The Lorca Earthquake

Effects on buildings

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Contents

Foreword

On the occasion of the Mula earthquake on 2 February 1999 and further to the experience gained in the appraisal of the damage caused, the Consorcio de Compensación de Seguros (CCS) published a book authored by Higinio Arcos and Mª Cristina Porcu entitled *Movimientos Sísmicos y Estructuras Murarias: Origen, Efectos y Evaluación de Daños en la Vivienda Tradicional.* That book, richly illustrated with photographs and drawings, was widely distributed among both professional damage surveyors and architects and engineers.

Seismic damage is fortunately infrequent in Spain, although the record shows that earthquakes are not unknown in this country, as the Lorca quake on 11 May 2011 made painfully clear. This was the most severe seismic event with which the CCS had been faced since its inception. In light of the quake's huge impact on people and property, the CCS deployed its full operational capacity from the outset, sending over 200 surveyors to the area and setting up a victims' support office to attend to and pay claims with the least possible delay. Given the complexity of the losses in events of this nature, the staggering number of claims lodged (over 32 500) and the sizeable indemnities paid (over 462 million euros), at CCS we believe that our objective was met and that the applications for indemnity and respective damage appraisal were handled as speedily and efficiently as possible under the difficult circumstances.

The lessons learnt from those damage and indemnity surveys may be applied in future to minimise seismic damage wherever an earthquake strikes. The knowledge deriving from an analysis of the experience gained in damage appraisal and post-quake reconstruction at Lorca since 2011 will consequently contribute to improvements in building techniques and advancements in structural capacity to resist future events. That is the aim pursued with the present book, the outcome of INTEMAC's in situ research into structures, materials and construction methods.

The CCS has published this study in the framework of its assigned role in prevention, in the awareness that knowledge and research on how earthquakes are generated and act are essential to mitigating risks and to adopting measures and procedures designed to reduce seismic vulnerability.

We trust that it will prove useful to both decision-makers and professionals whose task it is to define construction methods and criteria in seismic areas.

Consorcio de Compensación de Seguros

Introduction

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Introduction

The first question that a would-be author should pose before writing about any subject is whether it will be of interest to his or her intended readership. In the case of the Lorca earthquake, the specific question might be, “what is it about this earthquake that merits study, investigation or description?”

The immediate answer is obvious: it was the earthquake that caused the greatest loss of human life and property damage in Spain in over a century, the quake that gave rise to the highest acceleration ever recorded in the country, the quake, in a word, that reminded us that the peninsula’s seismicity is greater than many Spaniards may have realised.

Although those reasons suffice to justify the present analysis, other arguments could also be wielded, as listed below.

- **The earthquake affected a very representative town**, that meaning a medium-sized city with a layout similar to any other we might find in this part of the world. Generally speaking, the buildings in Lorca are no different from the ones in Murcia, Alicante or Granada. In fact, they do not differ from buildings in Madrid, where the seismic risk is practically nil. The urban layout in many of its quarters is similar to the zoning observed in the cities mentioned. Even the town’s soil and geotechnical features are sufficiently varied to be representative of many other places on the peninsula. The Lorca earthquake is particularly significant in this regard, because while other events of some consequence are on record, they occurred in less representative places. The Mula earthquake, for instance, affected mainly low-rise buildings whose structure consisted of traditional masonry walls.

  At Lorca, by contrast, the quake affected conventional steel or reinforced concrete buildings designed to contemporary practice, over 10 storeys tall in some cases.

- **The Lorca quake was no exceptional seismic event**. As discussed below, given the fairly moderate magnitude of this quake, its repetition is plausible at any time in many other places on the peninsula. Clearly, the likelihood of a tremor almost directly underneath a city, as in Lorca, is not high, but unfortunately on the grounds of current knowledge the possibility cannot be ruled out. At this time, that knowledge is too sketchy to chart a map of the earthquake threat to every town and city in Spain.

- **The damage caused by the Lorca event is no different from what could be expected anywhere else.** As discussed in the chapters below, the most severe casualties and material damage were largely due to the poor structural or architectural performance of a small number of structural and non-structural systems, all perfectly identified and analysed in prior studies on such earthquakes. Some of these systems have proven to be clearly inadequate to resist not only seismic, but also any other kind of action. The earthquake revealed what in all likelihood constitutes a simple question of quality in Spanish construction.

In short, investigating the Lorca earthquake is not an exercise involving a unique, isolated and unrepeatable event, one of indisputable technical interest but of scant practical application. On the contrary, any such analysis addresses real threats to surrounding areas and may lead to better construction practice and higher quality standards associated not only with seismic safety, but more general concerns.

Obviously, the interest that a subject may carry does not in itself justify writing about it. Writing about the Lorca quake is important, but more important than that is furnishing useful and relevant information. In this regard, the authors have benefited from the invaluable support of the experience accumulated by many professionals seconded to the city for over a year to conduct surveys and analyses.

The present text is the result of an initiative of the Consorcio de Compensación de Seguros, which deemed that the experience gained by INTEMAC's architects and engineers working in the city merited publication.

Field work was begun just a few hours after the quake subsided. INTEMAC's advisory role in support of the consortium's adjusters in the weeks that followed afforded us the opportunity to repeatedly survey approximately 350 buildings, drawing up the respective damage reports and recommendations for any necessary action. In subsequent stages (some of which are ongoing at this writing), the scope of INTEMAC's involvement was broadened to include the analysis of the suitability of the repairs proposed and even assistance in drawing up the repair designs for some of the buildings affected by the quake.

A sizeable team of architects and engineers worked full time in the city for over a year, generating vast documentary evidence. Their accounts were supplemented by the contributions of many non-INTEMAC professionals with whom we were fortunate enough to cooperate in solving the specific problems identified in each building and each type of damage. This book aims to compile some of the many lessons learnt from them.

Synthesising so much information in a text of limited length entails the application of subjective and consequently debatable criteria. Ensuring its accessibility to non-specialist architects and engineers also conditions the final wording of some of the chapters, which may not be as rigorous as many readers might expect. Obviously, then, despite our efforts, we shall not have attained the ideal objective of conveying all the experience accumulated by all concerned. No matter how poor the results, however, failure to make the attempt would be inexcusable.

We can hardly end this introduction without expressing our sincere gratitude to the Consorcio de Compensación de Seguros for entrusting us with this endeavour. More specifically, we gratefully acknowledge the firm and untiring support received from its Chief Operations Officer, Alejandro Izuzquiza Ibáñez de Aldecoa and the assistance provided during the surveys by Carmen Sánchez Rodríguez, Pablo López Villares and Juan Manuel Peraza Domínguez. We are also thankful to Alfonso Manrique Ruíz, most particularly for editing the original text with the constructive attention to accuracy that proves to be so useful in such cases.

Lastly, we wish to acknowledge the generosity of our fellow INTEMAC technicians, who for many months maintained the enthusiasm and devotion with which we all initially rose to the challenge posed by the work to be done at Lorca. Their names are listed below:

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Carlos Vergara Pérez
Alberto Blanco Zorroza
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David Fernández Montes
Mireia García Toro
Miguel Ibáñez Mayayo
Miguel Ángel Liébana Ramos
Laura Menéndez Martínez
Carles Pou Esquius
Mikel Remacha Mangado
Tiago Teixeira Martins

General description of the area

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1. General description of the area

On the afternoon of 11 May 2011, the earth in and around Lorca, a town in the Spanish province of Murcia, quaked twice. The first tremor, recorded at 17:05 (local time), had a magnitude of 4.5 and was felt throughout the region of Murcia and in some of the towns in neighbouring provinces. Some non-structural elements collapsed in the urban core.

The second quake occurred at 18:47. It reached a magnitude of 5.1 and could be felt in places as far away as Madrid. This quake caused nine fatalities, injured over 300 people, required the evacuation of over 10 000 and prompted the relocation of hospital patients.

This quake occasioned extensive damage in façades and roof parapets (whose collapse was the cause of the most severe casualties). Only one residential building collapsed altogether, although some masonry buildings and a few historic buildings underwent partial collapse.

The two epicentres were very close to one another, just a few kilometres northeast of the city and at similar depths, around 2 kilometres, according to the information published by the National Geographical Institute [49].

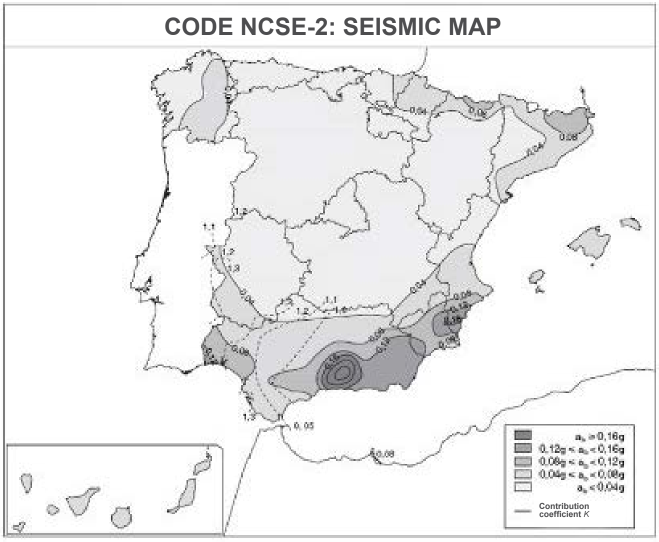
The second quake, hereafter referred to simply as the Lorca earthquake, generated the highest accelerations ever recorded in Spain. Its magnitude, however, was no greater than in two twentieth-century events, at Albolote, Granada in 1956 and São Vicente, Portugal in 1969. The nineteenth-century quakes at Vega Baja, Alicante (1829) and Arenas del Rey, Granada (1884) were more destructive, with hundreds of victims (over 1 000 in Granada). Lisbon's 1755 quake, in turn, is regarded as one of the most severe seismic events in history.

1.1. Seismicity

Lorca is sited almost exactly over one of the major arms of the Alhama de Murcia fault, which crosses nearly the entire province for 80 km along the northwestern boundary of the River Guadalentín basin.

In this extensive area, medium magnitude earthquakes such as the May 2011 event are fairly frequent. At Mula (1999), Bullas (2002) and La Paca (2005), the magnitudes were not much lower than at Lorca, ranging from 4.8 to 5. In fact, all that distinguished the Lorca event from the others and caused such severe damage was the city's proximity to the epicentre.

This region has higher earthquake-induced loss than any other area in the country. Further to the findings compiled in an interesting study authored by Ferrer Gijón *et al.* [30], in 1987-2001 the figures for the Murcian region were comparable to the data for Andalusia. Province by province, however, Murcia heads the list of losses (30.2 % of the nationwide total), followed by its neighbour Almería (23.2 %) and, surprising as it may seem given its location in northwestern Spain, Lugo (15.2 %). In loss projections for 2004-2033, Murcia continues to hold the



▲ **Figure 1-1**

lead position (29 %), followed by Granada (25 %) and Alicante (9.8 %).

According to the Spanish code on the seismic design of buildings presently in force, NCSE-02 [10], Lorca is in an intermediate seismic hazard zone, measured in terms of basic acceleration (Figure 1-1[1]). That value is practically identical to the figure listed in preceding codes.

The striking difference between seismicity as assessed by two such simple criteria, losses inflicted and the acceleration envisaged by the legislation, is indicative of the difficulty of defining the term itself further to a unique and general criterion. A distinction would probably need to be drawn between seismicity in areas affected by frequent earthquakes of moderate magnitude and areas characterised by more significant but less frequent quakes. Murcia and Granada would be examples of those two types of seismicity, respectively, although many others could be cited.

1. *This map will in all likelihood be obsolete when this text is published. The new version is actually available, although not published, at this writing. Nonetheless, the NCSE-02 map has been kept as the reference throughout the book, essentially because it was the one in effect when the earthquake hit.*

▲ **Photograph 1-1**

That notwithstanding, due to the proximity factor, the Lorca quake renders such an approach only partially applicable. It shows that events of modest magnitude may prove to be highly destructive locally if close to the surface and near an urban centre, such as the May 2011 quake.

A number of conferences dealing specifically with seismicity on the Iberian Peninsula are compiled in Capote *et al.* [33].

1.2. Description of the surrounds

Lorca is located in the southwestern part of Murcia, one of Spain's autonomous regions. It's very large municipal district is home to a population of over 92 000, nearly 60 000 of whom live in the urban centre. It lies between the Las Estancias and La Tercia mountains and the Guadalentín River basin, at 350 m above sea level.

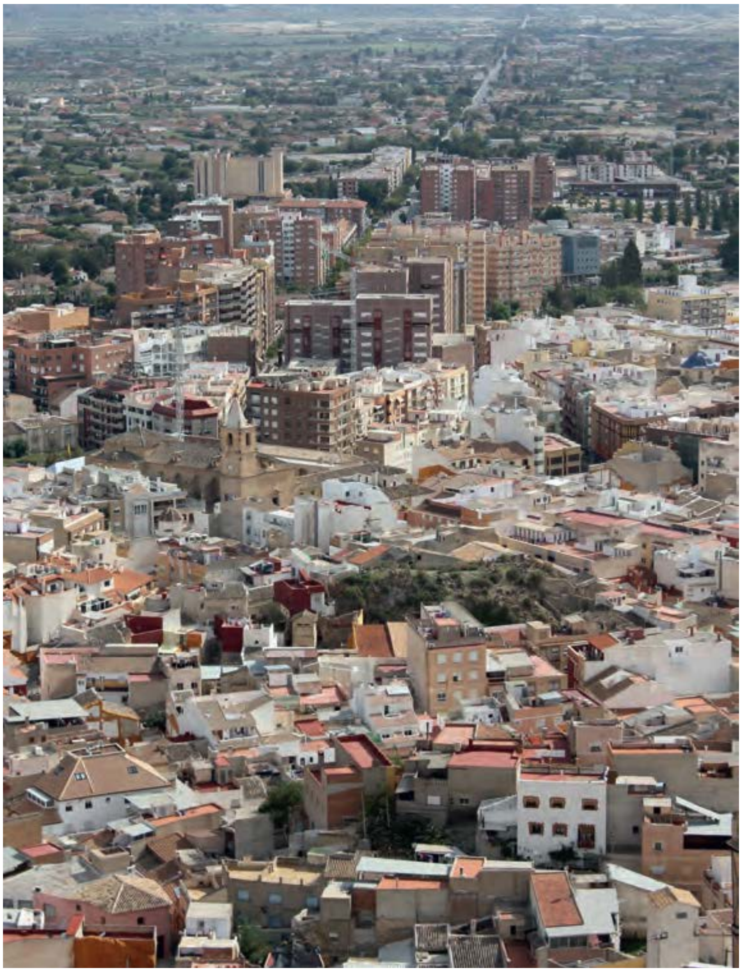
Photograph 1-1 shows an overview of the centre-west area of the urban core. The photo, taken from the foothills at the site of the town's castle, depicts the end slopes of the mountains, the Guadalentín basin and, in the background, the coastal mountains. La Viña, one of the quarters most severely affected, is located in the centre of the picture.

Given its location at the foot of a mountain, the cityscape is continuously albeit irregularly sloped, a situation that conditions the structural layout of many of its buildings, as discussed below.

The foothills location also explains the area's complex soil structure. While the upper-lying quarters alongside the castle are built on rock or hard soil, the newer quarters stand on sedimentary soils formed by erosion materials. Moreover, as the city grew, the original profile was smoothed by its residents, who filled in watercourses and flattened the steepest hills.

1.3. General description of the city

Part of the cityscape is depicted in Photograph 1-2. The town grew by phases, downward from the foot of the castle to the valley, which explains the differences visible in the photo. The oldest quarters, shown in the foreground, follow an obviously irregular pattern.



▲ Photograph 1-2

2. *Industrial engineer Francisco Javier Rojo wrote an extensive end of course dissertation at INTEMAC under the supervision of Ramón Álvarez, identifying buildings and damage.*

▲ Figure 1-2

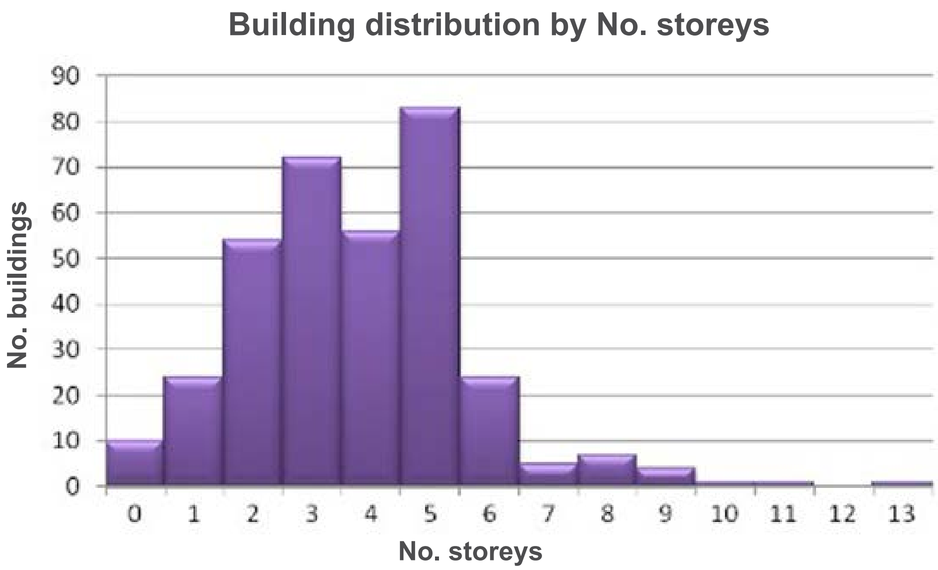
▲ Photograph 1-3

▲ Photograph 1-4

The buildings are lower lying and generally smaller in volume. Behind them in the middle ground, construction is more modern, the street layout is more regular and building volumes are larger. The following group is characterised by stand-alone mid-rise (over 10 storeys) towers. The valley, in contrast, is populated by detached single family units.

Photograph 1-3 shows a street in the oldest part of town, characterised by masonry buildings with no more than two or three storeys and a similar number of bays.

The part of the town occupying the lar-gest area was built in times of fastest economic growth, probably in the nineteen seventies. That would explain why most of the buildings surveyed dated from that decade, as shown in the graph in Figure 1-2[2], formulated with data on only the buildings for which reliable information was at hand.



▲ Figure 1-3

The layout in this part of the town is recognisable and comparable to that of many other Spanish cities (Photograph 1-4). The streets are wider and fairly straight, forming city blocks whose medium (six to nine storeys) and lower-rise buildings are practically contiguous (Photograph 1-5). Storey elevations are often uneven: i.e., the floor slabs in adjacent buildings are set at different heights.

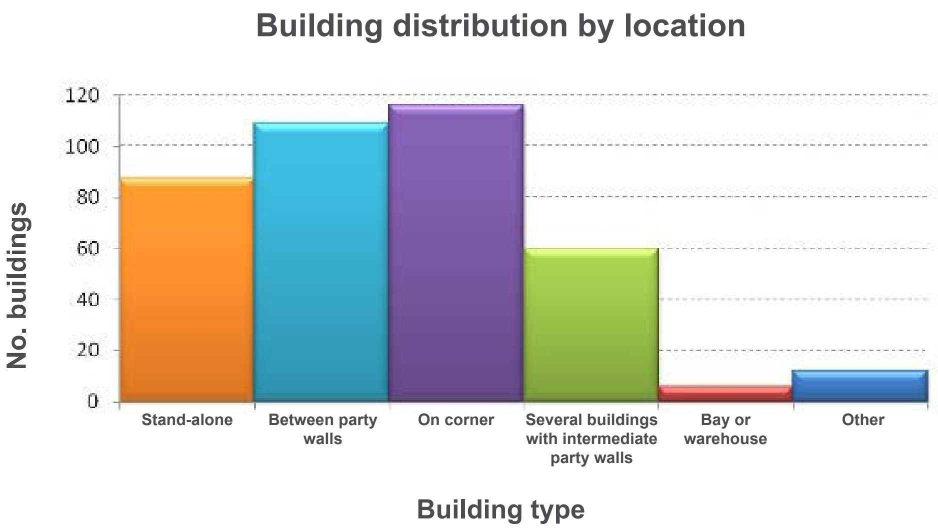
The relatively large area covered by this part of the city also explains why most of the buildings surveyed had more than three storeys, as shown in Figure 1-3.

As a rule, these buildings have reinforced concrete or steel (or combined RC-steel) portal frame structures.

That same typology is found in many recent buildings (Photograph 1-6), although the latter are sited farther from the town centre in areas where building volumes are much greater (often occupying entire city blocks).

▲ Photograph 1-5

▲ Photograph 1-6



▲ Figure 1-4

▲ Photograph 1-7

A number of likewise recently erected stand-alone towers, normally high-rises as shown in Photograph 1-7, also have conventional (portal frame and shear wall) reinforced concrete structures.

Outside of the above general description, the town also has an occasional distinctive development. The Alfonso X and San Fernando quarters (Photograph 1-8), built to house the victims of the 1973 floods, are typical examples. The buildings in these quarters are built around variations on a single model.

▲ Photograph 1-8

Most of the buildings surveyed in Lorca are located between party walls or on corners (Figure 1-4), in keeping with a conventional urban layout.

In most cases no real joint is visible between the party wall buildings. The separation generally consists of polystyrene panelling (Photograph 1-9), which in the most recent buildings is sealed with silicone along the façade. At roof level, however, the joints are sealed with mortar to prevent leaking (Photograph 1-10).

1.4. Masonry buildings

These are the town's oldest buildings. Barely any of the recent buildings surveyed had bearing wall structures, not even in the case of detached single family brick units. That short number in the sample may be due solely to the fact that this type of construction is found in the Guadalentín Valley, outside the urban centre and hence at a greater distance from the epicentre. Another factor may be that since such buildings are rarely insured, they would neither be the object of claims adjustment or lie within the scope of the INTEMAC survey.

As a rule such dwellings are small, with two or three storeys (the latter being much less frequent). Most were built over 50 years ago with masonry walls that support timber flooring and roof. An example of this type of construction is portrayed in Photograph 1-11.

▲ Photograph 1-9

▲ Photograph 1-10

▲ Photograph 1-11

▲ Photograph 1-12

▲ Photograph 1-13

A few recent buildings were nonetheless exceptionally found to follow this structural pattern. Most were homes built or enlarged by their owners outside established standards and without the technical expertise of specialised personnel.

The construction quality in these cases is generally poor. The materials used are inappropriate and laid to no bond or pattern and the general form is scantly suited to resist horizontal action. Moreover, given their age, these are the buildings that have undergone the most intense and often misguided remodelling. Cracking in some walls revealed that new openings had been made, former openings walled up (Photograph 1-12) or enlargements built with little or no regard for the existing structure.

Likewise due to their older age, these are the buildings where weathering-induced deterioration is most intense. Capillary moisture rising from the ground (clearly visible in Photograph 1-12 above), damp at the floor-wall abutment (where, moreover, problems due to insufficient support length -Photograph 1-13- or insect infestation are frequently observed) and want of satisfactory maintenance are the reasons for the less than optimal state of conservation of many of these buildings.

1.5. Portal frame buildings

Beginning in the mid-twentieth century, masonry construction gradually gave way to reinforced concrete construction. At first, singular construction procedures were used, such as in the Alfonso X el Sabio quarter (Photograph 1-14), in which the masonry walls were used as formwork (Photograph 1-15). This procedure results in a perfect connection between the masonry and the concrete portal frame, forming a mixed structure that works as a single unit. This construction system, known as confined masonry in which the masonry walls probably account for most of the structural strength, is used fairly frequently in some Latin American countries.

▲ Photograph 1-14

The horizontal structure consists of suspended beams (usually laid one way only, forming flat portal frames with the columns) and floor slabs initially featuring precast reinforced concrete, and shortly thereafter prestressed, joists. These slabs were not topped and the inter-joist filling was made from an assortment of materials. The notion of monolithic construction as the basic property to be required of any floor slab had not yet been developed.

These horizontal members have small spans, around 4 metres. The buildings are no more than four or five storeys high (Photograph 1-16) with clearances of normally not much over 2 metres per storey (with the exception in some cases of the ground storeys).

The enclosures, while not as fully restrained as described above, are integrated in and rest fully on the structure. They clearly constitute an example of enclosures with an obvious structural component (irrespective of the fact that they were not designed as structural members).

▲ Photograph 1-15

▲ Photograph 1-16

▲ Photograph 1-17

▲ Photograph 1-18

The soundness of these masonry infills may very likely have contributed to the earthquake resistance of buildings whose carelessly built structure, consisting of materials (smooth, inappropriately positioned reinforcing bars and concrete prepared in situ without the necessary control, Photograph 1-17) of questionable quality, does not in most cases meet the minimum standards of performance laid down in today's codes and regulations.

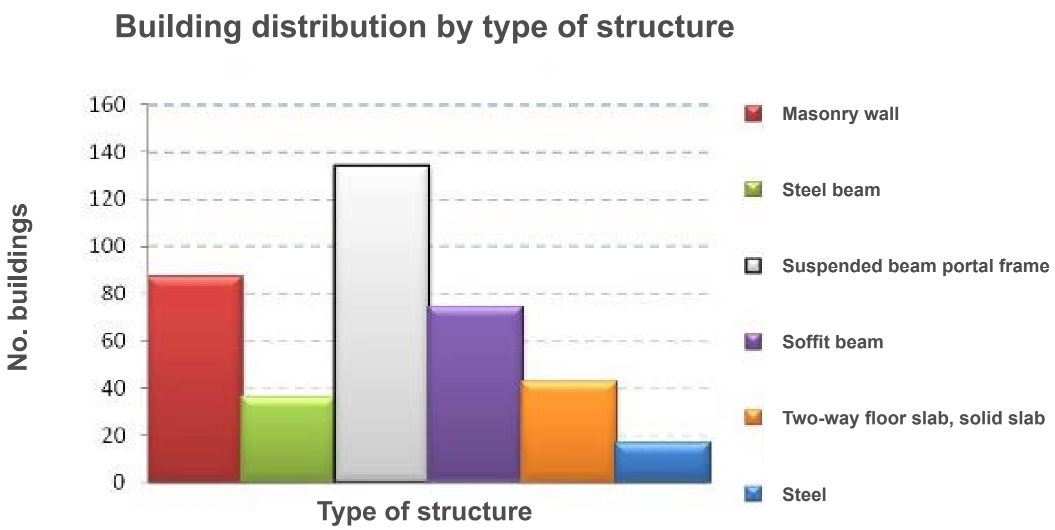
In a few years higher rise buildings were being built, with longer beam and floor slab spans and new construction systems. One of these consisted of the use of mixed solutions: steel beams resting on reinforced concrete columns (Photograph 1-18).

Today this solution is known to be wholly unsuitable, but it was used fairly profusely at the time. Such systems in fact account for a non-negligible portion of all the structural types identified (Figure 1-5).

The façades in such buildings rest only partially on the slabs (Photograph 1-19) and rise to form parapets around the (nearly always flat) roofs.

The buildings erected in the late nineteen seventies had eight storeys or over and structures consisting of flat portal frames inter-connected by one-way floor slabs with no topping. These buildings are wholly unable to guarantee the stiffness needed to resist horizontal action (in the direction perpendicular to the portal frames, at least). Column dimensions were engineered to accommodate gravity loads only. Enclosures were implicitly relied upon for bracing.

In the nineteen eighties the horizontal structure was modified: the use of soffit beams



▲ Figure 1-5

and topping on the floor slabs became standard practice that is still in place today. Floor slabs were built to greater depths to prevent the deformation associated with very slender solutions.

Façades began to be built in front of the floor slabs, resting on steel shapes often unable to provide sufficient anchorage in the event of horizontal action.

Many of these buildings are in less than ideal condition. The mere inspection of some of the damage presumably caused by the earthquake revealed that the actual origin was prior structural deterioration. One widespread problem is reinforcement corrosion, which may be attributed to poor quality concrete and careless workmanship. The concrete cover thicknesses measured in different sections diverges widely (undersized in some cases and oversized in others), with some of the steel found to have absolutely no protective cover. Cracks of all types are also common: due to plastic settlement (marking the position of the tie bars in the columns) or thermal contraction and shrinkage (generated in all likelihood by inadequate dosing) and along construction joints. Bug holes are also common.

▲ Photograph 1-19

The top of the column in Photograph 1-20 obviously looks different after the quake than it did before, but it is equally obvious that the tremor only caused the collapse of material that had already loosened. Consequently, the structure is no less safe after than before the earthquake.

Similarly, the earliest surveys revealed another widespread problem, deterioration at the bottom of the columns and walls due to capillary moisture from the soil.

▲ Photograph 1-20

Many columns were found to look like the one depicted in Photograph 1-21.

Flat slabs are often in place in the most recent buildings, normally with sacrificial lightening elements. Nonetheless, the soffit beam-one-way portal frame system described above is still found in some of these newer structures.

The structural layouts of these buildings is not always much better than in their older counterparts. Only new construction in recently developed areas exhibits a certain degree of formal regularity (in which the plan view logically reflects the regularity of the plot), although core problems such as differences in floor slab elevation in adjacent slabs and mechanical irregularities persist.

One especially serious flaw in the most recent developments is the widespread practice of setting the façades off the floor slabs. This, as discussed below, proved to be a key issue during the quake.

▲ **Photograph 1-21**

1.6. Conclusions

Generally speaking, buildings in Lorca are no different from what, in our experience, would be expected of any other Spanish city. Neither the materials used nor the construction systems or procedures are distinguishable from what is observed in other cities. This cannot be regarded in a positive light, however, because given the area's seismicity, its buildings should have distinctive features. Problems such as securing the masonry façade to the structure have not been wholly solved in Spain and the solutions deployed, which are similar across the country, are not always safe. The earthquake merely proved that they entail a certain hazard.

In this same vein, the scant quality of structures built with very poor materials by insufficiently skilled workers at times when the main objective was to build (quality mattered less) casts serious doubts on the suitability of a housing stock barely able to resist extreme loads. The problem, of course, is neither new nor exclusive to Spain. Articles published on recent earthquakes in Italy describe an essentially similar situation (Vicente *et al.* [57]).

Lastly, the deterioration of structures exposed for years to highly aggressive weathering with no maintenance to speak of has jeopardised building safety.

Such an unfavourable scenario might be thought to be inconsistent with the results of the severity of the earthquake. The city's structures were exposed to an action much more severe than what they were designed to resist and most passed the test.

This, in our opinion, is a misleading argument, for in most cases neither the structures were exposed to any load whatsoever (the non-structural elements actually bore most of the weight of the tremor), nor the forces were as great as might be assumed. Both these issues are addressed in the chapters that follow.

Description of the damage observed

2

2. Description of the damage observed

This chapter contains a listing of the damage initially observed in the city, which will be described in detail, enlarged upon and explained in the chapters that follow. The aim here is to simply provide an overview of the city immediately after the quake.

▲ Photograph 2-1

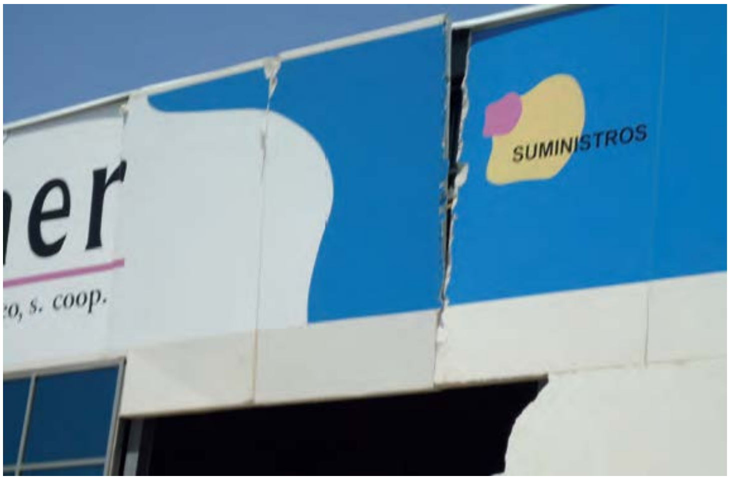
Many of the streets in Lorca looked like the one depicted in Photograph 2-1. Sidewalks, automobiles and roadways were littered with masonry rubble, the occasional whole section of façade or parapet and all manner of debris. Although this litter was found in all the city's quarters regardless of location, it was particularly abundant in areas with a prevalence of modern buildings, understood to mean the ones whose structure is separate from the enclosure, as opposed to older, bearing wall buildings.

Structural damage, however, was less obvious in this first impression. Aside from historical structures, only one building collapsed entirely (Photograph 2-2), although several collapsed partially (Photograph 2-3).

▲ Photograph 2-2

▲ Photograph 2-3

▲ Photograph 2-4



▲ Photograph 2-5

▲ Photograph 2-6

2.1. Damage to architectual elements

Observation of the buildings involved led to the conclusion that architectural elements collapsed due essentially to two mechanisms. Roof parapets (Photograph 2-4) and, to a lesser extent, some sections of enclosures, primarily on the upper storeys, seemed to have collapsed under loads normal to their plane, i.e., inertial bending moments that the bearing on the floor slab (the sole strength mechanism in the case of the parapets) was unable to absorb.

This problem appeared in many conventional masonry façades, but also in precast enclosures on some of the industrial bays (Photograph 2-5).

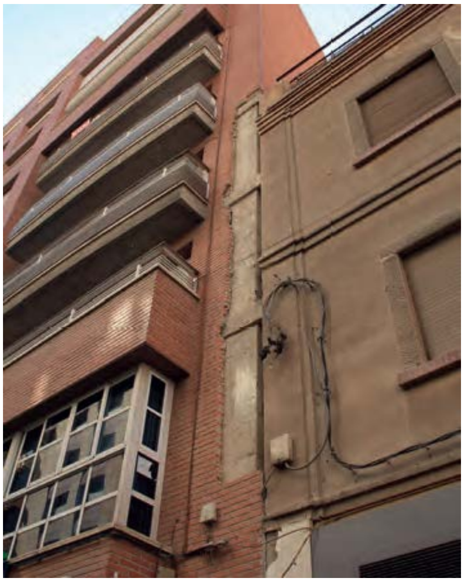
Lower façade failure followed a different pattern, consistent with collapse due to in-plane action originating at the edge. Such actions induce the characteristic “X”-shaped crevices (Photograph 2-6) observed in many of the town’s buildings.

Subsequent indoor surveys revealed the same damage pattern in both the main (around community areas) and secondary partition walls (Photograph 2-7). Here the “X”- shape was not always as clear as in the enclosures because the cracks tended to follow along the cableways and other conduits built into the walls.

Many of the provisional ground storey enclosures around store fronts in the more recent buildings also collapsed. Most consisted of very slender masonry infills (Photograph 2-8) ineffectively anchored to the structure and carelessly built.

▲ Photograph 2-7

▲ Photograph 2-8



▲ Photograph 2-9

This type of failure due to in-plane action also affected the wall sections closest to an adjacent building, where the full impact of pounding was felt (Photograph 2-9). This was particularly (although not exclusively) visible in buildings with storey divisions at different heights.

A high percentage of façade panelling, especially decorative panels on ground storeys, fell away (Photograph 2-10), revealing attachment systems of such dubious quality in most cases that they would be regarded as unacceptable even in non-seismic regions. This issue will be revisited throughout the present discussion: much of the post-earthquake damage is attributable less to the quake itself than to the use of unsafe construction systems.

▲ Photograph 2-10

▲ Photograph 2-11

▲ Photograph 2-12

▲ Photograph 2-13

2.2. Structural damage

On the masonry buildings, again excepting historic structures, the most obvious damage consisted of wall separation at the abutments (Photograph 2-11), indisputably triggered by the earthquake but originated by the lack of any effective inter-wall connection.

In some cases this separation induced the collapse of one of the sections of wall or of the ceilings resting on them (normally roofs consisting of rolled timber with lath and plaster membrane inter-joist fillings).

Slanted cracks of variable widths were observed in many surfaces, often stretching between openings in wall sections (Photograph 2-12).

Subsequent surveys revealed other types of indoor damage, frequently associated with remodelling undertaken with little regard for the original structure. One consequence, shown in Photograph 2-13, is lintel failure.

In some floors, joints were observed to widen along the separation between slab elements (Photograph 2-14), normally precast reinforced or prestressed concrete joists with any of a wide variety of inter-joist fillings. These slabs had no topping.

In the initial surveys, the most obvious damage observed in reinforced concrete buildings was associated with short columns (Photograph 2-15). Entire lines of façade columns, all short, collapsed in several buildings.

▲ Photograph 2-14

▲ Photograph 2-15

Severe column damage was also observed, generally on the ground storeys, attributable to the interaction between the structure and the masonry fillings (Photograph 2-16). In other cases the damage was due to pounding between adjacent buildings, especially where the floor slabs in the buildings involved were at different heights.

Other types of failure, such as in some unsuitably reinforced shear walls (Photograph 2-17) or in stairwell enclosures and slabs, appeared in subsequent more detailed surveys of the buildings.

▲ Photograph 2-16

▲ Photograph 2-17

2.3. Conclusions

Non-structural elements, essentially masonry infills and staircases, exhibited the most visible post-quake damage. The implications of that observation are of major consequence, in our opinion.

From the standpoint of personal safety, this sort of damage (in particular, parapet collapse) rather than structural damage caused the most serious losses. That clashes with the widespread belief that preventing structural collapse suffices to save lives during earthquakes. Certain interpretations of the codes tend to classify all damage to non-structural elements within the realm of serviceability limit states, implying that damage to such elements would be admissible under the ultimate limit state criterion (associated with personal safety).

The Eurocode on seismic design [3], however, is very clear in this regard. Its listing of ultimate limit states explicitly states that:

> *...“It shall be verified that under the design seismic action the behaviour of non-structural elements does not present risks to persons and does not have a detrimental effect on the response of the structural elements.”*

In the item on non-structural elements, the code specifies the elements that must be verified:

> *...“Non-structural elements (appendages) of buildings (e.g. parapets, gables, antennae, mechanical appendages and equipment, curtain walls, partitions, railings) that might, in case of failure, cause risks to persons or affect the main structure of the building or services of critical facilities, shall, together with their supports, be verified to resist the design seismic action.”*

Serviceability limit state verification would appear, in short, to be limited to the non-structural elements whose failure would not entail personal injury, such as lightweight plaster board-type partitions or similar. All other elements, such as heavyweight partitions or enclosures, would have to be engineered to ensure their stability during the most severe earthquake.

The problem is that in most cases, structural performance in earthquakes (and in the ultimate limit state, where the legislation envisages that alternative) is calculated assuming that the structure yields entirely, with the concomitant formation of a stable plastic mechanism, which entails substantial deformation incompatible with the stiffness of masonry walls.

This point will be re-addressed in subsequent chapters.

While less consequential, the economic issue also merits some attention, because repairing non-structural elements carries a very high cost (much higher than repairing structural members, at least in Lorca). This is due not only to the difficulty involved in the repairs (which in some cases consist of simple replacement: the sections damaged must be removed and new ones built), but also to the extent of the problem (most of the lower storey masonry infills failed in Lorca) and to the fact that the repairs also involve re-building M&E services, or at least the components built into the walls (such as cableways and plumbing and heating pipes).

