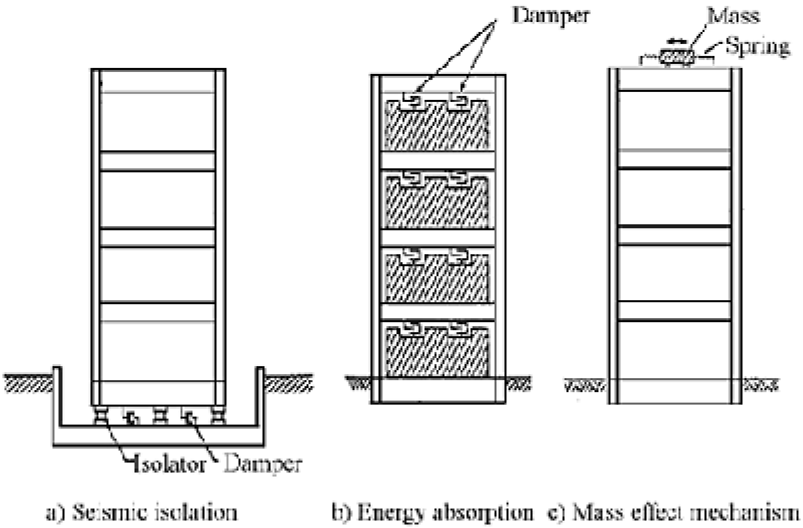


Figure 1.6: Examples of schemes of response control systems

The response control systems are used to reduce floor response and interstory drift. The reduction of floor response may ensure seismic safety, improve habitability, ease mental anxiety, protect furniture from overturning, etc. The reduction of inter-story drift may decrease the amount of construction materials, reduce damage to non-structural elements and increase design freedom.

During the Northridge and Kobe earthquakes, while many conventional structures suffered excessive damage and even partial or total collapses, some base isolated buildings located in zones close to the epicenter experienced successfully the first severe field tests (De Luca and Mele, 1997).

The concept of response control is a very promising strategy, but there are some limitations in using this system:

-there are situations where more than one source are depicted in the same region and, generally, these sources have different characteristics. It is very difficult to design a control system which has a variable response in function of ground motion type;

-it is not technically possible to design a control system which assures that the structure remains elastic during a strong earthquake. An open question is the behaviour of a structure when it falls in the inelastic range. The development of plastic hinges could in fact reduce the difference in period between fixed base and isolated schemes, reducing the effectiveness of isolators and leading to fast deterioration of dynamic response. In some cases, a sudden increase of damage is observed at some level of acceleration (Ghersi, 1994, Mazzolani and Serino, 1993);

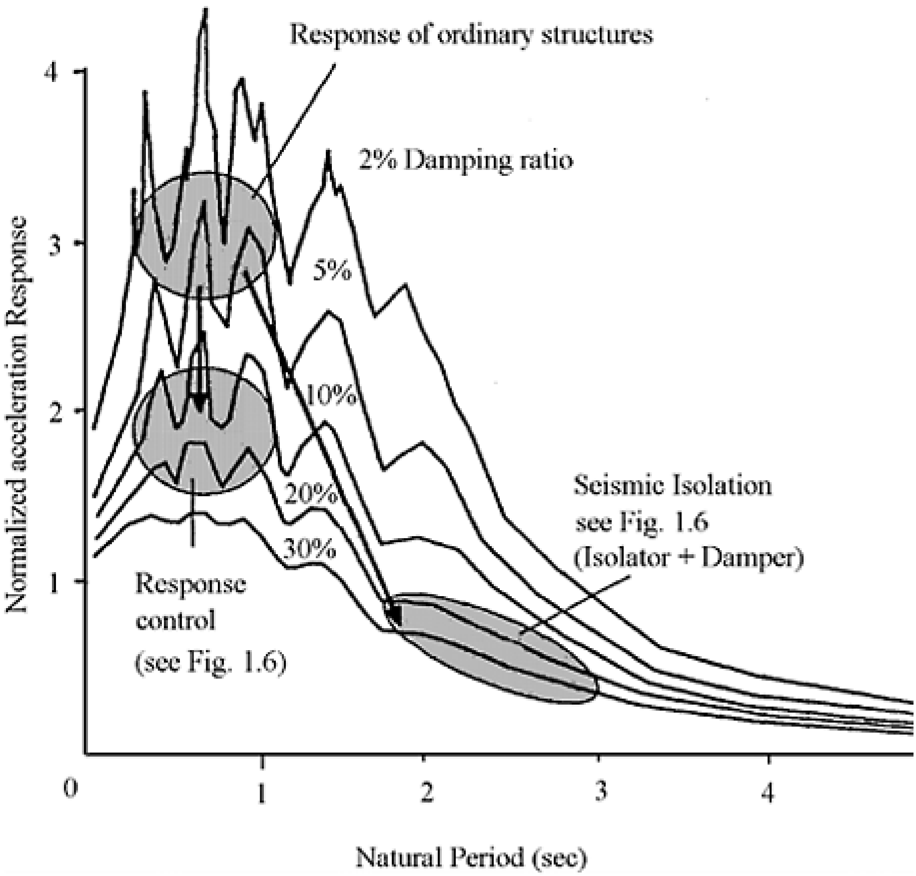


Figure 1.7: Examples of schemes of response control systems

-in case of near-field ground motion, as the energy content and velocity are very high, the required isolator displacements are very large and very often exceed the available displacements of the used isolators. In these cases, a high impact load to the isolated portion of building results (Iwan, 1995);

-when the vertical displacements are very high (as for near-field zones), the efficiency of devices for response control is disputable.

Thus, even in cases of response control, the ductility control remains a very important method of preventing any unexpected behaviour of a structure during severe earthquakes.

1.3
Leading Role of Ductility in Structural Design

1.3.1.

Ductility Definition

Before the 1960s the ductility notion was used only for characterizing the material behaviour. After the Housner's studies of earthquake problems and Baker's research works on plastic design, this concept has been extended to a structural level.

In the common practice of earthquake resistant design, the term *ductility* is used for evaluating the performance of structures, by indicating the quantity of seismic energy which may be dissipated through plastic deformations. The use of the ductility concept gives the possibility to reduce the seismic design forces and allows to produce some controlled damage in the structure also in case of strong earthquakes.

In the practice of plastic design of structures, ductility defines the ability of a structure to undergo deformations after its initial yield, without any significant reduction in ultimate strength. The ductility of a structure allows us to predict the ultimate capacity of a structure, which is the most important criteria for designing structures under conventional loads.

The following ductility types are widely used in literature (Fig. 1.8) (Gioncu, 1999):

-material ductility, or deformation ductility, which characterizes the material plastic deformations for different loading types;

-cross-section ductility, or curvature ductility, which refers to the plastic deformations of the cross-section, considering the interaction between the parts composing the cross-section itself;

-member ductility, or rotation curvature, when the properties of members are considered;

-structure ductility, or displacement ductility, which considers the overall behaviour of the structure;

-energy ductility, when the ductility is considered at the level of dissipated seismic energy.

A correlation among these types of ductility exists. The energy ductility is the cumulation of structure and member ductilities; the member ductility depends on cross-section and material ductilities. There are many disputable problems in the above definitions, due to the fact that they have a precise definition and quantitative meaning only for the idealized case of monotonic linear elasto-perfectly plastic behaviour. Their use leads to much ambiguity and confusion in actual cases, where the structural behaviour significantly differs from the idealized one (Bertero, 1988).

A very important value in seismic design is the ductility limit. This limit is not necessarily the largest possible energy dissipation, but a significant changing of structural behaviour must be expected at ductilities larger than these limit ductilities. Two ductility limit types can be defined (Gioncu, 1997, 1998):

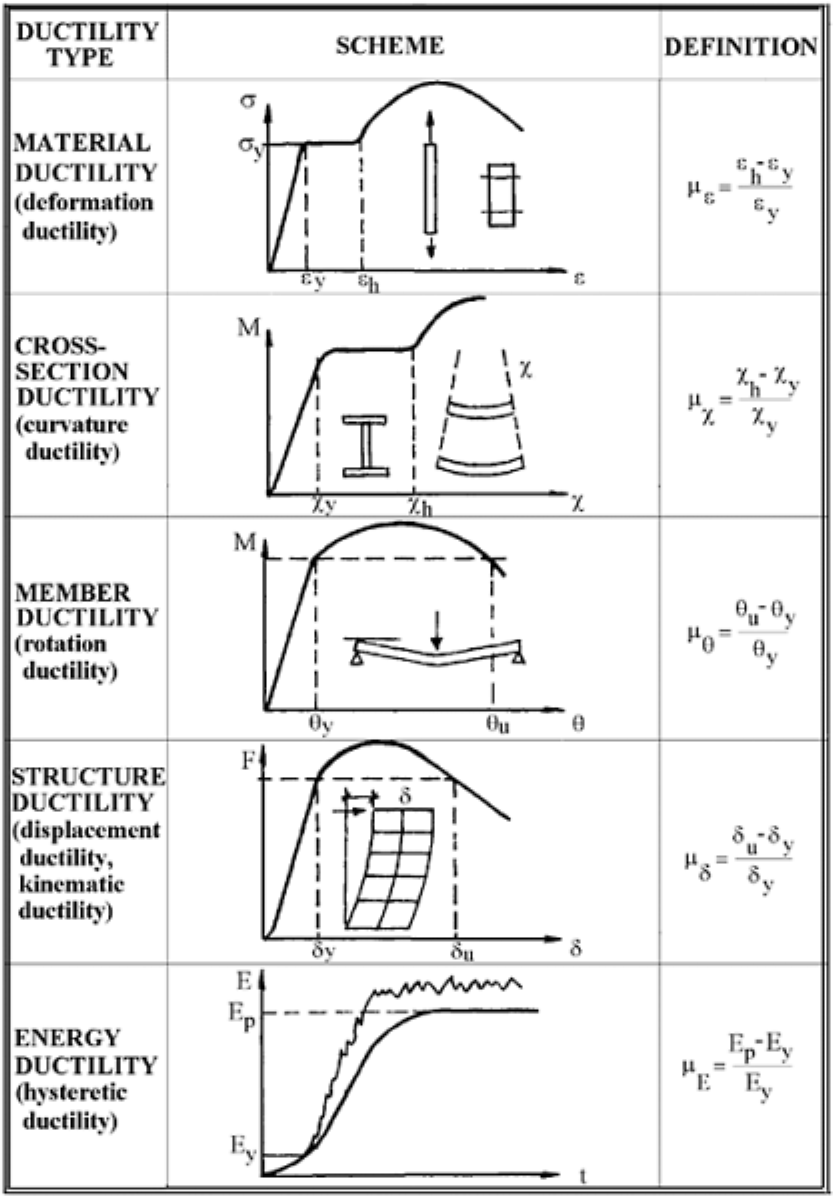


Figure 1.8: Ductility types

-available ductility, resulting from the behaviour of structures and taking into account its conformation, material properties, cross-section type, gravitational loads, degradation in stiffness and strength due to plastic excursions, etc.;

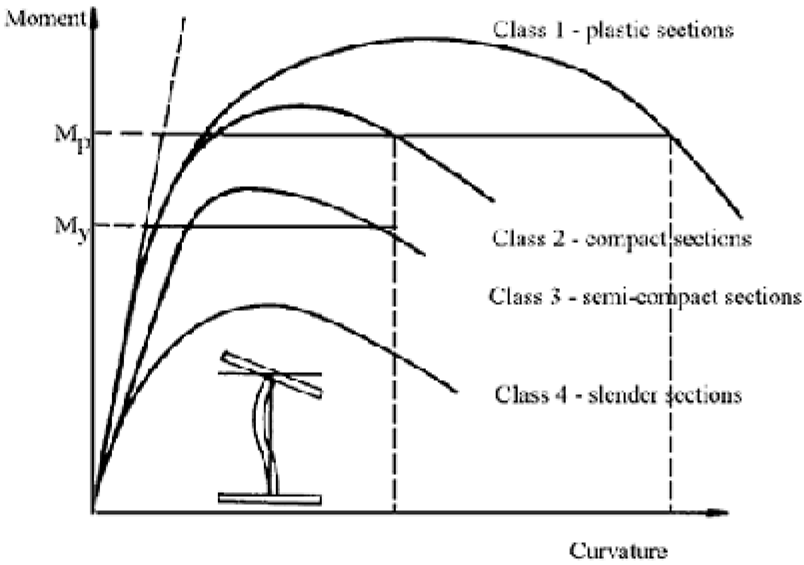


Figure 1.9: Cross-sectional behaviour classes

-required ductility, resulting from the earthquake actions, in which all factors influencing these actions are considered: magnitude, ground motion type, soil influence, natural period of structure versus ground motion period, number of important cycles, etc.

1.3.2. Ductility for Plastic Design

The plastic behaviour of a structure depends upon the amount of moment redistribution. The attainment of the predicted collapse load is related to the position of plastic hinges, where sections reach the full plastic moment, and to the plastic rotation which other hinges can develop elsewhere. Hence, a good behaviour of a plastic hinge requires a certain amount of ductility, in addition to its strength requirement. The plastic rotation capacity is the more rational measure of this ductility.

The basic requirement for plastic analysis of statically undetermined structures is that large rotations (theoretically infinite) are possible without significant changes in the resistant moment. But these theoretical large plastic rotations may not be achieved because some secondary effects occur. The limitation to plastic rotation is usually given by flexural-torsional instability, local buckling or brittle fracture of members. Due to this reduction in plastic rotation, the cross-section behavioural classes are used in design practice (Fig. 1.9):

-class 1 (plastic sections); sections belonging to the first class are characterized by the capability to develop a plastic hinge with high rotation capacity;

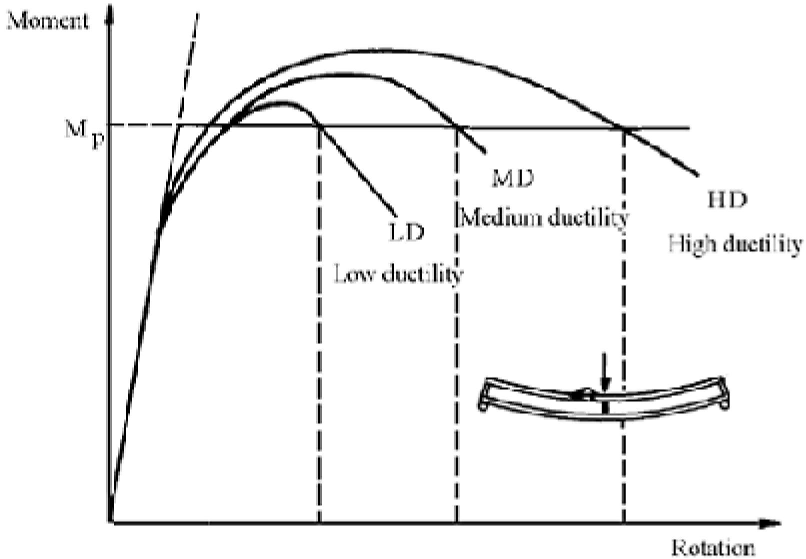


Figure 1.10: Member behavioural classes

-class 2 (compact sections); second class sections are able to provide their maximum plastic flexural strength, but they have a limited rotation capacity, due to some local effects;

-class 3 (semi-compact sections); sections fall in the third class when the bending moment capacity for the first yielding can be attained, without reaching the plastic moment;

-class 4 (slender sections); sections belonging to this class are not able to develop their total flexural resistance due to the premature occurrence of local buckling in their compression parts.

Evidently, only the first two classes have sufficient ductility to assure the plastic redistribution of moments.

This classification is limited at the cross-section level only, so it has many deficiencies. Another more effective classification at the level of a member has been proposed by Mazzolani and Piluso (1993) (Fig. 1.10):

-ductility class HD (high ductility) corresponds to a member for which the design, dimensioning and detailing provisions are such that they ensure the development of large plastic rotations;

-ductility class MD (medium ductility) corresponds to a member designed, dimensioned and detailed to assure moderate plastic rotations;

-ductility class LD (low ductility) corresponds to a member designed and dimensioned according to general code rules which assures low plastic rotations only.

These classifications used for the plastic design are also very useful for earthquake design. But some corrections must be introduced, due to the

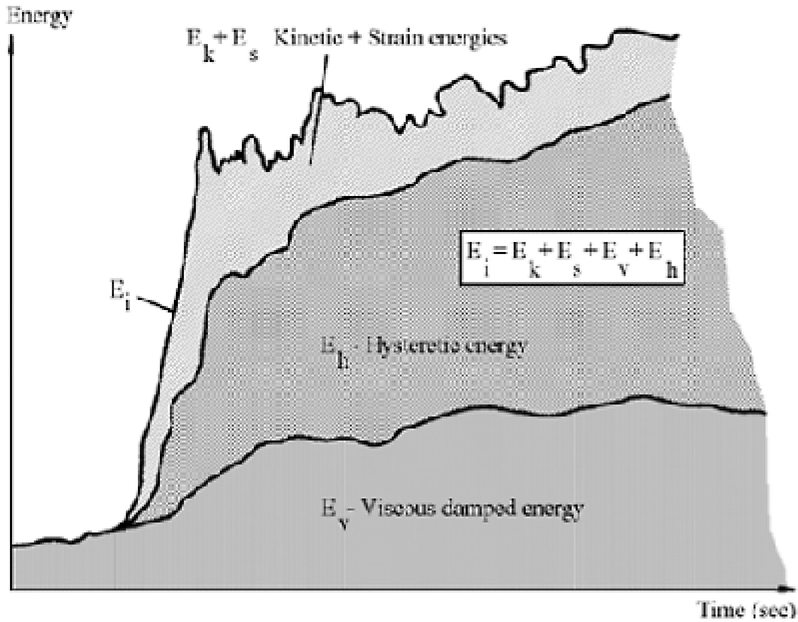


Figure 1.11: Seismic input and energy balance

fact that in the plastic design the loading system is monotonic, while in the case of earthquakes the variation in time is cyclic and an accumulation of plastic deformation occurs.

1.3.3. Ductility for Earthquake Design

The analysis of dynamic responses of structures to a seismic input is based on the application of energy concepts through the use of an energy balance among kinetic energy, recoverable elastic strain energy, viscous damped energy, and irrecoverable hysteretic energy. From Fig. 1.11 it is very clear that at the beginning of an earthquake or for a moderate earthquake, all input energy is balanced by damping. For severe earthquakes, when the input energy is greater than damping, the difference is balanced by hysteretic energy, which implies the ductility of the structure.

Analysing whether it is technically and economically possible to balance the seismic input, the designer may decide to adopt one of the following alternative approaches to protect the structure against severe earthquakes:

-to rely on the elastic behaviour of the structure only;

-to consider the viscous damping given by the non-structural elements and to increase the plastic hysteretic energy, namely to increase the structure ductility, by using appropriate constructional details, but accepting some damage during severe earthquakes;

-to increase the viscous damping, by using some damper devices, which decrease the hysteretic energy, protecting the structure against damage;

-to increase both viscous damping and hysteretic energies;

-to decrease the input energy using base isolation techniques and to balance the remaining energy through elastic vibrations only.

Generally, the absorption of input energy by elastic behaviour only is restricted for those facilities, whose failure may lead to other disasters, affecting man and/or the environment, e.g. nuclear power plants, dams, petrochemical facilities, etc. The common practice is to increase the hysteretic energy as much as possible through inelastic behaviour, using the ductile properties of the structure. Only recently has it been recognized that it is possible to increase the dissipated energy through dissipation devices. But the efficiency of these devices is very doubtful in many situations, so the dissipation of input energy through plastic deformation remains the most realistic measure of protection. So, even in using new control systems, it is absolutely necessary to provide the structure with some given level of ductility.

1.4 Progress in Design Methodology

1.4.1. International Activity

Today we have probably reached the stage when the actual structural performance during strong ground motions can be satisfactorily explained. Even now, the most important effects on the inelastic structural behaviour can be quantified (Fajfar, 1995). This significant progress which has been recently achieved in the earthquake design methodology is due to following factors:

-a great amount of information concerning the features of earthquakes has collected and important databases are operative. For instance, the database for the European area and Middle East of the Imperial College of Science and Technology of London (Ambraseys and Bommer, 1990) contains almost 1000 records for earthquakes of all magnitude and depths. Similar databases exist in Italy, Greece, USA and Japan;

-important activity in macro and micro-zonation has been recently carried out all over the world to identify and characterize all potential sources of ground motions. Important national and international conferences on zonation have been held recently;

-a wide activity in research works concerning the behaviour of structures in seismic areas has been made contemporary. This activity is materialized by a sequence of World Conferences (WCEE), European Conferences (ECEE), and National Conferences, the proceedings of each scientific event contains hundreds of very important papers;

-for steel structures, a sequence of STESSA Conferences on the Behaviour of Steel Structures in Seismic Areas initiated in 1994 in Timisoara (Mazzolani and Gioncu, 1995), followed by the 1997 Kyoto Conference (Mazzolani and Akiyama, 1997) and the 2000 Montreal Conference (Mazzolani and Tremblay, 2000). The proceedings of these Conferences present the state-of-the-art research works for these structures and underline the lessons to be learned from the last great earthquakes;

-important international associations such as the International Association for Earthquake Engineering (IAEE) and the European Association for Earthquake Engineering (EAEE), are involved in promoting the development of research activity. The result was the important initiative to establish the International Decade for Natural Disaster Reduction (IDNDR), with the aim to limit the destruction produced by natural phenomena, among them the earthquakes playing a leading role;

-the interest in the problem of earthquake engineering is manifested also by the European Convention for Constructional Steelwork (ECCS), with the preparation of the first proposal of codification “European Recommendations for Steel Structures in Seismic Zones” in 1988. A manual for using these recommendations for practical purposes has been elaborated by Mazzolani and Piluso, (1993b). Today, this text is incorporated in the Eurocode 8, for the steel buildings Section, with just some editorial changes;

-after each great event, extensive international activity is performed to characterize and understand what happened to buildings. So, after the Northridge earthquake, an SAC Joint Venture research program was elaborated upon by Structural Engineering Association of California (SEAOC), Applied Technology Council (ATC) and California Universities for Research in Earthquake Engineering (CUREE) (Ross, 1995, Krawinkler and Gupta, 1997). After the Kobe earthquake, a JSSC Special Task Committee was organized to analyse the impact of this earthquake on steel building frames (JSSC Technical Report, 1997). The European research project dealing with the “Reability of Moment Resistant Connections of Steel Building Frames in Seismic Areas (RECOS)” has been recently sponsored by the European Community within the INCO-Copernicus Joint Research Project (Mazzolani, 1999). The aim of this project is to examine the influence of joints on the seismic behaviour of steel frames, bringing together knowledge and experience of different specialists from several Countries (Italy, Romania, Greece, Portugal, France, Belgium, Slovenia and Bulgaria). The results of these research works were published by Mazzolani (2000) as editor.