While this section is not intended to justify the ultimate significance of masonry infills, it does aim to identify the imbalance between the effort devoted to engineering the various types of (structural or non-structural) elements and their actual relevance in earthquake scenarios.

The earthquake

3

3. The Earthquake

This section analyses the earthquake itself, with a review of legal provisions and the response of conventional buildings to the Lorca quake.

3.1. Description

According to data published by the National Geographical Institute, the epicentre of the two main earthquakes recorded on 11 May was located a mere 2 km northeast of Lorca.

For all practical purposes, such a short distance is tantamount to a tremor directly underneath the town. The term "*epicentre*" is derived from an abstract model that reduces to a single point what is actually an extensive area affected by a slip along the fault. In this case, if projected onto the surface, it would spread across more or less the entire municipality.

In addition to their proximity, another essential characteristic of the Lorca quakes was their scant depth, at around 2 000 m. These facts explain the intensity of the damage wreaked by earthquakes whose magnitude, at 5.1 (National Geographical Institute), can hardly be regarded as more than moderate.

Magnitude is a measure of the energy released by an earthquake, which in turn depends on the size of the area along the fault affected by the slip and the relative displacement of the rock on the two sides of the fault plane. Magnitude, in a nutshell, refers to the "*size*" of the quake or, from another perspective, the amount of energy released.

Intensity is a measure of its effects at a given site and depends, among others, on the distance between the site and the epicentre. The effects of an earthquake are logically attenuated with distance. Consequently, a quake of great magnitude and distant from a given site may result in the same intensity as an event closer to the site but of much lower magnitude.

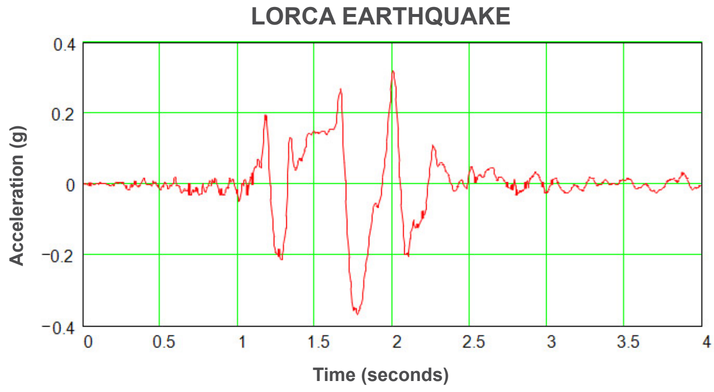
On average, more than two earthquakes of a magnitude similar to the one recorded at Lorca occur daily somewhere in the world, but since their surface effects are highly localised, the likelihood of damage to urban centres is small.

The foregoing explains why such a relatively moderate earthquake proved to be so intense at Lorca. The accelerations logged[1] by the town's seismograph (Figure 3-1) were the highest ever recorded by the national seismic network.

As the accelerogram shows, the peak acceleration recorded came to around 0.37 g, or triple the requirement laid down in the existing legislation for normal buildings. Moreover, since the instrument that recorded this value was sited on rock, acceleration may have been even greater in areas of the city standing on several layers of topsoil.

1. *Motion tends to be expressed in engineering as acceleration because the forces acting on a body in motion are known to be proportional to acceleration.*

2. *For a clearer understanding of the meaning of these values, suffice it to say that they can only vary upward, for otherwise the building housing the instrument would rise off the ground. People would momentarily float in thin air because the ground would drop faster than gravity could accelerate their bodies.*



▲ **Figure 3-1**

3. *The seismograph installed in the town was not oriented exactly along the cardinal axes, but at a slight angle, so that orientation 1 is approximately 30º off the north axis.*

That such high acceleration can be recorded for moderate earthquakes should not be surprising, for it depends only on the coincidental presence of a seismograph close to the epicentre. In fact, as the worldwide instrumental network grows denser, records of acceleration values that are surprisingly high in comparison to quake magnitude are becoming more and more common. In the 2011 earthquakes at Christchurch, New Zealand, the acceleration, which was vertical[2], doubled gravitational acceleration during events with a magnitude of only 6.2, i.e., greater than at Lorca but much lower than observed in many other quakes (Yuen Kam and Pampanin [61]). Such a large number of instruments had been installed in the surrounds that some were (unsurprisingly) positioned directly over the epicentre.

Nonetheless, according to the National Geographical Institute's report on the Lorca events, the acceleration was too high to be consistent with normal attenuation models.

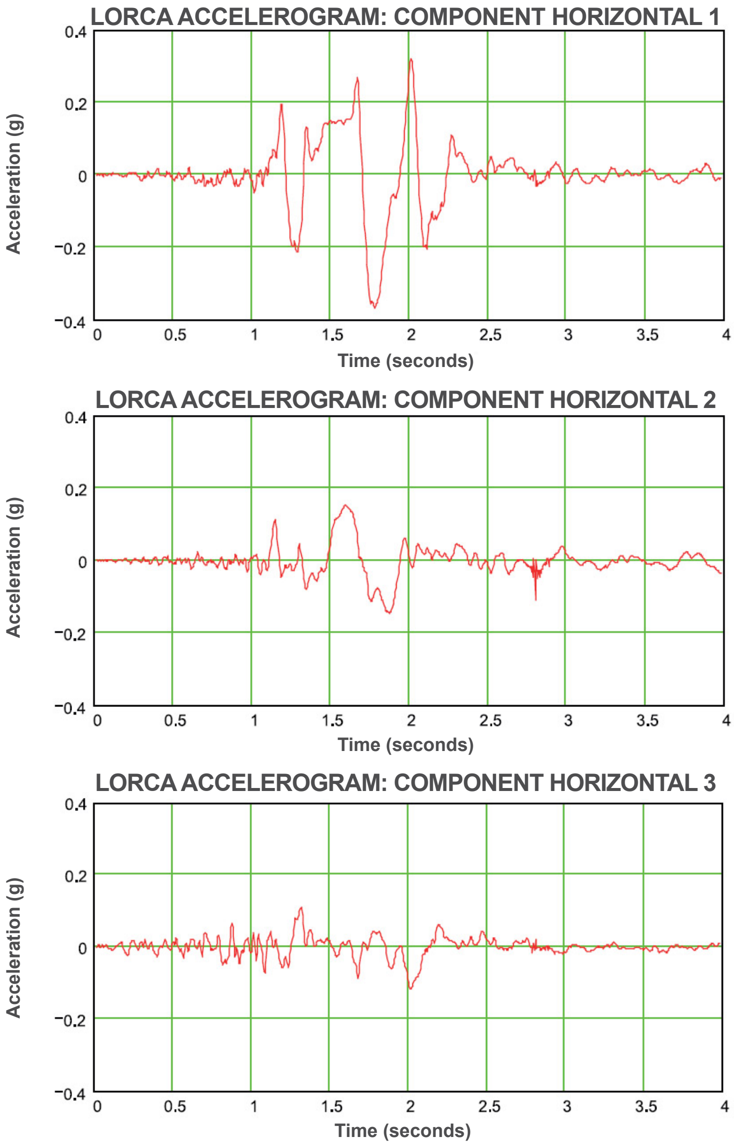
The effects observed are, then, characteristic of what is normally termed a "*near earthquake*".

Figure 3-2 shows the three seismographic components. One graph, designated[2] "HOR. COMPONENT 1" shows values much greater than in the seismogram for the other horizontal component: this behaviour is typical of near earthquakes.

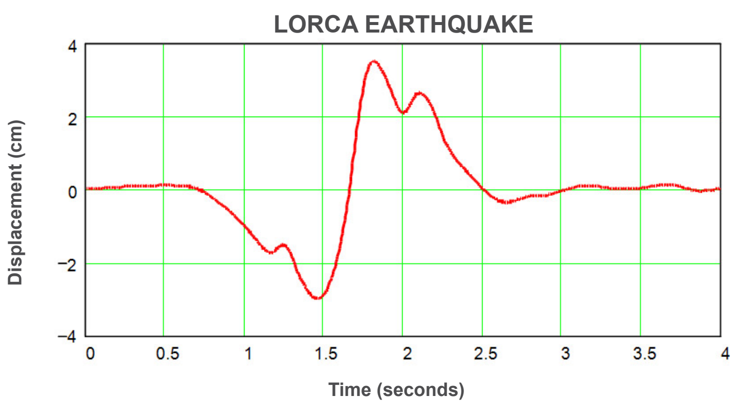
The vertical component is likewise observed to be of scant significance, a development not at all typical of near earthquakes.

If ground desplacement instead of acceleration is plotted, the result is the graph reproduced in Figure 3-3, which shows more clearly that the tremor, a single pulse, lasted scantly more than 1 second and had an amplitude of 3 cm in each direction.

This evidence clashes with many townsfolks' accounts, according to which both duration and amplitude were much greater. The explanation for this apparent inconsistency may quite simply lie in the fact that the witnesses' reports reflect the visible effects of the tremor, such as moving lampposts and traffic signs, structures with scant damping capacity that remain in motion after the excitation disappears.



▲ **Figure 3-2**

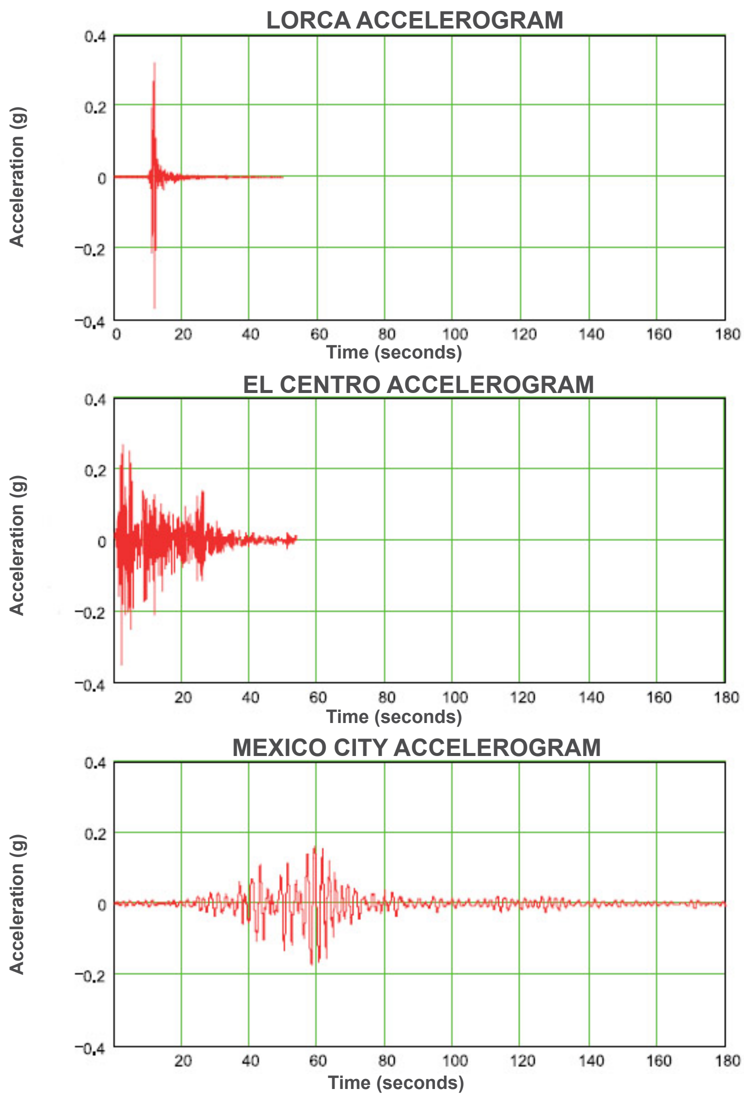


▲ **Figure 3-3**

The severity of the Lorca quake can be better visualised if compared to others. In Figure 3-4 it is compared to a very well known tremor that shook California in 1940, known as El Centro. This accelerogram was of particular notoriety in its time because, as in Lorca, it showed higher values than hitherto recorded (although in Lorca the record was only national).

Note that the time scale here is much smaller than in in the preceding figure, on which the duration of the El Centro quake could simply not be represented. In contrast, since the acceleration recorded in the two quakes was very similar, the vertical scale required no modification.

The difference in duration is obvious, despite the fact that the El Centro accelerogram is not typical of an especially distant quake. The 1985 Mexico City earthquake, whose accelerogram, also shown in Figure 3-4, is discussed in the following item, lasted for over 60 seconds.



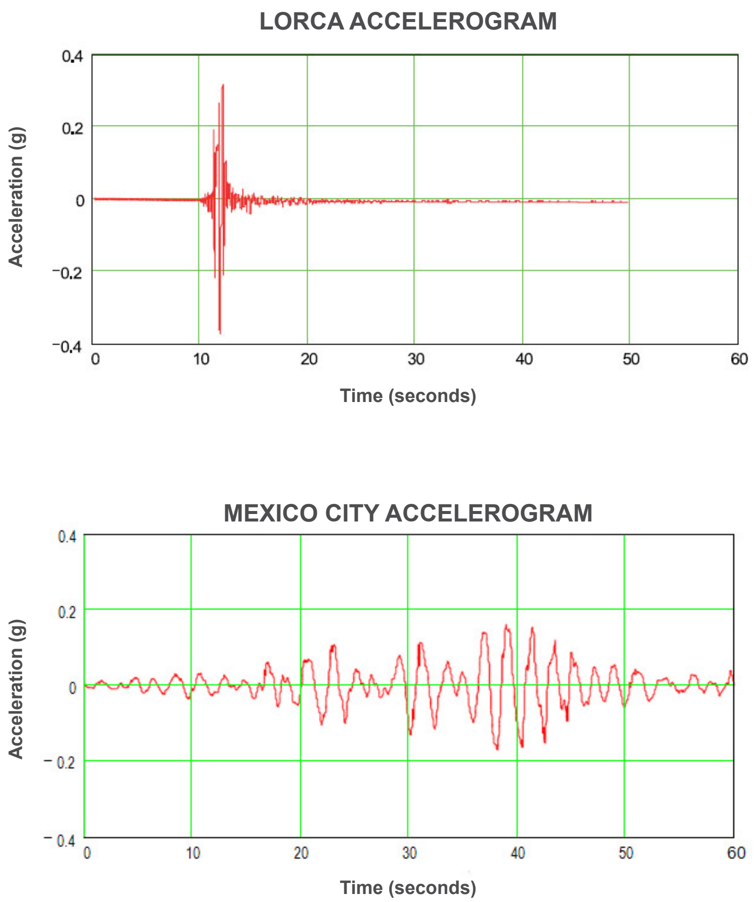
▲ Figure 3-4

3.2. Soil effects

On 19 September 1985, an earthquake with a magnitude of 8.4 occurred in the Pacific Ocean off the coast of Lázaro Cárdenas, Mexico. When the seismic waves reached Mexico City, 400 kilometres away, their amplitude had been attenuated to the point that some instruments recorded an acceleration of scantly 0.05 g (compared to the 0.36 g recorded in Lorca). Those instruments were located in areas of the city where the topsoil over the bedrock was very thin or even non-existent.

The seismographs sited on thick layers of topsoil, however, logged the acceleration values shown in Figure 3-4, referred to above.

These accelerograms exhibit singular characteristics. Firstly, the amplitude, at nearly 0.2 g, trebles the values recorded on rock. Secondly, the shape of the accelerogram is very characteristic. Figure 3-5 is a window in the horizontal time scale shown in Figure 3-4. Whereas the Lorca pulse is extremely short but with a very broad amplitude, the accelerogram for the Mexico City quake has a clearly defined frequency (with a



▲ Figure 3-5

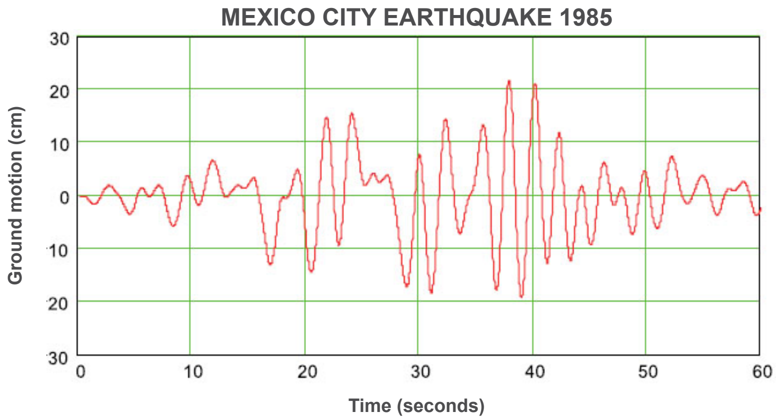
period of around 2 seconds). As a result, the most slender buildings, with natural periods of approximately 2 seconds, were the ones most severely affected. Stiff buildings (the cathedral, for instance, whose structure consists of thick masonry walls) went unharmed.

These effects are more clearly visible in Figure 3-6. Moreover, ground motion reached 20 centimetres in each direction (compared to 3 cm in Lorca).

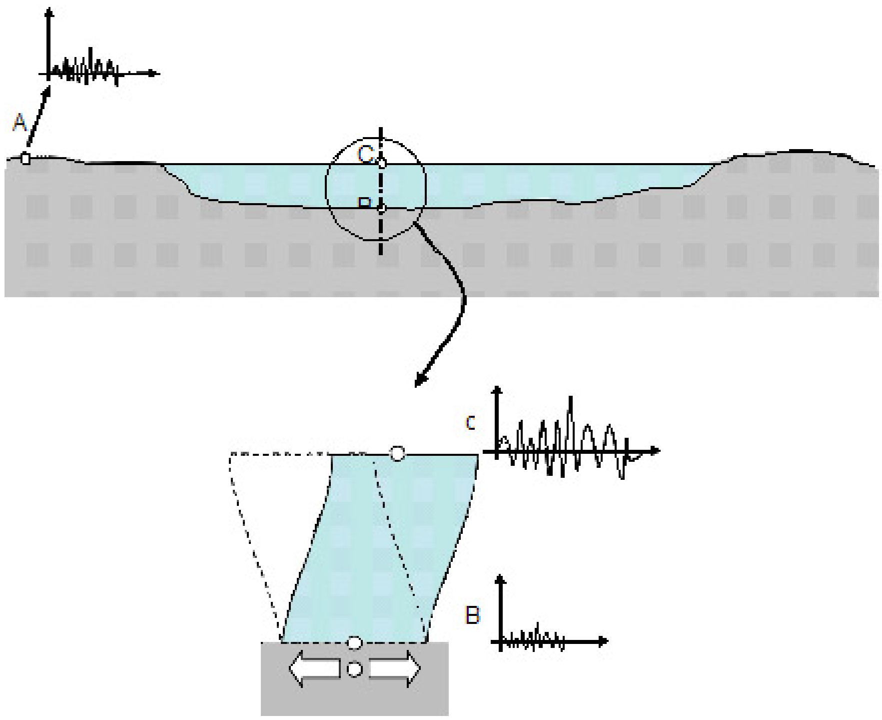
The combination of the characteristics described, namely duration, amplitude and scant frequency content, proved to be particularly lethal. Thousands of people died (7 000 to 10 000 according to most references) and hundreds of buildings collapsed.

The Mexico City quake is an extreme case of soil-induced amplification.

For readier understanding, this occurrence is simplified in the diagram in Figure 3-7. When the base of a soil layer is subjected to horizontal motion, the layer moves like any other elastic system: i.e., displacement at the top (the surface in quakes), is always different from (and often greater than) at the base. Such behaviour is similar to that of buildings undergoing displacement (quaking) at the base. The major differences are attributable to the soil, in which stiffness is much more distinctly non-linear than in buildings. Álvarez *et al.* [56] provide a simple analytical formulation of the problem. The use of software such as *EduShake*, the cost-free demo version of *Shake*, is likewise recommended.



▲ Figure 3-6



▲ **Figure 3-7**

Similar circumstances were present in Lorca. While the initial city core, in the foothills, stands on very hard ground or rock (the seismograph that recorded the quake was located in the basement of the former municipal jailhouse, whose foundations are bedded in rock), much of the town's growth has been absorbed by the valley, where sedimentary soil prevails. In addition, buildings have been erected on all manner of intermediate soils and in some cases on less than sturdy landfills covering old watercourses.

With such a complex geotechnical profile, amplification would clearly be expected, although to date and to our knowledge, those effects have not been precisely quantified.

The thorough and rigorous field work conducted by the Institut Geológic de Catalunya, the Universitat Politécnica de Catalunya, the Asociación Española de Ingeniería Sísmica and the Regional Government of Catalunya [50] failed to deliver any such precise quantification. Layers of topsoil that induced amplification were identified, along with clear discontinuities between rock and soil, but the exact magnitude of the amplification could not be established.

The acceleration reached in some quarters, then, may very possibly have been higher than the values recorded, and the shape of the accelerograms themselves may also have been different in those quarters (due to the frequency filtering induced by the soil). That effect, if any, may nonetheless be masked by other equally significant parameters, such as the distance from the epicentre, which is of cardinal importance in such near tremors.

The La Viña quarter may be regarded as a typical example. Located in the southwestern part of the town, which stands on soil, it may well have undergone an earthquake of different characteristics than recorded by the seismograph as a result of two equally important effects: the presence of topsoil and the quarter's greater distance from the epicentre.

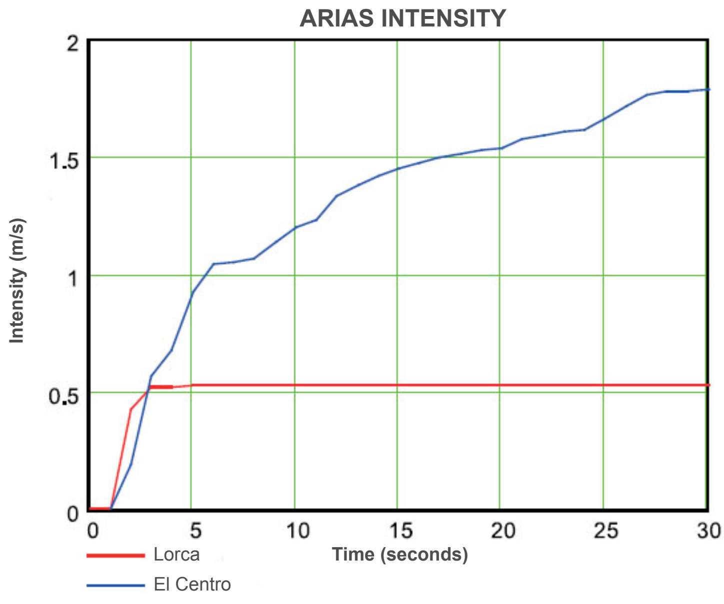
3.3. Other ways to describe earthquakes

As noted in the preceding sections, the acceleration recorded at Lorca trebled the legislative requirement. The possible conclusion might be that the Lorca quake was three times more damaging than officially envisaged. That in fact is not entirely true. Ground acceleration is only one, and according to some authors, not the best, parameter for defining the destructive power of earthquakes (see for instance, Orosco and Alfaro [51] or Schmidt and Quirós [59]).

In the foregoing description, the Lorca earthquake was compared to others with lesser acceleration but particularly devastating effects. The duration or frequency content of earthquakes may be much more consequential than their acceleration.

This idea is so obvious that from the outset, seismic engineering defined a wide range of parameters with which to measure the destructive potential of earthquakes. The Housner intensity, established in 1952, was one of the first, although perhaps the best known, proposed by Arias in 1970, is formulated as follows:

$$I_A = \frac{\pi}{2 \cdot g} \int_0^{t_t} a(t)^2 \, dt$$



▲ Figure 3-8

While strictly speaking the integral extends across the entire duration of the quake, it is usually plotted against time. In other words, the equation plotted is:

$$I_A(t) = \frac{\pi}{2 \cdot g} \int_0^t a(\tau)^2 \, d\tau$$

The plots for the El Centro and Lorca earthquakes are reproduced in Figure 3-8. Note that the former was three times more intense than the latter.

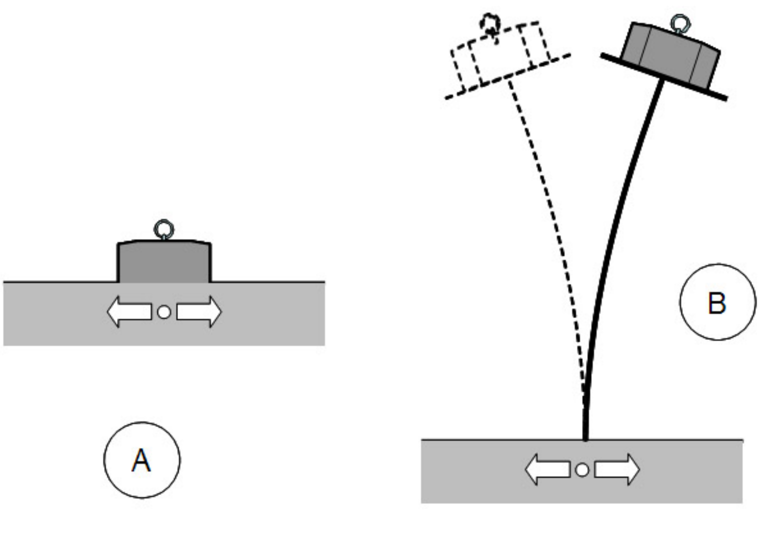
The above clearly shows that ground acceleration is not the best indicator of earthquakes' destructive power. It may be argued that, while not the best, it is the only parameter used in Spanish legislation which, like many others, bases all safety criteria on ground acceleration. From that standpoint, the initial assertion that the Lorca quake was three times worse than envisaged in the legislation would hold.

Even that assertion can be fine-tuned, however. All seismic legislation aims not only to ensure that structures are able to bear the horizontal actions generated by the peak ground acceleration, but also that they do so stably, i.e., with no loss of strength. That consideration is the sole justification for checking the structure for a set of static actions only, i.e., for using a mere snapshot instead of the repeated actions (the successive tremors present in any earthquake) involved in the actual events. The objective of many legal provisions is just that, to ensure the stability of the structural response, although such provisions have not been translated into design practice.

3.4. Effects of the earthquake on structures

The preceding item describes one of the features of the Lorca earthquake, the ground motion involved. That discussion addresses properties such as ground acceleration and motion and the frequency content of the accelerogram.

In actual fact, none of these parameters is particularly significant, at least directly. The accelerogram is, of course, essential because the forces acting on the objects involved are proportional to the acceleration and the proportionality factor is mass. That would only be applicable, however, to objects standing on and stiffly secured to the ground (as in the sketch in Figure 3-9 A). If the objects are flexibly attached to the ground (Figure 3-9 B), all the foregoing is meaningless and the forces acting on the objects, and their displacement, are quite different.



▲ Figure 3-9

This idea is illustrated in Photograph 3-1. All the top pieces on the masonry pilasters fell to the ground. The inference is that the stress generated between these blocks of stone and the pilasters on which they rested was stronger than friction and the mortar bond together (this would have been highly unlikely if the blocks had rested on the ground, in which case friction alone would have sufficed to absorb the horizontal force amounting to 36 % of its weight, the peak ground acceleration reached during the quake). In this same vein, once the bond was broken, the relative displacement between the blocks and the pilasters must have greatly exceeded the 3 centimetres recorded at ground level.

▲ Photograph 3-1

▲ Photograph 3-2

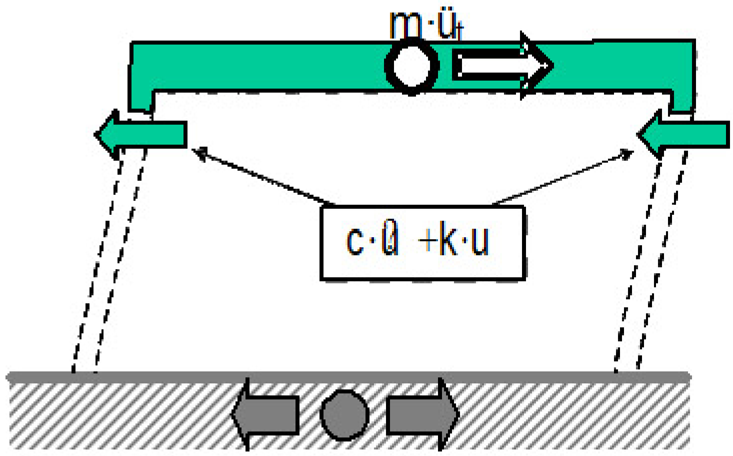
The IGC report [49] describes even more spectacular examples. The dome on Nuestra Señora del Rosario Chapel became detached from the walls on which it rested after shifting by around 15 cm. A less striking but equally illustrative example is given in Photograph 3-2: a façade which at the top may well have exceeded the aforementioned values.

These examples, while apparently simple, can actually only be analysed with much more elaborate models than the ones described below. Such models not only lie beyond the scope of the present text (and its authors' expertise!) but also deviate from the object of this chapter, because they are more concerned with the characteristics of each specific structure than of the quake originating the respective loads.

3.4.1. Response spectra

This term refers to one of the most commonly used procedures to describe the effect of earthquakes on structures.

The behaviour of the system diagrammed in Figure 3-9 B or of the pilasters depicted in Photograph 3-1 is not actually much different than would be exhibited by any building.



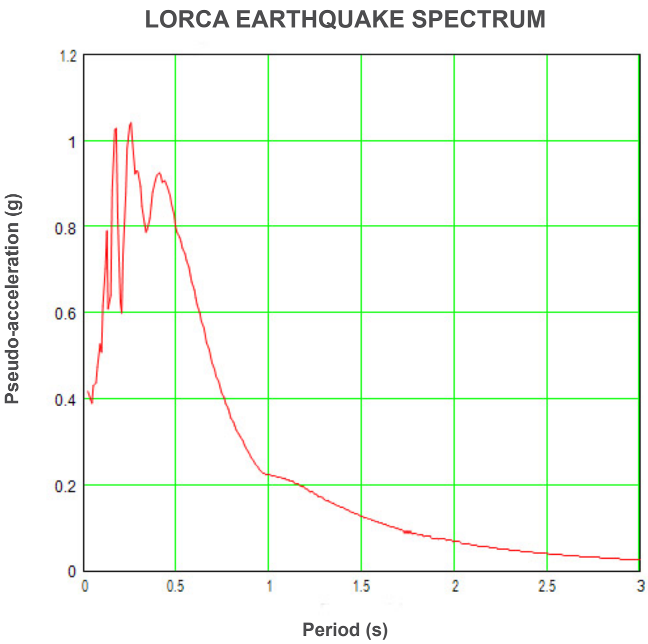
4. *If after applying a given lateral deflection, the lintel is released, the period is the time that it takes to return to its original position after swinging back (assuming very low damping). Note that if the initial motion is increased, when the lintel is released, it takes exactly the same time to return to the initial position: that means that it needs to move at higher speed, for the distance is greater.*

▲ **Figure 3-10**

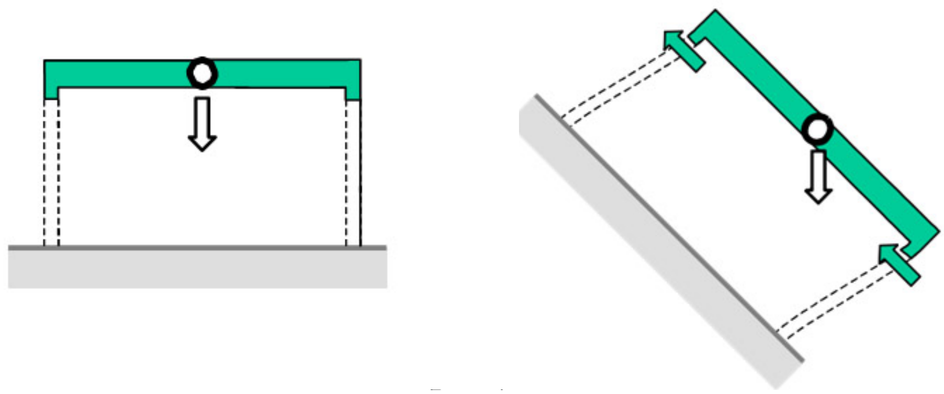
Simply defined, building behaviour in response to horizontal actions can be likened to that of a heavy mass (floor slab) separated from the ground by a flexible element (columns). The lateral loads exerted on the slab by the earthquake would be the product of its mass times the acceleration (not the ground acceleration, but another, normally much greater, value), together with the elastic forces transferred to the slab by the columns and any others due to damping (Figure 3-10).

The peak forces generated by a given earthquake would be the result of the peak floor drift, although as discussed in Annex 1, these forces are normally expressed as the result of the product of the floor mass times a term expressed as acceleration. That term depends on building characteristics and more specifically on its period[4]. The relationship that expresses this dependence is known as the "*pseudo-acceleration response spectrum*" (often simplified as the "*response spectrum*" or simply the "*spectrum*"). The values for Lorca are shown in Figure 3-11.

The interpretation of this graph, as for any other spectrum, is quite simple. For structures with a very small natural period, very stiff columns or lightweight floor slabs, the slab motion equals the ground motion; the acceleration is therefore the same on both and the peak horizontal force is the product of the mass times the peak ground acceleration (the 0.36 g cited above).

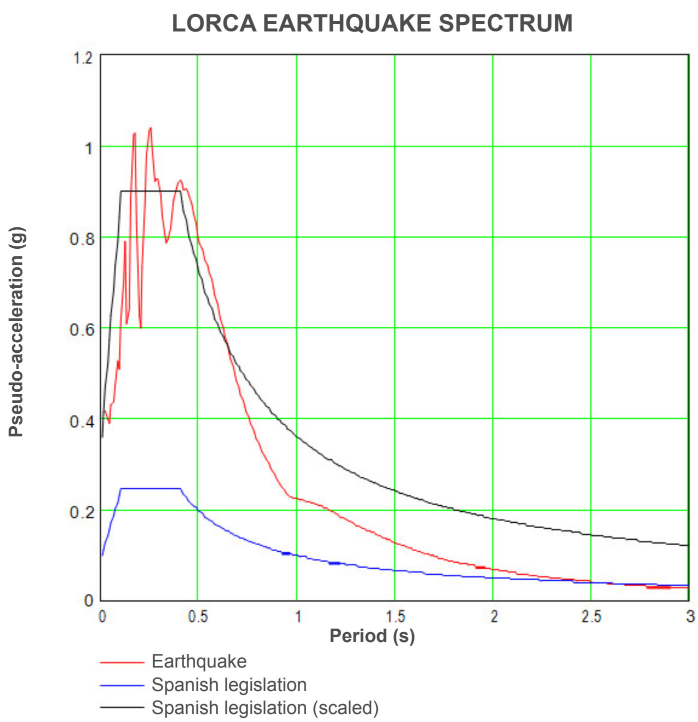


▲ **Figure 3-11**



▲ Figure 3-12

On the other extreme, in structures with very large natural periods, flexible columns or very heavy floor slabs, the slabs do not actually move in absolute terms (the ground moves but the columns are so flexible that they fail to transfer the motion).



▲ Figure 3-13

At the typical period values found in buildings, amplification is substantial. For a building with a natural period on the order of 0.5 seconds (typical, for instance, of 5 or 6-storey concrete portal frame structures), the peak forces generated by the Lorca earthquake would be the product of multiplying the mass by an acceleration of 0.80 g.

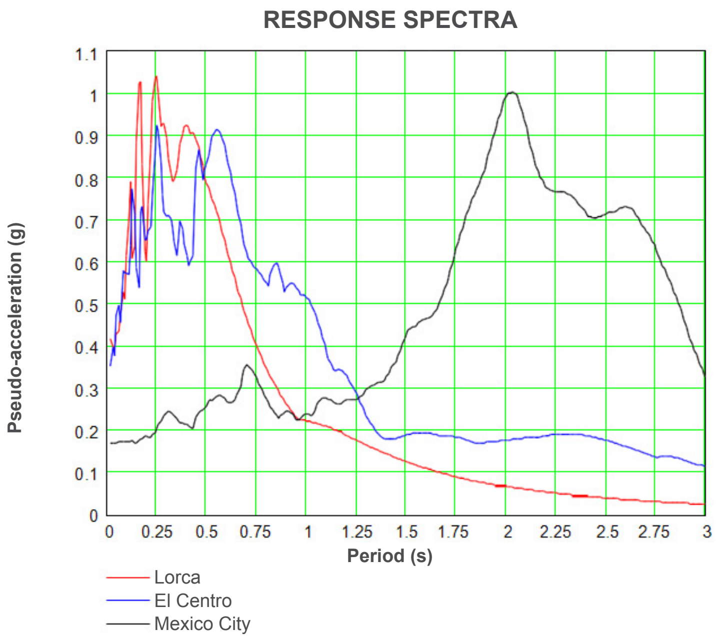
To visualise what such forces mean, suffice it to say that the building would have to be slanted at an angle of slightly over 50º to obtain a component in the slab direction equivalent to the loads induced by the quake (Figure 3-12).

Figure 3-13 reflects the differences between the actual spectrum (red line) and the spectrum envisaged in the code (blue line), i.e., the equivalent forces actually acting on the building and the forces laid down in the legislation. Note the substantial differences between the two. Even when the spectrum as envisaged in the code is scaled up to the acceleration recorded, substantial differences are observed, with amplifications concentrated in the lowest periods (stiff buildings).

Figure 3-14 shows both the Lorca spectrum and the spectra for the El Centro (1940) and Mexico City (1985) quakes to which it was compared in earlier items. The graph clearly shows that the widespread notion that seismic action is unique and clearly defined hardly holds up under the evidence. The Lorca earthquake had very little in common with the Mexico City event. Not even the building types affected in the two cases would be the same.

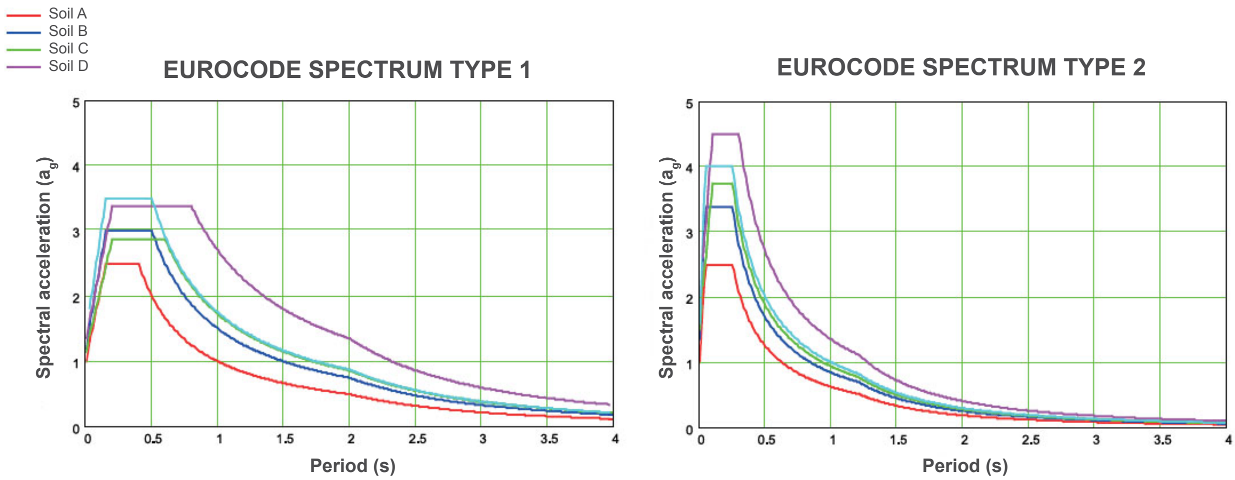
Similarly, Figure 3-15 shows the two types of spectra defined in the Eurocode on earthquakes. The first, type 1, would be applicable when the quakes most likely to occur at the site are of moderate to high magnitude. The second would be applicable to sites where low magnitude, near earthquakes are the most likely events.