-as a consequence of the significant economic losses that resulted during the last decade, a pressing need has been identified to develop a set of new approaches for improving the civil engineering facilities during severe earthquakes. So, the Structural Engineering Association of California (SEAOC) established the “Vision 2000 Committee” with the aim to develop a conceptual comprehensive framework for seismic design (Bertero et al, 1996, Bertero, 1997). This framework, which is called “performance-based seismic engineering” involves the conception, design, construction and maintenance activities. Consequently, the performance-based seismic engineering is defined as “…consisting of the selection of design criteria, appropriate structural systems, layout, proportioning, and detailing for a structure and its non-structural components and contents, and the assurance and control of construction quality and long-time maintenance, such that at specific levels of ground motion and with defined levels of reliability, the structure

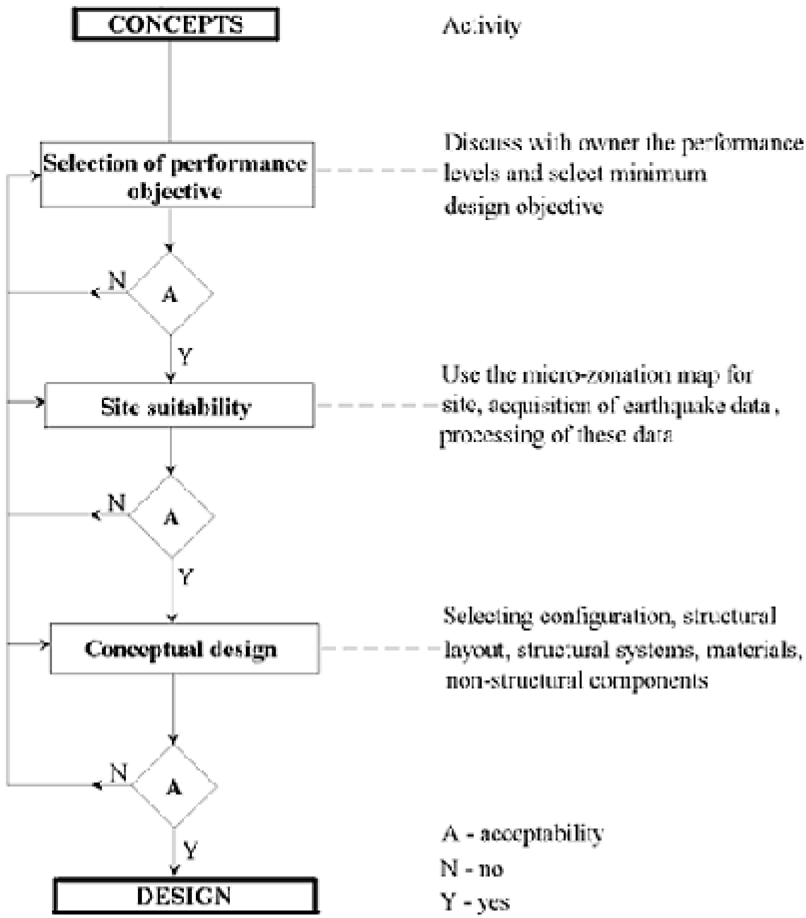


Figure 1.12: Flow chart for the concept development

will not be damaged beyond certain limiting states or other useful limits” (Bertero, 1997).

Thanks to this intense international activity in earthquake engineering, a marked progress in conception, design and construction has been observed recently.

1.4.2. Progress in conceptions

The flow charts for the concept development and its progress are presented in Figs. 1.12 and 1.13 and contain the following steps:

(i) *Selection of performance objectives.* For a long time the main objective of seismic design has been to protect the life losses and consequently to

ACTIVITY	CONVENTIONALCONCEPT	PROGRESS
SELECTED PERFORMANCE	One level performance: - ultimate limit state	Multi-level performance: - fully operational; - operational; - life safety; - near-collapse.
SITE SUITABILITY	Macrozonation: - determination of seismic action based on a singlespectrum type	Microzonation information - magnitude; - return period ; - distance from source; - site soil stratification; - attenuation low; - duration.
CONCEPTUAL DESIGN	Traditional concepts based on strength demands	Using the new concepts considering the "rigidity-strength -ductility" triade

Figure 1.13: Progress in conception

satisfy only strength requirements by preventing structural collapse. Thus, the engineers have great difficulties in explaining to the owners what they are buying only according to the minimum code requirements and that in case of strong earthquake the code designed structure could suffer important damage, which must be repaired by using supplementary funds. After the last earthquakes, the level of economic losses has been socially and economically unacceptable, thus the owners have begun to understand that they must accept supplementary cost for additional protection. So, an important conceptual progression is achieved by introducing in design activity the possibility of protecting the structure at different levels of seismic action, in the frame of the so-called multi-level performance concept. Four levels of protection are, therefore, defined: fully operational (serviceable), operational (functional), life safety (damageable), near collapse (preventing collapse). These performance levels are associated to specific probabilities of occurrence: frequent, occasional, rare and very rare. The structural engineer, together with the owner, can establish whether the performance objectives remain at the code level, accepting damage occurrence in case of severe earthquakes or requesting additional protection and accepting paying supplementary costs.

(ii) *Site suitability.* The code provisions give a macro-zonation at the level of Country, which is insufficient for a proper design. The code analysis method is based on a single design spectrum, with some corrections considering the site soil type. This conception is proved to be unsatisfactory in many cases, because it does not catch the actual site feature. So, progress in this subject can be achieved by underlining the great importance of site conditions: actual magnitude, return period for each level of performance, distance from potential source, attenuation low, site soil stratification, direction, duration, etc. If the site conditions are bad from a seismic point of view, the designer can suggest to the owner to change the location or to accept supplementary cost for improving the soil conditions by using specific constructional methodologies.

(iii) *Conceptual design.* The conceptual design consists in the establishment of the general configuration of building (form, regularity, masses and stiffness distribution, gaps, etc.), foundation types (shallow or deep), structural materials (steel, composite, r.c., etc.), structural systems (moment resisting frames, braced frames, dual frames, etc.), joint types (rigid or semi-rigid), non-structural elements (type of interior and exterior walls), etc. Progress in this aspect is due to the large amount of new information obtained after the investigations of the behaviour of structures during the last severe earthquakes, and, from the impressive results of theoretical and experimental research works carried out all over the world. The designers must be conscious of the fact that a good concept design, rather than complex numerical analysis, has permitted many buildings to survive severe earthquake ground motions.

1.4.3. Progress in Design

The flow charts for design process and its progress recorded in the last time are presented in Figs. 1.14 and 1.15, and contain the following steps:

(i) *Preliminary design.* The conventional methodology of preliminary design is mainly based to satisfy the strength performance. But in many cases, the stiffness or ductility demands are more important than strength and the preliminary design must be improved through a series of analyses. Thus, the importance of a proper preliminary design should be overemphasized, because, if the design procedure has a poor preliminary design, the number of iterations for improving it is very high. By introducing the multi-level performance demands, it is necessary to establish, from the beginning, the most drastic requirement and an attempt of optimization is required, in order to smooth these demands. This step requires the evaluation of structure periods, design forces for different performance levels, critical load combinations, torsional effects, inter-storey drifts, required ductilities, expected overstrength, foundation behaviour and, finally, the beam and column preliminary sizing.

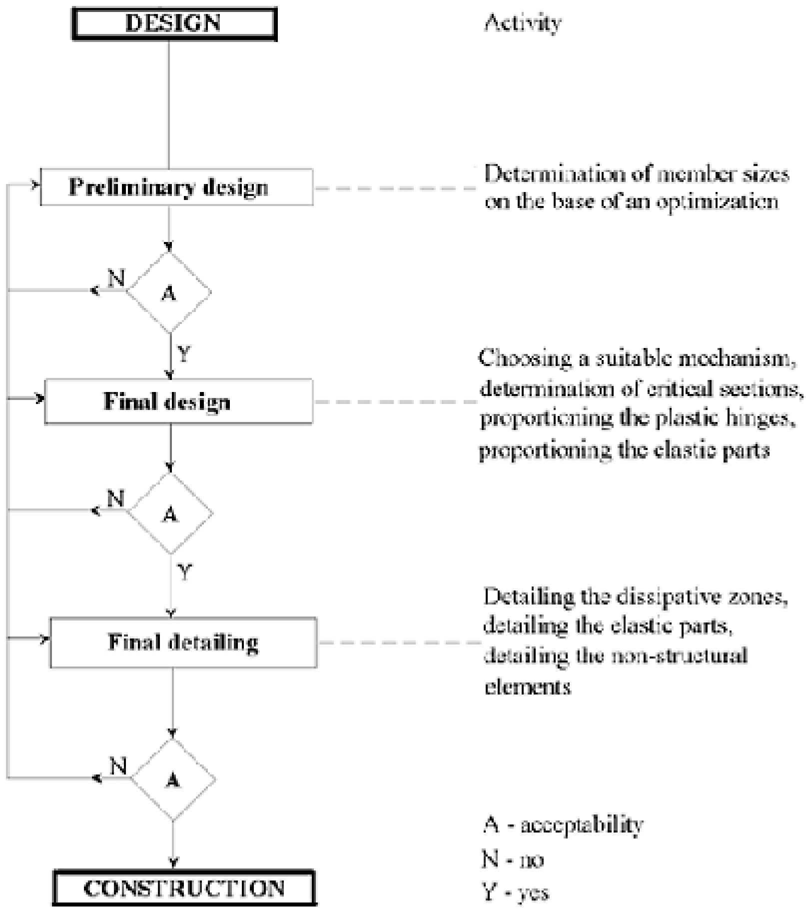


Figure 1.14: Flow charts for the design process

(ii) *Final design.* The conventional methodology considers only the classic verification of internal forces derived from the analysis, but we have to consider that plasticizations are possible everywhere, the pattern of plastic hinges is random, the overall seismic behaviour is difficult to predict and that the local ductility demand cannot be estimated. Due to these shortcomings of the conventional methodology, a new capacity design method is proposed (Pauley and Priestly, 1992, Bachmann et al, 1995). This methodology has been developed for reinforced concrete structures, but its principles can also be very useful for steel structures. Firstly, a complete and admissible plastic mechanism is chosen by locating the potential plastic hinge positions, and, an inelastic redistribution of design actions is carried out. Subsequently, adequate member dimensions are derived, by considering the

ACTIVITY	CONVENTIONAL CONCEPT	PROGRESS
PRELIMI-NARY DESIGN	On the basis of strength performance	Determination of member sizes on the basis of all level performance and an attempt of an optimization. Calculus of: - structural periods; - design forces for each performance level; - loads combination; - torsional effects; - ductility ratio; - expected overstrength; - foundation behaviour.
FINAL DESIGN	• Dimensions of structural elements • Verification of inter-storey drifts	Design of structural components based on: - choosing a suitable plastic mechanism; - determination of the critical sections; - verification of plastic hinges; - determination of overstrength of plastic hinges; - proportioning of elastic structural parts.
FINAL DETAILING	Few specific details	Improving of details after the last earthquakes for: - assuming a good local ductility; - improving the welded connections.

Figure 1.15: Progress in design

critical sections in the members selected for the eventual development of plastic hinges. The consideration of the overstrength of these critical sections, where plastic hinges occurs, is an important feature of the capacity design method. Finally, the other members or parts of them are designed to resist in elastic range, considering the overstrength of adjacent potential plastic hinges. This procedure ensures that the system may dissipate seismic energy with some local damage, but without global collapse and should allow a very useful design method to obtain the compatibility between ductility demands and available ductilities.

(iii) *Final detailing.* The suitable achievement of design concepts mainly depends on the simplicity of detailing of members, connections and supports. We can say that a good design is the only one which can be constructed (Bertero, 1997). The respect of detailing requirement assures a good behaviour of structure during severe earthquakes, according to the design concept. The failures produced during the ground motions indicate more deficiencies in structure detailing than in structure analysis. However, the code provisions contain only a few details directly involved in the protection against local damage of members and connections. After the last strong earthquakes, a great amount of information has been obtained and real progress in improving the detail conception is now possible.

1.4.4. Progress in Construction

Design and construction are two intimately related phases of the birth of a building. A good design conception is effective only if the building erection is qualitatively good. After each earthquake, field inspection has revealed that a large percentage of damage and failure has been due to poor quality control of structural materials and/or to poor workmanship. The flow charts for constructional aspects and their progress in recent years are represented in Figs. 1.16 and 1.17 and they contain the following steps:

(i) *Quality assurance during construction,* referring to the rules for verification of material qualities and proper workmanship. For instance, the analysis of structural steel specimen tests shows a considerable variation in material characteristics. In view of this variability, many present seismic code provisions specify only the minimum value for yield stress, which can lead to an unsafe design because a random overstrength distribution can modify the global ductility. An upper bound for yield stress must be given in the code and a more severe control of the variation factor is required.

(ii) *Monitoring and maintenance.* In many cases the damage or failure of buildings may be attributed to a lack in monitoring and improper maintenance. Due to this fact, the deterioration of mechanical properties of material and elements undermines the seismic response of the structure. The progress in this field would consist of the elaboration of severe rules for monitoring and maintenance of buildings during their life.

(iii) *Refurbishment, repair and strengthening.* The building may require some functional changing, which claims proper modification of the structural system. If this change is performed without suitable seismic rules, the modified structure could be victim of future earthquakes. On the other hand, there are a lot of buildings which were built many years ago, before the introduction of seismic design. Today, the building industry is looking with particular interest at the restoration, repairing and consolidation of

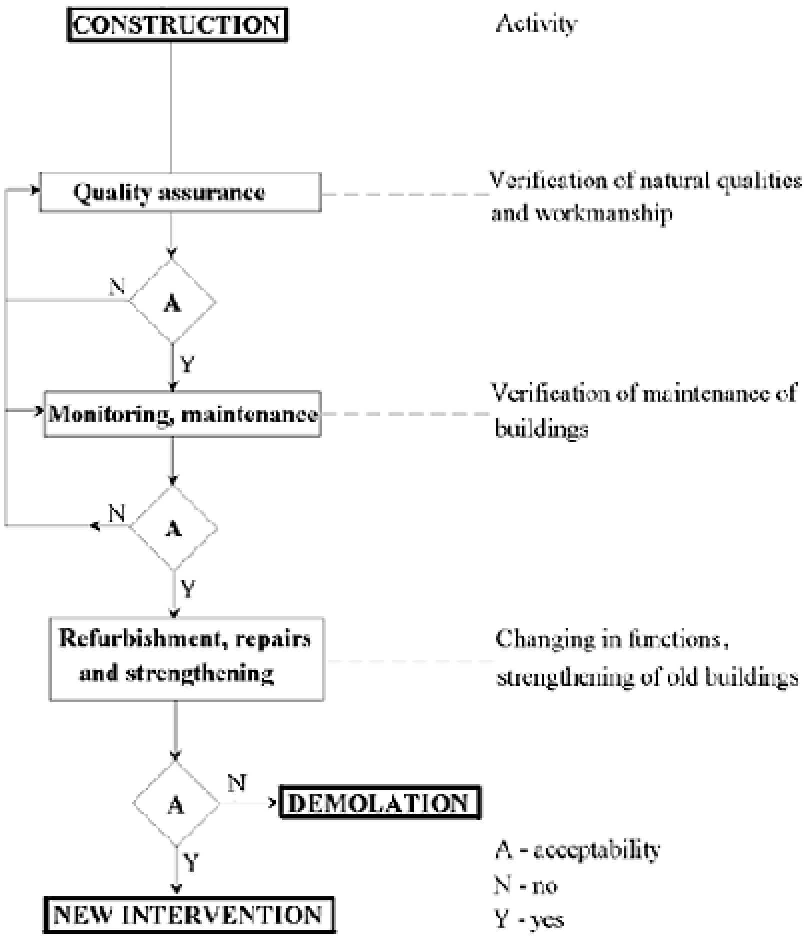


Figure 1.16: Flow charts for constructional aspects

old buildings. In future this industry will be extensively concerned with the refurbishment and strengthening of existing buildings, many of which are very important from an historical and architectural point of view. In all cases steel is an ideal material for refurbishment and important progress is marked in recent years in using some specific technologies (Mazzolani, 1990, 1996, Ivanyi, 1997).

1.5 Progress in Codification

1.5.1. The long way from theory to practice

The general methodology presented in the previous Section is in agreement with the new design philosophy. However, current code design methodology

ACTIVITY	CONVENTIONAL CONCEPT	PROGRESS
QUALITY ASSURANCE	Poor provisions on verification of material quality and workmanship qualification	Very careful control of - material qualities - workmanship qualities
MONITORING AND MAINTENANCE	Without rules	Rules for maintenance and monitoring
REFURBISH-MENT, REPAIRS AND STRENGTH-ENING	Poor rules	Rules for refurbishment repairs and strengthening

Figure 1.17: Progress in construction

fails in realizing goals and objectives of this philosophy (Bertero, 1997). The main reason for this can be explained as follows:

(i) Structural engineers are professionals and not researchers; their activities are driven by the need to deliver the design in a timely and cost effective manner. They may also resist new concepts, unless these concepts are put into the context of their present mode of operation (Krawinkler, 1995). On the other hand, in many cases the research works are performed by professors and researchers who are more interested in publishing their results for their colleagues than in the transmission of new knowledge to those who will apply it (De Bueno, 1996).

(ii) The loading condition on a structure during a major earthquake is very difficult to model. The definition of a design earthquake load inevitably calls for a series of engineering judgments of seismology, safety policy as well as structural engineering matters (Studer and Koller, 1995). For this reason design earthquakes should be elaborated upon in close collaboration between seismologists and engineers. Unfortunately, there are some difficulties in communication between these two professions. Engineers, in design-ing structures, must rely on proven principles, the engineering approaches being quantitative. In contrast, seismologists, when investigating geologic processes, have the privilege of proposing, testing and discarding erroneous hypotheses (Seeber and Armbuster, 1989). This difference in approach may be in part responsible for difficulties in collaboration.

(iii) The design philosophy for earthquake loads is totally different from the design methodology for the other loading conditions. The admittance of plastic deformations during severe earthquakes implicitly anticipates the occurrence of structure damage, which is not so easily accepted by the building owner. So the most relevant performance criterion for a building structure that has survived an earthquake is the total cost of damage. In this perspective, clear attention to damage control for structural and nonstructural elements should be a central concept. This damage control is very difficult to be quantified in a simple manner to be introduced as provision in a design code.

(iv) The implementation of new concepts in codes is constrained by the need to keep the design process simple and verifiable. Today, the progress of computer software has made it possible to predict the actual behaviour of structures subjected to seismic loads. But now the availability of powerful computational tools at relatively low cost does not imply that the most complicated models must always be used. During the elaboration of codes it must be kept in mind that the engineering community tends to be conservative. So the code provisions must always be a compromise between new and old knowledge and procedures, otherwise the new methodology will be rejected by the designers.

(v) Recognizing the need for code development based on a transparent methodology, we must also recognize that it is necessary to underline some dangers of this operation: over-simplification, over-generalization and immediate application in practice of the latest research results, without an adequate period of time during which these results can be verified.

1.5.2.
Required Steps in Code Elaboration

Considering the above observations, it is possible to establish the following steps in the elaboration of a performance code (Bertero, 1997):

-establishing a conceptual methodology, which must contain the problems of practical design and construction;

-elaboration of the first draft, in which the methodology is developed in detail in such a way as to be directly applied in practice;

-designing of some buildings by using different regular and irregular configurations, structural layouts and structural systems, which preferably have been designed and constructed according to current seismic codes, and whose response to earthquakes have been recorded or predicted;

-selection of conclusions after using the first draft and establishment of some simplifications of code provisions requested by the designers;

-elaboration of the final draft, in which all the conclusions coming from the previous steps are introduced.

The importance and advantage of developing of code elaboration is due to the fact that it is based on a transparent methodology which considers and checks the selected or desired performance objectives, in a very clear manner for the designer. This methodology assures that the improved provisions can be easily introduced in the code, when new or more reliable data become available, because it is not necessary to change the philosophy or the format of code. Another important advantage is the clear establishment of a program for focused research on the need to improve the code.

In a good codification for seismic design the following major objectives should be considered:

-codification for macro and micro-zonation;
-codification for design of structures, including the foundation design;
-codification for design of non-structural elements.

1.6
Challenges in Design Methodologies

1.6.1.
After the Last Severe Earthquakes

For a long time it was generally accepted in design practice that steel is an excellent material for seismic resistant structures, thanks to its performances in strength and ductility at a material level. But very serious alarm signals about this optimistic view arise after the last severe earthquakes of Michoacan (1985), Northridge (1994) and Kobe (1995). Beside a lot of steel constructions which have shown a good performance, at the same time a lot of others exhibited very bad behaviour. For these, the actual behaviour of joints, members and structures has been very different from the design expectation, so the traditionally good performance of steel structures under severe earthquakes has been recognized as a dogmatic principle not always respected in the reality. In many cases the damage occurred when both design and detailing have been performed in perfect accordance with the code provisions, it means that something new happened, which was not foreseen in the design practice (Mazzolani, 1995). The engineers and scientists want to know exactly the reasons of this poor behaviour:

-inaccuracy of ground motion modelling?
-modification of material qualities during severe earthquakes?
-shortcoming in design concept, especially concerning the use of the simplified design spectrum method?
-insufficient code provisions concerning ductility demand?
-shortcoming in accuracy in constructional details?

Today, concerning the measures which are necessary to eliminate the possibility of similar damage occurrance during the future strong earthquakes, the world of specialists is divided. Some of them consider that the actual design philosophy is proper and only some improvement of constructional details, especially concerning welded joints should be enough. But at the same time, there are many other specialists who consider that the abovementioned questions are real problems in the design and some pressing modifications in concept are required.

In the frame of this debate, the challenges in design methodology are presented.

1.6.2.
Challenge in Concept

The study of the structural response during an earthquake constitutes an important step for improving the methodologies of analysis and design of structures. In the past, due to the reduced number of records during the severe earthquakes (the famous El Centro record obtained in 1940 was for long time the main information about the time-history of an earthquake), the developed design methods are mainly based on simple hypothesis, with little possibility of verifying their accuracy. Today, due to a large network of instrumentation all over the world, several measurements of ground motions for different distances from the sources and on different site conditions are available. This situation gives the possibility to underline a new very important aspect which was previously neglected in the current concept: the difference in ground motion for near-field and far-field earthquakes. The near-field region of an earthquake is the area which extends for several kilometers from the projection on the ground surface of the fault rupture zone. Because in the past the majority of ground motions were recorded in the far-field region, the current concept refers to this earthquake type only. The great amount of damage during the Northridge and Kobe earthquakes are due to the fact that these towns are situated in a near-field region. So, the ground model, adopted in current design methodology on the basis of ground motions recorded in far-field regions cannot be used to describe in proper manner the earthquake action in near-field regions. The differences are presented in Fig. 1.18 (Gioncu et al, 2000, Mazzolani and Gioncu, 2000):

-the direction of propagation of the fault rupture has the main influence for near-field regions, the local site stratification having a minor consequence. Contrary to this, for far-field regions, soil stratification for travelling waves and site conditions are of first importance;

-in near-field regions, the ground motion has a distinct low-frequency pulse in acceleration time history and a pronounced coherent pulse in velocity and displacement time histories. The duration of ground motion is very short. For far-field regions, the records in acceleration, velocity and displacement have the characteristic of a cyclic movement, with a long duration;

-the velocities in near-field regions are very high. During Northridge and Kobe earthquakes, velocities with values of 150–200cm/sec were recorded at the soil level, while for far-field regions these velocities did not exceed 30–40cm/sec. So, in case of near-field regions the velocity is the most important parameter in design concept, replacing the accelerations which are a dominant parameter for far-field regions;

-the vertical components in near-field regions may be greater than the horizontal components, due to the direct propagation of P waves (see Chapter 3), which reach the structure without important modifications due to soil conditions, their frequencies being far from the soil frequencies.