According to the Eurocode approach, information would be needed on the most likely type of quake at each site to apply the respective spectrum to the design. Even in the best of cases, assuming that such information is available and reflected in the applicable code, the approach would render the engineering more complex.



▲ Figure 3-14

At this writing, work is ongoing on the definition of spectra for inclusion in the legislation.

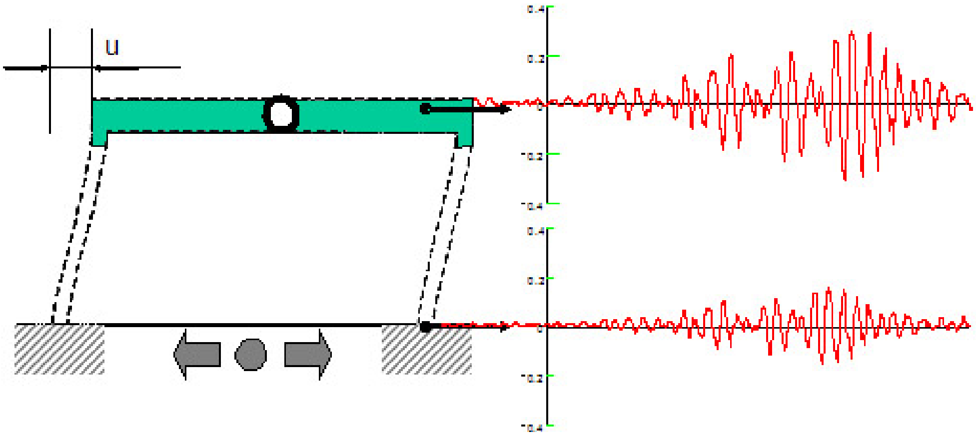


▲ Figure 3-15

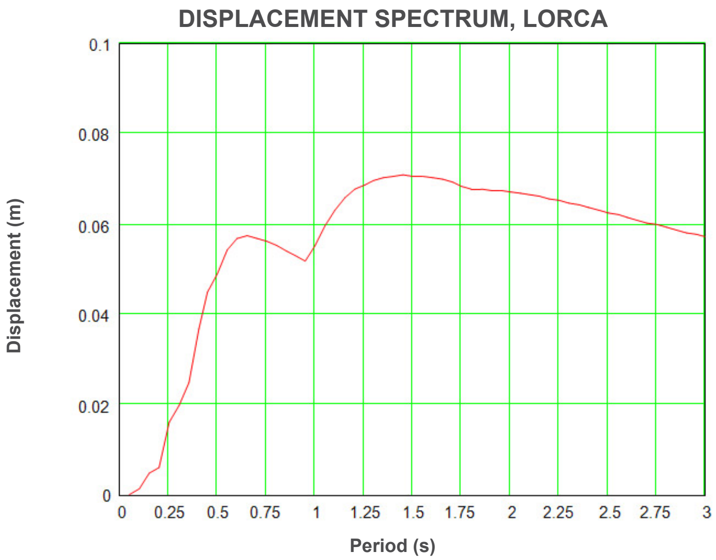
3.4.2. Displacements

Displacement spectra are analogous to pseudo-acceleration spectra. They show the peak values, in this case of relative displacement (between the floor slab and the ground), versus the period of the structure for a given earthquake (Figure 3-16).

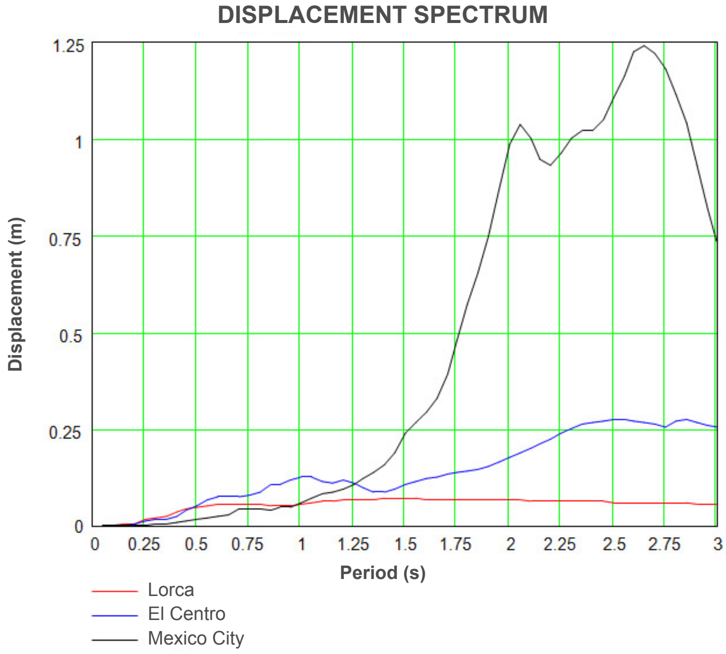
The displacement spectrum for the Lorca earthquake is reproduced in Figure 3-17. As in the case of the acceleration spectra discussed in the preceding item, the specific characteristics of this quake are most readily perceived by comparing its spectrum to those for other seismic events. Figure 3-18 is particularly illustrative in this regard: the displacement induced by the Mexico City earthquake in slender buildings was upward of 1 metre, compared to slightly over 6 cm in Lorca.



▲ Figure 3-16



▲ Figure 3-17



▲ Figure 3-18

3.5. Conclusions

In this chapter, the specific characteristics of the Lorca earthquake are illustrated by the simplest descriptive device, comparison with other quakes.

The most obvious conclusion may be the practical difficulty inherent in characterising actions so complex that it must be broached not from one but from several fields of science. The analysis of the origin, transmission and effects of earthquakes calls for specific expertise characteristic of different disciplines.

Even when the study is limited to the most immediate characteristic of quakes, ground motion, and such apparently clear and objective evidence as the instrumental record of that motion is at hand, answering even the simplest questions may prove to be difficult, for these authors at least. For instance:

Do we know what really happened during the earthquake? Or to put it another way: are the records available truly representative of the quake?

Yes, as regards the general (the truly important) characteristics, such as the severity of the tremor and its scant duration. Nonetheless, if the instrument had been sited in another part of the city, it may very likely have recorded different acceleration values (which would have been very high in any event) or the accelerogram may even have been shaped differently. When the slip causing the earthquake is so close and so shallow, ground motion may vary substantially within very short distances. That, in conjunction with the differences in the type and depth of topsoil in Lorca, means that the acceleration values very probably differed from one quarter to the next.

5. *One of the reasons for penalising the use of soffit beams, for instance, which in non-repetitive action can afford acceptable ductility, is the degradation of their behaviour curve after a short number or cycles, when the area within the hysteresis cycle practically disappears. See the study described in Benavent-Climent [18].*

Was the Lorca earthquake three times worse than envisaged?

No. Such an assertion is only valid where peak ground acceleration is accepted as the sole reference. This parameter is one, but not the only and in all likelihood not the best, of the several used to characterise the potential damage induced by a given earthquake. This chapter has aimed to show that the effects of a quake depend largely on other characteristics which, like duration, may not even be directly addressed in the legislation on design procedures (although they are implicitly included in many specifications[5]).

Is a similar earthquake possible at any other place on the peninsula?

Yes, inasmuch as the Lorca quake was not the most severe predicted for the area. What is less likely is that such an earthquake would occur directly underneath a major city.

Masonry buildings

4

4. Masonry buildings

This section analyses buildings whose vertical structure consists essentially of masonry walls, although often supplemented by an occasional linear member (such as intermediate columns, beams or mullions).

While formally similar to the non-structural walls used in the enclosures analysed in Chapter 5, structural walls exhibit a specific characteristic that determines their behaviour: the presence of compressive stress due to the effect of gravity loads.

Such a general assertion may naturally be nuanced. Enclosures and partitions also actually bear vertical loads that in theory should be borne by the structure. Consequently, they are likewise exposed to substantial compression that improves their response. The only difference is that these are unplanned forces whose value cannot be precisely quantified and which should not therefore form part of safety calculations.

At the other extreme, some structural walls, designed to brace the building, are intended to bear horizontal action only, and receive no contribution from gravity.

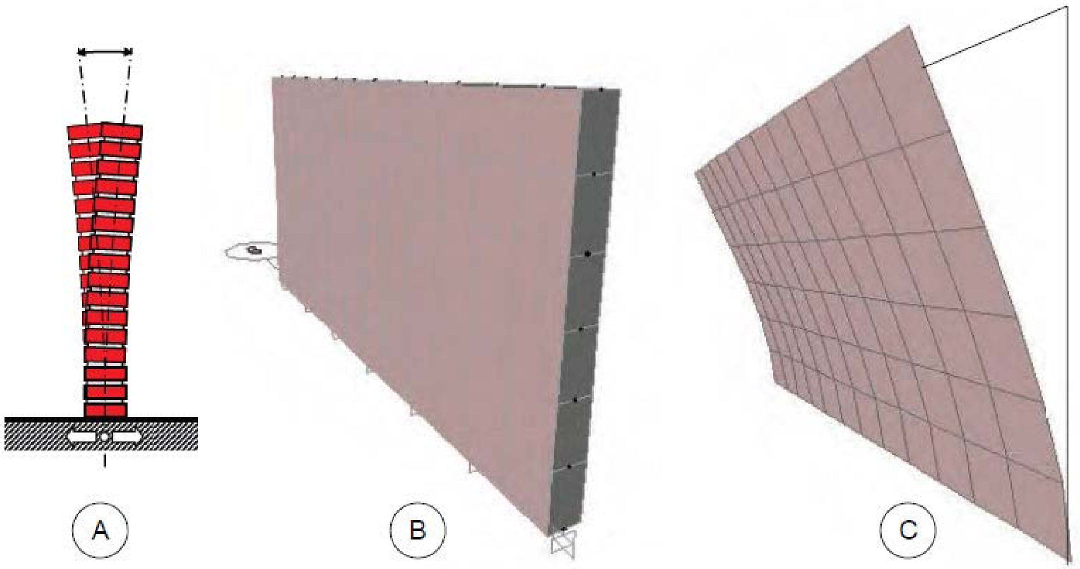
Arcos Trancho and Cristina Porcu [19] provide a detailed description of the masonry used in the area and its behaviour in the 1999 earthquake at Mula. The following sections discuss only a few particulars observed at Lorca.

4.1. Structural behaviour

Bearing walls are very stiff and resist in-plane actions, both horizontal and vertical, very well. Their stiffness and resistance to actions normal to their plane are much lower, however, and conditioned by the value of the respective vertical actions.

A wall affected by horizontal action normal to its plane (an earthquake, for instance, Figure 4-1) behaves like a simple cantilever. Normal tensile stress arises in any horizontal section, conditioning the bearing capacity of the wall, given the scant strength of such members.

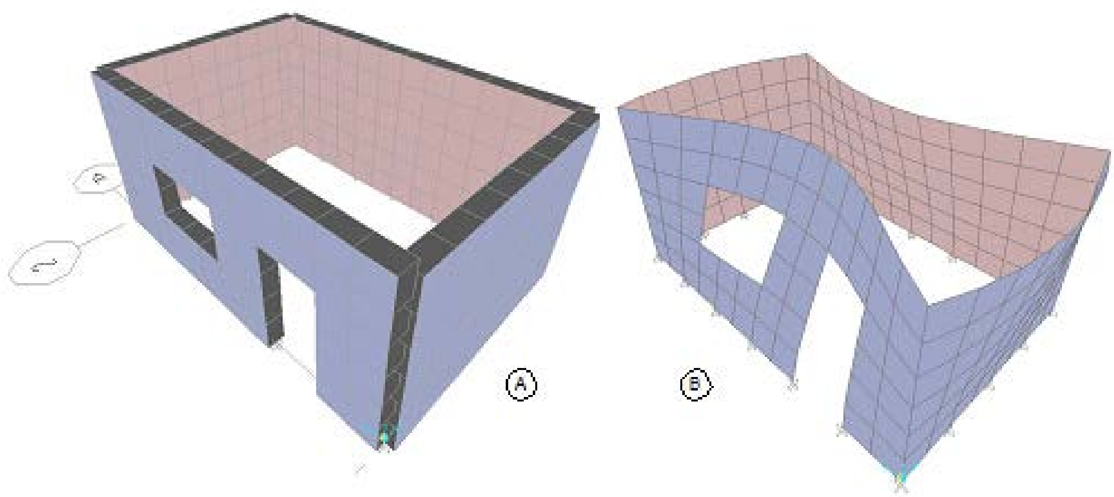
Traditionally, two stabilisation mechanisms have been used: the addition of a certain amount of weight at the top to enhance compression and compensate for bending-induced tension (pinnacles on Gothic cathedrals, for instance), and lateral bracing.



▲ **Figure 4-1**

Adding weight is not an acceptable solution in seismic areas because it increases the forces on the wall. As a rule, loads are practically proportional to the mass. The only other solution, then, is to brace the wall.

One obvious stabilisation procedure consists of building a second wall normal (or at least at an oblique angle) to the initial wall. This introduces a new strength mechanism in the system (vertical axis bending, like a plate resting on the bracing walls), which supplements the horizontal axis bending induced by the moment fixity at the base (Figure 4-2).



▲ **Figure 4-2**

4.2. Inter-wall connections

The effectiveness of wall bracing depends on the distance between walls and their bearing capacity. Traditionally, façades, stairwell walls and some partitions have been used as bracing walls; i.e., a number of wall sections contribute to structural stability.

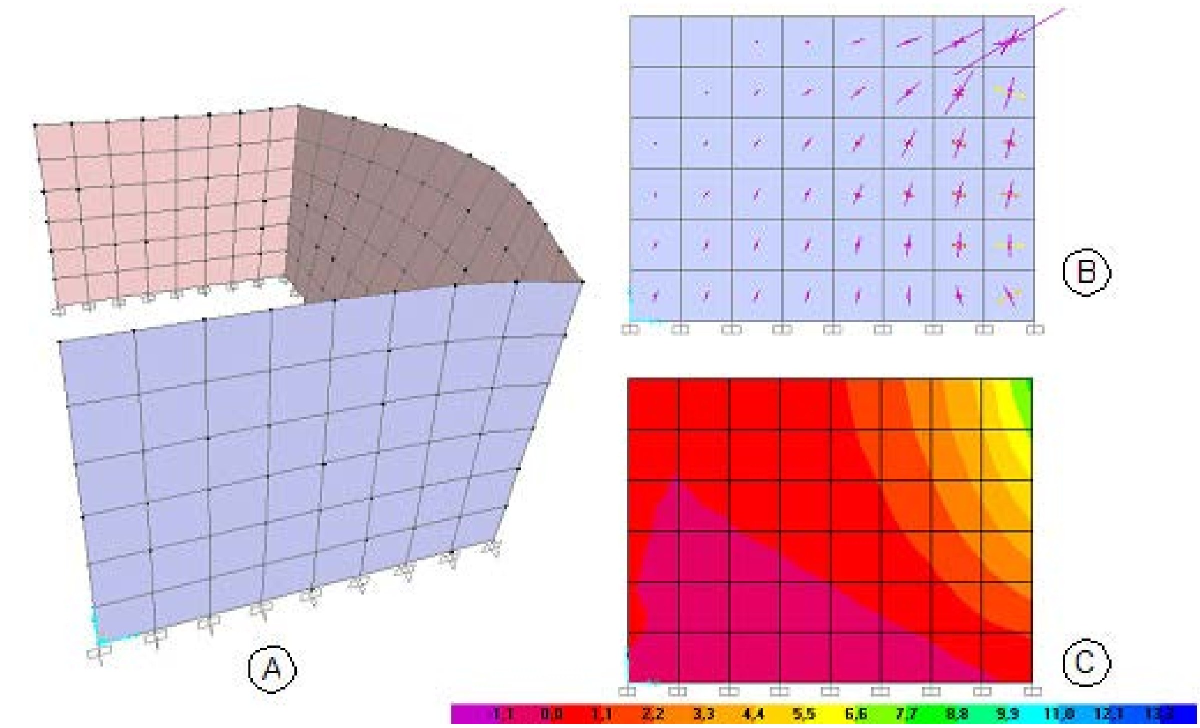
The problem, at least as observed at Lorca, is that many of these walls were unable to play this stabilising role due to prior connection failure.

Inter-wall abutments must bear not only direct axial loads, but also any bending stress generated.

A simplified simulation of this effect is shown in Figure 4-3. If horizontal loads are imposed on the long wall in Figure 4-3 A, the two bracing walls must absorb the resulting tensile stress (B in the figure) as well as the peak bending stress on the vertical axis (C) at the bond area.

That concentration of stresses at the corner connection justifies the care traditionally placed in its construction. In traditional bearing wall construction, this was achieved (even in walls subject to scant stress, such as the boundary marker shown in Photograph 4-1) by alternating ashlar headers and stretchers at the joint between walls made of poorer quality materials.

At Lorca, these traditional construction principles were often ignored, and as a result the walls pulled apart as in Photograph 4-2, which depicts a type of failure commonly observed at this site.



▲ Figure 4-3

▲ Photograph 4-1

▲ Photograph 4-2

While structural walls can be made of poor quality masonry, even coarse rubble, when greater stability is required restraint mechanisms must be provided (traditional lacing courses). These afford a mechanical solution for inter-wall abutments (alternating headers and stretchers) or to strengthen openings where the stress that inevitably concentrates at the corners can only be absorbed by careful bonding. The consequences of neglecting this requirement are illustrated in Photograph 4-3.

4.3. Floor-wall connections

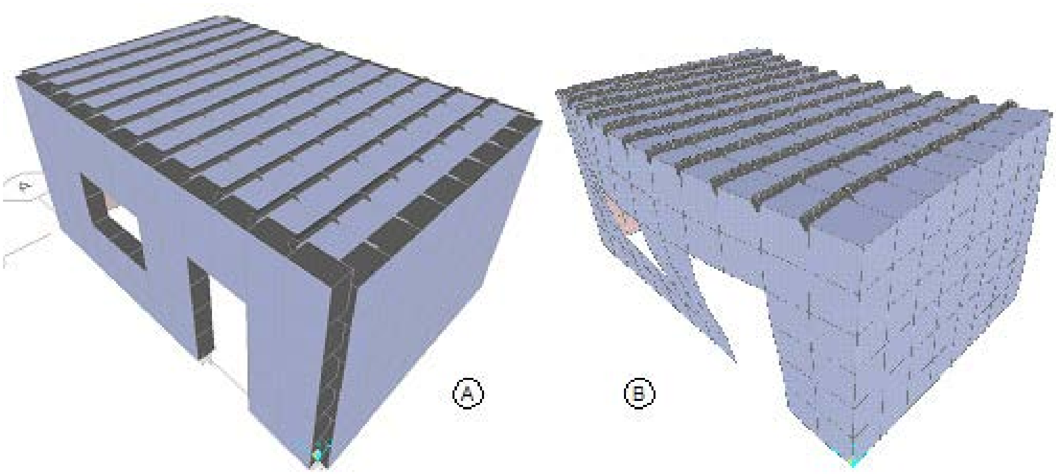
Another traditional system for bracing walls consists of tying them to floors.

For this system to be effective the floors must be monolithic, i.e., with sufficient in-plane stiffness and strength to act as an effective diaphragm. Only then can they perform like deep beams able to resist bending forces, receiving and transferring inertial forces to the bracing walls on which the beam rests (Figure 4-4).

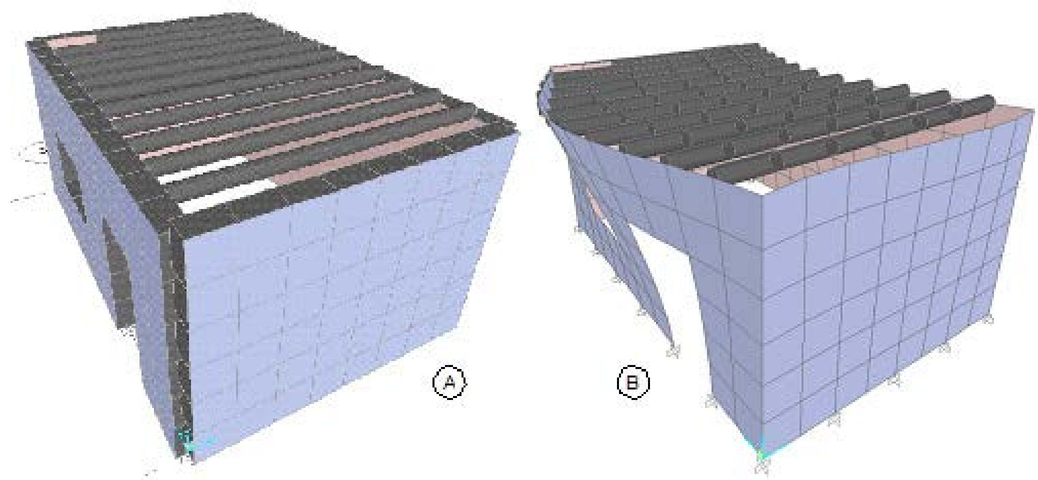
If the floor is not monolithic, as often observed at Lorca, no effective brace can be established.

In the best of cases, simulated with the numerical model depicted in Figure 4-5 A, the joists (all the ceilings observed in wall structures had them) merely joined the tops of similarly stiff sections of wall. In that case, the two sections move simultaneously and in the same direction, with neither actually restraining the other in any way whatsoever (Figure 4-5 B) In other words, the ceiling fails to fulfil its purpose as a brace, but at least the joists bear no loads.

▲ Photograph 4-3



▲ Figure 4-4



▲ Figure 4-5

▲ Photograph 4-4

Where the joists join walls with different stiffness values, the problem is more severe, because in that case the two walls move in different directions (the stiffer attempts to bear up the more flexible), inducing membrane stress on the floor. Since the wall-joist connection is unable to bear any tensile stress so generated, the joists slip at the abutment (Photograph 4-4).

In extreme cases, the joists collapse due to insufficient support length.

4.4. Structural arrangement

Like any other structure, masonry buildings must be constructed to a very precise structural arrangement. In their list of traditional structural systems, Arcos Trancho and Cistina Porcu [19] show that all these systems conform to the principles of simplicity, regularity and symmetry that should govern construction in seismic regions.

Those principles appear not to have been followed at Lorca, however. Functional needs have prevailed over structural criteria throughout buildings' life spans. Openings have been alternately cut out or walled up, storeys have been added, materials mixed. On occasion, the result was so chaotic that merely drawing up a sketch of the building (the survey team's first task) was a daunting endeavour.

Some of these problems are reflected in Photograph 4-5.

In certain cases, even identifying where the building began and ended was difficult, not only horizontally, due to the existence of shared party walls and the concomitant disappearance of the joint between buildings, but also and more surprisingly, vertically, as a result of enlargements in both directions.

The basic earthquake-resistant unit was not, in many cases, the building, but the city block (Photograph 4-6).

4.5. Inappropriate construction procedures

The preceding items contain a description and justification of the general response of masonry buildings in Lorca.

Our impression is that the flaws in bracing and tying mechanisms often prevented such a general response from materialising. Due to the premature failure of the bonds between different parts of buildings, each responded to the quake nearly independently. That translated into greater damage: collapse of elements for want of support, pounding damage and so on. In addition, structural members exhibited shortcomings which, in our opinion at least, lessened their bearing capacity. The problems most frequently encountered are described briefly in the following items.

4.5.1. Masonry

The poor workmanship in some of the essential details, described in the foregoing, was frequently aggravated by the use of unsuitable materials. In some cases, floor slabs rested on mere partitions.

Another common practice was to mix different materials in a given structural unit (Photograph 4-7), probably as a result of building enlargements.

The traditional dado or similar elements used to protect the masonry from moisture-induced deterioration was absent on many ground storeys.

▲ Photograph 4-5

▲ Photograph 4-6

▲ Photograph 4-7

▲ Photograph 4-8

▲ Photograph 4-9

4.5.2. Floor slabs

Timber creep generates long-term deflection in traditional floors. That effect was often countered by adding fillers whose weight induced further deflection, subsequently corrected by adding more filler and so on until the situation reached the extremes illustrated in Photograph 4-8.

4.6. Conclusions

Traditional construction, which includes bearing walls, is often regarded as quality building in some circles. Frequent literary reference to "*thick walls*" or the use of beams "*sturdy as trees*" to convey the notion of soundness stems from this commonplace.

Our perception in this regard is less straightforward. Traditional construction worthy of the name indisputably exists, but that is not what we saw at Lorca.

There we found "*thick walls*" literally shattered (Photograph 4-9) due to poor workmanship, deteriorated and inappropriate materials or the absence of protection against the elements. We also saw large, sturdy beams that failed for insufficient support length, moisture or insect-induced deterioration, or similar.

In the interim between the date of the earthquake and the writing of this report, many purportedly expert voices have extolled the good behaviour of traditional buildings, even contending that they responded better than modern buildings.

Except as regards one very specific matter, the collapse of façade elements, essentially parapets, we do not share that opinion. What determines buildings' earthquake resistant behaviour is less likely to be the type (traditional or contemporary) than the quality of construction.

Conventional buildings wall behaviour

5

5. Conventional buildings wall behaviour

One of the clearest lessons learnt from the Lorca earthquake is the role of non-structural elements, in particular masonry elements, in conventional (meaning reinforced concrete or steel portal frame) buildings.

Masonry walls, one such element, exhibit singular behaviour. While they are highly vulnerable when exposed to bending stress induced by actions normal to their plane, they are stiff and highly resistant to in-plane actions.

When exposed to bending loads, masonry acts as a passive element affected by building response but not strong enough to modify it in any material way. Its importance lies in the fact that its failure may cause the wall to collapse. Under in-plane loads, in contrast, these walls make a substantial contribution to the overall stiffness and strength of the building as active elements that modify its response.

Inasmuch as both factors proved to be instrumental to the response of conventional buildings in Lorca, they are discussed in some depth in two separate chapters hereunder.

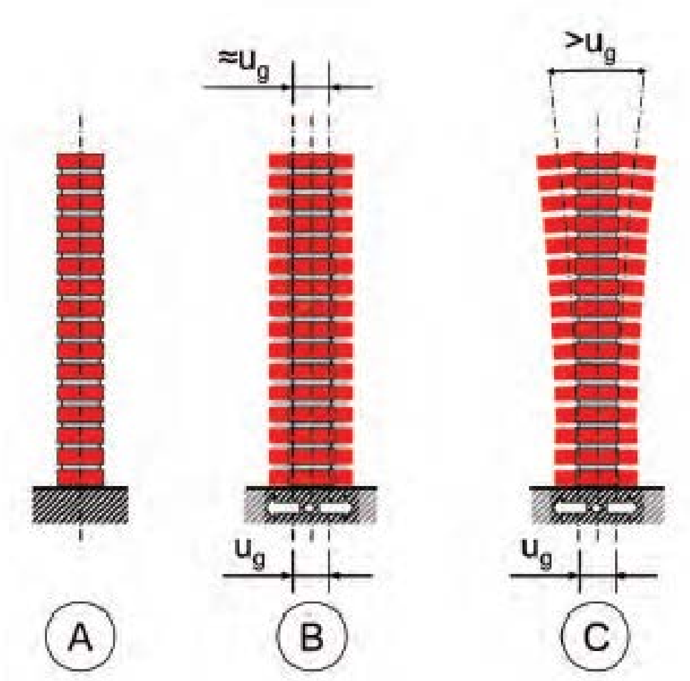
The present chapter analyses the behaviour of these walls as plates subjected to bending loads, a factor of cardinal importance because it constitutes the greatest hazard for personal safety. More specifically, parapet collapse proved to be the greatest hazard during the Lorca quake.

While some of the masonry elements collapsed because the construction was clearly unsuitable for the area, this analysis will focus on the behaviour of the elements built to standard and their capacity to resist the effects of an earthquake as severe as the Lorca tremor, an issue deemed to be of greater interest.

5.1. Seismic response of walls

This study of the response of masonry elements to seismic stress acting in a direction normal to their plane begins with the most elementary situation, illustrated in Figure 5-1 A. Here the element is a solid brick wall one half foot thick, 1.2 m high (the standard height for roof parapets), with continuous rendering and resting fully on the ground. Initially, the wall will be considered as an entity separate from the building.

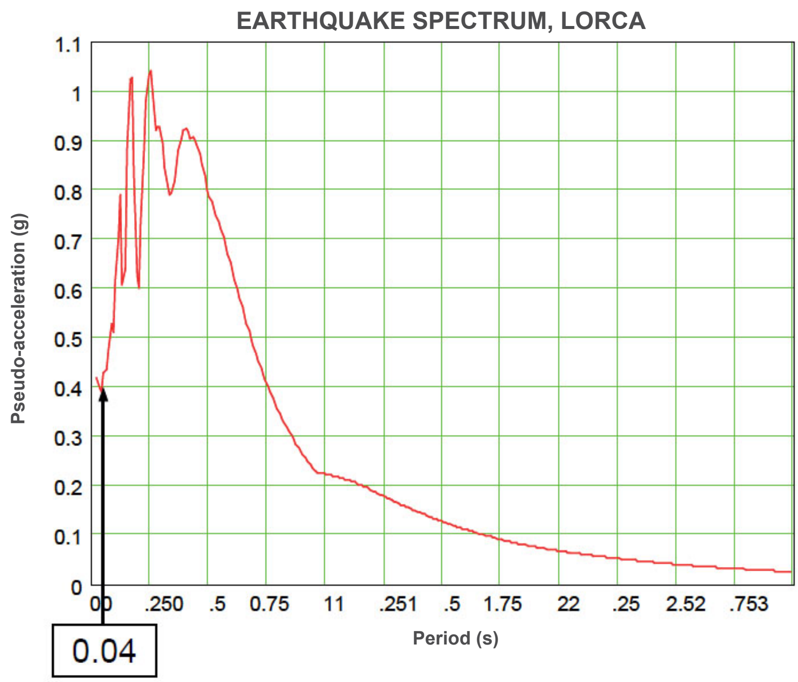
The natural period of masonry walls under bending stress is normally very small. For walls with the above geometry (Figure 5-1 A), the value is around 0.04 seconds. This means that until they fail, they move essentially like a stiff body anchored to the ground: motion in all their points is identical to the motion in the soil (Figure 5-1 B). No amplification (which would entail deformation of the element as shown in Figure 5-1 C) whatsoever occurs and the value of the horizontal loads on the parapet or masonry element is the product of its mass times ground acceleration.



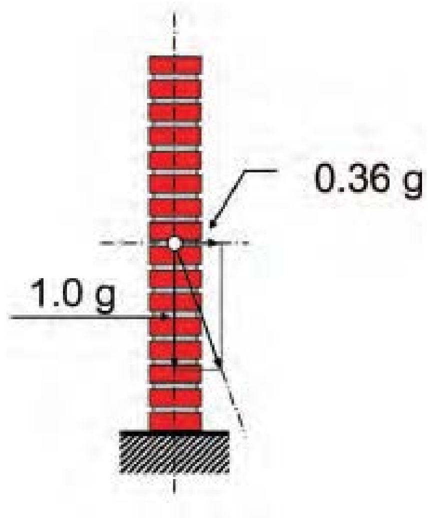
▲ Figure 5-1

This can be clearly seen from the spectrum for an earthquake with such a small period (Figure 5-2), whose spectral value would logically concur with the ground acceleration.

For a masonry element resting on the ground the major effect of the Lorca earthquake would therefore be a lateral force equivalent to 36 % of its weight (Figure 5-3). When the height of such a wall is over a certain value, the moment at the base obviously exceeds its bending capacity, inducing collapse (Photograph 5-1).



▲ Figure 5-2



▲ Figure 5-3

▲ Photograph 5-1

The existing legislation actually precludes considering bending capacity in seismic design. The “Documento básico de seguridad estructural” (basic document on structural safety), a chapter of Spain’s building code (Código Técnico de Edificación) [15], provides as follows:

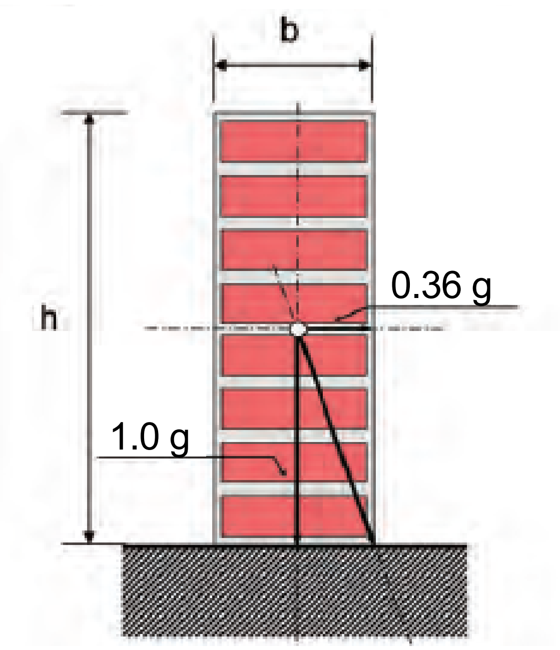
> ...*“Bed joint bending strength may only be used with load combinations that include variable actions normal to the surface of the masonry (such as wind). That strength may not be considered when bending failure in the masonry element induces the collapse or lessens the stability of the building or any of its parts, or in the event of seismic action.”*

Consequently, for these intents and purposes, the element must be regarded to be dry-joined, with no bond whatsoever. Only the walls slender enough for the resultant of the weight and the lateral acceleration to be contained within the bearing section would be stable under such conditions (Figure 5-4).

Stability condition

$$\frac{h}{b} \leq \frac{1.0g}{0.36g}$$

$h \leq 36\,\text{cm} \quad (b \approx 13\,\text{cm})$



▲ Figure 5-4

▲ Photograph 5-2

At a thickness of one-half foot including the cover (i.e., for a width of around 13 cm), only walls under 36 cm high would have been nominally safe during the Lorca earthquake. Even assuming the acceleration envisaged in the legislation, 0.12 g, only walls no higher than 1 metre would have been safe.

That building code limitation restricts the use of masonry walls in seismic zones considerably, for their design strength would be reduced to the strength in the perpend direction. They would work like one-way members resting against pilasters, a scantly effective and seldom applicable mechanism because of the greater distance in that direction (pilasters are rarely spaced at the small distances that would be required).

These factors are addressed more thoroughly in other legislations. Photograph 5-2, taken on the Atlantic coast of Mexico, a moderately seismic zone, depicts the construction of a masonry parapet less than one metre high. Note the use (systematic in the region) of reinforced concrete columns spaced at short intervals.

5.2. Walls

In earthquakes, building walls are exposed to much greater forces than walls resting on the ground.

A regularly shaped, five-storey building (representative of many buildings in the city, Photograph 5-3), whose structure consists of flat reinforced concrete portal frames spaced at 5 m, forming four bays for a total width of 16 m, was modelled to determine the acceleration affecting the infills on each storey. The storeys in the shear portal frame model used were all 2.75 m high, except the ground storey, which measured 3.5 m. The masses were estimated in keeping with standard construction practice and the building codes normally applied.

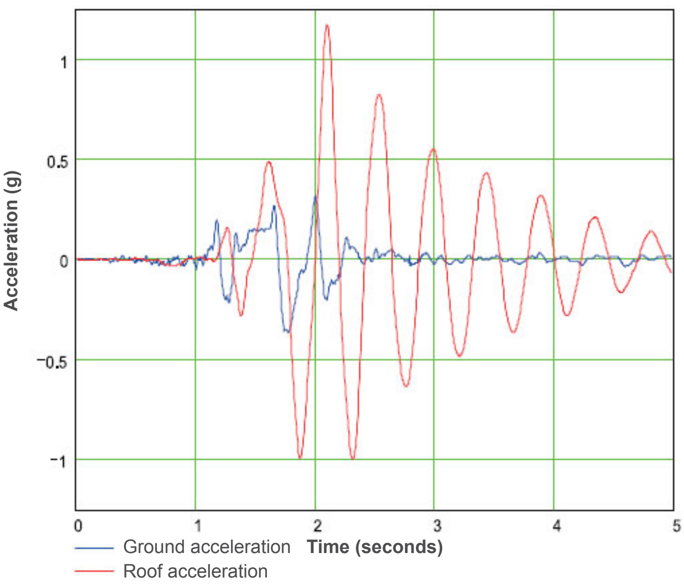
Model stiffness was adjusted so that the period for the first vibration mode would concur with the period found calculated from the expression given in the legislation:

$$T_F=0.09\cdot n$$

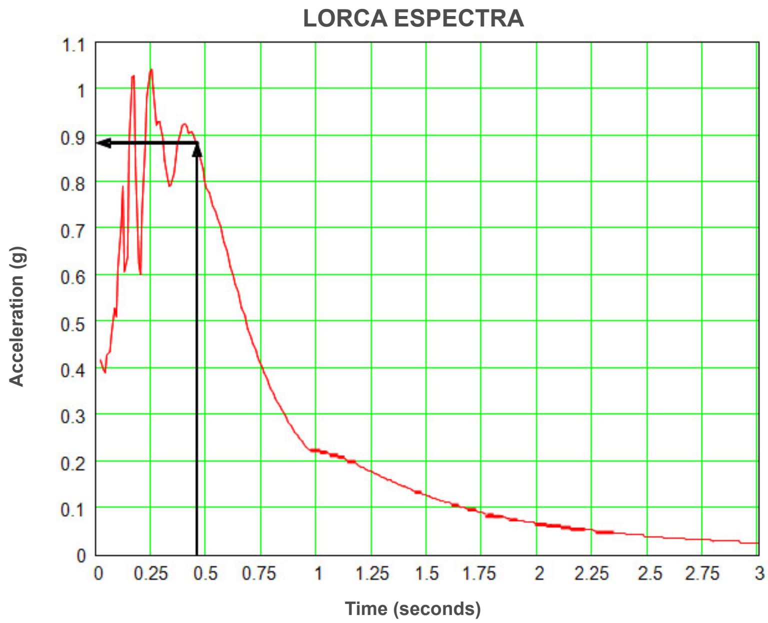
In this equation, applied to buildings with reinforced concrete portal frames unstiffened by shear walls, n is the number of above-grade storeys. For the present example, the period would be 0.45 seconds.