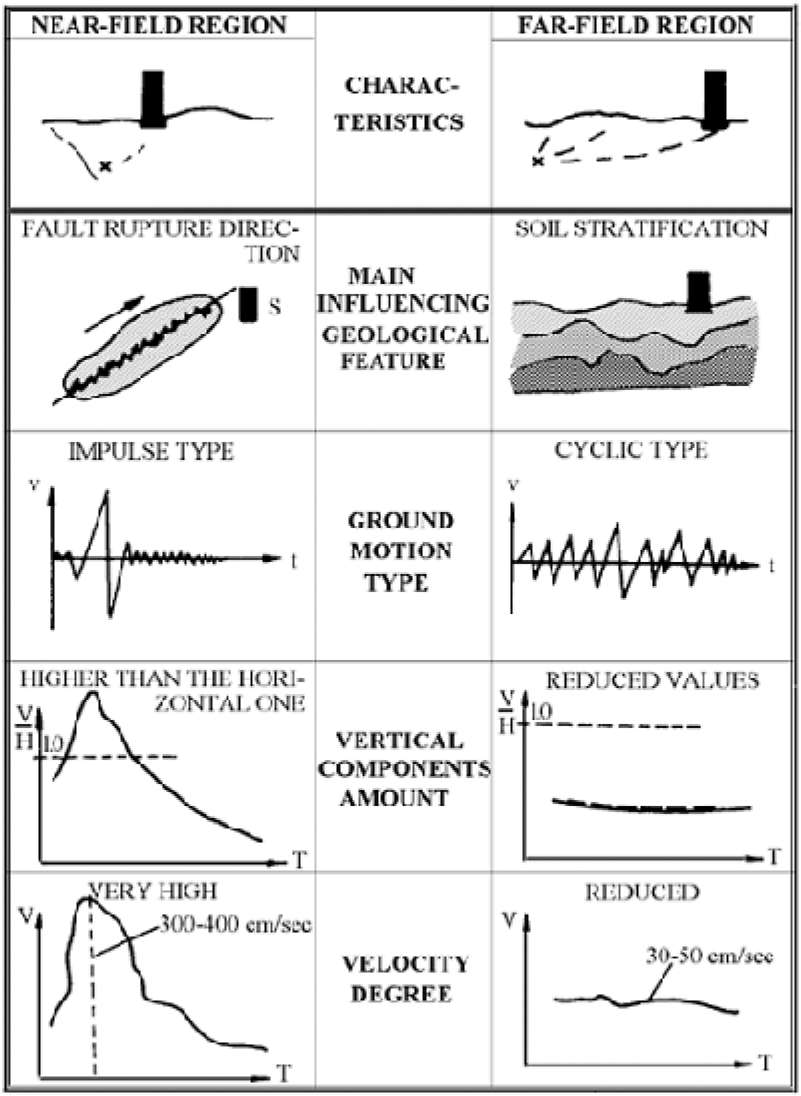


Figure 1.18: Near-field vs. far-field ground motion features

1.6.3. Challenge in Design

As a consequence of the above mentioned differences in ground motions, there are some very important modifications in design concept (Fig. 1.19) (Gioncuet al, 2000):

(i) In near-field regions, due to very short periods of ground motions and due to pulse characteristic of loads, the importance of higher vibration

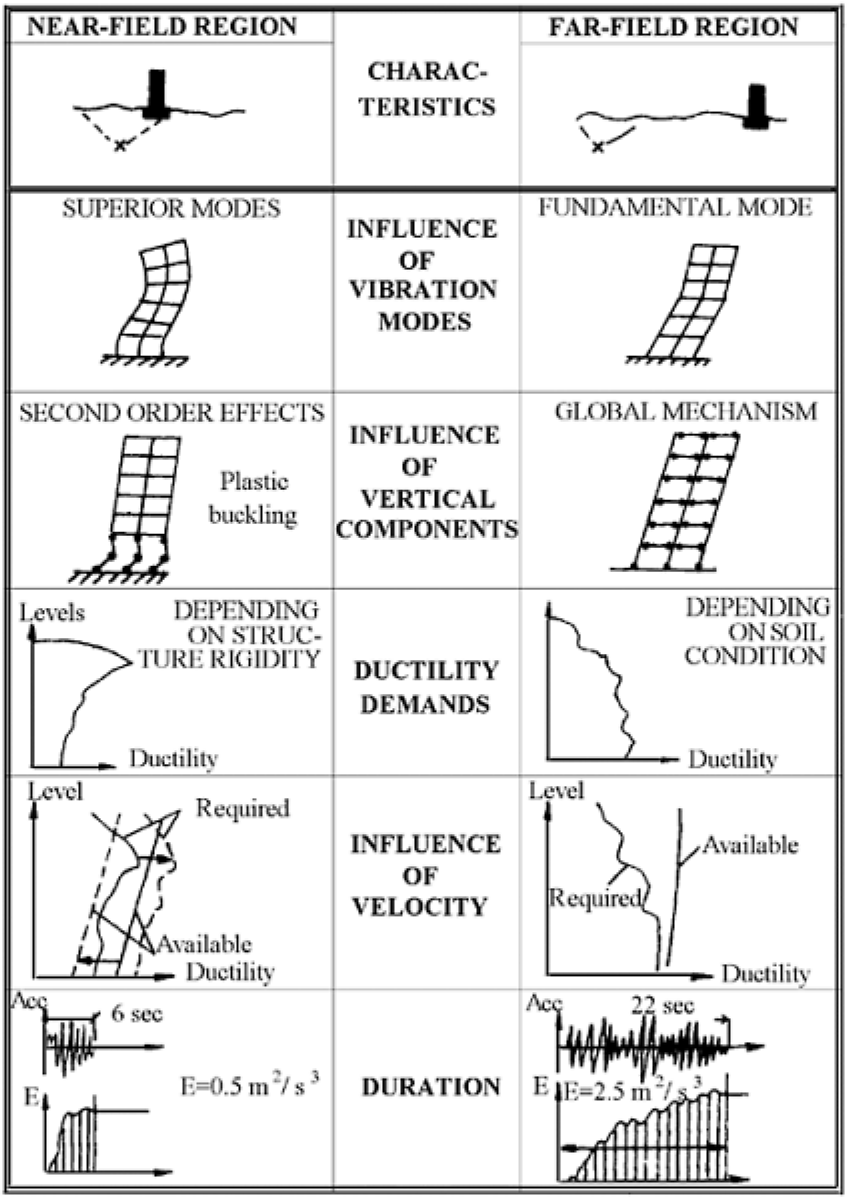


Figure 1.19: Near-field vs. far-field structure behaviours

modes increases, in comparison with the case of far-field regions, where the first fundamental mode is dominant. For structures subjected to pulse actions, the impact propagates through the structure as a wave, causing large localized deformations and/or important inter-storey drifts. In this situa-tion the classic design methodology based on the response of a single-degree-of freedom system characterized by the design spectrum is not sufficient to describe the actual behaviour of structures. Aiming to solve this problem, a continuous shear-beam model is proposed and the design spectrum is the result of the shear strain, produced by the inter-storey drift (Iwan, 1997).

(ii) Due to the concordance of frequencies of vertical ground motions with the vertical frequencies of structures, an important amplification of vertical effects may occur. At the same time, taking into account the reduced possibilities of plastic deformation and damping under vertical displacements, the vertical behaviour can be of first importance for structures in near-field regions. The combination of vertical and horizontal components produces an increase in axial forces in columns and, as a consequence, increases in second order effects.

(iii) Due to the pulse characteristic of actions, developed with great velocity due to the lack of important restoring forces, the ductility demand may be very high. So, the use of the inelastic properties of structures for seismic energy dissipation must be very carefully examined. At the same time, the short duration of ground motions in near-field regions is a favourable factor. A balance between the severity of ductility demand, due to pulse action, and the effect of short duration must be analyzed.

(iv) Due to the great velocity of seismic actions, an increase in yield strength occurs, which means a significant decreasing of available local ductility. Due to this increasing demand, as an effect of the impulse characteristic of loads, together with the decreasing of response, due to the effect of high velocity, the demand-response balance can be broken. The need to determine the ductilities as a function of the velocity of actions is a pressing challenge for research works.

(v) If it is not possible to take advantage of the plastic behaviour of structures, due to this high velocity, it is necessary to consider the variation of energy dissipation through ductile fracture. The fact that many steel structures were damaged by fracture of connections during the Northridge and Kobe earthquakes without global collapse, gives rise to the idea that the local fracture of these connections can transform the original rigid structure into a structure with semi-rigid joints. The positive result of this weakening is the reduction of seismic actions at a level which can be supported by the damaged structure, taking into account that the duration of an earthquake is very short. This is not the case of far-field earthquakes, for which the effect of long duration can induce the collapse of the structure.

1.6.4. Challenge in Construction

Pictures showing failures of structural members, in which details were grossly wrong, are very frequent in the post-earthquake reports. Very often, these mistakes were due to the fact that the detailed analytical calculations were not accompanied by a consistent set of structural details. This reality claims that special parts of codes must be elaborated in order to give constructional requirements for details (Corsanego, 1995). After the joint damage produced during the last earthquakes, when welded connections behaved very badly, the challenge in construction is to establish some provisions to improve the joint behaviour and to eliminate any source of brittle fracture. A lot of important experimental tests (JSSC, 1997) and a very careful examination of the connection behaviour (Miller, 1995) have been performed and the output of these investigations can be used directly in building erection.

1.7 Conclusions

After the careful examination of the state-of-the-art conception, design and construction, by analysing the most important challenge for future developments, it is possible to underline the following conclusions:

-seismic design is one of more intricate activities of the structural engineer due to the impossibility to accurately predict the characteristics of future ground motions that may occur at a given site and the difficulty in evaluating the complete behaviour of a structure when subjected to very large seismic actions. The best way to solve these unknowns is to give to the structure the necessary ductility;

-without any doubt the design concept based on the ductility properties of structures remains for the moment the main method to assure a suitable behaviour against strong earthquakes. The capacity to predict ductility demand and available ductility under seismic loads is a key-point in seismic design;

-even in the case of new concepts of response control, the structures must be provided with some given ductility level to prevent the situation in which the control measures do not react to the seismic input as it has been foreseen in design;

-the methodology for checking the structure ductility must be carried out with the same degree of importance as given for fulfilling the other criteria, as strength, stability and deformation checks. It means that, instead or in addition to the general rules, especially for construction details, some direct analytical formulations must be introduced in practice for ductility verifications.

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2
Learning from Earthquakes Seismic Decade 1985–1995

2.1
Damage of Steel Structures during the Recent Earthquakes

The occurrence of earthquakes, their consequent impact on people and on the facilities they live and work in, the evaluation and interpretation of damage caused by severe ground motions, are the principal items for structural engineers designing buildings in seismic areas. The attempts to find the answer to the question: "why does damage occur, after a wide amount of research work?", is an ethical duty of the specialists. The paradox of structural engineering is that while engineers can learn from the structural mistakes of what not to do, they do not necessarily learn from successes how to do (Petroski, 1985). The failure of a structure contributes more to the evolution of design concepts than structures standing without accidents, on the condition that the engineers have the capability to understand what happened. So, the damage of a structure during an earthquake represents a challenge for structural engineers to improve the design methods.

This aspect can be very well illustrated with the example of steel structures. For a long time it was accepted in design practice that steel is an excellent material for seismic-resistant structures, thanks to its performance in terms of material strength and ductility. To exemplify this good behaviour, in many papers the excellent performance of the Torre Latino-Americana building in Mexico City was mentioned, without showing that the fundamental period of this building is much larger than the predominant period of the Michoachan ground motion and the structure has piles supported by rock. Therefore, the seismic demand was relatively low (Osteraas and Krawinkler, 1989, De Buen, 1996). This is an example of the danger of generalization, mentioned in the previous Chapter. Contrary to this, the last severe earthquakes of Michoacan (1985), Northridge (1994) and Kobe (1995) have seriously compromised this image of steel as the most suitable material for seismic resistant structures. It was a providential sign that in the same place, Mexico City, where the case of Torre Latino-Americana building was assumed as example of good performance of steel structures, the first overall collapse of a steel structure occurred: the Pino Suarez building. The bad performance of joints in steel structures, both in Northridge and Kobe earthquakes, having the same characteristics of damage, shows that there are some general mistakes in design concept. And the fact that in both cases damage arises also when the design and detailing were performed in perfect accordance with the design philosophy and code provisions, amplifies the challenge addressed to structural engineers.