The movement induced by the N-S component of the Lorca quake was applied to the base of the model building and a step-by-step integration algorithm was used to obtain the accelerogram for the top storey, shown in Figure 5-5.

▲ **Photograph 5-3**



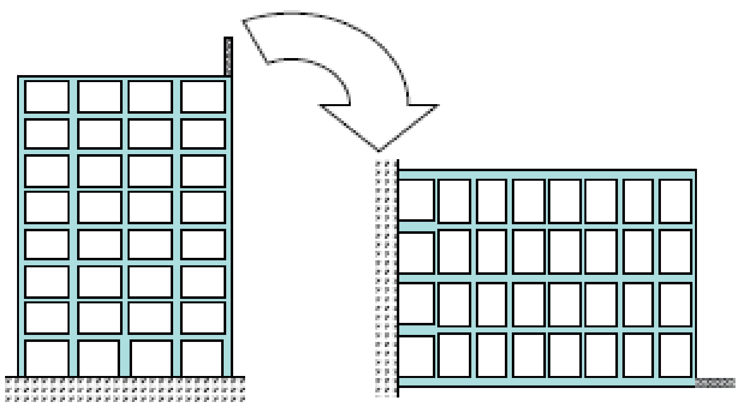
▲ **Figure 5-5**



▲ Figure 5-6

The roof acceleration values were clearly greater than g. That should come as no surprise: entering only the first mode into the model and therefore 0.45 seconds as the initial value on the spectrum yields pseudo-acceleration values of 0.9 g (Figure 5-6) which, multiplied by a distribution factor whose value is over 1.2, leads to a similar result.

Horizontal acceleration on the order of 1 g is equivalent to rotating the building by 90º. The parapet, which would thus cantilever horizontally, would have to bear its own weight (Figure 5-7).



▲ Figure 5-7

Consequently, the moment on the floor slab bearing at the base of a half-foot brick masonry parapet (rendered on both sides) 1.2 m high (further to the Spanish building code the clear height must be 1.1 m) would be:

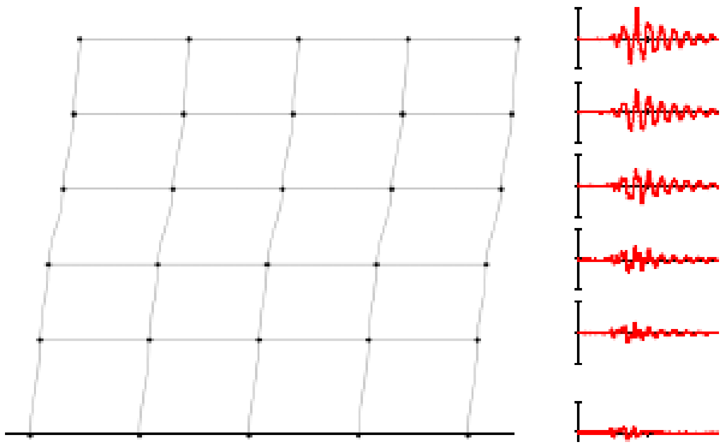
$$M = \frac{q \cdot h^2}{2} = \frac{0.13\text{m} \cdot 1\text{m} \cdot 15\text{kN/m}^3 \cdot (1.2\text{m})^2}{2} = 1.4\text{kN} \cdot \text{m}$$

Which is equivalent to stress on the masonry on the order of:

$$\sigma = \frac{M}{W} = \frac{1.4\text{kN} \cdot \text{m}}{\frac{1\text{m} \cdot (0.13\text{m})^2}{6}} = 0.5\text{N/mm}^2$$

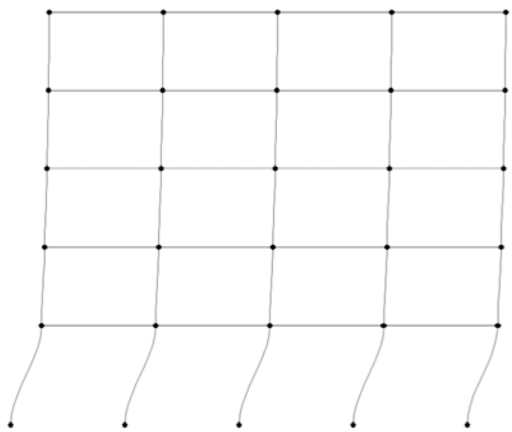
That value is much higher than the characteristic strength (0.1 N/mm^2) of masonry laid down in the same code and the Eurocode on masonry [2].

Theoretically, the acceleration value and therefore the inertial force perpendicular to the masonry walls is lower on the lower storeys. Building deformation during the earthquake (Figure 5-8) widened the difference between the acceleration on the higher and ground storeys. As a result, while the forces on the higher walls would be comparable to the forces acting on the parapets, on the ground storey they would be much smaller and similar to the forces on a masonry wall resting directly on the ground.



▲ Figure 5-8

In practice, however, in many buildings deformation varied widely from the pattern shown in Figure 5-8 due to what is known as "*soft-storey deformation*", characterised by the concentration of building displacements on the ground storey (Figure 5-9).



▲ Figure 5-9

As discussed in the following chapter, the masonry itself and more specifically its general failure on ground storeys, induces this effect, whose most immediate consequence for the present intents and purposes is to equalise displacement on all storeys and therefore the actions on all the masonry infills.

If the enclosures on all the storeys were exposed to similar actions, the reason that damage concentrated at the parapets (and some upper storey walls) must lie in the greater strength of the masonry infills in the lower storeys, as discussed in a later item.

The existence of more flexible members than parapets, such as bulkheads or roof enclosures consisting at times of very slender walls (Photograph 5-4), complicates matters.

In such cases the actions are much greater than the mere product of roof mass times its acceleration. That value is amplified depending on the period of the element (and the building itself, logically). These elements are often analysed with storey spectra such as in Figure 5-10, which are formulated in the same way as for ground spectra, except that the initial value is the acceleration of the storey at issue (in this case, the one depicted in Figure 5-5). Note the obvious amplification in the range of periods around the building period (0.45 s in the example).

▲ Photograph 5-4

▲ Photograph 5-5

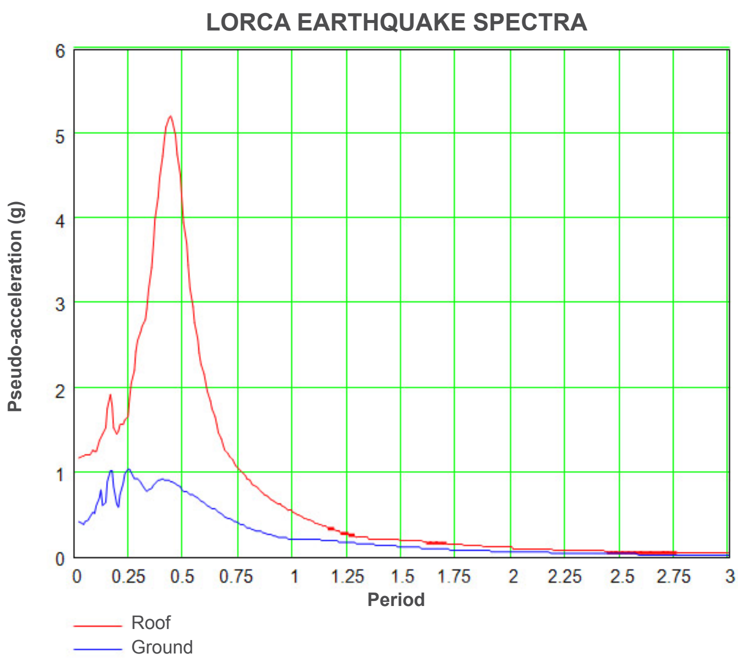
All the foregoing on masonry walls can naturally be applied to the rest of the roof elements, such as chimneys (Photograph 5-5), bulkheads and antennas.

The Eurocode on earthquake-resistant design proposes a simplified expression to avoid having to formulate a spectrum for each storey:

$$S_a = \alpha \cdot S \cdot \left[\frac{3 \cdot \left(1 + \frac{z}{H}\right)}{1 + \left(1 - \frac{T_a}{T_1}\right)^2} - 0.5 \right]$$

Where:

- S_a: the pseudo-acceleration times which the mass of the non-structural element would have to be multiplied to obtain the equivalent force directly
- $\alpha \cdot S$: site design acceleration
- z: height of the storey housing the non-structural element
- H: total building height
- T_a: natural period of the non-structural element
- T_1: building natural period

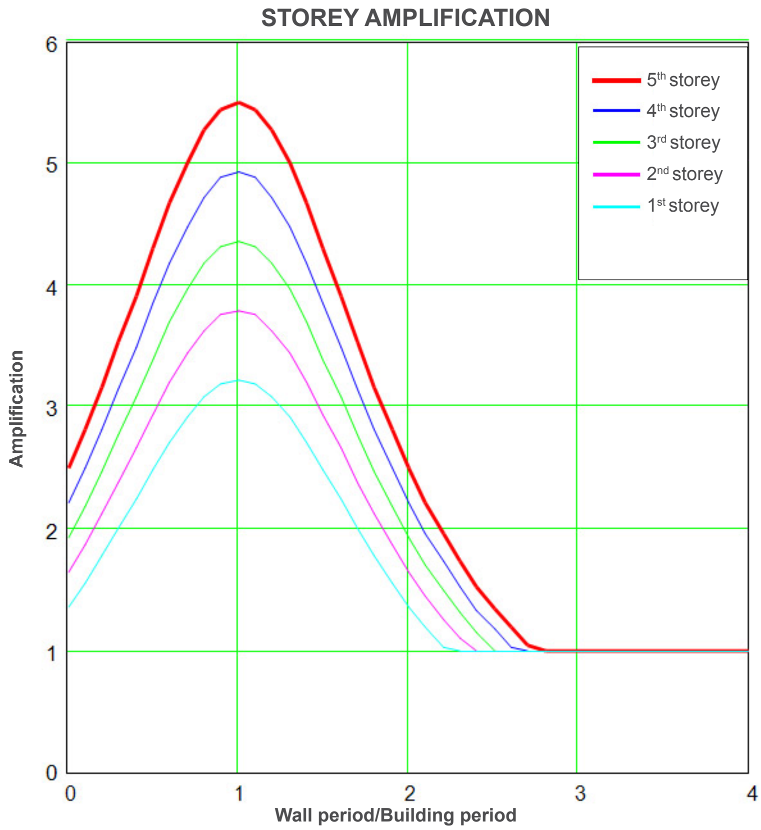


▲ Figure 5-10

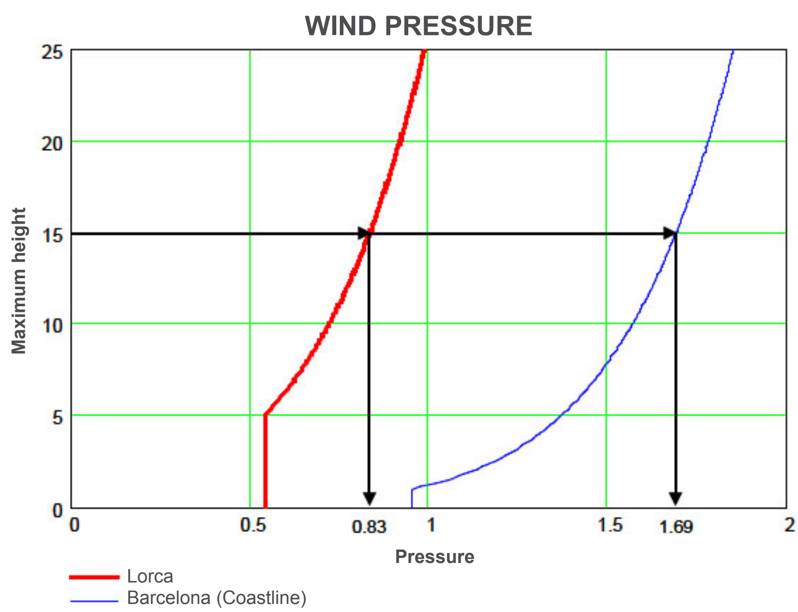
Plotting the storey amplification, $S_a/\alpha \cdot S$, for each value of the ratio between the period of the non-structural element and the building period yields the graph for all five storeys shown in Figure 5-11. The amplification in the highest storey (thick red curve) for stiff elements such as parapets is 250 %, a value similar to the result calculated above.

More surprising yet than these findings (which as noted are equivalent to building a horizontally cantilevered parapet) is the fact that this is not the highest value deriving from the live loads envisaged in the legislation.

Further to the existing code on actions ("Documento Básico SE-A", Código Técnico de Edificación [14]), the variation in wind-induced pressure with height for a building in an "A" climate zone, where the minimum basic wind velocity value (26 m/s) is applicable, and type "IV" (urban) surroundings follows the pattern shown in Figure 5-12. Note that in this case the values are in keeping with the Lorca data but much lower than would be applicable to many other buildings in Spain, located in more demanding climates or less favourable surrounds. As the figure shows, in a building located on the coast in Barcelona, the pressure would be double.



▲ Figure 5-11



▲ Figure 5-12

For a building of the height defined in the example, 15 m, the pressure amounts to 0.83 kN/m². Multiplying this value by the pressure factor, 1.4 (as per Spanish and European standard UNE-EN 1991 [1]) and by the load factor for variable actions, 1.5, yields a total pressure of 1.75 kN/m², which would generate a moment at the base equal to:

$$M = \frac{q \cdot h^2}{2} =$$

$$\frac{1.75 \mathrm{kN/m^2} \cdot 1\mathrm{m} \cdot (1.2\mathrm{m})^2}{2}$$

$$= 1.3 \mathrm{kN \cdot m}$$

The above value is not much smaller than induced by the earthquake and more than sufficient to prompt masonry failure.

These findings are far from original. The monograph on housing in the section of the Spanish building code on masonry walls [9] reaches similar conclusions when calculating the maximum safe height of a parapet exposed to wind: the values found are much smaller than the minimum required by the code itself in its document on safe usage (1.10 m). These ideas are discussed more fully by Dávila *et al.* [43] and [44] and addressed as well by Puertas and Blanco Perrín in their AEC (Spanish quality association) monograph [42].

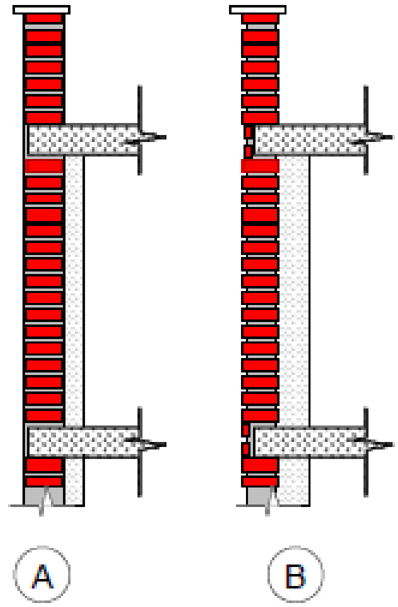
The experimental results reported by Gutiérrez *et al*. [47] are similarly conclusive.

That notwithstanding, the most surprising result was obtained when considering service loads, whose minimum value (roofs accessed for maintenance only) at the top, a uniform force of 0.8 kN/m, generates a moment at the base of the parapet similar to the moment induced by an earthquake:

$$M = \gamma_Q \cdot (P \cdot h) =$$

$$1.5 \cdot (0.8\mathrm{kN} \cdot 1.2\mathrm{m})$$

$$= 1.4 \mathrm{kN \cdot m}$$



▲ Figure 5-13

Even the former Ministry of Housing's initial code, MV-101-1962, in effect from the date it was decreed in the early nineteen sixties until it was superseded by the present building code, defined a linear load of 50 kgf/m, which would give rise to internal forces only slightly lower than the forces generated by seismic action.

In short, seismic action is not the most demanding force acting on masonry walls positioned on top of buildings.

The actual situation is even less favourable, however. While the example considered assumes that the entire section of the infill rests on the frame (Figure 5-13 A), building enclosures are often only partially supported (Figure 5-13 B). Under former building codes, only 2/3 of the section had to be supported.

One noteworthy fact observed in Lorca was that, while the internal forces on parapets and façade walls were similar and much greater than their respective bearing capacities, the parapets were much more severely damaged. That, in INTEMAC's opinion, may be due to two beneficial effects more frequently present in infills:

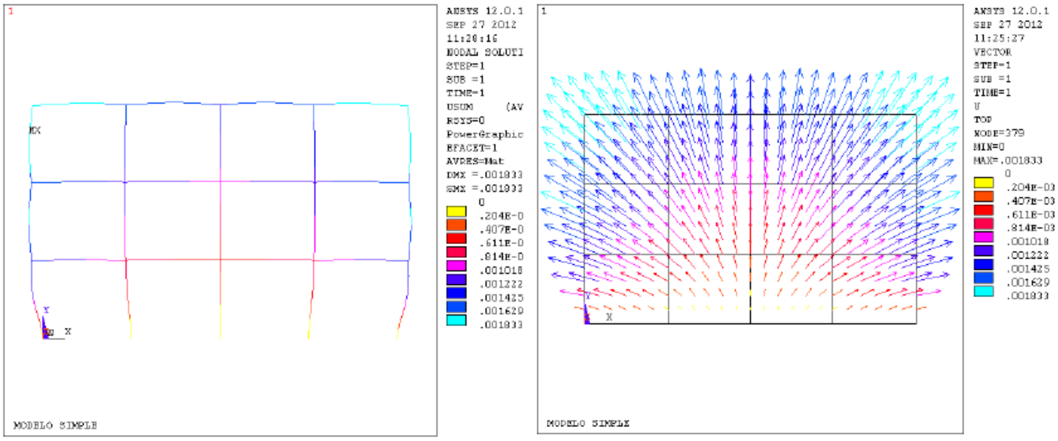
- Vertical loads advene on the horizontal loads, inducing compression forces on the wall that lower the stress on its tensioned side and thereby raise its bearing strength; in parapets, the compression is a result of their self weight only and consequently very small:

$$\sigma = \rho \cdot h = 15\mathrm{kN}/\mathrm{m}^3 \cdot 1.2\mathrm{m}$$
$$= 18\mathrm{kN}/\mathrm{m}^2 \approx 0.018\mathrm{N}/\mathrm{mm}^2$$

Such values are negligible compared to the loads induced by the earthquake (which is why they were disregarded in the preceding calculations). In infills, by contrast, the vertical loads are much greater because in addition to their own weight they bear part of the heavier loads acting on the floor slabs. This effect is greatest in standard flat portal frames positioned parallel to the façade and supporting one-way slabs. While the values are not overly high, their contribution would always be more significant in upper storey infills than in parapets. In the lower storeys, on the contrary, the accumulation of gravity loads transferred downward across the enclosures could explain the buckling observed in some infills exposed to earthquake-induced lateral action (Photograph 5-6).

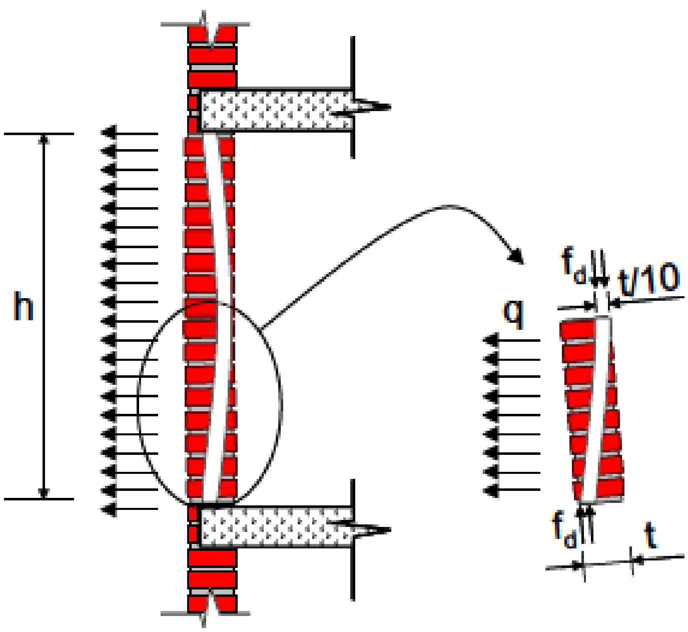
▲ **Photograph 5-6**

Vertical loads are also the result of the shrinkage in the concrete frame around the panels and the damp-induced expansion of the infills themselves, which would together generate considerable compressive stress. The simplest numerical simulations of these effects yield surprising results. One, shown in Figure 5-14, was detected by Industrial Engineer Lucía Sánchez Marta and de-



▲ Figure 5-14

scribed in her end-of-course dissertation, prepared at INTEMAC. Her study found that the axial loads on the columns induced by even minimal values for these two events (masonry expansion and concrete shrinkage) would exceed the gravity load, at least in the upper storeys, thereby subjecting some of the respective columns to tensile stress.



▲ Figure 5-15

- According to the provisions of the aforementioned building code document, arching may occur when the enclosure is confined between floor slabs stiff enough to absorb the reactions along the edges. Assuming the arch thickness to be one-tenth of the useful width of the wall, the masonry to have average compressive strength (as per building code tables) and deformation to be negligible, the resulting expression would be (Figure 5-15):

$$q_d = 0.72 \cdot f_d \cdot \left(\frac{t}{h}\right)^2 =$$

$$0.72 \cdot 5000\text{kN}/\text{m}^2 \cdot \left(\frac{0.12\text{m}}{2.75\text{m}}\right)^2$$

$$\approx 6.9\text{kN}/\text{m}^2$$

That value is much higher than the forces (which are equivalent to the weight of the section, i.e., under 2 kN/m^2). The actual conditions are less favourable, of course. Neither the infill rests fully on the frame (while in the above example the effective width was 2/3 of the total, in actual standard construction practice it is less than half, under which circumstances the effect of deformations cannot be disregarded as negligible) nor are floor slabs stiff enough to absorb the reactions in most cases.

- Consequently, the model would only be applicable in the areas of the infills adjacent to the columns (which would act like arch ties). In practice, the greatest problem is that it is impossible to ensure that even 2/3 of the infill actually rests on the slabs (Photograph 5-7).

5.3. Inappropriate construction procedures

The foregoing discussion attempted to show that even under the best of circumstances, when masonry walls are properly built (using the word “properly” with all due reservations), they are unable to withstand the actions envisaged in the legislation.

In some real-life situations, conditions are even more precarious due to construction practice which, while less than appropriate, has become so widespread that it might be thought to be standard.

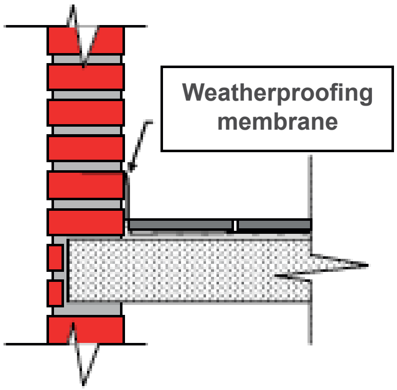
One clear example of such practice is the careless workmanship at the abutment between roof pavements and parapet masonry. Sometimes the joint (not understood here to mean the standard strip of polystyrene) required to accommodate pavement expansion is lacking. The resulting thrust ultimately breaks the mortar bond at the base of the parapet, pushing the element outward to form the characteristic uneven surfaces seen on many façades (Photograph 5-8).

At other times the lack of joints to absorb masonry deformation induces vertical cracking that interrupts the continuity of the wall, countering the beneficial effect of the pilasters. Note the cracking on the wall to the right of the corner in Photograph 5-9 (the adjacent wall also foreseeably cracked prior to collapse).

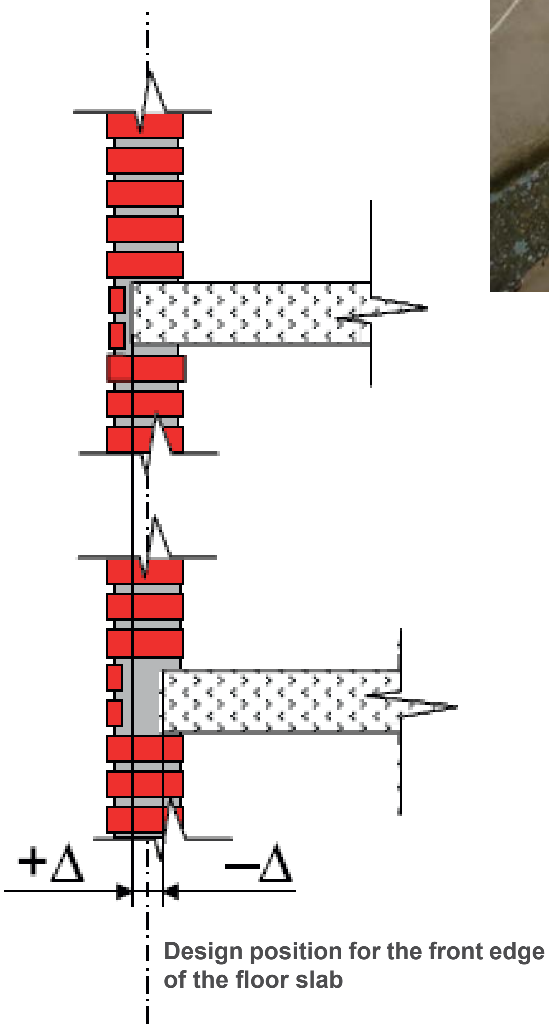
▲ Photograph 5-7

▲ Photograph 5-8

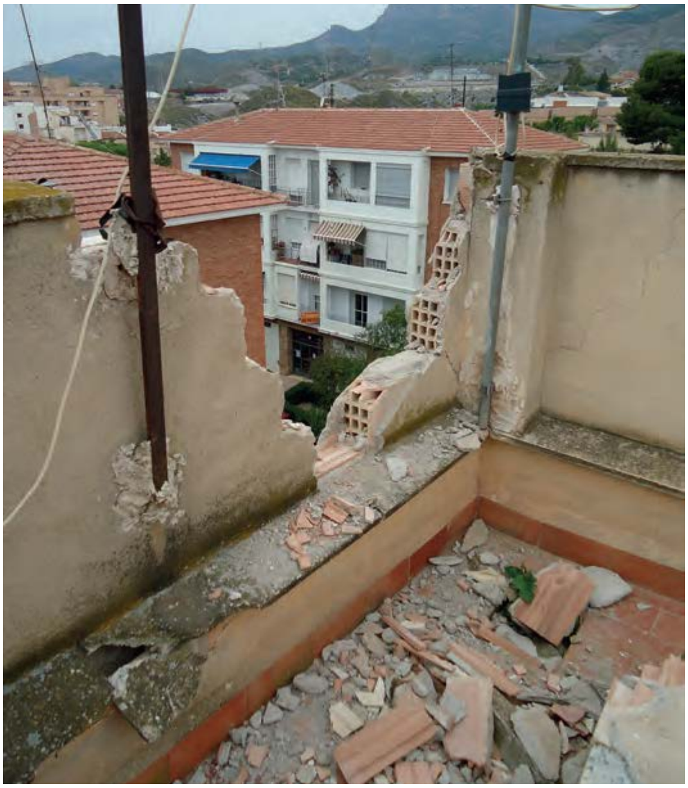
Another potential source of trouble is the weatherproofing membrane, for on occasion design detailing calls for securing it along the edge of a bed joint, interrupting section continuity, as shown in Figure 5-16.



▲ Figure 5-16



▲ Figure 5-17



▲ Photograph 5-9

The figure also illustrates the standard procedure for resting the wall on the floor slab. If, as is usually the case, continuity is sought between the plane of the façade and the cladding over the front of the slab, the infills can rest only partially on the slab. The problem posed by this situation is the lack of any practical way to ensure that the required minimum proportion of the wall is actually supported, within normal tolerance limits. Since façades must be very strictly vertical (the human eye can detect the lack of plumb with surprising precision) and aligned, logically, with the slab that protrudes the most, the slab that protrudes the least cannot provide sufficient support. Since conventional walls are 11.5 cm thick and approximately 3 cm must protrude beyond the slab to clad it, 8.5 cm at most would rest on the frame.

▲ **Photograph 5-10**

That means that on the slab with the greatest recess from its design position (Figure 5-17, the bearing under the wall would be (8.5 -2·Δ), where "Δ" is the allowable deviation according to the applicable tolerance table. Given that such deviation varies in the existing structural concrete code from 2.4 to 5 cm for conventional buildings, the obvious inference is that even in correctly constructed buildings, some of the infills may be wholly unsupported.

A clear example is shown in Photograph 5-10, which depicts intermediate storeys after their façade collapsed. The enlarged detail reveals that the façade on the second highest rested on no bearing whatsoever.

Nonetheless, the greatest hazard exists when the masonry façade is positioned off a slab resting on a steel shape, in turn hung from mere tie rods (Photograph 5-11: note the supporting angle clip and above it, the tie bands attached after the earthquake to provisionally secure the façade). This arrangement is scantly reliable even for gravity loads.

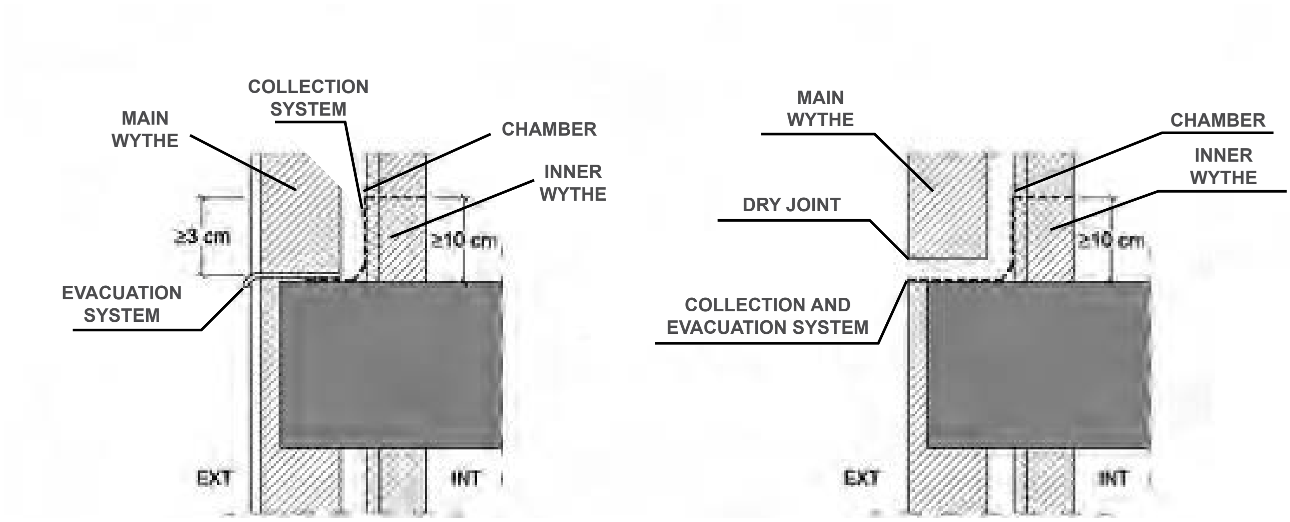
▲ Photograph 5-11

Another example of these problems is illustrated in Photograph 5-12: the outer wythe was laid in front of the floor slab, leaving only the inner wythe to rest correctly on the slab. The weatherproofing installed across the entire thickness of the latter prevented any satisfactory bearing, however.

Standard construction practice and even the applicable legislation exhibit such diversity that specific problems are difficult to pinpoint. The risk of section loss and with it strength due to the insertion of weatherproofing has been mentioned. Significantly, however, this is at least partially the result of statutory regulations presently in force that not only call for weatherproofing, but require laying it across the entire thickness of the wythe (see the technical building code basic document on healthful habitats).

The need for some of these provisions is obvious from the standpoint from which they are instituted. Details such as shown in Figure 5-18 (extracted directly from the aforementioned basic document) are imperative to eliminate damp from the air chamber, certainly. Nonetheless, the need for measures to compensate for the loss of support under the façade walls involved is equally obvious.

▲ Photograph 5-12



▲ Figure 5-18

5.4. Façade collapse due to adjacent building pounding

Pounding between adjacent buildings (Photograph 5-13) during earthquakes generates in-plane forces in the masonry walls, inducing failure different from the type discussed in the preceding items. The problem is nonetheless addressed in this chapter because its consequences are similarly severe. Moreover, in some cases at least, drift in buildings perpendicular to the common façade plane also generates forces normal to the masonry walls for, while in theory the separation joint between them should prevent that happening, in practice many such joints are mortar-filled, maintaining continuity even in the face of this type of actions.

Standard practice at Lorca appears to have been to abut adjacent façades and even press mortar into any gaps between them. It should come as no surprise, then, that on occasion the quake created new joints positioned off the theoretical separation between buildings (Photograph 5-14).

▲ Photograph 5-13

▲ Photograph 5-14

▲ Photograph 5-15

According to Spanish code NCSE-02, these separation joints should be wide enough to allow the building displacement within its plot boundaries in seismic events. For a five-story building such as the one taken as an example here, further to the equation set out in item 4.2.5 of the code, maximum drift is around 2 cm (a value amply exceeded if the Lorca accelerogram rather than the spectrum defined in the code is applied to the sample structure).

In other words, in the worst case scenario, i.e., adjacent and standard but differently structured buildings, these separation joints should be able to accommodate drift on the order of 4 cm.

For more favourable circumstances, i.e., buildings having similar characteristics that would move in phase with the motion generated at the base, the Eurocode on earthquake-resistant design makes provision for a reduction in joint width of up to the square root of the sum of the squares of each drift value, which in this case would yield widths of around 3 cm.

Adding the earthquake-induced drift to the displacement caused by environmental action (temperature, humidity) and the rheological effects of materials (masonry swelling), however, yields large values that constitute a hindrance to both joint construction and the sealing system and its subsequent maintenance. Sealing is normally the operation that poses the most serious problems.

While joints between structural members (in independent buildings with no continuous M&E services, finishes or similar) should entail no significant difficulties, sealing the gap between the enclosures on the two buildings is especially complex.

A separation joint between façades that allows for fairly wide displacement (around 4 or 5 cm, by way of reference) calls for the deployment of specific solutions.

Mere caulking with a standard silicone bead at the back of the joint is actually not at all suitable, because the displacement accommodated by such solutions is very small. In fact, such displacement (which should not be confounded with the apparent width of the joint) is measured in millimetres, whereas the displacement that can be expected between building façades, even without earthquakes, is measured in centimetres.

Since even the best silicones cannot guarantee stable deformation of over 20-25 %, a conventional 2-cm joint such as in Photograph 5-15 would accommodate movements of no more than 5 mm. Assuming that the distance between façade joints does not exceed the 12 m laid down in the building code and that total humidity-induced masonry expansion is no greater than 0.5 mm/m (some authors propose much higher values), in a matter of only a few years wall swelling would deplete the joint, which would be able to accommodate no additional deformation whatsoever.

Even a quick look at Spanish cities unfortunately reveals that with very few exceptions, separation joints have not been suitably constructed, resulting in a fair amount of readily visible damage: open joints and the concomitant leaking, closed joints that fracture adjacent finishes and so on.

▲ Photograph 5-16

It would appear to be more cost-efficient to repair periodic joint damage than to make suitable allowance for these elements from the outset (note the “repair” of the joint depicted in Photograph 5-16 taken, like Photograph 5-15, in Madrid and thus unrelated to the earthquake).

This obviously unsolved problem generates considerable uncertainty. While in buildings located in low seismic risk areas its effects are essentially related to building functionality, in seismic areas it is associated with the no less important issue of enclosure stability.

▲ Photograph 5-17

5.5. Conclusions

Anchorage for masonry walls in general and especially for façade parapets should be a basic priority in seismic-resistant building design. Present practice, however, deviates from that ideal to the point of systematically failing to comply with the legislation. The need to design parapets to resist horizontal action has been explicitly addressed in the legislation since its earliest editions, in the nineteen sixties. Standard PGS1, published in 1968, contained a specific section on the subject and even included calculation methods.

The problem sometimes stems from the use of wholly inappropriate construction systems. Many of the masonry walls at Lorca collapsed simply because they were not secured, i.e., as a result of faulty construction.

The inference might be that the solution is simple and merely consists of building to standard. That, however, while absolutely necessary, is not sufficient. Indeed, the preceding discussion shows that the provisions on masonry wall construction, even where "*to standard*" (with the infills suitably supported), fail to guarantee resistance to plane-normal actions, not only in earthquakes, but even against wind or other live loads. An intermediate sub-structure is always required to receive and transfer the loads to the main structure. This idea is implicit in standards such as the NCSE which, for parapets, requires the construction of a frame around the entire wall, consisting of a tie beam at the top and short columns to which it transfers the load.

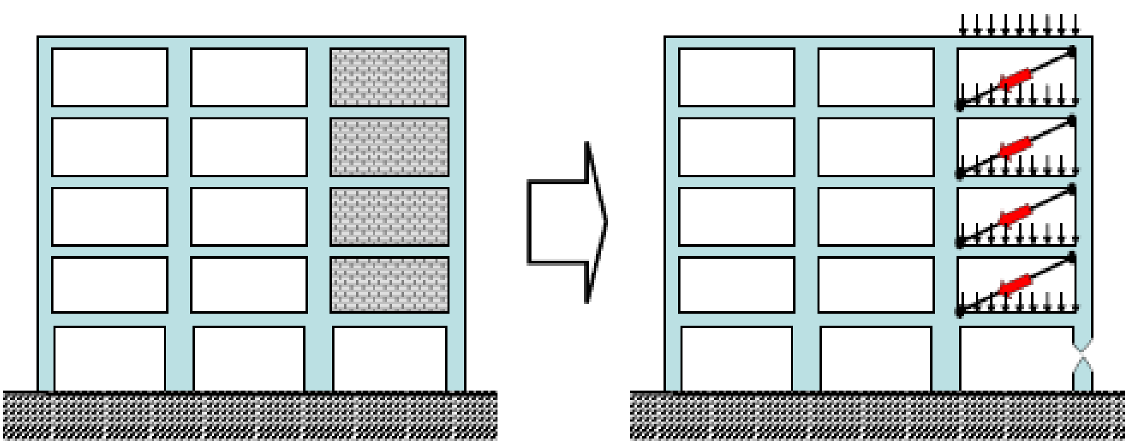
The complexities involved in applying such solutions may explain why these masonry parapets were not rebuilt after the quake, but replaced by steel railings. Photograph 5-17 shows a building damaged by the earthquake before and after repair.

▲ Photograph 5-18

This chapter began with the assertion that masonry wall collapse caused the largest number of severe casualties at Lorca. In all due fairness, it should end with remarks on how these elements also helped save lives. This was the case of buildings where a whole row of columns collapsed, such as in Photograph 5-18.

In these buildings the partitioning remained as the sole resistant mechanism, as per the standard compressed strut system depicted in Figure 5-19. Simple calculations show that the bearing capacity of the masonry was more than sufficient to withstand the gravity loads on the building as a whole, which were much smaller than the forces induced by the earthquake in the lower storeys.

These questions will be addressed in the following chapter.