Generally, when such failures occur, the observed damage depends on following factors (Corsanego, 1995):

- general characteristics of the earthquake;
- local soil behaviour;
- seismic vulnerability of buildings;
- incomplete knowledge in seismic behaviour of structures;
- inadequacy of code provisions;
- wrong design, in opposition with code provisions;
- bad construction;
- lack of maintenance.

From the structural engineers' point of view the most important aspects of post-earthquake analysis are the lessons to be learned after each event, having the conviction that the best teacher is the full-scale laboratory of nature. No theory or mathematical model can be accepted, unless they correctly explain what happens in nature (McClure, 1989).

In the following Sections, the best known failures of steel structures during the last earthquakes are presented and a careful examination of the above mentioned factors is carried out. Almost exhaustive references are presented in order to have the possibility of finding supplementary information. Anticipating the results of this analysis, it is necessary to underline that the main conclusions are the existence of some differences between the ductility concept incorporated in codes and the actual ductility demand. So, there are some situations when the code ductility concept does not work (Eisenberg, 1995) and the task of improving these provisions is a crucial problem for structural engineers.

2.2
Michoacan Earthquake

2.2.1.
Earthquake Characteristics

On 19 September 1985, a major earthquake of magnitude 8.1 occurred, with an epicenter located in Zacatula City, about 350km from Mexico City, in the South of Michoacan State, due to the subduction activity of the Mexican Pacific coast. This earthquake was the most severe among a succession of ground motions with magnitudes between 5.8 and 8.1, which were produced in the South-West part of Mexico. Fig. 2.1a shows some of these earthquakes and the area of rupture of 1985 event (Reinoso et al, 1992, Iglesias and Gomez-Bernal, 1992).

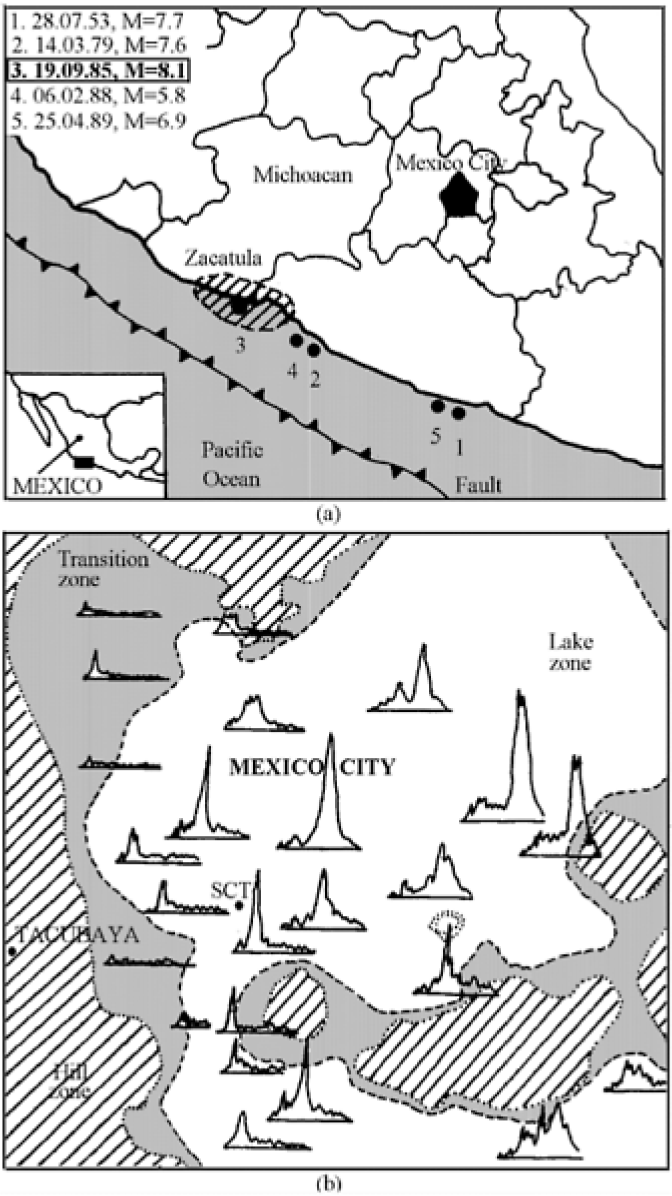
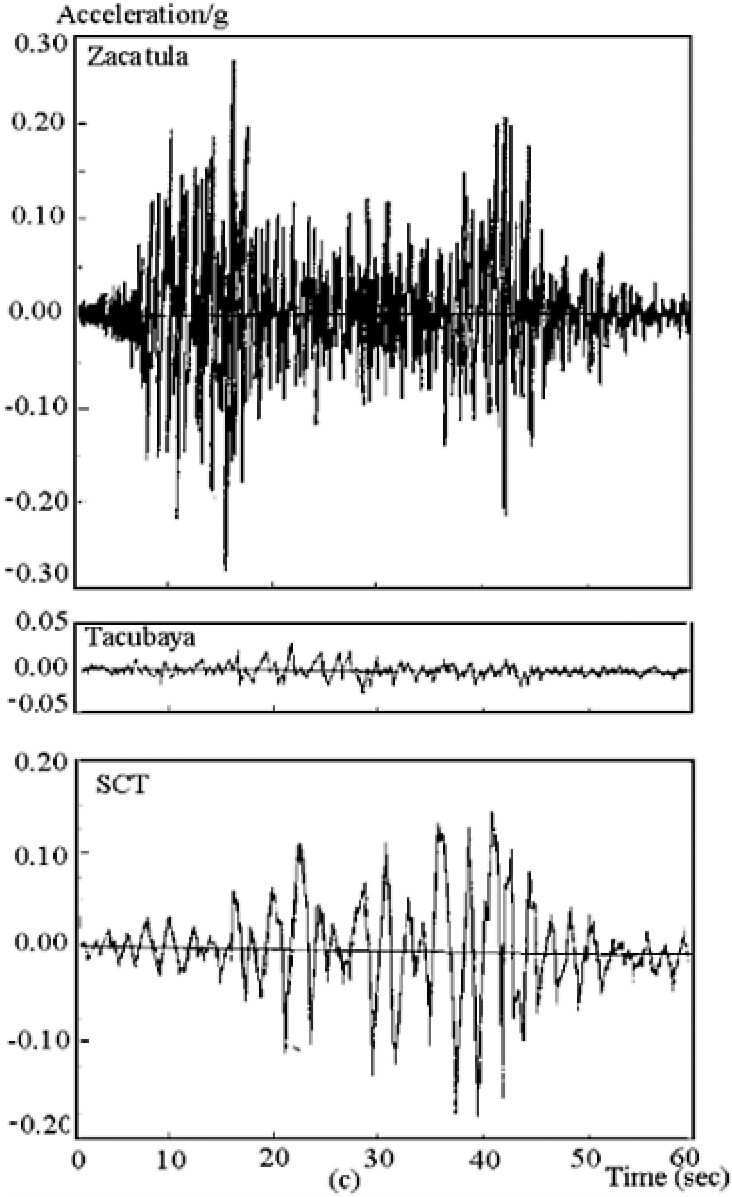


Figure 2.1: Michoacan earthquake: (a) Magnitude and epicenter of some Mexican earthquakes; (b) Response spectra for Mexico City Valley (after Reinoso et al, 1992)



(c) Accelerogram at Zacatula (near epicenter), Tacubaya (hill) and SCT (lake) (after Fischinger, 1997)

Fig. 2.1c presents the accelerations recorded at Zacatula City (near to epicenter) and Mexico City (Tacubaya and SCT Stations). One can see the great differences on peak ground accelerations and natural periods of these three records. The great question of this earthquake is to explain why there were so many important differences in recorded ground motions and why so much damage was produced at such large distance from the epicenter, when normal attenuation laws would suggest that much lower levels of acceleration would be expected at such distance. The first answer results from the fact that the earthquake is of interplate type, with a deep source location, so the area of influence was very large. The characteristics of the soil in Mexico City also played a key role in the disastrous event. The valley of Mexico City is a closed basin which was filled by water and wind-laid transported materials during the ancient periods. Due to the disintegration of rocks, the surrounding hills were gradually eroded and the finest elements were transported by water into the basin (Diaz-Rodriguez, 1995). The subsoil of Mexico City has been divided into three zones from the point of view of foundation engineering: hill zone, transition zone and lake zone (Fig. 2.1b). The lake area consists of a layer of clays of 20 to 30m deep. The site effects are characterized by large amplification at the resonant frequency of the clay layer. This amplification and the dominant period are also presented in Fig. 2.1b (Reinoso et al, 1992, Chavez-Garcia, 1995, Fischinger, 1997). These periods vary between 0.5sec for hill zone, up to 5.2sec for lake area, explaining the amplification of 12.7 times in the some zones (Abbiss, 1989) and the great energy spectra (Fajfar, 1995). The duration was also very different: for the lake area, about 140sec. and for hill zones, about 30sec. So, this earthquake was one of most devastating event for structures with long fundamental periods, such as the multi-storey steel moment resisting frames.

2.2.2.
General Information about Damage of Steel Structures

Buildings in Mexico City are shaken on average once every two years by an earthquake with magnitude 7 or larger. So it is expected that Mexico City structures be badly affected by degradation not only during one earthquake but of accumulated damage during several earthquakes (Reinoso et al, 2000). More than 100 steel structures were subjected to the 1985 severe test. Among them, 59 buildings were built after 1957, having from 7 to 22 stories (Osteraas and Krawinkler, 1989, Teran-Guilmore and Bertero, 1992). This was the first very important in-site verification of the behaviour of steel structures during a strong earthquake, showing generally a very bad performance. The main cause of this unexpected behaviour was the double resonance phenomena, seismic wave-soil and soil-structure, which gave rise to a required ductility exceeding the normal demand. The influence of higher modes, which were more active than the first one, caused damage on the upper stories and also collisions between adjacent buildings.

In Mexico City the most frequently used steel structures were the moment-resistant frames (MRFs). Typically, this system consisted of box columns (2 channels and cover plates, or four welded plates), H-section columns and beams (either hot rolled or welded) or truss girders built up with angle sections. The MR frames behaved generally well. Of the 41 buildings of this type, one underwent severe structural damage requiring

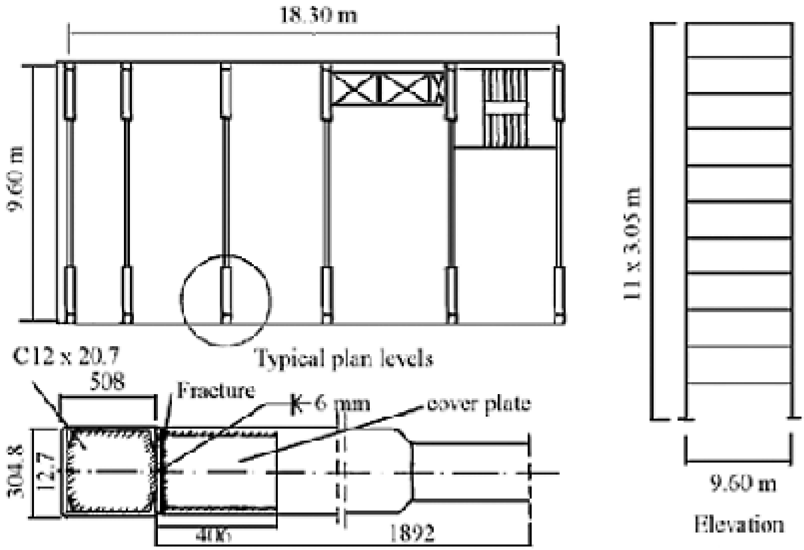


Figure 2.2: Damage of Amsterdam Street building (after Osteraas and Krawinkler, 1989)

partial demolition, one was affected by repairable damages and three sustained minor structural damages. All these damaged buildings were 10 or more stories high, having a long fundamental period. The damage was concentrated at welded beam-column connections or in truss girders, by buckling of compression diagonals.