▲ Figure 5-19

Conventional buildings Infill-frame interaction

6

6. Conventional buildings Infill-frame interaction

In standard practice, the analysis of conventional (portal frame) building behaviour with respect to external actions such as gravity loads, wind, earthquake and so on is confined to the structure, with no regard for the effect of enclosures, partitions, elevator shafts or similar elements.

That approach is often contended to be conservative on the grounds that while the adverse effects of those elements (normally their self-weight) is taken into consideration, their contribution to building strength is not.

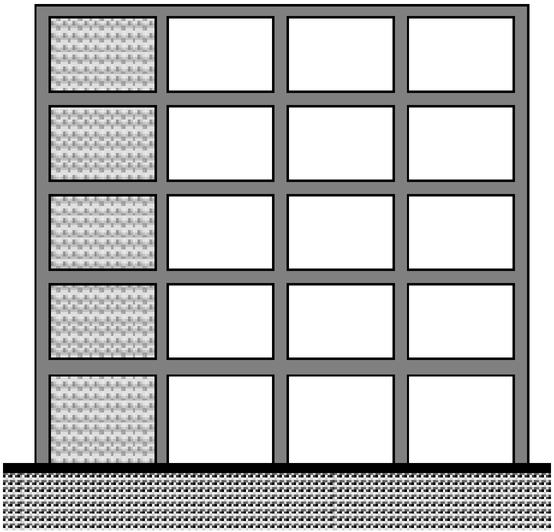
Such reasoning is nonetheless flawed because it omits any regard for the stiffness afforded by these elements, which modifies the internal force distribution across the structure entirely, rendering structural analysis alone futile. The logic in performing rigorous calculations with sophisticated software (even though the accuracy of such calculations is often questionable) for buildings subsequently fitted with elements that redistribute stiffness, and consequently internal forces, is elusive at best.

In seismic design, such reasoning is particularly perilous because greater stiffness entails greater forces, which may climb to levels several times higher than in the bare structure.

The in-plane stiffness and strength of masonry infills often exceed the values for the frames that confine them, conditioning the behaviour of the latter. The problem is more complex when the walls form part of a structural frame, as is often the case (Photograph 6-1). The complex behaviour exhibited by the infill-frame assembly differs from what is observed in the two elements separately and has yet to be fully quantified.

The foregoing can be substantiated by a simple example. The exercise consists of calculating the increase in horizontal stiffness in the five-story, four-bay portal frame building described in item 5.2 above, assuming just one of the bays to have a masonry infill.

▲ Photograph 6-1



▲ Figure 6-1

The horizontal stiffness of a wall 4 m long and 3.5 m high (Figure 6-2), such as in the ground storey of that portal frame building, can be readily found as:

$$K=\frac{F}{\Delta}=\frac{F}{\gamma\cdot h}=\frac{F}{\frac{\tau}{G}\cdot h}=\frac{F\cdot G}{\frac{F}{e\cdot \ell}\cdot h}=$$

$$G\cdot\frac{e\cdot\ell}{h}=1500\frac{N}{mm^2}\cdot\frac{130mm\cdot 4000mm}{3500mm}$$

$$=2.2\,E\,5\,\frac{N}{mm}$$

The lateral stiffness of the frame (Figure 6-3) would be:

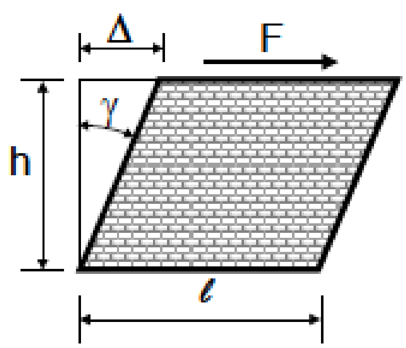
$$I=\sum_{i=1}^{5} I_i=\sum_{i=1}^{5}\frac{b_i\cdot h_i^3}{12}=3.375E9\,mm^4$$

(five 30x30 columns)

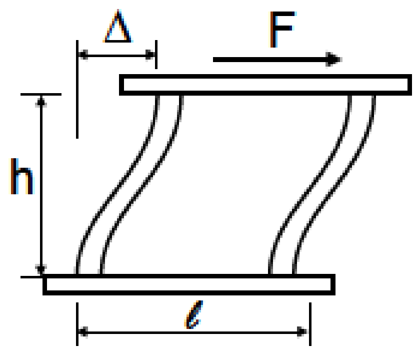
$$K=\frac{12\cdot E\cdot I}{h^3}=$$

$$\frac{12\cdot 25000N/mm^2\cdot 3.375E9\,mm^4}{(3500\,mm)^3}$$

$$=2.4\,E\,4\,\frac{N}{mm}$$



▲ Figure 6-2



▲ Figure 6-3

In other words, a single masonry wall affords more stiffness than all the columns in the storey together.

In fact, such walls are often the sole source of a building's stiffness and strength relative to horizontal actions. One of the structural designs frequently used in Spain consists of successive parallel rows of flat portal frames joined by one-way slabs (Photograph 6-2). Such a system cannot ensure sufficient stiffness or strength to counter horizontal loads in the direction of the slab, particularly in the case of steel frames, in which the joists simply rest on the beams with no connection whatsoever.

In such cases, building stability in that direction relies exclusively on the masonry infill. Nonetheless, contrary to expectations, in Lorca these buildings were not found to have an especially high damage rate, an observation that would infer that it was actually the non-structural elements that absorbed the seismic action.

▲ Photograph 6-2

This additional stiffness and strength may lead to serious problems if it is not taken into consideration in the design. Some of these problems are discussed below.

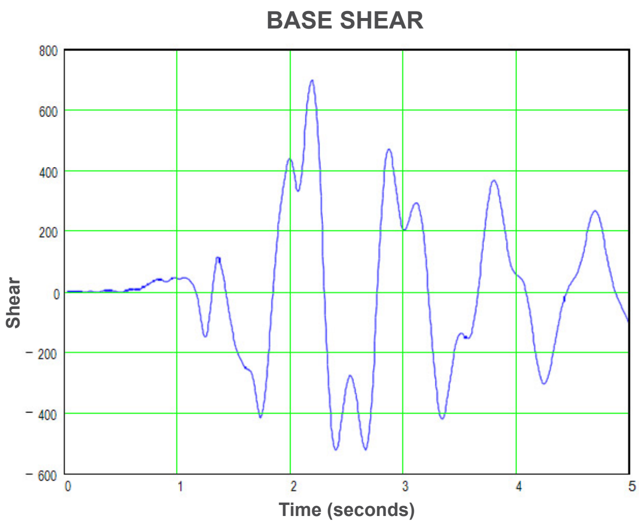
6.1. Increased loads

In the example in the preceding chapter, the stiffness of the portal frame building was adjusted so that its natural period would concur with the code value for similar buildings. Such adjustments are not made in standard design practice, however. Rather, the value used is the period resulting from modal analysis of the structure (i.e., the value calculated by structural engineering software).

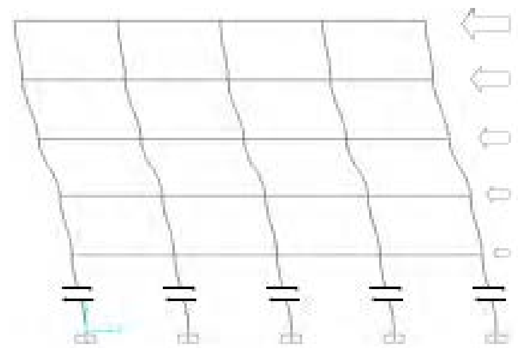
That raises the natural period from 0.45 to 0.85 s (or even to 1.0 s if instead of a shear shape, a conventional frame model is used). Any building is necessarily stiffer than its bare structure.

When this portal frame building was subjected to the Lorca accelerogram and analysed with specific software, the base shear obtained was as shown in Figure 6-4, with a peak value of 700 kN.

The base shear is simply the sum of the shear forces on all the ground storey columns (Figure 6-5). That value logically concurs with the horizontal reaction in the foundations and the total horizontal load induced on the building during the quake.



▲ **Figure 6-4**



▲ **Figure 6-5**

That peak value, 700 kN, is slightly less than 30 % of the total frame weight, 2 536 kN. Pursuant to the provisions of the Eurocode on earthquake-resistant design, the structure should actually have been modelled with the stiffness values for the cracked sections or, where that was not possible, with a stiffness no greater than half of the value for the unaffected section. See Álvarez [38] and Dávila *et al.* [43] for an analysis of the implications of that requirement.

To quantify the effect of the infill, the above exercise was repeated with the stiffness provided by a single panel (i.e., 4 m wide). The variations in the base shear yielded by the model simulating the earthquake are shown in Figure 6-6. Note that the peak value trebled to 2 150 kN, or 85 % of the total weight of the building. The increase in actions is readily explained when the periods for each situation are overlain on the acceleration response spectrum, as in Figure 6-7. While the period for the bare structure was 0.85 s, the added stiffness provided by the masonry lowered that value to 0.29 s, multiplying the pseudo-acceleration.

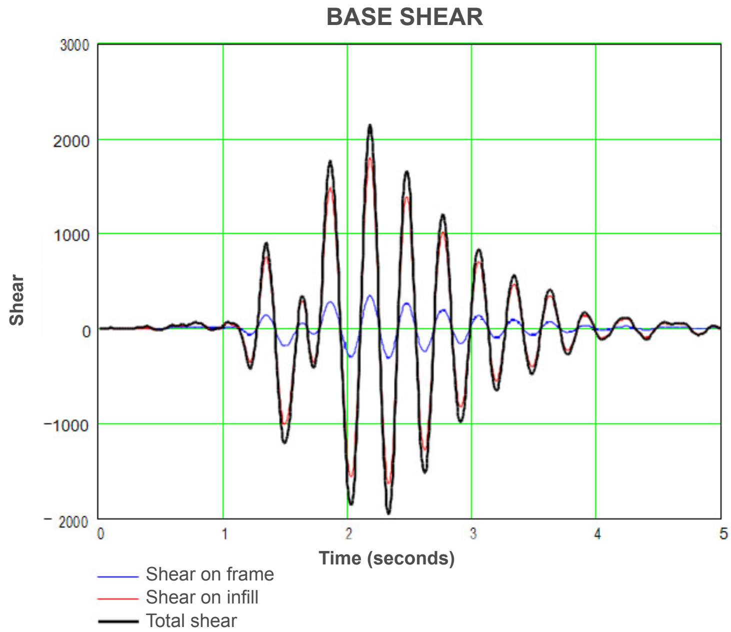
Although the aforementioned values may appear to be extreme, similar results were reported in the literature consulted. The effect of masonry was even addressed in classic texts (Dowrick [23], Paulay and Priestley [33], etc.). In tests conducted on a full-scale model at the EU's Joint Research Centre at Ispra (Fardis [46]), masonry infills raised building lateral stiffness 16-fold. In tests conducted and reported by Bertero [41] 30 years ago, the masonry quadrupled the capacity of structural portal frames. According to Taranath [20], experimental frequency measurements in New York City's Empire State Building revealed that the building was 4.8 times stiffer than its bare structure...

Actually, the theoretical peak shear at the base, the 2 150 kN mentioned above, is a statistic of minor interest. As Figure 6-6 shows, most of that stress, 1 805 kN, would have to be transferred by the masonry, which obviously is not strong enough to do so. In fact, the resulting tangential stress would be:

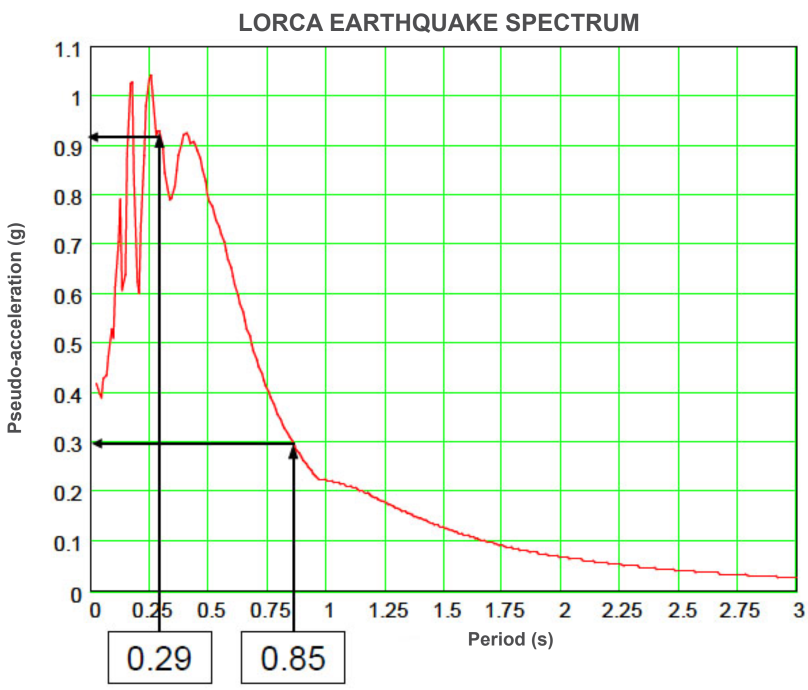
$$\tau = \frac{V}{\ell \cdot t} = \frac{1805kN}{4m \cdot 0.13m} =$$

$$3471 kN/m^2 \approx 3.5 N/mm^2$$

where ℓ and t are, respectively, wall length and thickness (including rendering).

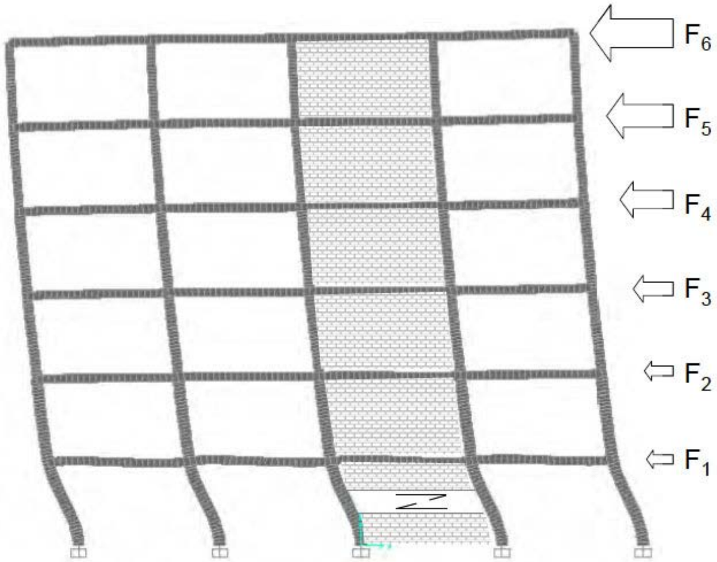


▲ Figure 6-6



▲ Figure 6-7

▲ Photograph 6-3



$$\sum_i F_i \leq V_{Masonry} + V_{Columns}$$

▲ Figure 6-8

The value found is more than a full order of magnitude higher than the strength of any masonry wall. That would explain the failure in so many of the lower storey infills (Photograph 6-3) in Lorca and the aforementioned near irrelevance of the theoretical shear value.

The peak horizontal force induced by an earthquake on a building can never exceed the strength of its ground storey elements (such as masonry walls, columns and stair slabs, Figure 6-8), which is the value that truly matters. In other words, the actions on buildings often, and in Lorca especially, depend much more on the strength of the (structural and, more essentially, non-structural) ground storey elements than on seismic action.

In short, calculating seismic forces where building stiffness, which includes the stiffness afforded by non-structural elements, is taken as the initial value on the spectrum is not entirely realistic. That, however, does not justify standard practice whereby the effect of these elements is omitted, in light of the significant contribution they may make to total actions.

All the references consulted concurred in highlighting the importance of non-structural elements (not only enclosures and partitions, but also stair slabs and ramps) in a building's response. The subject has not been broached from a consensus position, however. While some authors (Pujol *et al*. [55]) stress the lesser displacement and greater damping that can be attributed to masonry, others (Álvarez, [38]) emphasise the uncertainty it induces in calculations to determine building response.

These issues are addressed in greater depth in the items that follow.

6.2. Vertical irregularities

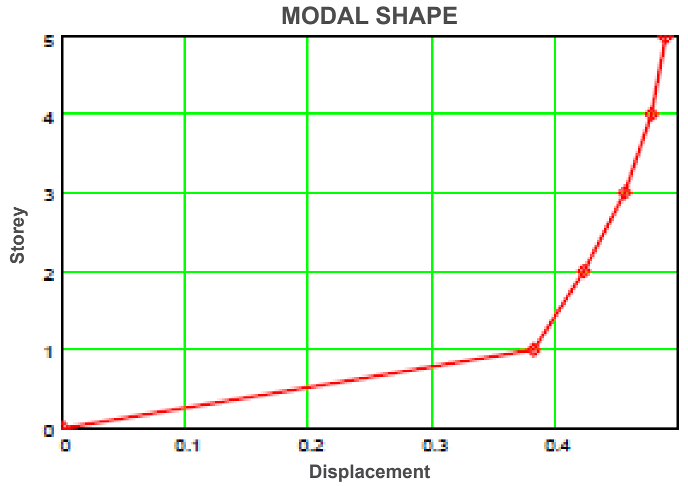
Masonry failure can be simulated by running the model without the stiffness provided by the ground storey wall. This is not wholly accurate because, depending on the type of failure, wall strength may not be entirely or immediately depleted, but it serves to describe the effects.

As might be expected, in this case the natural period rose, to 0.52 seconds, and the base shear declined slightly, to 1 900 kN. Given the theoretical absence of infills to absorb most of the shear, in this exercise the shear had to be borne by the ground storey columns.

More interesting, however, was the change observed in the first mode shape (Figure 6-9), which was indicative of the formation of a flexible or soft storey mechanism which, as discussed below, may be highly unfavourable. Its effect is so consequential that the Eurocode on earthquake-resistant design provides that it must be taken into consideration:

> *"...Account shall be taken of the high uncertainties related to the behaviour of the infills (namely, the variability of their mechanical properties and of their attachment to the surrounding frame, their possible modification during the use of the building, as well as their non-uniform degree of damage suffered during the earthquake itself)..."*

The code also provides that the entire length of the ground storey columns must be regarded as critical and reinforced accordingly (by raising the respective secondary reinforcement ratio).

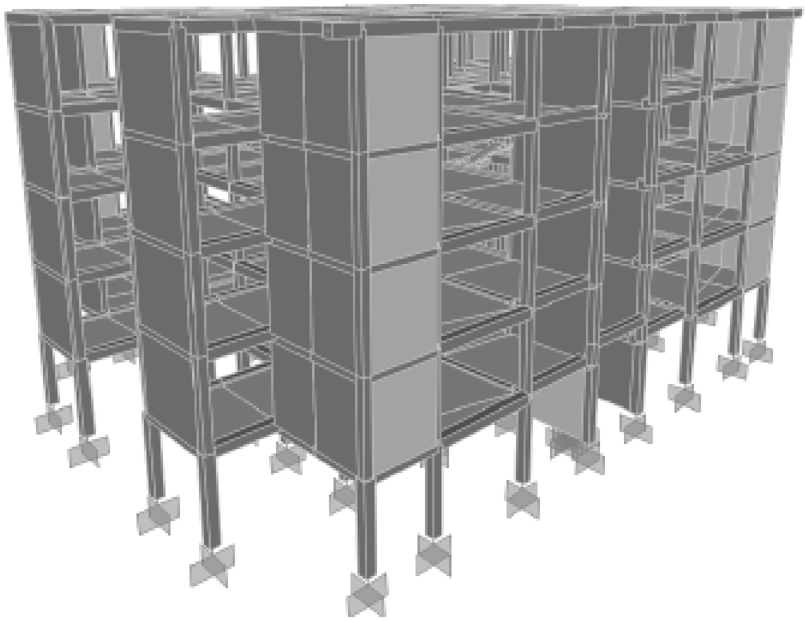


▲ Figure 6-9

The aforementioned flexible storey mechanism often arises not as a result of ground storey masonry failure but of the lack of any infill on that storey. An example found in one of the buildings in San Fernando quarter at Lorca is depicted in Photograph 6-4.

The building was analysed with several numerical models, one of which included the effect of some of the enclosure walls, as shown in Figure 6-10.

▲ Photograph 6-4



▲ Figure 6-10

Table 6-1 summarises the findings, which are discussed below:

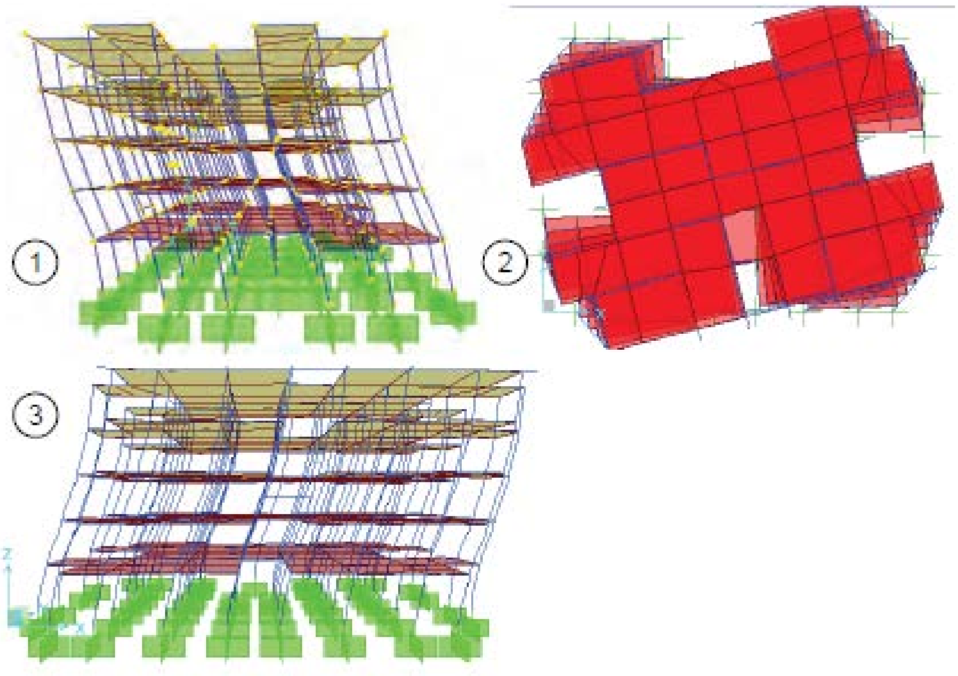
- When the structure was modelled in three dimensions, three instead of only one natural periods were applied, one for bending in each of the plan directions (short and long sides) and the third for torque. The first three modes for the bare structure are shown in Figure 6-11.

- The inclusion of non-structural elements in the model not only lowered the values of the periods in each direction, but also altered their order.

- The reduction was sizeable. The first mode in the short direction declined by more than half (stiffness in that direction rose more than four-fold due to the effect of the non-structural elements). Note that the numerical model only covered the effect of the blind façade walls, excluding all open sections and indoor partitions.

	1st Mode	2nd Mode	3rd Mode
Bare structure	1.00 Short side	0.936 Torque	0.896 Long side
Structure and staircases	0.932 Torque	0.853 Short side	0.846 Long side
Structure, staircases and façades	0.605 Long side	0.455 Torque	0.419 Short side

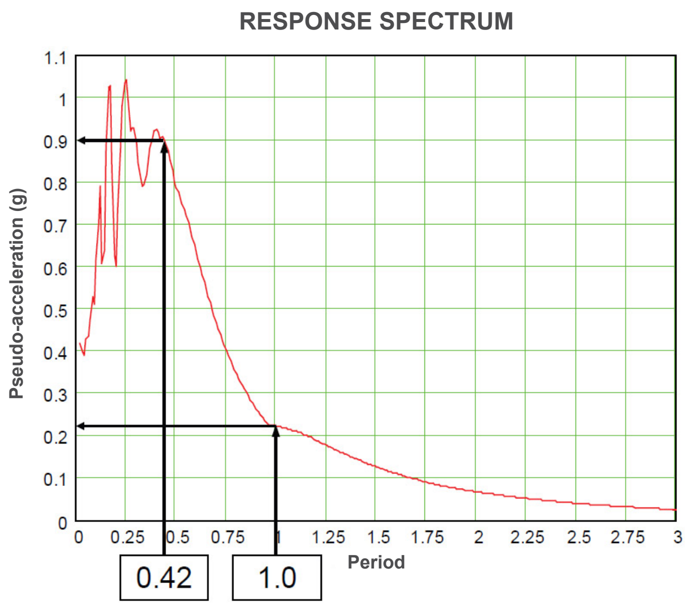
▲ Table 6-1 Periods (in seconds) by mode



▲ Figure 6-11

- The two short side mode values must be placed on the spectrum to perceive their significance. For buildings oriented in that direction, the effect of the earthquake was found to be four times greater with than without the enclosures (Figure 6-12). In this specific case, moreover, the proportion would be realistic, because the façades were not severely damaged and actual building stiffness would be close to the model value.

- The soft storey effect could be clearly seen in the first mode for the short side (Figure 6-13).



▲ Figure 6-12

▲ Photograph 6-5

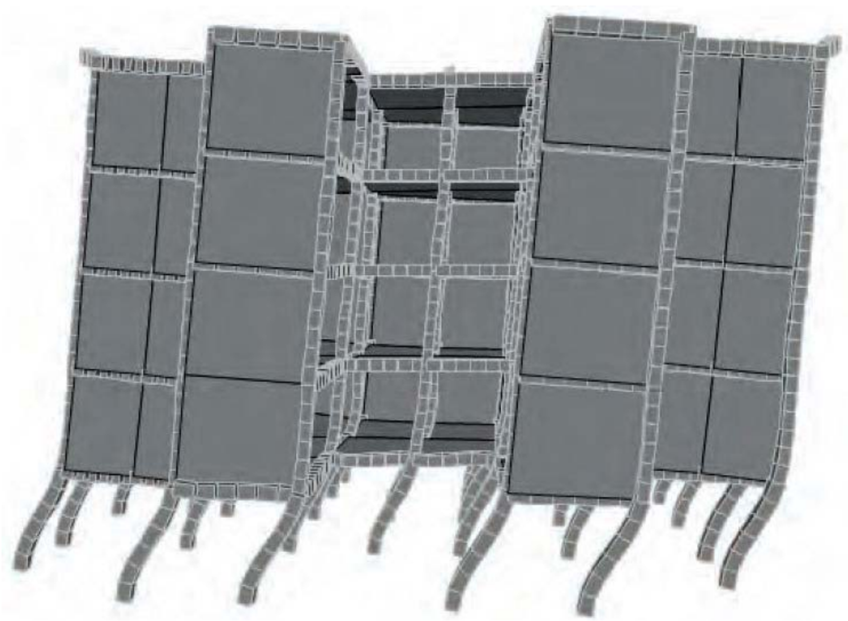
6.3. Plan irregularities

The symmetrical arrangement of masonry walls found in the San Fernando quarter buildings referred to in the preceding section was not the general rule by any means.

Buildings were much more frequently observed to have thick masonry infills as separation walls (Photograph 6-5). In buildings located on the corner of a city block, such walls would clearly generate in-plan asymmetries.

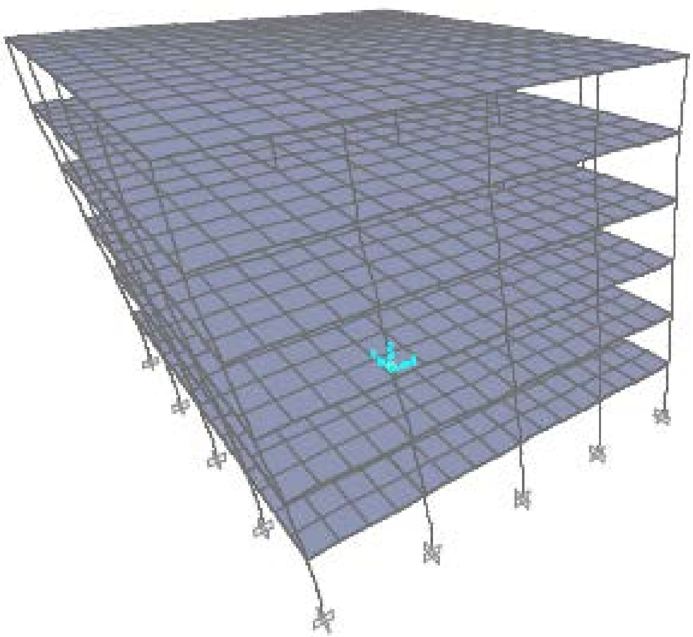
The building in Figure 6-14 is an example of a conventional six-storey apartment complex consisting of two-way reinforced concrete portal frames.

The shape of the first vibration mode is shown in Figure 6-15. The columns would be stressed solely in one direction. When separation infill masonry was added on one of the sides, as in Figure 6-16, the mode shape changed, as shown in Figure 6-17.



▲ Figure 6-13

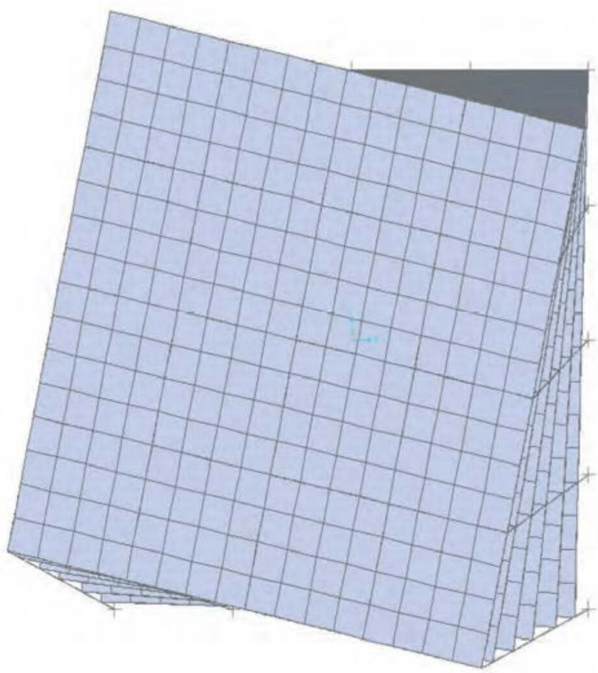
▲ Figure 6-14



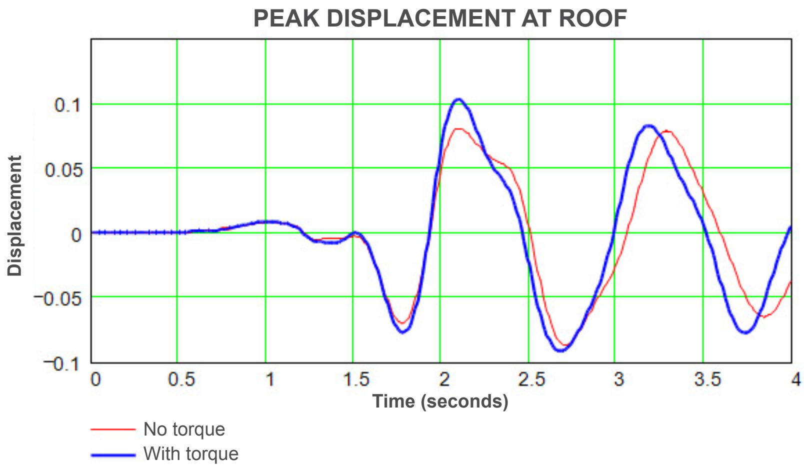
▲ Figure 6-15

▲ Figure 6-16

The ultimate effect was that the displacement in the direction shaken was slightly more intense on the corner columns (as shown in Figure 6-18) and, much more importantly, those columns were also stressed perpendicularly.



▲ Figure 6-17



▲ Figure 6-18

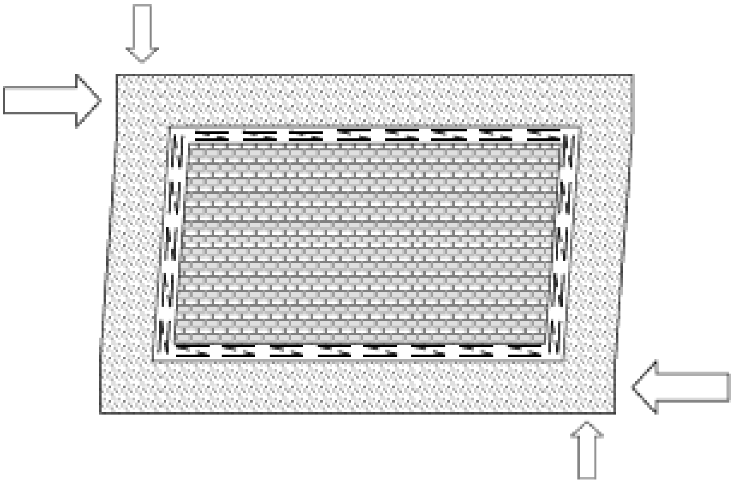
6.4. Structural damage

As discussed in item 6.1, "Increased loads", when masonry infills were added to the portal frame model the horizontal loads on the building trebled.

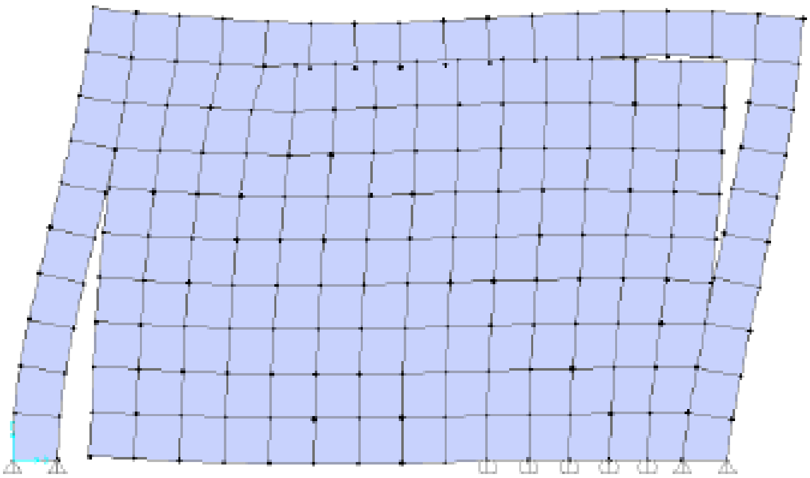
It might be contended that, regardless of how realistic these values are, such conditions may not necessarily be detrimental to the structure. The load increase induced by the masonry infills would be absorbed by the walls themselves, independently of the structure. A direct reading of the findings in Figure 6-4 and Figure 6-6 reveals that the shear on the columns declined, a situation that might even be interpreted as structurally beneficial.

Actual conditions, however, are much more complex and less favourable. The frame-infill interaction model used in the aforementioned item assumed linear behaviour in which the masonry worked under pure shear (Figure 6-19). Further to that model, only the axial load on the columns rose.

That is accurate for the lowest stress levels only, however. As those levels rise, the frame-infill interface fails and the two elements pull apart as in Figure 6-20 (drawing taken from a very elementary numerical simulation).



▲ Figure 6-19

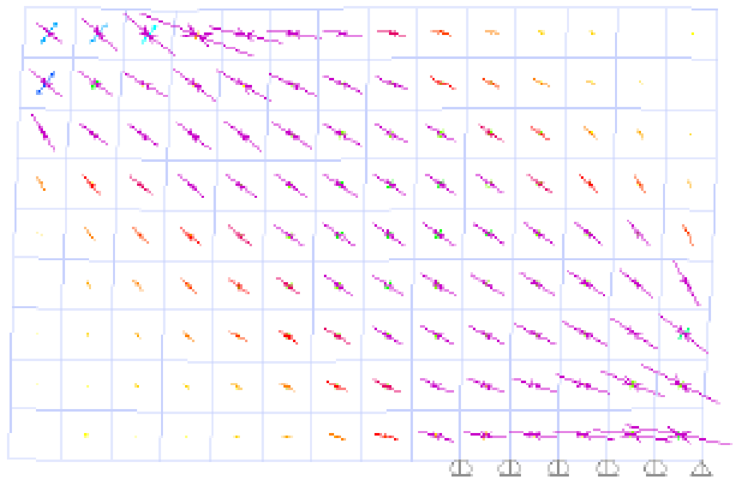


▲ Figure 6-20

As a result of the local separation between masonry and frame, the former begins to work like a strut driven into the opposite corners of the frame, pursuant to the stress diagram in Figure 6-21.

Such considerations have prompted many authors to propose using equivalent struts instead of masonry walls. In its simplest form, put forward by Paulay and Priestley, the brace would consist of a masonry element with a width one-fourth of its length, Figure 6-22. Crisafulli [22] described more elaborate models.

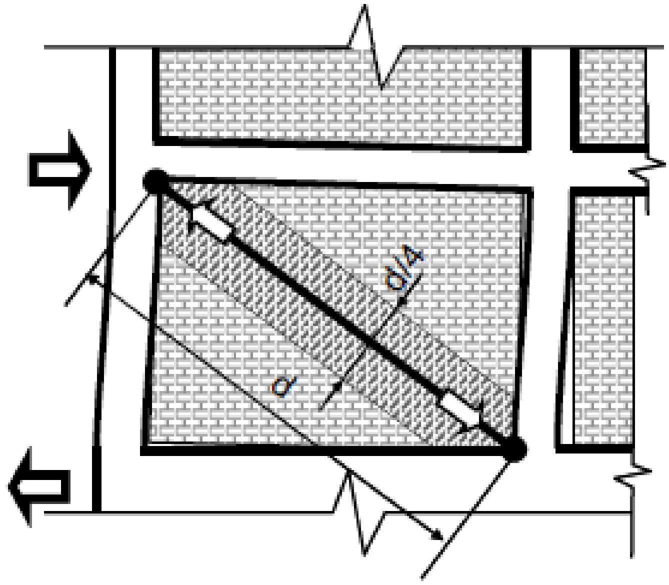
In this much more complex mechanism, shear stress is indeed transferred to the column, because in addition to the tangential stress, the interface is affected by normal loads whose resultant (Figure 6-23) is actually the shear stress.



▲ Figure 6-21

Obviously, the horizontal load transferred by the wall is not absorbed entirely as shear by the columns. Part of that load continues to adopt the form of tangential horizontal stress between the wall and the beams. Since the issue here is to safely assess how the two types of stress are distributed, in practice the conservative assumption, i.e., that the entire load is transferred to the column as shear, is often adopted.

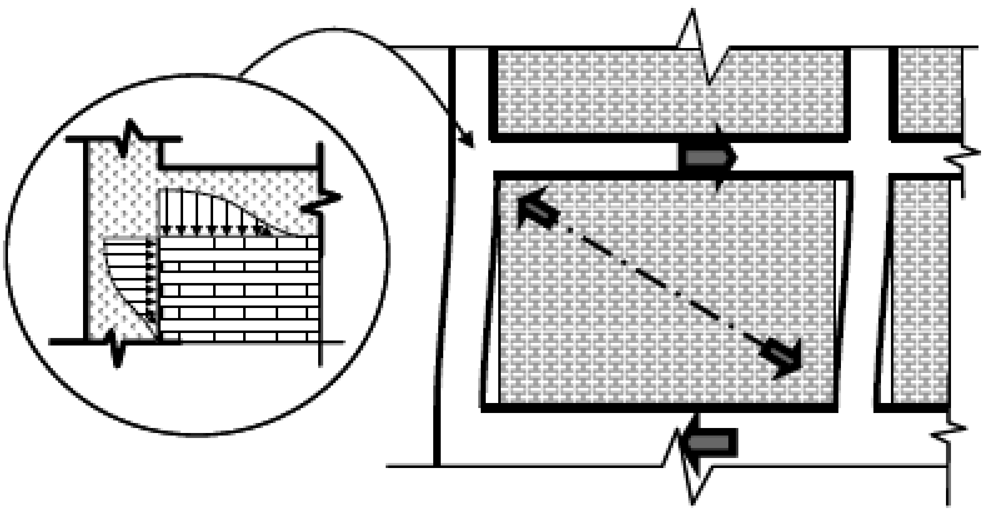
As the ends of beams are also stressed perpendicularly, shear rises in the respective sections (as shown in Figure 6-23). This does not generally pose serious problems, however, because such higher values appear at points where shear stress, due to the design gravity loads, conditions dimensioning. In other words, the stirrups are engineered to withstand the shear resulting from the worst case scenario, factored upward to accommodate gravity loads. The latter are much greater than modelled for earthquakes, for which only the permanent (unfactored) loads and a given percentage of the service loads are taken into consideration.



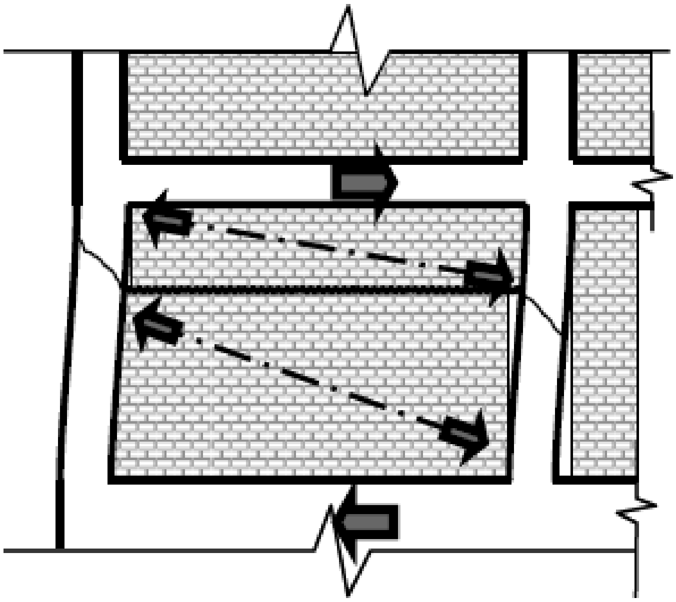
▲ **Figure 6-22**

In Lorca, no beams were observed to fail for that reason.

Further to the discussion in this and the preceding section, then, the following precautions should be taken in connection with the types of structures observed at Lorca.



▲ **Figure 6-23**



▲ Figure 6-24

- Masonry infills, at least in the lower storeys, must be expected to fail in the event of moderate seismic forces, when built as in standard construction practice.

▲ Photograph 6-6

- When failure occurs (in the infill or simply at its interface with the frame), masonry walls must also be assumed to transfer all the shear generated by their failure to the columns that confine them.

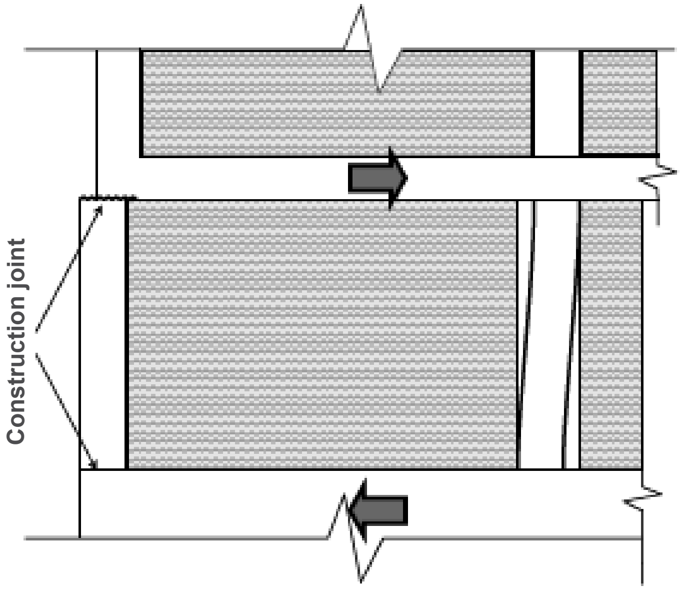
Consequently, columns must be checked against the maximum shear strength of the adjacent masonry. This is one of the essential precautions prescribed in the Eurocode on earthquake-resistant design, item 5.9, “Local effects due to masonry or concrete infills”.

Performing such verification is not straightforward, however, because the columns, when stressed by the masonry infills, may fail via any one of three mechanisms and the walls themselves may fail in one of several ways. The mechanisms possibly involved in column failure are listed below.

The most conventional mechanism would entail the formation of diagonal cracks, as shown in Figure 6-24 and Photograph 6-6. Column resistance in this situation can be assessed using the standard equations proposed in the legislation for linear elements (Spanish concrete code EHE, for instance [13]).

A second type of failure would be induced by construction joint sliding between columns and beams, as illustrated in Figure 6-25.

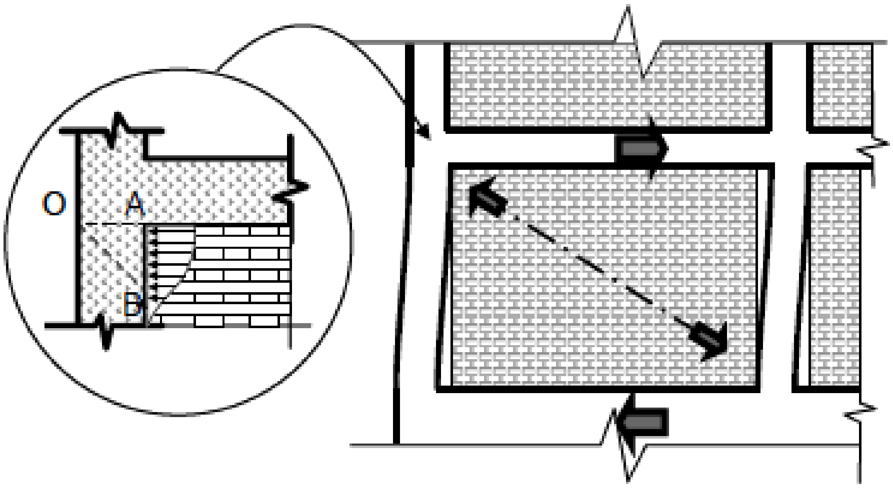
This type of failure was observed fairly frequently in Lorca, particularly in corner columns, such as in Photograph 6-7.



▲ Figure 6-25

A concentration of stress at the top of the column (Figure 6-23) would prevent the development of the conventional failure mechanism because most of the load would be exerted at less than one measure of depth from the end of the column, i.e., between the conventional fracture surface, line OB, and the joint, line OA (Figure 6-26).

▲ Photograph 6-7



▲ Figure 6-26

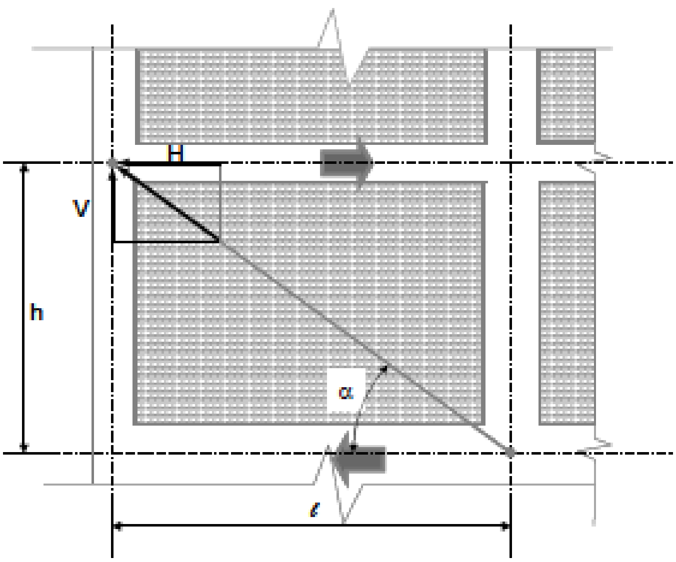
Under such conditions the construction joint, whose strength mechanisms are not always reliable, may slide.

The most effective of those mechanisms, friction, is not readily calculated because the value of the axial load borne by the column cannot be precisely estimated due to the difficulty involved in assessing the vertical component of the force applied to the joint by the wall, or to put it differently, in assessing the slant on the equivalent strut. That component, which is logically subtracted from the compressive load travelling down the column, can (theoretically at least) induce tensile stress in the column itself.

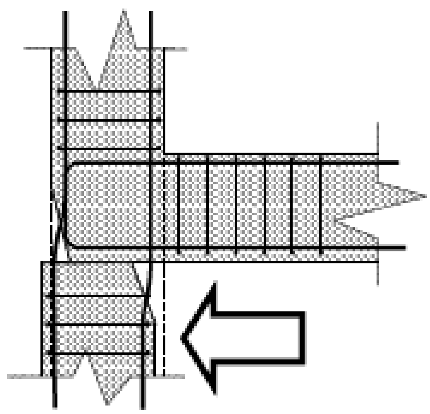
In a rough estimate, assuming that the equivalent strut would be sited between the node axes as in Figure 6-27, the vertical component would be:

$$V = H \cdot \frac{h}{\ell}$$

Where H is masonry infill strength.



▲ Figure 6-27



▲ Figure 6-28

As noted earlier, determining the distribution of gravity loads between the frame and the infill constitutes an additional difficulty. The simplest interaction models discussed above showed that columns may even be subject to tensile stress before an earthquake hits.

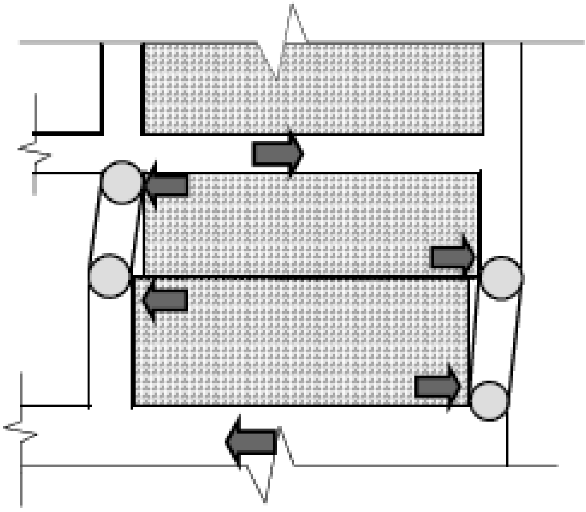
Another strength mechanism identified by the Eurocode on earthquake-resistant design is based on the dowel effect in the column reinforcement that bonds the construction joint.

Calculations to determine the bonding steel ratio cannot include all the bars that cross the joint, because the ones at the ends are located too close to the free edge. Since all such bars do is expel the concrete cover (Figure 6-28), they are unable to provide reliable strength. For that reason both the Eurocode and the Spanish building code require at least one intermediate bar between the ones on the ends of each side.

On occasion simple visual inspection of the damage at Lorca revealed negligent workmanship in the joint (note the smooth finish on the joint in Photograph 6-8 and the use of only two bars per side to reinforce the column).

The third type of column failure, depicted in Figure 6-29, is due to the formation of plastic hinges. This is the least hazardous of the types of failure described, because as it would be the result of flexural forces, some deformation could be accommodated. It is also the least likely, however, because to prevent prior shear failure the columns would need to be very heavily reinforced (because the column is effectively divided into short sections).

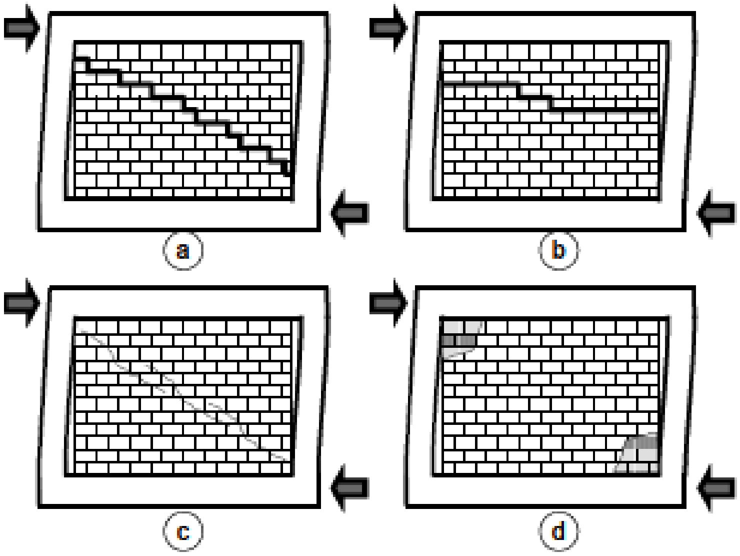
▲ Photograph 6-8



▲ Figure 6-29

This issue is addressed more fully in Hermanns *et al.* [52].

Crisafulli, in turn, describes four types of masonry failure observed under laboratory conditions.



▲ Figure 6-30

As depicted in Figure 6-30, they are: a, debonding; b, mortar joint sliding; c, diagonal tension; and d, crushing at the corners.

These types of failure are the result, of course, of uniform, controlled experimental set-ups. Less uniform and less regular real-life construction using variable materials would give rise to further mechanisms. At Lorca, new openings had been made in some infills, while in others former openings were covered up (Photograph 6-9). Walls were built with all manner of brick, highly disparate bonds and a very wide variety of fills, especially in the upper rows. As a result, failure followed a completely different and in all likelihood less hazardous pattern than observed in frame-infill tests (although the latter assertion would need to be verified).

▲ Photograph 6-9

▲ Photograph 6-10

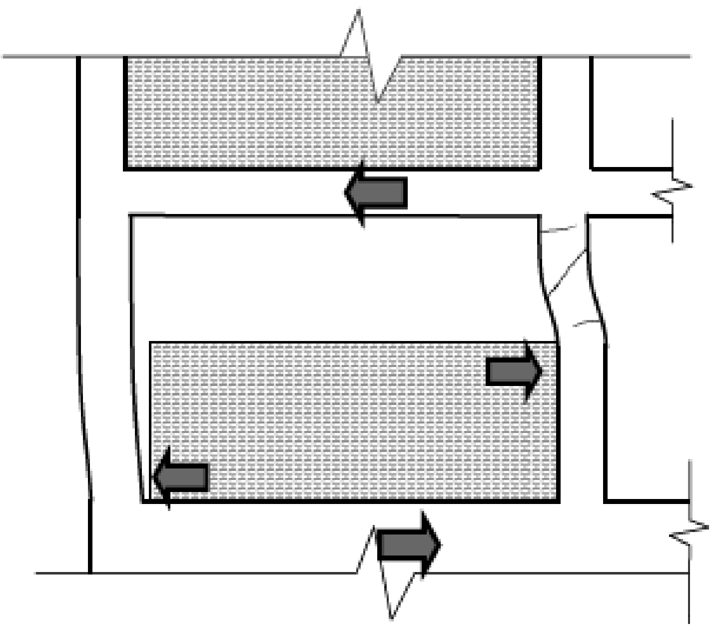
These real-life walls are probably weaker than test walls, which is beneficial in the present context, i.e., the structural damage induced by these walls because of their strength.

At Lorca, damage was consistently observed at the frame-infill interface, as shown in Photograph 6-10, although this is not, strictly speaking, a failure mode, for it involves a loss of stiffness but not of strength. Failure types similar to the ones described earlier were also identified. Photograph 6-11 depicts a masonry wall crushed at the corners and Photograph 6-12 mortar joint failure.

▲ Photograph 6-11

▲ Photograph 6-12

▲ Photograph 6-13



▲ Figure 6-31

A more common type of failure, shown in Photograph 6-13, may be a combination of several of the basic types described by Crisafulli.

The issue, then, proves to be particularly complex, despite the apparent simplicity of the initial premise. Something as straightforward as determining whether the columns are stronger than the masonry (to prevent the inevitable wall or interface failure from inducing column failure) is rendered tremendously complicated due to the variety of ways in which these elements may fail. The solution, in principle, would call for calculating each and every possible failure combination to reach a conclusion.

Under the Eurocode on earthquake-resistant design, calculations need only be performed to determine wall resistance to mortar joint sliding, for which Paulay and Priestley proposed the following expression:

$$V = \frac{\tau_0}{1 - \mu \cdot \frac{h}{\ell}} \cdot d_m \cdot t \cdot \cos\alpha$$

Where:

- τ_0: masonry shear strength in the joint direction
- μ: coefficient of friction
- h, ℓ, t: wall dimensions
- d_m: length of the wall diagonal
- α: diagonal angle off the horizontal

Here the crux is to determine the strength of the masonry infill. According to Paulay and Priestley themselves, strength may range from 0.1 to 1.5 MPa and friction from 0.3 and 1.2.

▲ **Photograph 6-14**

Entering those values into the above formula yields strength values of 74 to 3 214 kN...

The above discussion assumes the most elementary situation, in which a blind masonry wall covers the entire span. The contrary, illustrated in Figure 6-31 and Photograph 6-14 and known as the "*captive column*" in the literature, induces short column failure, as discussed at some length below.

The Eurocode on earthquake-resistant design warns against this mechanism very clearly. When the gap at the top of the column is less than one and one half times its depth, the code requires the use of diagonal reinforcement to resist the shear stress generated.[1]

6.5. The legislation: remarks

While, as explained in the preceding chapter, the problem of masonry as a passive element (when under bending force) has been explicitly addressed in all earthquake-resistant legislation for many years, its active role in building response has not been the object of specific attention in codes until fairly recently.

The first direct reference[2] in Spain appeared in item 4.1.4 of the NCSE-94, "*Non-structural elements*":

> *"... Non-structural elements such as enclosure and partition walls, which may develop sufficient stiffness and strength to alter structural conditions, must be taken into consideration in the structural analysis model and the calculations relative to the respective actions..."*

1. Beware of the possible consequences, even in terms of legal liability, of such prescriptions, so common in the Eurocode.

2. Actually, the effects of partitions are implicitly taken into consideration natural period assessments.

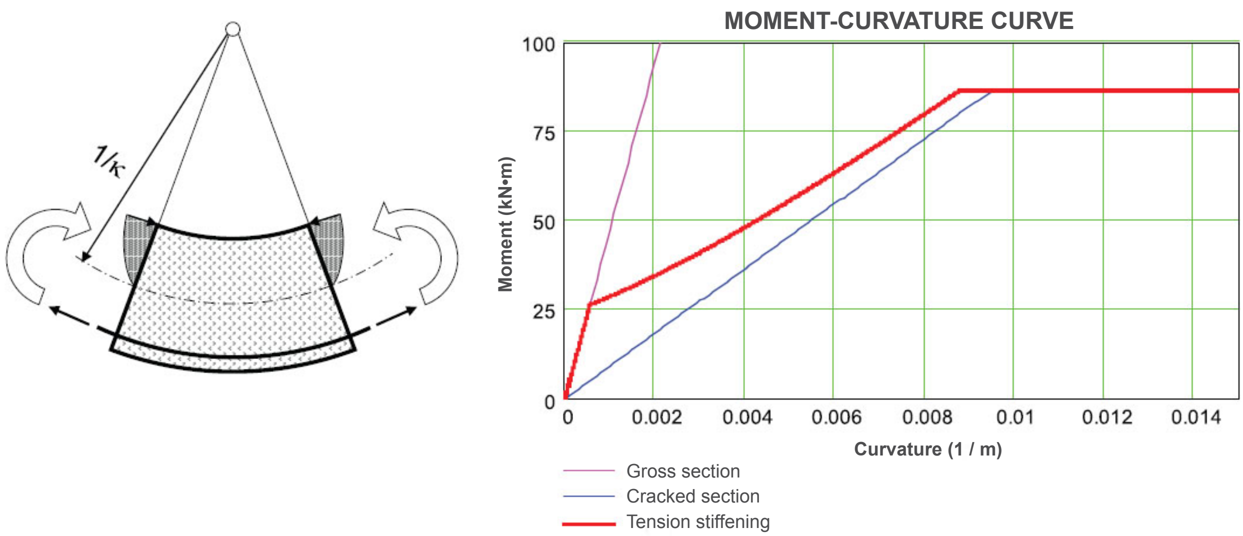
That article was maintained intact in the code presently in effect, NCSE-02.

The existing code on structural steel [12] deals with this question more fully and even proposes an acceptable analytical procedure:

> *"...When the effect of such elements on structural stiffness cannot be assessed with sufficient precision either because their behaviour is not fully understood or because it may vary throughout the life of the structure, conservative values must be adopted.*
>
> *The foregoing will generally call for conducting more than one analysis. Internal forces will be assessed with models in which stiffness is not lower than the actual values. Conversely, the stiffness values used to assess displacement will be no higher than the actual figures ..."*

Since its initial, early nineteen eighties editions, the Eurocode on earthquake-resistant design has contained provisions requiring the inclusion of the role of masonry in building response evaluations. They call for internalising the contribution of infills to stiffness directly in the assessment of the building's natural period, calculated as the arithmetic mean of the values for the bare structure and the full building, and applying the initial stiffness of all the enclosures.

In its present version, the Eurocode is very explicit in this regard. It not only contains whole sections on non-structural elements and their implications, but even conditions such essential aspects as the ductility reduction factor, which in the case of masonry-infilled steel portal frames is limited to 2 (Table 6.2 of the code), or column design in the case of concrete structures, to their presence.



▲ Figure 6-32

Perhaps one of the reasons why the code devotes so much attention to non-structural elements and specifically to the need to include the stiffness afforded by their presence in the building model is the reduction in the stiffness of the structural members that its approach entails.

The code clearly specifies that when analysing the structure with the response spectrum method (the procedure applied by a vast majority of architects and engineers and in virtually all structural engineering software), cracking in concrete structures must be taken into consideration. More specifically it provides as follows:

> *"...In concrete buildings, in composite steel-concrete buildings and in masonry buildings the stiffness of the load bearing elements should, in general, be evaluated taking into account the effect of cracking. Such stiffness should correspond to the initiation of yielding of the reinforcement.*
>
> *Unless a more accurate analysis of the cracked elements is performed, the elastic flexural and shear stiffness properties of concrete and masonry elements may be taken to be equal to one-half of the corresponding stiffness of the uncracked elements..."*

The diagram in Figure 6-32, showing the relationship between moment and the curvature induced, illustrates the importance of this consideration.

The purple line represents the behaviour of a concrete section with no tensile cracking, disregarding the contribution of the steel to stiffness in this case. That is obviously inaccurate because the concrete is unable to withstand large tensile stresses and consequently cracks, but all the existing numerical models ignore that reality: structural engineering software calculates element stiffness from the nominal geometry of their sections.

The blue line depicts a wholly cracked section in which the steel yields.

The thickened red line assumes the section to be intact until it cracks and, for moments greater than the value that induces cracking, includes the effect of the concrete between cracks, which has been termed "tension stiffening". It represents intermediate behaviour closer to the performance of uncracked sections for small loads and of cracked sections for near yield loads.

The point made is that the stiffness of the cracked sections is lower than the value for the design section. In the example given in the figure, which depicts a typical beam section, the ratio is 1 to 5.

In other words, the values entered in structural design software should envisage structures around five-fold more flexible than normally modelled.

All the foregoing has very substantial consequences.

- It renders the imbalance between the stiffness actually afforded by the frame and the infill even more dramatic, both globally and locally. While the preceding chapters called attention to the soft-storey effect generated when the ground storey enclosures fail, here the emphasis is on the implications of reducing the stiffness of the respective column sections so drastically.

- All the results yielded by commercial software would be erroneous, strictly speaking. The displacement calculated would be much smaller than the real values, while the equivalent loads would be overestimated.

The present approach, which requires suitable modelling that appraises actual structural stiffness and includes enclosures as elements in the model, appears to be more rigorous.

All the foregoing stands as proof of the growing importance attached by the legislation to the interaction between structural and non-structural elements.

6.6. Conclusions

Masonry wall stiffness and strength in connection with in-plane actions conditioned building response during the Lorca earthquake.

That was often due to the lack of a structure able to withstand horizontal actions, which had therefore to be transferred across the masonry. Many of the buildings in Lorca (and the rest of the country) were designed for gravity loads only, with a structure lacking sufficient stiffness to withstand horizontal actions. This will be discussed at greater length in the following chapter. Be it said again here that in the authors' opinion not all the buildings were correctly constructed.

The problem is not that simple, however. Many other buildings that did have a satisfactory bearing structure (from the standpoint of horizontal action) also exhibited damage in all the masonry walls and essentially similar behaviour. The mere presence of a structure does not suffice: it must afford stiffness at least comparable to the stiffness of the masonry. That, however, is extraordinarily costly and difficult to achieve in practice. It nonetheless constitutes an essential issue that will be addressed in detail in a later chapter.

Conventional buildings
Structural issues

7

7. Conventional buildings Structural issues

The following discussion attempts to explain some of the structural problems encountered in buildings. The basic aim is to substantiate one of the ideas reiterated throughout this text: the effects of the Lorca earthquake on structures were not very different from the effects that have long been described in the literature on seismic action and can be catalogued under just a few specific headings.

7.1. Short columns

Lateral stiffness in columns is proportional to the cube of their length. One column half as tall but with the same section as another is eight times stiffer.

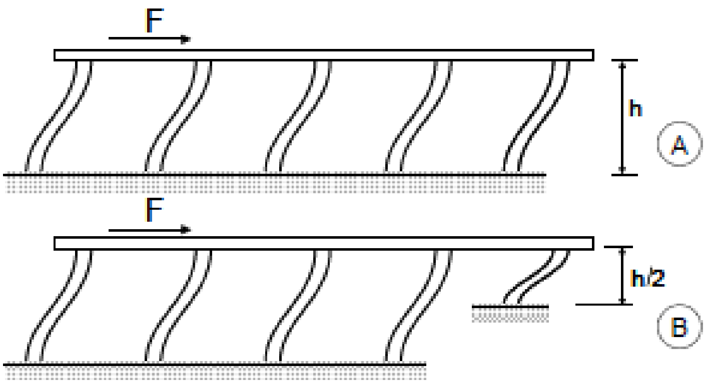
The inference is that if an earthquake imposes shear of a given value at the base of a building, and all the columns are similar, the proportion of shear absorbed is also similar (Figure 7-1 A). If on the contrary, one of the columns is shorter than the rest (assume half as tall), it absorbs a much (eight-fold) greater load. The short column in Figure 7-1 B would have to absorb two-thirds of the total shear, compared to the 20 % it would bear if it were of the same length as the others.

Although the foregoing estimates may appear to be greatly exaggerated, actual observations at Lorca revealed even less favourable situations.

One such situation, depicted in Photograph 7-1, was found in columns aligned with semi-basement façades.

▲ Photograph 7-1

Such storeys (normally used as car parks) were higher than the rest and consequently their indoor columns were likewise taller. The façade columns, however, did not spring from the same elevation as the indoor members, but rather from the street level perimetric wall.



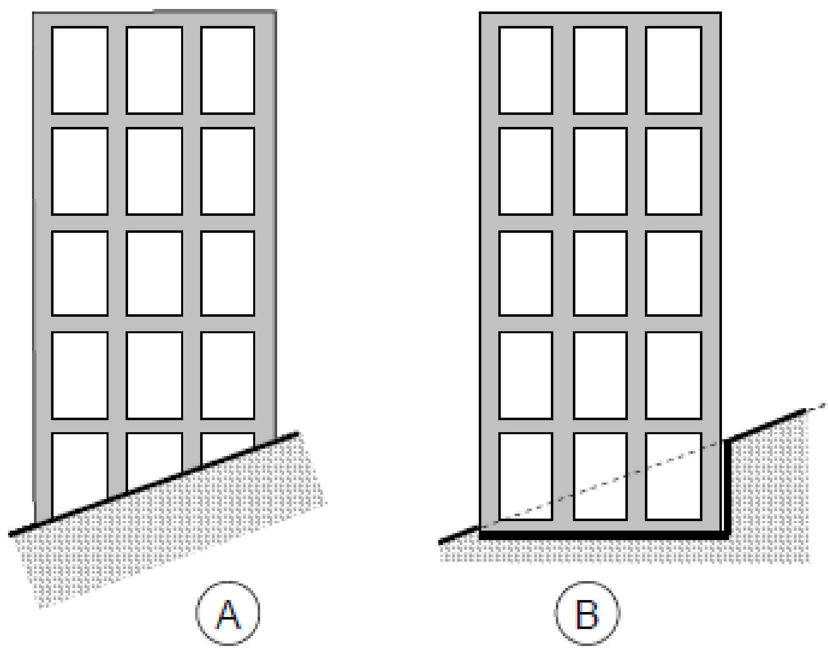
▲ Figure 7-1

Such walls were built not to abut with the floor slab above, which would be structurally rational, but rather to enhance indoor lighting and ventilation. The columns used were consequently very short. In some cases they were not even tall enough to be regarded as linear members, which precluded the feasibility of estimating their stiffness or strength, given the inapplicability of the standard expressions that refer exclusively to such linear elements.

Briefly, when the clear lengths of columns differ so disproportionately, estimating shear distribution is a senseless exercise: the short members absorb the entire load until they fail, after which the loads are transferred to the remaining columns.

Short columns springing from perimetric walls is not the only, although it is the most hazardous, example of this situation. Similar problems arise when a building is sited on a slope and the first storey elevation is maintained by varying the length of the columns (Figure 7-2 A). The problem is naturally the same if the end column springs from the same elevation as the others but is built adjacent to the basement wall (Figure 7-2 B). The slabs forming the access ramps in car parks are subject to an identical hazard. The damaged columns in Photograph 7-2 (the photograph does not depict the damage itself, but it does show the bracing) were sited under such a ramp.

The "*captive*" columns discussed at length in the preceding chapter entail even greater risk because the damage is unforeseeable. Captive columns are restrained by non-structural elements such as masonry infills and staircase slabs.



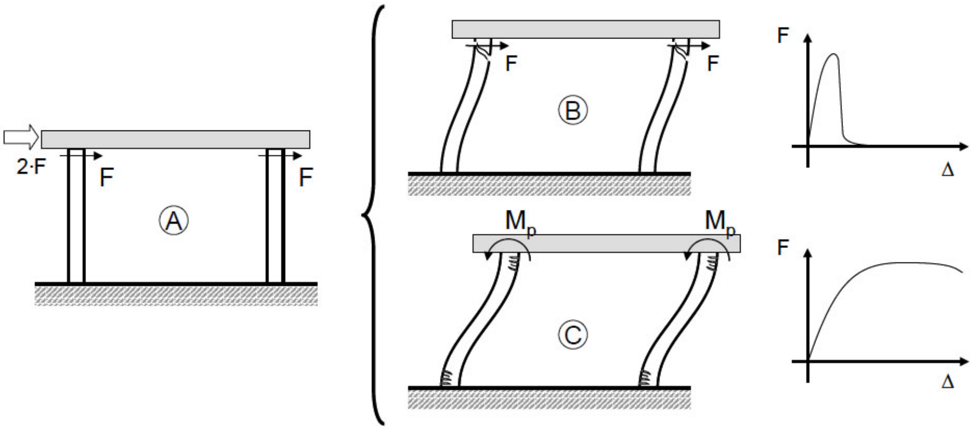
▲ Figure 7-2

The conclusion drawn from the foregoing is that all the columns in a given storey must be built to a similar length (to ensure uniformity, an idea repeated throughout this book) and that overly short columns should never be used. Exactly what constitutes the minimum allowable length would have to be quantified, of course.

An initial criterion would be to follow the traditional premises of design according for capacity principles. The design principle states that elements subjected to seismic forces should reach their ultimate bending strength before failing under shear. This general idea, applicable to any beam or column of whatsoever length, aims to guarantee the formation of a ductile failure mechanism.

▲ **Photograph 7-2**

The maximum value of force "*F*" that can be applied to the top of a column in a direction normal to its axis (Figure 7-3 A) is delimited by column shear failure (Figure 7-3 B) or the formation of plastic hinges at the top and bottom (Figure 7-3 C). Shear failure is brittle and allows for neither redistribution of stress nor a residual value that can be relied on. It is consequently a type of failure that should be avoided. In contrast, the formation of plastic hinges at the top and bottom provides for a plastic mechanism which, while not ideal (as discussed below), affords some ductility.



▲ **Figure 7-3**

The maximum shear force that a column can resist (or the greatest force "*F*" that it can transfer as per Figure 7-4) depends on its top and bottom bending strength, for simple static equilibrium leads to the following expression (Figure 7-4):

$$\sum M = 0 \Rightarrow -2 \cdot M_p + V \cdot h = 0$$
$$V = \frac{2 \cdot M_p}{h}$$

Therefore, given a shear strength of over $2{\cdot}M_p/h$, a column would never be subject to shear failure.

An example may make this clearer. Assume an HA-25 reinforced concrete (the lowest strength structural concrete) column with a 40-cm square section, reinforced with eight 16-mm bars and 8-mm tie bars every 100 mm, loaded to less than one-third of its bearing strength. This is a powerful member, apt for a building of some size. Its bending strength would be around 200 kN·m and it would be able to resist shear forces to approximately 190 kN. Consequently, with such a section the column would be regarded to be "short" if its length were less than:

$$h = \frac{2{\cdot}M_p}{V} = \frac{2{\cdot}200\ \text{kN}{\cdot}\text{m}}{190\ \text{kN}} = 2.1\text{m}$$

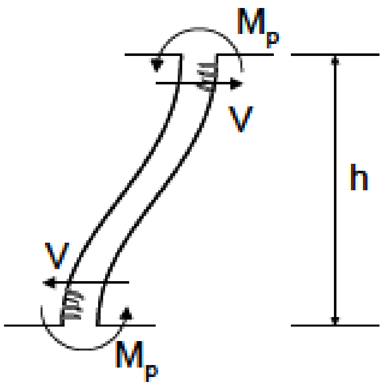
Note that this and the preceding criterion are complementary. Here the issue is not a short column in a line of normal columns, but one that is short enough to exhibit shear failure. In fact, all the short columns (and the vast majority of columns, short or otherwise) observed to fail in Lorca failed in shear (Photograph 7-3).

Shear failure may not necessarily follow the standard pattern. If masonry infills are present, the construction joint may slide first (Photograph 7-4), providing further proof of the stiffness and strength of such elements.

7.2. Horizontal bracing

As in masonry buildings, in their RC counterparts, horizontal bracing of all the joints is essential because otherwise the differential displacement between column heads on each storey would generate unforeseen forces (bending on the vertical axes and torque) on the beams.

One-way concrete floor slabs were the type most commonly observed at Lorca, a scheme devised after the Spanish Civil War to meet reconstruction needs in a context of World War II-related scarcity of steel and the lack of skilled labour. Together, these circumstances inevitably led to designs based on joist and pan form floor slabs, a procedure that would be irreplaceable for many years.

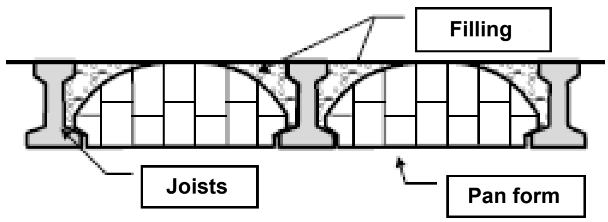


▲ Figure 7-4

▲ Photograph 7-3

▲ Photograph 7-4

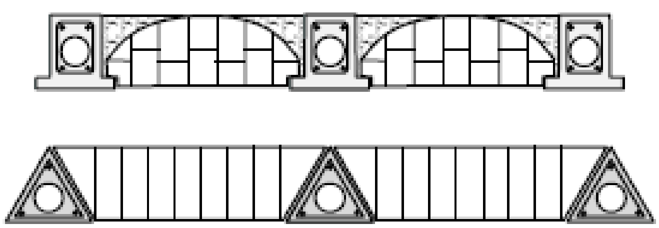
The earliest versions were based on the use of reinforced concrete joists that rested directly on the beams or masonry bearing walls, at times very precariously secured to the structure itself (beams were seldom built around the perimeter). The inter-joist filling consisted either of hollow clay or mortar blocks, pan forms, or rows of thin hollow brick, while the rest of the space was filled and levelled with a variety of materials (Figure 7-5).



▲ Figure 7-5

Solutions designed for power poles, based on centrifuged elements, were also used with some success. Some such solutions are depicted in Figure 7-6.

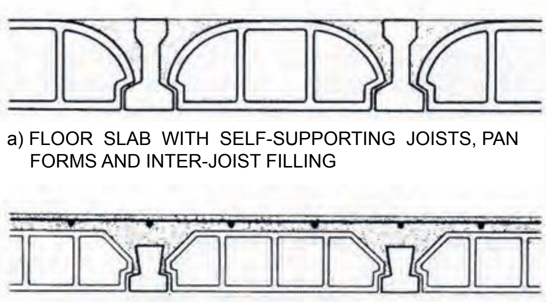
Despite the paucity of resources, in some cases considerable efforts were made to publicise structural engineering procedures and recommendations to facilitate the work of architects authoring building designs. One of the brochures circulated at the time, which advocated the use of design based on plastic behaviour, a fairly progressive approach in 1942 when it was published, is shown in Figure 7-7.



▲ Figure 7-6

▲ Figure 7-7

While these slabs obviously failed to comply with such basic principles as monolithic design and attachment to the structure (fundamentals that substantiate the definition of floor slabs as diaphragms characterised by in-plane shear stiffness and solidarity with the structure that today constitutes the basis of building design), in fact such slabs formed part of thousands of buildings erected in Spain.



a) FLOOR SLAB WITH SELF-SUPPORTING JOISTS, PAN FORMS AND INTER-JOIST FILLING

b) FLOOR SLAB WITH SEMI-SELF-SUPPORTING JOISTS, PAN FORMS AND INTER-JOIST FILLING

▲ Figure 7-8

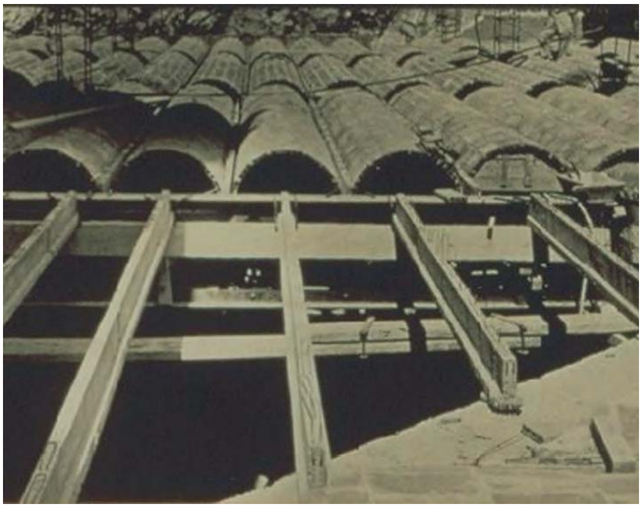
Two innovations appeared at mid-century that quickly came into widespread use: "semi-joists" and prestressing.