The second type used in Mexico City area was the steel dual system, some bays of MR frames being braced. Of the 25 buildings of this type, two collapsed totally, one partially, four sustained various degrees of damage, the rest were undamaged. The collapse of the Pino Suarez building is the most famous case.

The third type was the mixed dual systems, consisting of steel frames and concrete shear walls. Of the 6 surveyed buildings, one sustained serious structural damage and one suffered minor structural damage, concentrated primarily in the truss girders.

The 77 Amsterdam Street Building (11-storey building) was built around 1970, and it is a one bay MR frame (Osteraas and Krawinkler, 1989), (Fig. 2.2). The columns consist of two channels and two welded plates and the beams are welded I-section. The damage reported in this building was severe cracking of masonry infill walls in the two longitudinal walls and connection failure in the first four stories of the transverse frames. The connection type constitutes a very weak link, because the only reliable force transfer from beam to column appears to take place from the beam flange splice plate through the full penetration weld to the column cover plate. The filled welds between continuity plate and cover plate were fractured, due to moments generated in the connections by the earthquake and the force transfer from beam to column shifted to vertical welds where cracks immediately ocurred. In these conditions, the connections worked as semi-rigid joints. Despite the large number of inelastic reversals experienced during this earthquake, the deterioration was not sufficient to cause the total collapse of structure due to second-order effects. This is due to the redundancy properties of the structure, which permitted a redistribution of moments. These connection damages were the first alarm signal to structural engineers, confirmed by the next Northridge and Kobe earthquakes, about the wrong behaviour of poor conceived joints and the need to provide connections with a sufficient ductility, assuring a second line of force transfer.

2.2.3.
Pino Suarez Building

The Pino Suarez complex shown in Fig. 2.3a, comprised five high-rise steel buildings: two identical 15-storey structures (A and E buildings) and three identical 22-storey structures (B, C, and D buildings) (Osteraas and Krawinkler, 1989). The complex is standing on a two level reinforced concrete subway station, which acts as a rigid foundation common for all five buildings. The two first stories are also common.

During the earthquake, the building D collapsed on the building E and buildings B and C were very seriously damaged. Because the building C was close to collapse, it represented a very rare occasion giving the possibility to study a building just before failure, at its ultimate state level. The layout and typical details of buildings C and D are shown in Fig. 2.3b. The structural system consists of moment-resisting frames and a bracing system around the service core, consisting in two X braced bays in the transverse direction and one bay in the exterior longitudinal frame. The beams are truss girders, built up with angle sections and plate elements. The trusses are double in the longitudinal direction and single in traverse direction. All columns are welded box sections, built-up with four plates of equal thickness. The braces consist of double T cross-sections, built-up with three plates welded together.

The most evident localized failure observed in building C was the severe local buckling in the fourth storey box columns (Fig. 2.4a). The four plates of the cross-section were separated due to failure of welding, thereby causing significant reduction of column stiffness. The shortening of these columns of about 25cm was responsible of large deflection of the girder supporting the V-braces (Fig. 2.4b). Buckling was observed also in the X-bracing system. Local failure was present in almost all the truss girders of longitudinal and transverse directions; many of the lacing members buckled (Fig. 2.4c) (Fischinger, 1997).

A very well conducted analysis of this failure has been performed by Cheng et al (1992), Ger and Chang (1992), and Ger et al (1993). Experimental tests were carried out for girders and columns. For transverse truss girders, a ductility factor of 2.3 was obtained, the failure being caused by

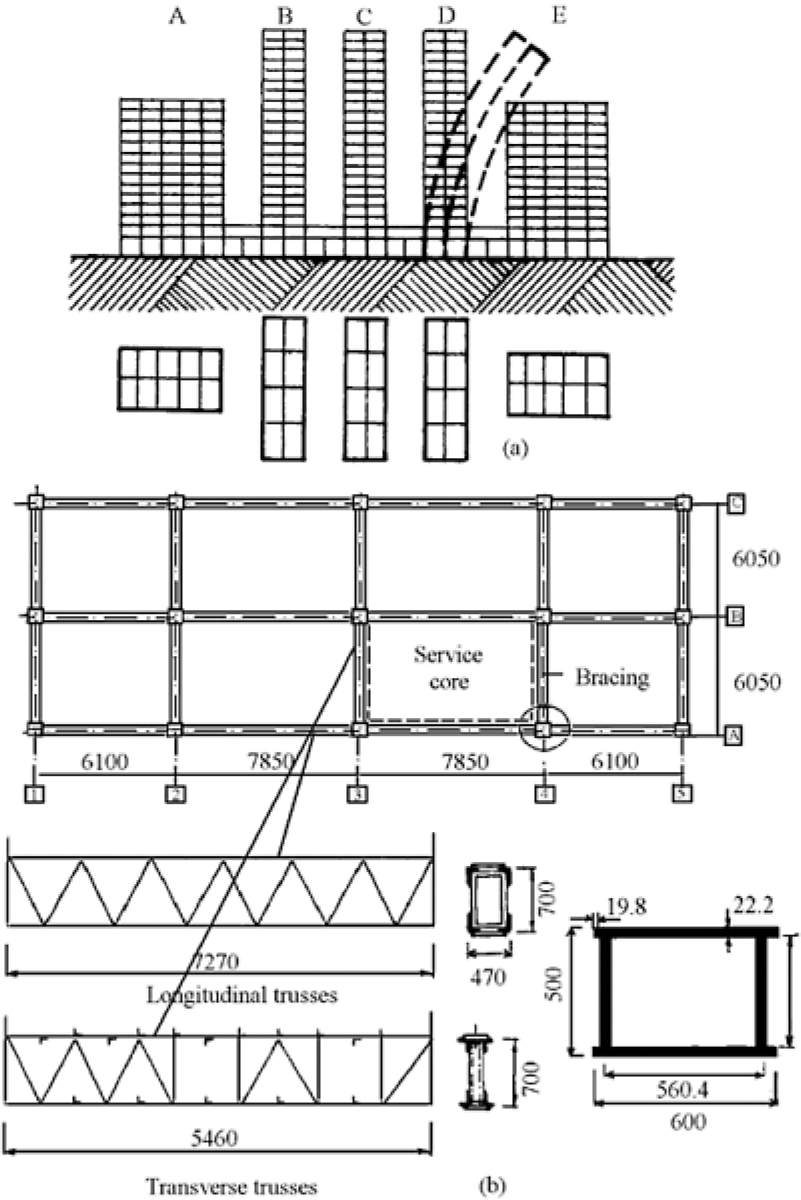


Figure 2.3: Pino Suarez complex (a) Elevation, plan and collapse of building D; (b) Plan view and typical framing details (after Osteraas and Krawinkler, 1989, Ger et al, 1993)

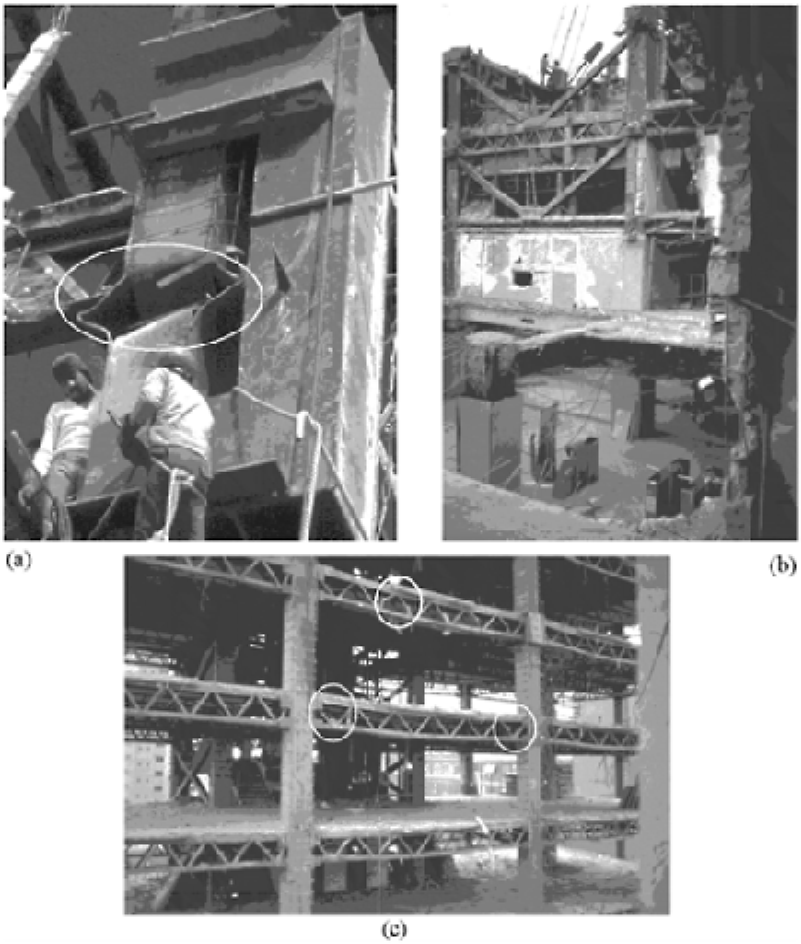


Figure 2.4: Collapse of Pino Suarez buildings: (a) Column buckling; (b) Collapse of bracing system; (c) Buckling of truss girder members

buckling of web members. For longitudinal truss girders, only 1.72 and 1.71 ductility factors were obtained, the failure being produced by local buckling and cracks of top chords. For columns, after the local buckling, a very unstable behaviour was observed, with a very bad ductility factor. The column failure was due to high axial force and low moment combination, due to the presence of bracing systems.

According to the code requirements, which do not consider the specific situation of soil in Mexico City and the actual ductility of structural mem-

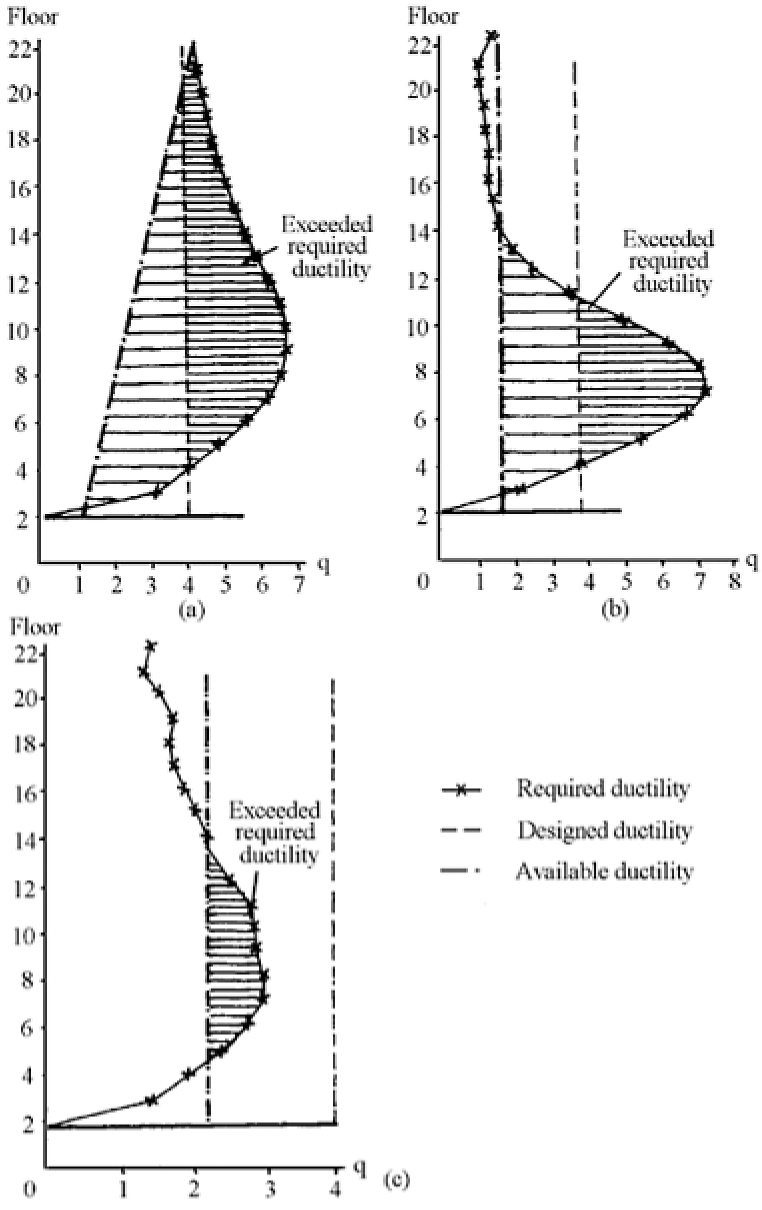


Figure 2.5: Designed, required and available ductilities: (a) For columns; (b) For long direction girders; (c) For short direction girders (after Ger et al, 1993)

bers, the structure was designed for a ductility factor equal to 4. The overall structure analysis, incorporating the specific behaviour of members and the peculiarities of the

Mexico City earthquake, allows to evaluate the actual behaviour of the structure and to determine the required ductility for the structural members. Due to soil conditions, the required ductilities were greater than 7 for columns, 7.5 for long direction girders and 3 for short direction girders, the maximum of these ductility demands being obtained at the 9th storey (Fig. 2.5). Comparing these required ductility factors with the experimental values, very large differences may be noted. Thus, it is very clear that the collapse of the structure occurred due to insufficient ductility of columns and girders.