"*Semi-joists*" or "*semi-self-supporting joists*" are no more than low-strength joists that call for additional cast-in-place concrete not only to support total design loads, but in most cases even the loads induced by the weight of the slab itself. They consequently had to be braced during construction. Figure 7-8, taken from Calavera [21], illustrates the difference.

The advent of prestressed reinforcement, developed in large industrial plants, provided for the inexpensive, mass manufacture of joists and semi-self-supporting joists that capitalised on the benefits of prestressed sections. Figure 7-5, which dates from 1945, depicts one of the earliest applications.

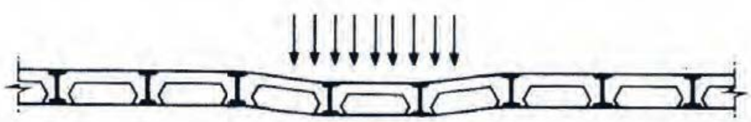
Between the nineteen fifties and early seventies, precasting manufacturers engaged in an unfortunate price war. Cost-cutting was based on using less material, with a concomitant reduction of thickness and the deployment of pan forms designed to minimise the amount of cast-in-place concrete and avoid the need for perimetric beams. In the example in Figure 7-6 the façade rested directly on the edges of joists and pan forms.

It was not until 1973 that the reinforced concrete code first required the use of a concrete topping such as shown in Figure 7-8 b).



▲ Photograph 7-5

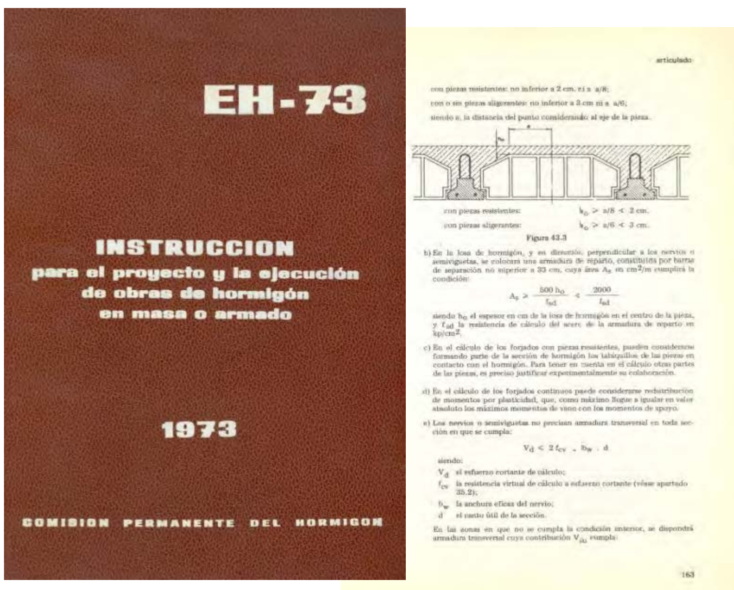
▲ Photograph 7-6



▲ Figure 7-9

Despite its obvious importance in seismic zones, that was not the reason behind the provision, which merely aimed to prevent the concentration of loads induced essentially by partition walls that were built on the top side of the floor slab or that unexpectedly rested on the partitions on the lower storeys (if no joint was built between the top of the partition and the bottom of the slab). Such loads had been causing damage to floors and pan forms due to differential movement in the loaded and the adjacent joists ("piano key" deformation, Figure 7-9, taken from Calavera).

The new code also required the use of perimetric beams, defined the need to secure joists to the structure and specified minimum depth values. Its appearance marked a turning point in floor slab quality.



▲ Figure 7-10

▲ Photograph 7-7

▲ Photograph 7-8

While many of the buildings inspected at Lorca had no slab topping, the damage observed was not as severe as might have been expected, at least in collective housing. No slab deformation or damage attributable solely and directly to non-monolithic construction was in fact observed. The possible symptoms were confined to the widening of joints between floor tiles, whose origin in many cases may very likely have been prior and unrelated to the earthquake. In fact, in some cases inspection holes in the floors revealed that the joists and fill seemed to have pulled apart (Photograph 7-7) for reasons not attributable to the quake.

Flooring was found to have been deformed in some single family dwellings, with gaps between joists and structure or even out-of-phase inter-joist displacement. In most cases, the floors in question were made of timber in which the joists, mere logs, lacked any strong connector at the bearing. In concrete slabs, the same problem was only observed where the structural configuration was particularly deficient and formally identical to the timber flooring, probably due to a lack of professional advice (Photograph 7-8).

The whole slab was observed to have slid across the beams in some cases, but only where the beams were made of steel and unconnected to the slab (Photograph 7-9).

7.3. Structural system

7.3.1. Lack of structure

Although the heading to this item may appear to be overly dramatic, it reflects a situation frequently observed in Lorca. Structure was not entirely lacking in these buildings, of course: what was absent was any horizontal structure. As discussed in preceding chapters, this was observed in buildings where the floor slabs were laid over but not connected to the flat portal frame structures (Photograph 7-10).

Other cases were found in which the structure was only effective vertically, due the absence of any real connection between the bars (beams that simply rested on columns, as in Photograph 7-8 above).

The same problem appeared in a structural scheme found fairly frequently in the city, characterised by the use of steel beams and concrete columns (Photograph 7-11). In such cases, the beam-column joint cannot guarantee the minimum stiffness required and the assembly fails to constitute a true portal frame. In response to lateral loads, the structure behaves like a series of cantilevered columns. The actual configuration was even less favourable, for column strength was nullified around the joint, where the steel shape hindered proper concrete casting, interrupting the continuity of stress flow across the member. A more serious consequence, however, was that it prevented concrete confinement and the application of tie bars to hold the reinforcing steel in place.

▲ Photograph 7-9

▲ Photograph 7-10

▲ Photograph 7-11

▲ Photograph 7-12

Moreover, in a few cases the slabs were not connected to the beams in any manner whatsoever.

Lastly, mention should be made of the many additions to the original buildings, in which new storeys were built over the original roof with no identifiable structure whatsoever. These additions consisted of mere floor slabs lying on upward extensions of the façades, in turn no more than brick walls with no bearing capacity.

7.3.2. Inappropriate distribution of mass

One of the most surprising observations from the very outset was the large number of tanks, some very sizeable (Photograph 7-12), housed in conventional buildings. Their existence was the outcome of legal provisions that call for a minimum guaranteed volume of water in certain types of public premises. While these deposits were highly unlikely to have ever been filled to capacity, probably because that would have induced the collapse of the underlying floor slab, they constituted an inadmissible hazard in a seismic zone.

Another equally astonishing finding was the number of fuel tanks positioned on roofs, systematically in some quarters (Photograph 7-13). Inasmuch as seismic acceleration on the roof was observed to be much greater than at ground level and usually on the order of 1 g, the magnitude of the actions induced by these tanks in the structure can be readily deduced (bearing in mind that such actions are the resultants of the self-weight not only of the deposit, but also of its base).

▲ Photograph 7-13

Moreover, the possible leaks due to anchorage or pipe failure associated with these deposits constitute an additional hazard.

Yet another factor worthy of note was the effect of the very thick flooring on floor slabs due to the routine practice of re-flooring without removing the previous layer (Photograph 7-14). Curiously, one of the most common post-quake repair measures consisted of laying new floors over the existing material.

▲ Photograph 7-14

7.3.3. Failure contrary to strength hierarchy

Seismic design has been based for years on structural ductility, i.e., on the capacity of a structure to form deformable plastic mechanisms able to bear earthquake-induced displacement for as many cycles as induced by the seismic action.

Ensuring such ductility is no easy matter. It calls for a prior understanding of structures not presently at hand, at least not for all types of structures.

Ductility can only be guaranteed in certain structural types (portal frame or shear wall systems, essentially), built with uniform materials (concrete or steel, but not concrete and steel) using very specific detailing (such as steel ratios, anchorages and stirrups in concrete structures).

In addition to the foregoing, these structural types must be designed to very specific guidelines that strictly ensure strength hierarchy: joints must be stronger than the connecting bars, columns must be stronger than beams, and any element must have greater shear than bending strength...

The actual situation observed at Lorca was exactly the opposite.

Not a single case was found to comply with that hierarchy. Beams were not observed to form hinges (which should be the first effect of seismic forces), while shear failure, which should not have occurred under any circumstances, was detected in dozens of columns. Nor, of course, should joints have failed like the one in Photograph 7-15. Note, moreover, the lack of tie bars and the absence of any joint with the adjacent building.

▲ Photograph 7-15

A comparison between that and the following picture, Photograph 7-16, taken in a low seismicity area in Mexico, is revealing. Note the ratio of tie bars at the connections.

In short, many of the city's structures were not compliant with the classical "*weak beam / strong column*" principle, and their geometry (Photograph 7-17) suggested the exact opposite.

7.3.4. Inappropriate use of shear walls

Shear wall-based structural systems constitute one of the classical solutions to counter horizontal, and more specifically seismic, actions. Some designers deem them to be the sole effective solution not only to prevent building collapse, but also to lessen the damage produced by an earthquake: only shear walls can afford sufficient stiffness to reduce differential inter-floor drift, which is the direct cause of damage to partitioning and façades.

Shear walls are, in short, a valid solution for dealing with earthquakes. Nonetheless, building a shear-walled structure entails much more than erecting the wall (or several, where a building as a whole is concerned). The elements on each storey that receive the seismic forces at each point where they are exerted and transmit them to the wall are as important as the shear wall itself. The rest of the structure must be designed around the "*hard point*" established by the respective shear wall at each level and in each plan direction. A conventional slab cannot be relied upon to accommodate diaphragm actions.

▲ Photograph 7-16

▲ Photograph 7-17

▲ Photograph 7-18

▲ Photograph 7-19

▲ Photograph 7-20

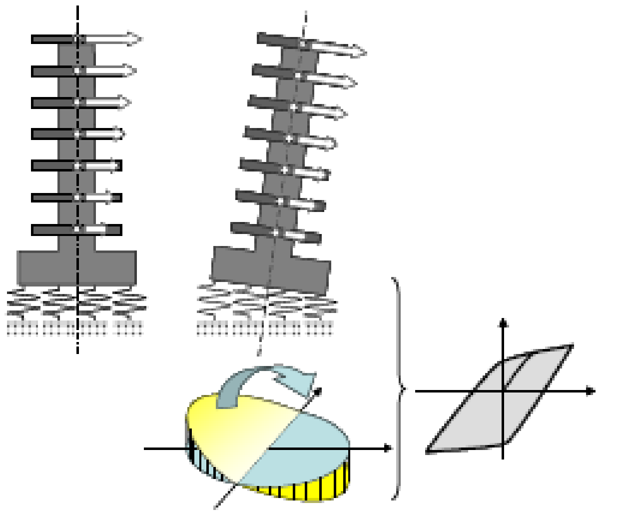
Since such elementary precautions were not always adopted at Lorca, in some cases the shear wall literally cut through the floor slab. Note the 45º angle cracks in the slab alongside the shear wall in Photograph 7-18, and the crushed waffles along its entire length. A continuous beam would obviously have been needed to extend the bracing plane.

Another factor that governs shear wall behaviour is the reinforcement used. Meeting the basic requirement of earthquake-resistant design, i.e., preventing brittle failure, leads to extraordinarily costly reinforcement, with steel ratios, particularly for horizontal forces, much larger than observed at Lorca (Photograph 7-19). The shear failure of many such walls could be attributed to these reinforcement shortfalls (Photograph 7-20).

Lastly, building shear walls over very powerful foundations is an obvious necessity if they are to furnish the entire reaction to horizontal loads. In high-rise buildings, designing such foundations is rendered more complex by the need to verify not only strength but stiffness.

If the soil is not stiff enough (a parameter difficult to quantify, for soil has a clearly non-linear behaviour and consequently stiffness depends on the amount of stress), the foundation rotates and with it the shear wall, which would move like a stiff solid (Figure 7-11). The ultimate consequence is a loss of effective stiffness.

In short, while shear walls can effectively counter earthquakes, they are neither a simple nor an inexpensive solution.



▲ Figure 7-11

7.4. Inter-building structural joints

The problem of joints, at least as observed in Lorca, is that there were none.

Standard practice seems to have been to cast the concrete for new directly against the adjacent buildings, using the latter's side wall as formwork. The sole interface between buildings usually consisted of a few polystyrene plates.

Where floor slab elevations did not concur, in some cases the more recent building literally penetrated the perimeter of the older one, up to the outer leaf. In such situations the slabs on the newer building were found to embrace the columns on the existing one (see Photograph 7-21, in which the slab in the building on the left can be seen to surround the columns in the building from which the photo was taken on the right).

That practice naturally induced very severe damage in the structures and the very robust outer masonry walls, whose collapse constituted a serious hazard (Photograph 7-22).

7.5. Soft storeys

The following is an analysis of a number of ideas that are not always clearly stated.

Initially the term "*flexible*" or simply "*soft*" was used to mean a (normally ground) storey that was much less stiff than the others in the building. The term not only had no negative connotations, but in fact sometimes denoted beneficial situations. A flexible ground storey raises a building's period and lowers its equivalent loads. It is hardly surprising, then, that the apparent advantages led to the deployment of such solutions.

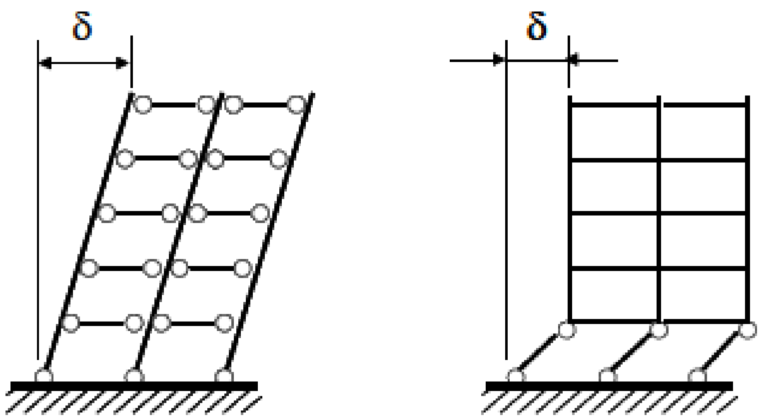
▲ Photograph 7-21

▲ Photograph 7-22

Unfortunately, however, the advantages were soon shown to be the misleading results of shortcomings in equivalent load-based structural engineering procedures, which did not fully represent seismic action. While the loads on the storey were smaller, earthquake shaking induced greater displacement and, more importantly, it did so in a way that concentrated deformation in only a few points, namely the top and bottom of the columns on the soft storey. If building drift during an earthquake is the result of deformation in similar proportions in all its storeys, in an ideal plastic mechanism, the plasticised sections would need to rotate less than if drift affects the soft storey only (Figure 7-12).

In the former case hinge rotation would be $\theta = \delta/(5h)$ and in the second, $\theta = \delta/h$, or five times greater. Moreover, the plastic mechanism associated with the soft storey system entails hinge formation in the sections of the structure where it is most difficult to attain ductility (simply because axial force is greatest in such sections). By contrast, in an ideal mechanism, the hinges would concentrate at the ends of the beams where it is easier to obtain large-scale, stable plastic rotation due to the absence of any significant axial force. In addition, hinge failure at the end of a beam is always less hazardous than failure at the top or bottom of a column, because the former induces only local collapse, whereas the latter would in all likelihood lead to the collapse of the entire building.

Naturally enough, none of the above reasons constitutes a definitive argument. Some authors contend that the aforementioned risks can be controlled affordably and even cost-effectively. They reason that repairing a building that behaved correctly during an earthquake is never feasible, whereas repairing a building with a soft storey probably would be.



▲ Figure 7-12

In other words, if damage is distributed globally across a building, its recovery would be much too costly, but if it is confined to the ground storey columns, recovery is inexpensive, particularly if the storey had no masonry walls that would need repair (such as the typical buildings in San Fernando quarter).

Nonetheless, the real problem posed by soft storeys is that they are seldom designed as such. As a result, insufficient attention is paid to the area where energy is dissipated, i.e., the top and bottom of columns. As noted from the outset, the standard, equivalent load-based calculation method used in conjunction with linear models is oblivious to all these considerations. Consequently, soft storeys have often led to the collapse of entire buildings; hence the use of the term "*weak*" to describe them which, from a purely conceptual standpoint, would not be justified.

Such situations were often observed at Lorca, where the storeys in question would indeed have to be designated as "*weak*" rather than "*soft*" because they constituted a sure hazard in the event of an earthquake. They were normally the result of architectural considerations and urban ordinances requiring storefronts to have greater clear heights. The outcome was ground storeys with insufficiently reinforced columns that were taller than the ones in the rest of the building, and often with provisional, fairly weak, hastily built brick or even glazed enclosures unconnected to the structure (Photograph 7-23).

Likewise as noted, unengaged ground storeys that exemplified this typology were also frequently observed (in San Fernando quarter, for instance). The formation of this type of mechanisms must also be mentioned in connection with the failure of ground storey masonry walls.

The clearest consequence of these situations in Lorca was the frequency with which pounding was observed on the first storey (Photograph 7-24).

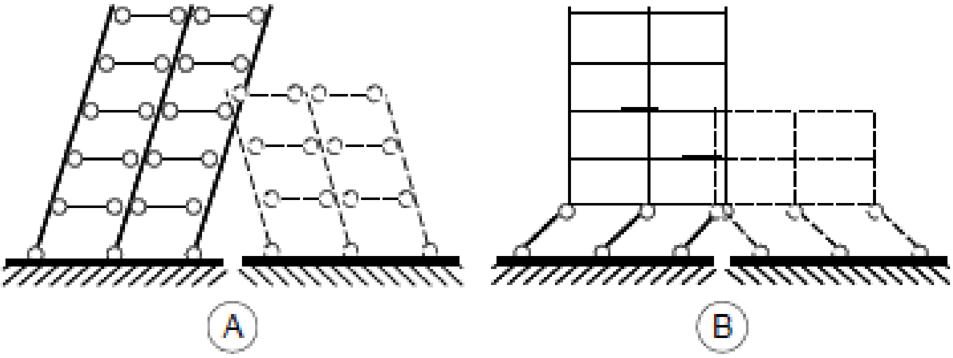
Logically, whereas in adjacent buildings that move in keeping with the usual and desirable pattern, pounding would take place on the highest storeys (Figure 7-13 A), buildings with soft storeys would collide equally along their entire height (Figure 7-13 B).

Outside of pounding, no major damage was found on the lower storeys of buildings that could be solely and directly attributed to weak storey behaviour. Some kinds of damage, such as reinforcement buckling at the top or bottom of some columns in San Fernando quarter buildings (Photograph 7-25), was clearly caused by a need for greater ductility than extant in the section. Nonetheless, failure could actually be more clearly attributed to inadequate reinforcement and careless workmanship (as shown in the scant concrete cover. Note the rust on the reinforcement bar, denoting cover loss prior to the quake.)

The explanation for the scant damage observed at Lorca lies in all likelihood in the very special characteristics of its earthquake. Soft storeys are very hazardous in longer-lasting quakes with a more distant epicentre and a spectrum less concentrated in the low range of periods. As noted at the beginning of this item, soft storeys raise a building's period, preventing it from reaching the areas of high amplification characteristic of this type of near quakes that impact stiff buildings more severely.

The short duration of the earthquake was another essential factor. The stability of the hinges formed is as important as their ductility: i.e., they should be able to withstand a sufficient number of alternating shocks while remaining plastic. At Lorca, given that the earthquake consisted of practically a single, very violent but short-lived shock, the sections did not have to bear the reiterated stress of a conventional quake.

▲ Photograph 7-23



▲ Figure 7-13

7.6. Staircases

Code NCSE-02 provides the following in connection with staircases.

...*"General evacuation structures, particularly vertical communication cores such as staircases, must be built to additional strength and ductility to ensure their usability even in the case of severe earthquakes"*

▲ Photograph 7-24

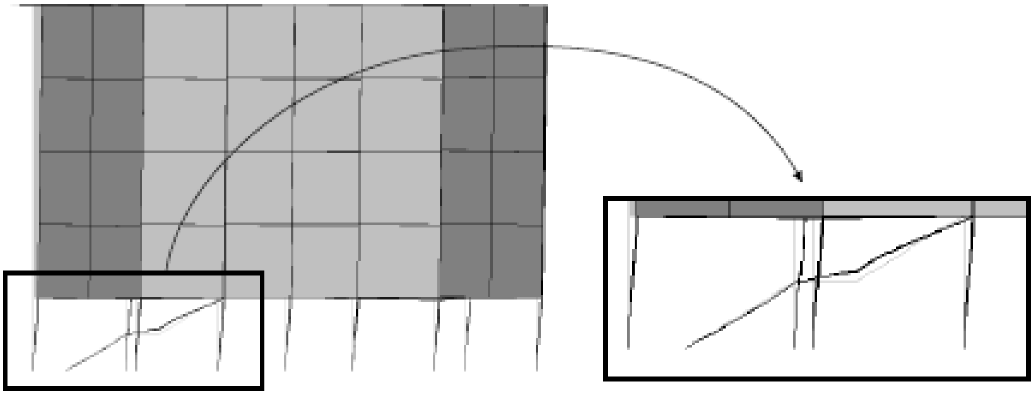
▲ Photograph 7-25

▲ Photograph 7-26

Many standard configurations obviously failed to comply with this idea, for they entrusted their stability to the support afforded by the stairwell masonry which, as discussed below, is one of the first elements to fail in earthquakes, leaving the stair slab in precarious condition (Photograph 7-26).

Staircases also exhibit the dual behaviour described in connection with masonry walls. On the one hand, as passive elements, their constituent slabs are stressed by building displacement, which induces forces for which they are not normally designed (Figure 7-14, often causing their collapse (Photograph 7-27). The consequences of staircase failure are severe and so obvious that no need is felt to insist on the importance of prevention.

Staircases are also extremely important as active elements that modify structural stiffness and strength. The discussion of the results on numerical modelling for one of the buildings in San Fernando quarter in item 6.2, "Vertical irregularities" showed that entering the staircase slab in the calculations modified not only the modal values but also their distribution, due to the stiffness introduced by the slab.



▲ Figure 7-14

That notwithstanding, the actual effect of staircases is greater than denoted by those numbers, because it necessarily includes the effect of the stairwell enclosures, which interact with the slab, forming a stiffer and more powerful assembly than resulting from the mere sum of their respective contributions. This factor could not be reflected in the very simple model used in the aforementioned item, however.

The entire discussion on masonry walls set out in the preceding chapters is naturally applicable to staircases. For instance, a stairwell located asymmetrically on the floor plan may condition the overall response of the structure, inasmuch as it constitutes an obvious irregularity not normally considered in the structural analysis of the building.

Lastly, the greatest impact on structural strength occurs when staircase slabs are attached to or simply abut against a column, for that induces a short column mechanism with the foreseeable consequences (Photograph 7-28).

The greatest uncertainty arises around the ways stair slabs are built, however. The introduction to this item described the hazard generated by the routine practice whereby staircases are built to rest against sometimes very poorly constructed masonry walls.

More surprising, given both the elementary nature of the problem and the frequency with which it was observed, was failure due to the thrust of the tensile reinforcement in the slab (Figure 7-15). The textbook principle calling for securing the reinforcement to the compression head often went unheeded, causing severe damage along the inter-flight or flight-landing angle (Photograph 7-29).

▲ Photograph 7-27

▲ Photograph 7-28

▲ Photograph 7-29

Photograph 7-30 shows the reinforcement arrangement in many staircase slabs, which is unable to ensure even minimum safety.

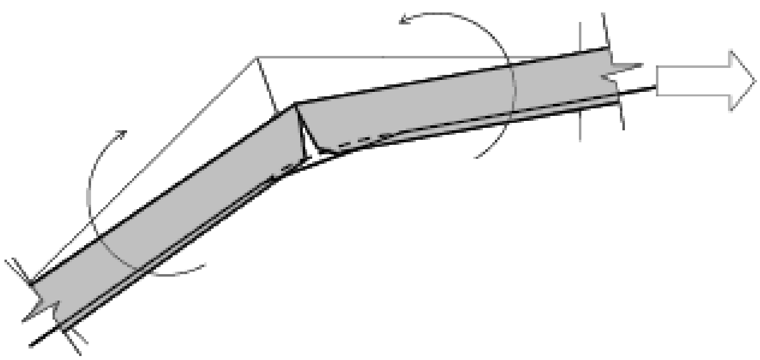
7.7. Inappropriate construction procedures

This item describes some of the features of the reinforcement details or workmanship observed at Lorca that may have conditioned structural behaviour.

The effects of these features are deemed to be fairly modest. If a building has no system able to effectively resist horizontal actions, as was often the case, it obviously makes little sense to labour the issue of concrete confinement in columns...

7.7.1. Lack of stirrups

Photograph 7-25 may help illustrate how demanding seismic loads can be. As noted earlier, ideal behaviour in earthquakes entails the formation of stable hinges. That implies that for part of the time the reinforcement on one of the sides of the member is elongated far beyond its yield point (Figure 7-16 A), a condition that it would normally be able to readily withstand (although this poses a problem for the compressed concrete, particularly if the member involved is a column). One instant later the direction of the action is inverted: the tensile-stressed bars (now longer than they were initially) must shorten not to their original (point B in the figure), but to a much shorter length (point C).

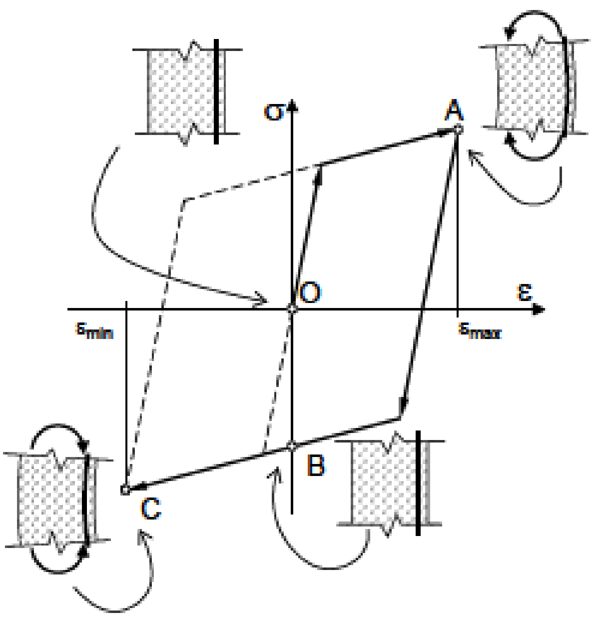


▲ Figure 7-15

In short, the reinforcement is subjected to strain leading to plasticity of a much greater amplitude.

It is hard to imagine that a circular bar 12 mm in diameter and more than 20 cm long can be compressed to much beyond its yield strength without buckling, but that was exactly what was observed in some lengths of columns at Lorca, where the stirrups were spaced at more than 20 cm and the concrete cover had disappeared years earlier.

▲ Photograph 7-30



▲ Figure 7-16

▲ Photograph 7-31

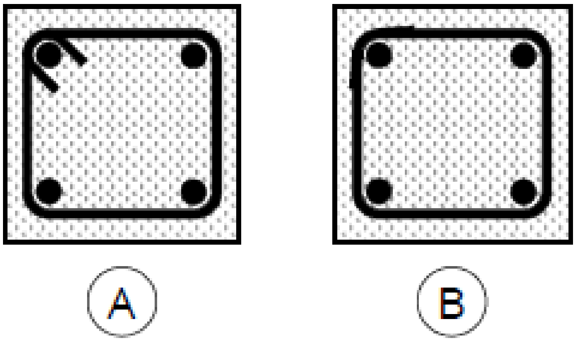
Transverse reinforcement also conditioned concrete confinement and consequently its ultimate strain under compression, an essential issue in compressed sections. Nonetheless, this type of failure could be clearly identified on only a few occasions in Lorca (Photograph 7-31).

By way of reference, code NCSE-02 provides that the maximum distance between tie bars should not exceed one-third of the depth or 10 cm if the main bar diameter is under 16 mm, or 15 cm otherwise.

Much wider spacing was systematically measured at Lorca, reaching an extreme at nodes, where tie bars were apparently not deemed necessary (Photograph 7-32).

7.7.2. Inappropriate stirrup anchorage

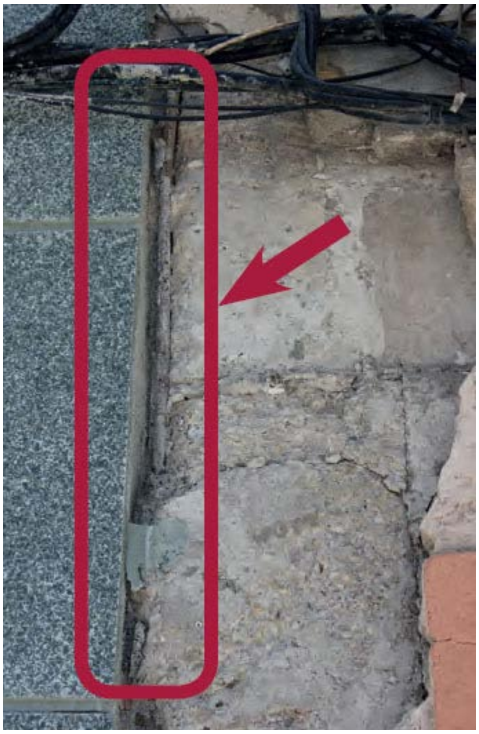
In seismic zones, tie bars and stirrups must be secured with hooks 10 diameters long, bent at a 135º angle (Figure 7-17 A) instead of by conventional lap-splicing (Figure 7-17 B).



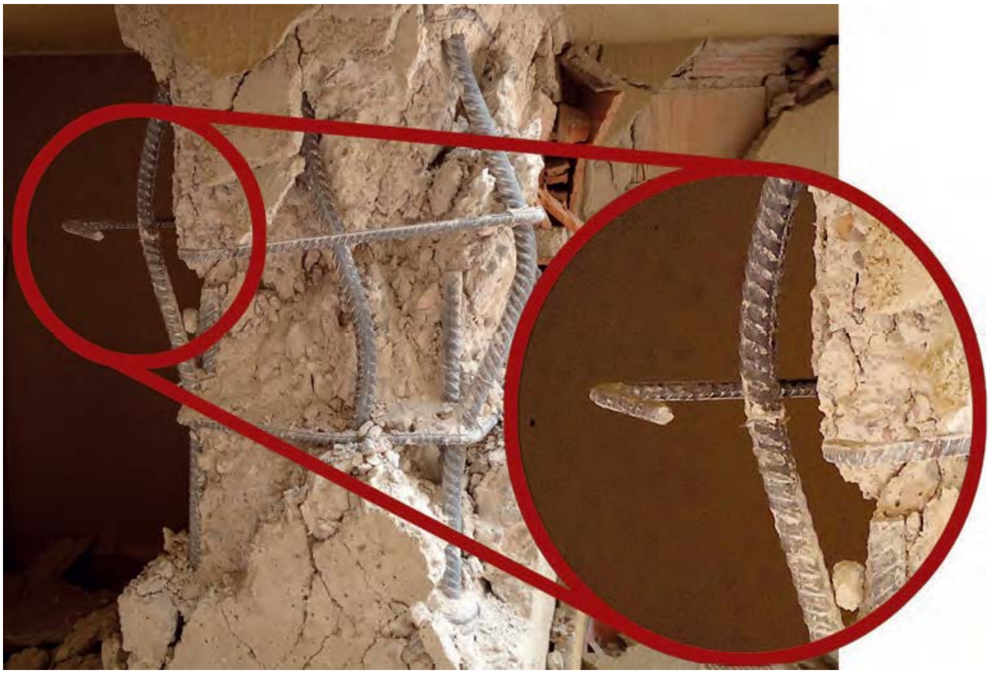
▲ Figure 7-17

Although this is an obviously more complex solution, it is the sole way to ensure that the tie bars continue to serve their purpose when they are most needed, i.e., in the absence of the concrete cover (Photograph 7-33, which also shows that the bar was spliced on the side under greatest stress).

All the codes presently in place stress the importance of this question. In some cases the use of continuous spiralled reinforcement is recommended to minimise the risk of anchorage detachment. Others prohibit that arrangement, however, because failure in any section would entail a loss of capacity in all the reinforcement as a whole, as opposed to the local failure that would occur if an individual tie bar fails.



▲ Photograph 7-32



▲ Photograph 7-33

Post-quake action

8

8. Post-quake action

This chapter addresses a particularly complex issue, the repair and strengthening of buildings affected by the earthquake.

A good deal of thought went into the inclusion or otherwise of this chapter, especially because the authors are aware that it will very likely give rise to controversial interpretations. INTEMAC's survey and assessment of building damage in Lorca may obviously be viewed with certain mistrust or even dismissed as partial, despite the efforts made to objectively quantify and record absolutely all the damage observed in buildings, both by INTEMAC's experts and by the technicians hired by the owners. A purely technical criterion is, moreover, not necessarily the only basis for the most important decisions (such as building demolition), which may be made on social, economic, or zoning grounds. Lastly, the criticism (which aims to be constructive) levelled against some of the strengthening solutions applied in Lorca may be misinterpreted, especially where no generally valid alternative is furnished. Nonetheless, the text would have been incomplete without at least a minimum mention of these absolutely essential matters.

8.1. Damage apprasaisal

This was the first and in all likelihood the most important action after the quake.

Several assessments, successively more detailed, are always necessary in such cases. The first is usually conducted by teams of technicians coordinated by the competent authority (normally Civil Defence). It is based on a visual survey and aims to provide a preliminary definition of the hazards posed by the damage to substantiate any emergency measures (such as the evacuation or possible restricted access or use of buildings).

Further to code NCSE-02, subsequent assessments must be conducted on each building by a qualified professional hired by its owners or the technician entrusted with its maintenance. Such assessments are the object of subsequent reports on the effects of the earthquake on the building and the type of measures to be taken.

All these activities should be performed by technicians with structural experience, able to identify the damage, its origin and implications.

While the appraisal of damage to architectural elements or building services does not normally pose any serious problem, the assessment of structural damage is much more complex.

The following is a discussion of a few ideas in this regard.

1. *Many of those reports were even sparser. The sole grounds for demolition was that the building had undergone severe seismic action. No description of damage was regarded necessary.*

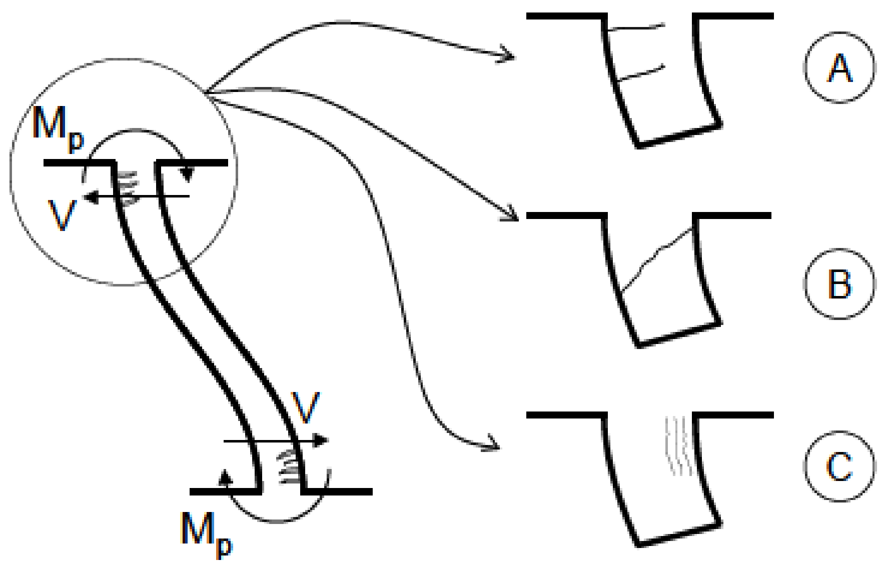
8.1.1. Identification

Some of the reports issued justified building demolition on the grounds of the mere existence of cracks[1] in the structure, for which no description (no indication of location, shape or width) or assessment of their impact on the strength of the member was given. Indeed, a mere read-through of these reports revealed that their authors clearly identified cracking as damage.

While defining the term damage lies outside the scope of this text, the error underlying such a conceit merits some comment. In most cases cracking is no more than the result of satisfactory behaviour (i.e., compliant with the strength model laid down in the legislation) of any reinforced concrete element. Damage is the presence of uncontrolled cracking that lessens bearing capacity or durability.

The column in Figure 8-1 affords a specific example. It may exhibit the bending cracks illustrated in detail A, which are represented on one side only for greater clarity but which, given the alternating nature of conventional seismic action, would actually arise on the two opposite sides of the section (although in Lorca, the shock was so impulsive that cracking was often observed on only one side).

In the ATC manual [5], to cite one reference, irrespective of their size, these cracks are not even regarded as indicative of hazard. Similarly, codes FEMA 274 [7] and 306 [8] (whose scope is nonetheless confined to shear walls) classify them under the heading "*insignificant severity*" when they are less than 3/16 of an inch (nearly 5 mm) wide.



▲ Figure 8-1

Although in Figure 8-1 these cracks are drawn at the top and bottom of the column in keeping with structural logic, they were also observed at mid-height (Photograph 8-1), where the moment is theoretically nil. In the column shown in the photograph, they were induced by the failure of the adjacent masonry infills (in other cases the origin may lie in staircase slabs or basement ramps).

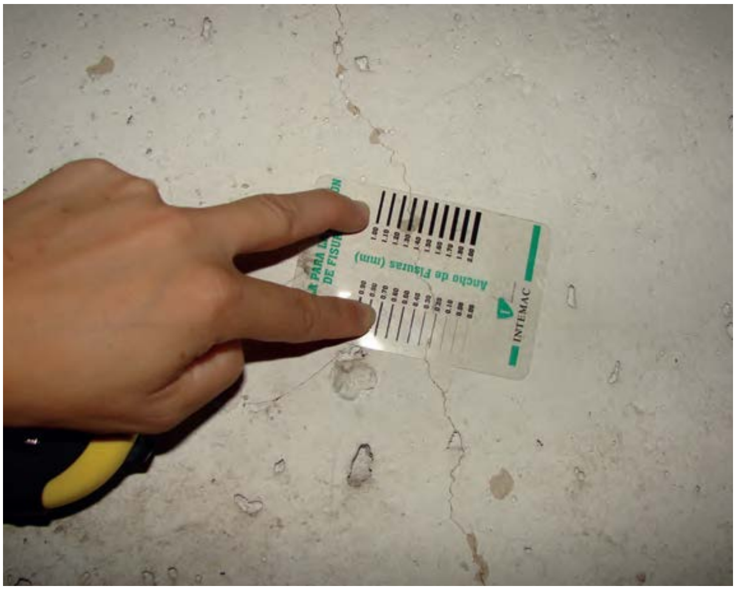
The FEMA codes also regard shear cracks (type B in Figure 8-1) as "*insignificant*" when they are less than one-eighth of an inch (slightly over 3 mm) thick, The ATC manual only considers such cracking to be hazardous when the cracks are large, without defining that term.