The Pino Suarez building collapse provided an excellent opportunity to underline what may occur if no concordance between required and available ductilities exists.

2.3 Northridge Earthquake

2.3.1. General Description of Californian Earthquakes

The largest earthquakes ever to hit the USA States were centered in the New Madrid seismic zone near Memphis City, Tennessee, in 1811–1812, with the magnitude of 8.6 (Basham, 1989). But the most active seismic regions are along the western shore of the Country, where the Pacific and the North American tectonic plates meet and a system of fault lines has developed (Popov, 1994). California is a part of the circum-Pacific seismic belt, which is responsible for about 80 percent of the world's earthquakes. The West Coast of USA is hit by thousands of shocks every year and earthquakes of destructive magnitude have occurred once a year in the past 50 years. California's crusted surface is crossed by many great fractures or faults, forming lines of weakness in the masses of rock. Some faults are known to be active, while others are presumed to be inactive, but they can give unexpected surprises. The most famous is the San Andreas fault in Southern California, along which the most frequent and dangerous earthquakes occur. The type of seism is an interplate motion, produced at shallow depth, which means that the main characteristic is given by the near-field ground motion type. The most important earthquakes produced in recent years along the San Andreas fault are presented in Fig. 2.6 (Grecu and Moldovan, 1994). Several major urban areas are located alongside major active faults and so could be subjected to near source ground motions from large earthquakes. The San Andreas fault runs 10km West of downtown San Francisco and extends south to Los Angeles metropolitan region. Among these earthquakes, Loma Prieta and Northridge are the most interesting from the structural engineering point of view.

Due to the fact that a very dense network of instrumentations is now available along the San Andreas fault, a lot of very important information was obtained. About 37,000 well-recorded earthquakes detected on the southern California seismic network between 1981 and 1994 provided data

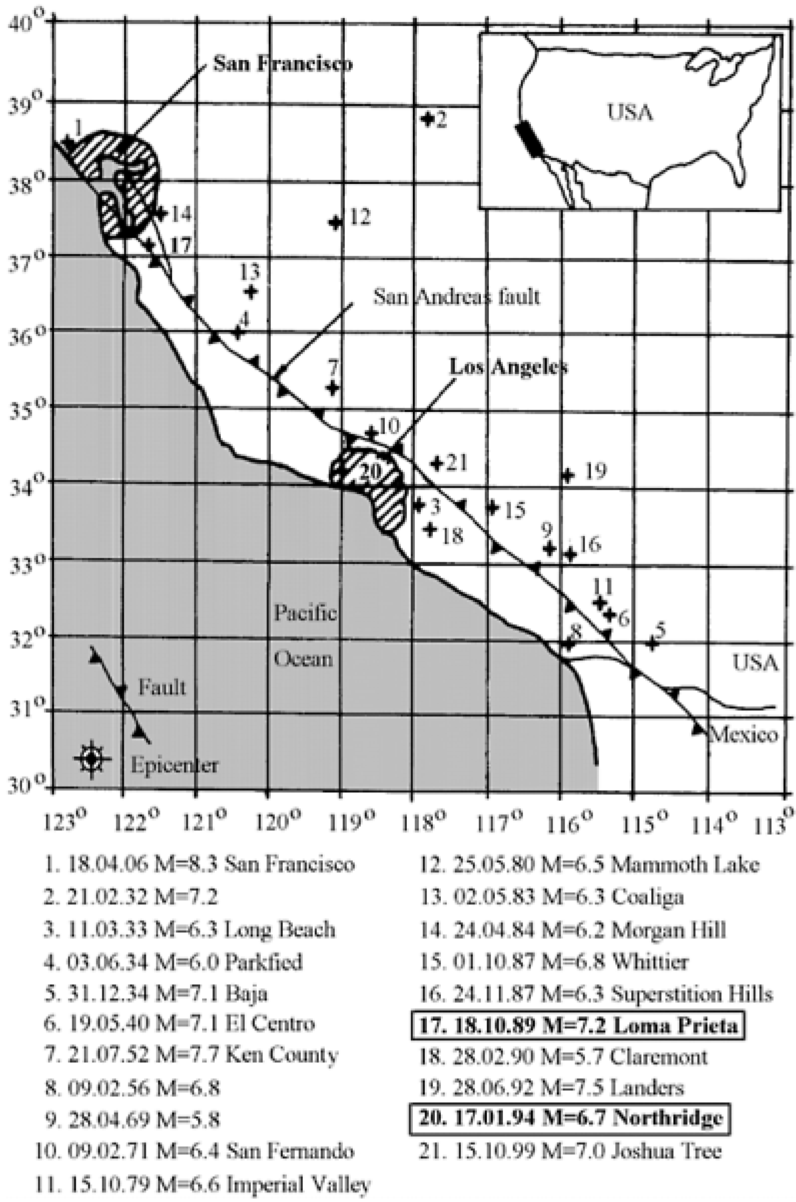


Figure 2.6: Californian earthquakes (after Grecu and Moldovan, 1994)

for calculating the features of these ground motions. During the Northridge event more than 200 strong-motion accelerograms were recorded in the metropolitan area (Magistrale and Zhou, 1996). The main characteristics of these earthquakes were as follows:

(i) *Pulse characteristic.* The analysis of records reveals that the aspect of the time-history variation of ground motions is qualitatively different from the other well known records, for instance the famous El Centro records. Fig. 2.7 shows the 1979 Imperial Valley records, which are typical for a nearfield earthquake record. The feature of these records is the low frequency pulses in the acceleration time-history, which translate into the pronounced coherent pulses in velocity and displacement histories.

(ii) *Vertical components of ground motions.* For a long time, the study of earthquake ground motions has been limited to the examination of the horizontal components. The vertical ground motions have been largely ignored, because until last time the recorded earthquakes were far from the source. But during the recent recorded earthquakes near the source, it has been observed that the vertical ground motions are sometimes greater in amplitude than the horizontal components. This remark was very evident in the strong ground motions which were recorded during the 1979 Imperial Valley earthquake (Lew, 1992, Chouw, 2000). From Fig. 2.8 it can be observed that the largest maximum vertical accelerations generally occur close to the fault rupture zone. In addition, the vertical movements are associated with frequencies higher than the horizontal ones, showing that different design spectra must be considered. So, for the Californian earthquakes, the vertical actions of the ground motions cannot be ignored.

(iii) *Combination of horizontal and vertical components.* It is generally accepted that the first waves which arrive to the structure are the vertical ones, as shown in the Fig. 2.9a for the Imperial Valley earthquake. But in other cases, as Morgan Hill earthquake, the vertical and horizontal motions are almost exactly coincident in time (Fig. 2.9b). Both cases must be considered in structure analysis, because it is not sure which situation will arise (Elnashai and Papazoglu, 1997).

(iv) *Velocity.* An increase of velocity near-field is marked. Velocities often exceed 150 to 200cm/sec in areas surrounding the source (Trifunac and Todorovska, 1998). The velocity histories of several Californian records are shown in Fig. 2.10. The ground motion has in the near source field a pronounced coherent pulse in velocity and displacement. The ground motion could be composed by only one pulse (Supersition Hills and Lucerne Valley) or more adjacent pulses (Tabas, El Centro and Loma Prieta). Fig. 2.11 shown a histogram of S-wave velocities of the sites where the seismic surveys were performed (Niwa et al, 1996). One can see that the main velocity is about 200m/sec, values for which the influence of asynchronism in horizontal and vertical ground motions may be important.

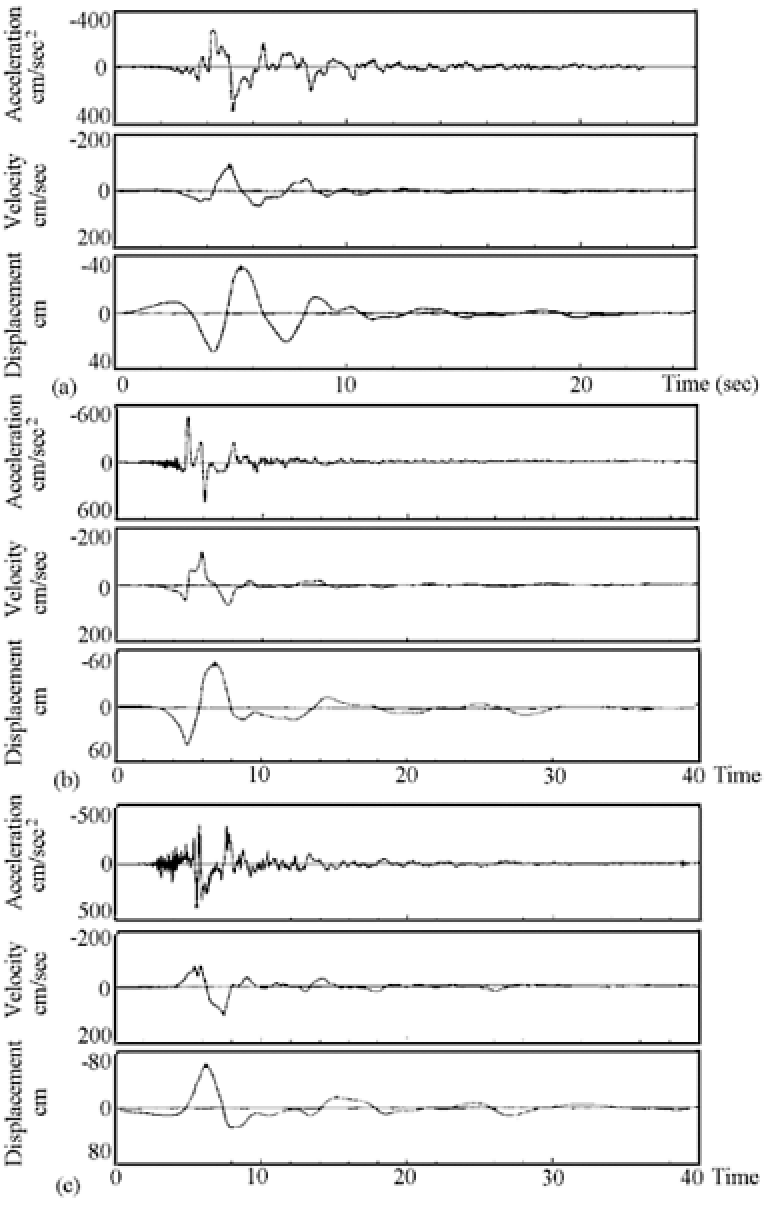


Figure 2.7: Pulse characteristic—Imperial Valley, 1979: (a) Meloland Overpass; (b) Array No. 7; (c) Array No. 5