The reports issued by INTEMAC recommended much more prudent damage thresholds than those cited.

The most hazardous type of cracking, further to the literature normally cited, are as shown in C in the figure, i.e., cracks that run parallel to the direction of the compressive stress, for they denote fatigue in the compressed block. They should not be confounded with the more usual cracking associated with cover detachment, often caused by the existence of prior cracks. INTEMAC identified no such cracks. In fact, bending could be singled out as the sole origin of damage in fairly few cases.

The foregoing infers that for cracks to truly entail a hazard, they must be so wide that no sophisticated measuring devices are required. INTEMAC systematically adopted the most elementary approach, a plastic template (Photograph 8-2).

▲ **Photograph 8-1**



▲ **Photograph 8-2**

▲ Photograph 8-3

▲ Photograph 8-4

That great precision is unnecessary in crack measurement does not excuse the failure to log readings, of course. All cracks associated with an earthquake must be duly recorded.

When a section is affected to the point of complete breakage, damage unquestionably exists. The respective records must specify the characteristics not only of the damage (including position on the structure, crack geometry, impact on reinforcement and so on), but also of the surrounds, for these data are often essential to determining the origin. Photograph 8-2 contains a clear example of column failure due to the forces induced by adjacent masonry infills (example of a captive column).

8.1.2. Origin

The preceding item began with a discussion of the identification of cracks as damage observed in some reports and an explanation of why such a conclusion is erroneous.

In fact, when joint surveys were conducted with the authors of those reports, the origin of many of the cracks was shown to have been prior to the quake and completely unrelated not only to it but even to any external action. Some cracks, such as the one labelled A in Photograph 8-4, which concurred with the tie bars in the columns, had appeared due to plastic settlement during construction. Others proved to be mere construction joints, such as B in the photo. Yet other cracks arose early in or throughout the life of the building (shrinkage cracks in beams and reinforcement corrosion cracks, respectively).

Nonetheless, as noted in item 2.2, "Structural damage", the origin of the damage observed was most frequently found to be prior deterioration of the materials.

8.1.3. Implications

Damage may be so regarded either because it lowers a structure's bearing capacity or because it has an adverse effect on its durability. While the latter is more common, it does not entail the immediate hazard associated with the former.

The width thresholds for uncontrolled cracking vary significantly from one type of damage to the other. While, as noted above, cracks under a few millimetres wide are not regarded to be detrimental to strength, allowable cracking in terms of durability is measured in tenths of a millimetre. More specifically, the Spanish structural concrete code [13] defines the threshold at three-tenths of a millimetre in moderately aggressive environments.

8.2. Demolition versus rehabilitation

The first decision stemming from damage surveying and appraisal is whether to demolish or restore a building.

Rehabilitation is meant here to mean both simple repairs and possible strengthening, although a distinction is drawn between them in a later item.

▲ Photograph 8-5

This is obviously a complex decision in light of the wide range of (technical, economic, legal) criteria to be considered, the diversity of agents involved (owners, government, insurers...), the plurality of many of these agents (flat owners' associations are the readiest example, but the various levels of government may also apply contradictory criteria) and the subjectivity that may inform some arguments, especially as regards the peace of mind of the dwellers of rehabilitated buildings. As that range of general criteria falls amply outside the scope of this book, the following discussion focuses on technical considerations arising around the Lorca findings and an analysis of the existing legislation and related references.

2. *INTEMAC's technicians are not overly impressed by the dramatic pathologies observed, which they have seen in previous and equally appalling events.*

8.2.1. Definition of ruin

Irrespective of the legal definitions (technical, economic or urban ruin), the present report contends that the vast majority of buildings can be rehabilitated.

Demolition was seldom recommended on the grounds of the surveys conducted. In all cases where it was, the condition of the buildings involved entailed a clear hazard for the survey or consolidation crews. Such a perception was not, of course, free of subjectivity, which was nonetheless clearly justified. A professional who would not feel safe in a building cannot recommend that others go into it[2]. The buildings involved (some uninhabited) had very low quality masonry walls that were severely damaged prior to the earthquake.

Barring those scantly representative cases, however, the authors' believe that with today's resources and knowledge, nearly any damage is reparable.

▲ **Photograph 8-6**

8.2.2. Severity or extent of damage

One of the paradoxes observed at Lorca was that a structural problem that usually causes the most severe damage in seismic zones, namely the existence of short columns, had a beneficial effect on building reparability.

Indeed, these members acted as unplanned damage concentrators (see Benavent-Climent for an especially enlightening description of the term), limiting the spread of the harm to the rest of the building to a certain extent. The same may be said of the buildings in San Fernando quarter (Photograph 8-6), repeatedly referred to throughout this text. Both their overall design and their detailing were particularly unsuited to a seismic zone and they have been cited as examples of the most severe problems that buildings may pose (weak storey, unsatisfactory reinforcement, steel corrosion and loss of cover, to name a few).

Paradoxically, however, repair of these buildings was particularly simple because all the damage, irrespective of severity, was concentrated in a few specific members (columns) that were, moreover, readily accessible (from the ground with no need for any scaffolding whatsoever), mostly unengaged (avoiding the need to demolish masonry walls) and cleanly (without rubble) and conventionally reparable.

While this should not by any means be interpreted to be a justification of the serious errors detected, such facts cannot be omitted from an account of this nature.

8.3. Repair versus strengthening

Photograph 8-7 depicts a staircase before (the same photograph is reproduced in item 7.6, "Staircases") and after it was repaired.

The repair solution deployed was the simplest possible: to rebuild the original structure. The problem is that the original structure was clearly flawed, as the earthquake itself showed.

Such approaches are only explicable economically: insurers cover the cost of repairing damage, but not of strengthening or replacing previously flawed construction.

Many other examples could be given. The most obvious may be the numerous buildings lacking any structure able to resist horizontal action, referred to in item 7.3.1, "Lack of structure", which only remained standing because their non-structural elements prevented collapse.

Does it make sense to limit restoration to the mere reconstruction of these elements, without fitting the building with a specific bracing system?

Such questions have no simple answers, among others because each possible answer poses further doubts. If the Lorca buildings are not simply repaired but strengthened, why not the buildings at Totana, Murcia or Granada? Who should defray the costs?

Lloret and Regalado [58] discuss these issues in greater depth.

The opinion defended here is that the foregoing is in order only in buildings such as schools and hospitals.

▲ Photograph 8-7

In those cases the repair-strengthening dilemma should not only not even be posed, but, in light of the findings at Lorca, the authors would even dare to suggest that the Government should conduct a specific campaign to survey all the buildings of this type in the areas of Spain at greatest seismic risk.

8.4. Repair criteria

These criteria depend on the type and degree of damage in need of repair.

Limiting the discussion to concrete structures, the most frequent actions refer to the damage described below, in ascending order of severity.

8.4.1. Cracks

The narrowest cracks, less than one- to two-tenths of a millimetre wide, require no more than purely cosmetic, normally surface repair. Any number of paint-like products are available on the market for this purpose.

Cracks that are somewhat wider but under 0.4 mm should be sealed. This consists of bonding an elastic, waterproof material to the concrete, able to cover the crack and allow for possible movement, to insulate the indoor space from the elements. Sealing is also sometimes interpreted to mean partially filling the crack to guarantee durability in even the most aggressive environments.

If the crack is over 0.4 mm wide, it should be injected (a recommendation that may rightly be branded as overly conservative).

Essentially the idea is to fill the crack with products, usually epoxy resins, able to re-establish mechanical continuity, i.e., so stress can be transferred across the crack.

The GEHO Bulletin [26] describes the modus operandi, a laborious process that includes prior sealing, product preparation and pressure injection into the crack. It calls for such specialised facilities (sealing compounds, pumps, nozzles such as shown in Photograph 8-8 and so on) and products (highly fluid resins) that it is best performed by authorised experts.

8.4.2. Spalling and partial loss of concrete cover

Repair consists of replacing the detached concrete with repair mortar to restore the initial section and with it the design safety and durability conditions.

It entails cleaning and texturing the surfaces as required and especially where the concrete has worked completely loose, exposing the reinforcement, or where cracks delimit its position. In these cases the cover must be chipped away down to the inner side of the reinforcing steel so the new and existing concrete can be stitched together.

Given the ease of application and bonding and drying shrinkage specifications of today's repair mortars, the requirement to expose the reinforcement entirely may appear excessive. Nonetheless, when a procedure is described in such general terms as in this case, it must be robust, i.e., it must be valid even where materials and workmanship are less than ideal.

That is deemed to be important because repairs are not always conducted by specialists aware of the care with which their work should be performed and of the very strict conditions under which their materials should be applied.

All too often, "*repairs*" are observed to fall away in a matter of weeks due to shrinkage of poorly batched mortar, constituting an additional hazard.

Perhaps the clearest example of this problem lies in the most common, systematic and (at least theoretically) specialised type of repair, namely the holes left by core samples.

The object of Photograph 8-9 is sure to be familiar to anyone who has worked at Lorca for any amount of time: the perimetric crack that clearly denotes detachment of the repair mortar.

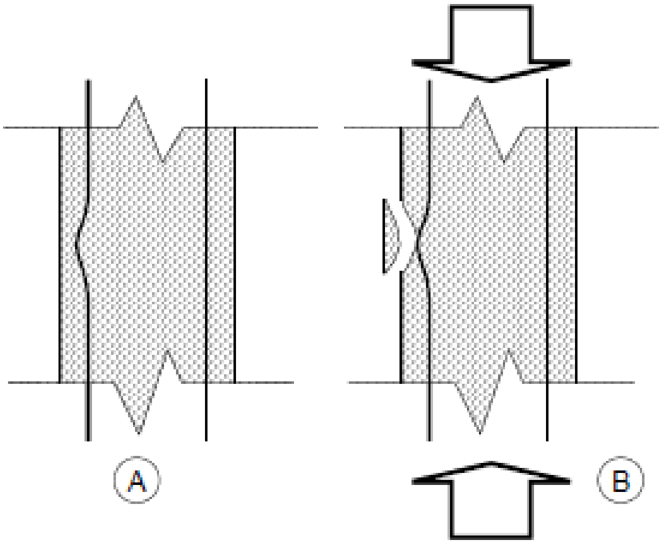
Sufficient importance is not always attached to this problem. Note the trajectory of the column failure in Photograph 8-10[3], which was indisputably conditioned by the position of the mortar used to fill a core hole.

▲ **Photograph 8-8**

▲ **Photograph 8-9**

▲ Photograph 8-10

3. *This photograph is included by way of reference only. It was neither taken at Lorca nor was its failure triggered by seismic action.*



▲ Figure 8-2

8.4.3. Partial section failure and reinforcement buckling (columns)

The present discussion is confined to columns because not a single beam was observed to fail in Lorca for this reason. Actually, as noted earlier, no significant damage was found in any beam.

This item describes situations such as depicted in Photograph 8-11.

In such cases the main concern is to eliminate the buckled length of bar to prevent the thrust that would be generated when the bar is re-loaded (Figure 8-2).

An alternative approach would be to mechanically straighten the deformed length (given that it need not be completely straight if new tie bars are wrapped around it to accommodate the effect of minor geometric imperfections). Nonetheless, with a view to a robust solution, replacing the length of bar is deemed to be the more suitable solution.

Annex II hereunder contains a more detailed description of a repair procedure for this type of damage. It is an adaptation of the proposal put forward by INTEMAC for some of the buildings at Lorca in which the section on materials was removed to omit commercial references.

8.4.4. Total breakage of the member (columns)

The reference here is to damage such as depicted in Photograph 8-12.

▲ Photograph 8-11

▲ Photograph 8-12

The most effective repair in this case is deemed to consist of rebuilding the damaged member as nearly as possible to design, respecting not only geometry but also material characteristics. The sole variation on the original would be to increase the number of tie bars and naturally anchor them with 135º hooks.

Replacement involves removing the broken segment, chipping away the surfaces of the adjacent segments to a sufficient reinforcement depth to splice the new bars, positioning the steel and casting the concrete.

The procedure is not particularly complex. In fact, it entails no more than the partial reconstruction described in the preceding item. The only real difficulty in this case is estimating the safety conditions of the rest of the structure during and especially after the repair operation.

Whereas in the preceding cases the column[4] could be assumed to be able to bear the service loads (which is not to say that safety need not be verified as specified in Annex II), here the column must be assumed to have no strength left whatsoever.

The inference is that loads are no longer flowing down the member. That in turn calls for determining where the load present at the time of the earthquake went: necessarily to the adjacent columns (or masonry walls, if any).

Consequently, repairing such columns may also require strengthening the adjacent members.

4. *According to tests conducted by Sezen and Moehle [48] on tightly reinforced columns exposed to bending, these members conserved their axial strength practically up to failure.*

From that perspective, the least important intervention would be the repair of the collapsed column, for it would recover only a minor share of the loads for which it was initially designed (the service loads present after the repair, i.e., the difference between the design loads and the quasi-permanent loads present during the quake). That difference would be the additional load for which the adjacent members would have to be calculated.

Alternatively, in theory these members could be unloaded by subjecting them to opposing loads (with hydraulic jacks, for instance).

The actual situation is more complex and in all likelihood more favourable. The concrete creep in the overloaded columns would force part of the prior axial load back to the repaired column ("*back and forth*" loads, according to the graphic term coined by Lloret and Regalado).

The problem is how to check that redistribution capacity, i.e., to verify whether the overloaded columns can be shortened (without imploding) enough to return the loads to the repaired column.

One factor that would compute in favour of that possibility is that concrete strain capacity to counter sustained loads is much greater than the 2 ‰ set out in the legislation as a general provision meant to cover all manner of loads. One that would compute against it is that concrete loss of strength under sustained loads would have to be taken into consideration.

Another adverse although fairly controllable question that must not be overlooked is possible shrinkage in the repaired column, which could become a conditioning factor if the part of the column replaced is of a sizeable length and the concrete used is not carefully dosed.

The foregoing is obviously no more than a listing of elementary ideas relating to any intervention on existing structures. They are discussed here merely to stress the care with which the repair of a collapsed column must be performed, and the need to broach intervention not from the standpoint of the member only, but of the entire structural system.

In other words, repairing a collapsed column entails repairing the structure and perhaps strengthening other columns.

8.5. Strengthening criteria

None are in place, or at least, none we know that can be generally applied with assured effectiveness.

Nonetheless, certain elementary ideas, discussed at the end of this item, can be regarded as valid.

Classical criteria laid down in a number of references can also be recommended. The ATC manual [6] and the FEMA codes [7] and [8] proved to be particularly useful for the present purposes. Eurocode part 3 [4], applicable in Spain, can also be cited. Of the books listed in the references, Fardis [24] contains a few very specific and helpful chapters. The difficulty faced here is that the application of these criteria to the actual circumstances prevailing at Lorca (which are not deemed to be very different from those found in other places affected by earthquakes) is not necessarily possible. Irreconcilable differences exist between those circumstances and the standards cited. Specifically:

- The object of strengthening in the references consulted is always a single building. At Lorca, as noted in earlier chapters, the basic unit is often a whole city block. In some extreme cases, buildings even share structural members, such as party walls. More often, the practice of casting the concrete for a new building against the side façades of the adjacent buildings, used as forms, forces the buildings to respond jointly to actions.

- The primary objective in such references is to improve the building's response to possible seismic action, defined to mean ground shaking. The primary objective of intervention in Lorca should often consist, rather, of protecting each building from the aggressive action of adjacent buildings.

  Such is the case of the building portrayed in Photograph 7-24 and included in Photograph 8-13 above. The latter also shows the result of the repair some months later. Note the (well-advised) effort made to demolish the end of the floor slab in the building on the left, which would have collided with the column on the one on the right, with visible results.

  In short, more than strengthening a given building, the aim would often be to intervene in the ones adjacent to it. Unfortunately, the global action that such circumstances would have required was seldom possible.

▲ **Photograph 8-13**


▲ Photograph 8-14

- While in the literature strengthening refers mostly to building structures, in Lorca that was less obvious. Often, weakening non-structural elements would have been more logical than strengthening structural members.

  Photograph 8-14, likewise shown in earlier chapters, illustrates the damage caused by masonry walls that were often stronger than the columns framing them. Strengthening those walls, as depicted in the photograph, will very likely weaken the structure. In some of the buildings in Lorca whose columns clearly fit the "*captive column*" failure pattern, after repairing the column, the enclosures were rebuilt with the same geometry but even stronger masonry.

- Although many references were found in the literature on strengthening non-structural elements (parapets, façade and staircase enclosures), they all called for interventions scantly compatible with standard construction systems.

That notwithstanding, a few elementary ideas on the possibility of improving buildings (which hardly qualify as strengthening criteria) are listed below. Actually, more than strengthening, the aim would be to simply correct the flaws most commonly observed.

- **Masonry parapets.** The action taken in some buildings in Lorca would appear to be particularly suitable: replacement with steel railings firmly anchored in the floor slab (necessarily, to bear the service loads). Why this solution was not generally adopted is not readily explicable. It seems to have been applied only where parapets failed. In the authors' opinion, the many masonry parapets that resisted the quake simply because they were located in the direction of the shock (i.e., the earthquake generated in-plane forces that such masonry can resist) remain highly vulnerable to, and are sure to fail during, a future quake acting in some other direction.

In any event, strengthening masonry parapets would only be justified in a few special cases and would call for an ancillary structure to secure them to the main structure.

In addition to the parapets, the stability of all other roof masonry (chimneys and bulkheads), particularly where located near the façade and liable to fall on pedestrians if they fail, would need to be checked. In some cases a structural member may be close enough to anchor such elements, deploying solutions as simple as shown in Photograph 8-15. These measures should be taken scrupulously and include even secondary elements such as chimney caps (note the trail of rubble between these elements and the edge of the roof in Photograph 8-16).

- **Joints between adjacent buildings.** Their actual existence should at least be ensured, i.e., the buildings must not be merely connected with the usual mortar fill (or fills consisting of the façade materials themselves). Where they are, the joint would have to be rebuilt and sealed with a commercial product.

- **Extra mass.** In light of the practice of housing very heavy objects (such as the tanks referred to in item 7.3.2 "Inappropriate distribution of mass") in the buildings, in apparent defiance of their capacity, the necessary structural calculations should be performed.

- **Short columns.** If they are built between the basement wall and the first storey floor slab, the most robust solution is to raise the wall along the entire perimeter of the storey and connect it to the slab. Moderately sized openings can naturally be made in such walls for lighting and ventilation. Alternately, the wall and slab can be connected only partially but sufficiently to ensure the stiffness, strength and symmetry of the connecting shear walls.

- **Staircases.** The first recommendation is elementary: to chip away some of the concrete to see the

▲ **Photograph 8-15**

reinforcement at the inter-flight or flight-landing connection. Erroneous construction in this respect, which induced thrust and hence cover detachment, was detected in too many cases (see item 7.6, "Staircases") to be deemed merely incidental. Nonetheless, outside of this prior measure, no simple solution to the problem of staircases built in the usual manner is at hand. Either an inter-flight joint must be built that allows for differential inter-storey drift, with all the complexity that would entail, or the building must be afforded sufficient horizontal stiffness to reduce such drift to values that can be borne by the slabs forming each flight.

The most serious problem associated with staircases, however, occurs when they are connected to building columns, meaning by this not the small columns sometimes used to support the landings but the members with vertical continuity that transfer gravity loads storey by storey. The result is the generation of one of the classic examples of short columns referred to earlier.

8.6. Actions on buildings

The following is a description of the general principles that seem to underlie some of the actions most commonly observed in Lorca.

One recurrent idea identifies damage with strengthening. As in the case of the parapets mentioned in the preceding item, where action was confined to the ones that collapsed during the earthquake, action has apparently been limited to the damaged structural members only, irrespective of whether such damage was the result of the quake or even actually existed (see the first item in this chapter on cracks).

Since much of the action clearly involved strengthening, the implicit premise would appear to be that the members that failed did so for want of bearing capacity and hence needed to be strengthened.

▲ Photograph 8-16

The authors believe that premise to be mistaken in earthquake scenarios, for a number of reasons.

- The singular nature of the Lorca earthquake precludes any generalisation of its effects. Other types of equally possible quakes (or even the same quake but in another direction) would induce other types of damage that would affect other elements. Therefore, strengthening only the ones affected now is no guarantee of building safety in future earthquakes.

- Strengthening damaged elements only can be likened to only strengthening the weakest links in a chain, but not the chain itself. If a link able to bear a load of 5 (for instance) exhibits brittle failure, strengthening it to bear 10 serves no practical purpose if the adjacent link can only bear 6. If its failure is ductile, strengthening would even be risky. Elements exhibiting ductile failure (unfortunately very few in Lorca) must never be strengthened, but only repaired.

- An example of this can be found in ground storey column and masonry wall strengthening, a solution generally observed in the city. In some cases the result was greater storey stiffness and strength both. The columns were thickened (Photograph 8-17) and the walls strengthened. But a similar earthquake would certainly generate greater stress in the non-strengthened rest of the building.

The above reasoning is even more clearly applicable to non-structural elements. If, as noted in item 6.1, “Increased loads”, the action on buildings often depended more on the strength of their non-structural

▲ **Photograph 8-17**

▲ **Photograph 8-18**

5. Further to a request received from the consortium, INTEMAC analysed the suitability of the detailed designs for the actions undertaken in many buildings in Lorca.

ground storey elements than on the quake itself, raising the strength of masonry walls whose failure acted like a damage-limiting fuse would appear to be illogical. In some cases, however, the ground storey enclosures were replaced with high performance masonry (thermo-clay block, as shown in Photograph 8-18).

- Viewed with an eye for greater detail, strengthening masonry may be even more hazardous. When the force exerted by the panel was what damaged the column that frames it, abutting the wall against the strengthened column makes little sense.

In short, strengthening damaged elements may ultimately prove to be counterproductive. The sole advantage would appear to be that it is a convenient approach because it calls for no structural analysis or estimation of force distribution or element strength.

This is the sole explanation for the fact that many of the designs reviewed[5] lacked a chapter on conceptual considerations. An essential component in any design, in the authors' opinion, it is imperative in structural strengthening proposals as substantiation for what is to be done, how and why, i.e., the aim of the action, the means to be deployed and the justification.

- **Objectives (what is to be done).** This is often confounded with "*how*". What should be done (i.e., the actual aim of the action) is to improve building performance in terms of safety or functionality by modifying specific elements, structural or otherwise. Such modifications, however, form part of the resources to attain the objective (the "*how*").

  Even at the simplest level, strengthening a section of a specific element calls for thinking in equivalent terms. The aim of the action should be to improve performance, expressed as strength or deformability, in respect of a specific force (axial, bending or shear) foreseen for a given design scenario (permanent or seismic).

  A specific example may help to express these ideas more clearly. One of the strengthening solutions used most frequently in Lorca was to retrofit columns with fibre.

In the most explicit designs this procedure was justified as a way to confine the concrete. Such confinement should not, however, be regarded as the aim of the action, but merely the means to obtain the actual objective, i.e., to improve the response of the section in the desired direction (raise axial strength or bending ductility for instance) in a given design situation.

That distinction is not academic, but strictly practical. If a column is strengthened to raise its capacity to carry axial loads that cannot be borne by adjacent collapsed columns, the material used should be calculated for its resistance to fire, which is not especially kind to fibre. That would not be the case, however, if strengthening is designed only for earthquake scenarios. Similarly, if the idea is to supplement a column's axial bearing capacity, it should be strengthened along its entire height (for the axial load is practically constant throughout), whereas if the objective is to increase bending deformability, strengthening should be positioned at the top and bottom (where bending is greatest).

- **Means (how).** A wide spectrum of ways to strengthen elements and sections is available: section confinement or encasement, element replacement and others.

  Less obvious, at least for INTEMAC, are the means for global action on a building as a whole. In fact, as mentioned above, the authors are reluctant to put forward generally applicable detailed solutions.

- **Justification (why).** They deem that the clearest approach is set out in the ATC 40 and FEMA codes.

  There justification is formulated in terms of performance, i.e., actions are made to depend (in both type and degree) on the overall objective sought for the building. This varies gradually from a minimum, which always refers to personal safety, to a maximum, which pursues building performance to a level that allows for its immediate occupancy. Procedures are also described to translate the definition of objectives to numbers and numerically analyse the building's response for comparison to such values.

8.7. Actions on structural members

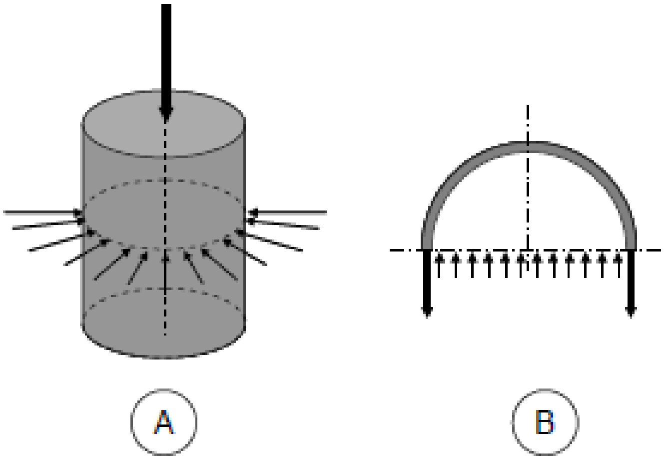
This item describes some of the specific actions most frequently observed in Lorca. Its scope is limited to columns, the structural members where most damage was recorded and most repair and strengthening performed.

In an initial, extraordinarily simplified classification, the actions taken can be divided into two types: those that seek to improve the characteristics of the existing section or member and those that merely attempt to enhance it with some manner of supplement.

The first type includes confinement by section wrapping, either with fibre (Photograph 8-19) or steel.

▲ Photograph 8-19

An understanding of the notion, confinement, is essential here (see Mander *et al.* [53]). When a concrete specimen is subjected to radial pressure (Figure 8-3 A), its capacity to resist axial loads rises in proportion to their value.



▲ Figure 8-3

One simple way to apply radial pressure is to wrap the specimen in a steel or fibre sheath. The axial force itself then induces specimen shortening and radial expansion, which in turn increases the circumference of the steel (or fibre) sheath and generates the desired radial stress on the specimen (Figure 8-3 B.)

In such actions, the wrapping is obviously not designed to receive any load whatsoever. It should not even abut with the floor slabs. Its sole purpose is to act as a tie and it should therefore be wrapped around the sections. This approach is well adapted to circular but less so to square sections. It is unsuitable for rectangular sections where one side is much longer than the other.

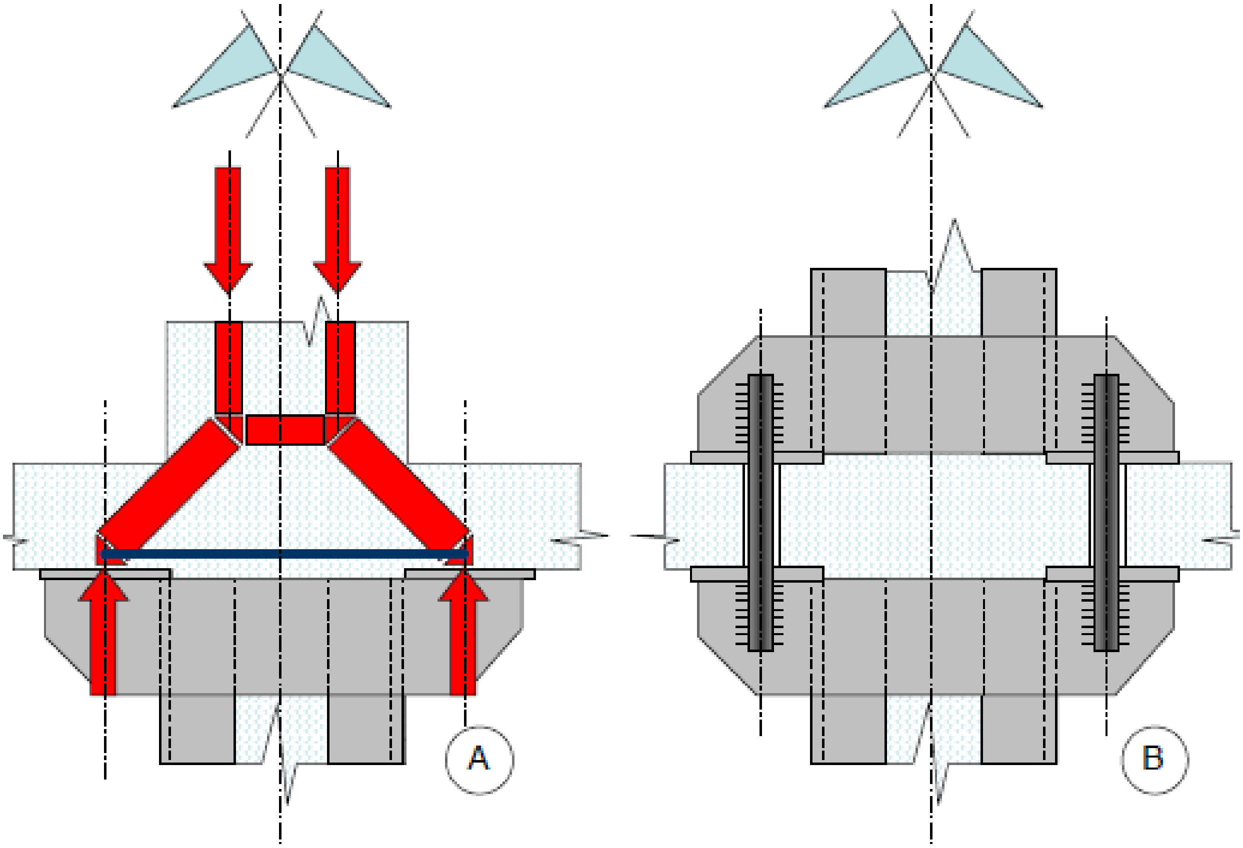
The second type of action involves elements intended to receive part or all of the loads that formerly flowed down the original column. The extreme example would be the construction of a new column embracing but unconnected to the existing member, which would pursue no improvement whatsoever of the initial section, and would usually disregard its contribution altogether. The most common version of this approach consists of positioning steel angles and battens around a column (Photograph 8-20).

Since no mechanical connection exists along the length of the shaft, this second type of actions, unlike the first, requires some manner of mechanism to transfer the loads to the top of the new column and from there to its base.

That calls for deviating the forces flowing across the member in higher storeys (Figure 8-4 A). That complex and not always feasible operation often entails extending the strengthening to those storeys and inserting steel elements in the intermediate slabs (Figure 8-4 B). If the segment of column involved is in an intermediate storey, the strengthening must obviously be extended both to the storeys above and below it.

Some actions, while formally similar, may actually derive from different criteria and therefore require very different detailing. Encasing columns with reinforced concrete (Photograph 8-21) is the clearest example.

▲ Photograph 8-20



▲ Figure 8-4

6. *Note that the original buckled reinforcement bar was not replaced. Further to the discussion in item 8.4.3, the authors deem that to be unsuitable.*

▲ **Photograph 8-21**

If the aim is to tie the existing section, the new concrete is reinforced most heavily in the transverse direction, while the longitudinal steel is kept to the minimum necessary for assembly and crack control. Under this approach, partial shaft strengthening would be possible, theoretically at least (Photograph 8-22).

In practice, such partial solutions are not deemed to be suitable because they induce an obvious discontinuity in the characteristics of the segment of column involved that multiplies the stress on the rest of the shaft.

In extreme circumstances, a short column mechanism might form (Figure 8-5). That effect is much greater in columns with a modest initial section. Given that a column can hardly be thickened by less than 10 cm per side, a cross-section initially measuring 30x30 cm would grow to 50x50 cm, raising section inertia nearly eight-fold.

If the aim is to erect a concrete column around the existing column, continuity across the top and bottom joints, which is essential, is attained by positioning the longitudinal reinforcement (which acquires a more prominent role) as shown in Photograph 8-23[6] and extending the operation to the adjacent floors as described earlier in connection with steel solutions.

Some authors contend that steel angle and batten solutions, like concrete encasement, can ensure confinement.

▲ **Photograph 8-22**

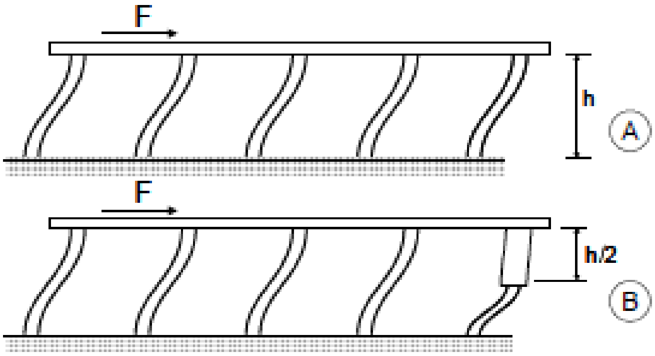
▲ Photograph 8-23

▲ Photograph 8-24

Some reputed manuals cite the possible use of this solution to confine an existing column and thereby improve its properties. They even recommend heating the battens before welding so that subsequent cooling and associated deformation induce horizontal pre-compression in the section.

The authors are unaware of any realistic construction process able to guarantee confinement of any magnitude.

Further to that premise, they find solutions such as shown in Photograph 8-24 to be incomprehensible, for they can neither confine the existing member nor receive loads.



▲ Figure 8-5

▲ Photograph 8-25

▲ Photograph 8-26

Steel jacketing (Photograph 8-25), in contrast, could provide an effective tie because the pressure injection of mortar in the gap between the column and the steel plate would provide the necessary fit.

The authors are unable to understand actions such as depicted in Photograph 8-26, however, where the beams prevent the perimetric confinement of the column (no tie is formed) and the angles appear to be positioned to support the beams (as if they were to receive the respective loads). Note also that the steel generates obvious longitudinal irregularity in the column, raising the stress on the non-strengthened segment.

In Lorca, partial confinement solutions at the top of columns were unusually common. Photograph 8-27 depicts a very typical example with fibres. The preceding photos portray the same type of solution but with other materials.

This type of action is not readily understood. It obviously constitutes strengthening, not only because it is so interpreted in the literature, but because it would be inconceivable to bond the fibre directly to the damaged member without first repairing it. Therefore, the action was subsequent to repair and logically purported to improve the design properties, i.e. to strengthen, a given part of the column.

But, for what kind of stress was it intended? Certainly not axial or shear loads because that would entail bonding the material to the entire column, for those loads are constant across the length of the member. Nor can it be envisaged as strengthening to better resist bending because in that case it would need to be applied at both the top and bottom of the column (where moments are consistently and design strength values usually very similar). Moreover, raising bending capacity is always very risky.

The aim, then, must have been to improve the ductility of the possible plastic hinge, the usual objective in bending confinement, but that would be tantamount to acknowledging the existence of plasticisation at the top and bottom of the column, a flawed mechanism observed in weak storeys. Moreover, the existence of plastic hinges at the top and bottom of columns can only be assumed when they are free across their entire length. If they are restrained by masonry walls the hinge may occur in any section and, according to the Eurocode on earthquake-resistant design, the segment involved must be regarded as a critical zone where identical ductility conditions must be maintained. Columns in contact with masonry walls must be strengthened across their entire length.

The importance of careful workmanship cannot be overstated. Any action of this nature calls for an exhaustive control of all the details, which always condition the result. Note the weld in the steel jacket in Photograph 8-28. Such a weld can obviously not guarantee any resistance whatsoever and yet it is supposed to withstand the tying stress. The weld will obviously fail long before the plate is loaded, nullifying the strengthening.

The same photograph shows that the steel plate was positioned directly onto the surface of the column[7], preventing satisfactory penetration of the mortar fill. A small gap should always be left between the two surfaces to favour the even distribution of the mortar.

Where polygonal section columns are to be fibre-wrapped, due preparation of the segment of column involved is very important and must include rounding the corners both to limit stress concentration in the fibres and to improve the effectiveness of the tie. That elementary precaution was not always taken in Lorca, however.

7. *In some extreme cases, the steel was attached directly to the architectural finishes on the column.*

▲ Photograph 8-27

▲ Photograph 8-28

▲ **Photograph 8-29**

Photograph 8-29 depicts a splicing detail in tie bars for column thickening. As noted in preceding chapters, in seismic zones tie bars must be anchored in the core with 135º hooks. Alternately, the laps may be welded, but simple lapping such as shown in the photograph is never admissible.

Lorca: the lessons learnt

9

9. Lorca: the lessons learnt

The work conducted in the city after the earthquake was enlightening, but not always in ways initially expected. The role of non-structural elements in building response came as a surprise, as did the vulnerability of the city at large to the earthquake, a finding that has called the suitability of standard construction systems into question. Some of these ideas are discussed at greater length in this chapter.

9.1. The role of non-structural elements

The preceding chapters stressed the importance of non-structural elements, particularly enclosures and partitions, in the Lorca quake.

One of the conclusions drawn in those chapters referred to the scantly singular nature of the construction systems in place. Many masonry walls in Lorca were supported in the same incorrect manner as observed in any other Spanish city.

The present item addresses the widespread use of solutions that have proven to be unsatisfactory and the pressing need to adopt measures in this regard. Situations such as depicted in Photograph 9-1 warn of the need, at least in buildings that house certain activities, to bear in mind the hazards inherent in these elements.

▲ Photograph 9-1

9.2. Vulnerability

An area's seismic risk is defined today as the combination of two basic factors: hazard and vulnerability.

Hazard refers to the likelihood of an earthquake occurring in a given place. Hazard levels are very high in some parts of Japan and the west coast of North, Central and South America, much higher than in any region in Spain.

Vulnerability is the likelihood that a given earthquake will cause damage and is associated with factors such as occupancy, construction quality and citizen sensitisation. As a rule, vulnerability in Japanese and North American cities is low.

▲ **Photograph 9-2**

▲ **Photograph 9-3**

1. *Taps were seen to be installed in columns.*

Risk, which is the truly determining factor, may be high in countries with moderate seismicity such as Spain if vulnerability is high, as the Lorca quake apparently revealed.

Generally speaking, the vulnerability of the Spanish housing stock is the result of low quality construction and insufficient maintenance.

The quality of both materials and workmanship observed in Lorca (Photograph 9-2) was so poor that it posed serious doubts about how to broach the necessary post-quake repairs. It is difficult to limit the scope of such repairs to the damaged portions of a column when the beams that rest on it are in the condition depicted in the photograph. The commonly repeated contention that "*it's been that way for 50 years*" is hardly reassuring.

Building deterioration over time also contributes to vulnerability.

In portal frame structures, earthquake resistance is based essentially on the premise that the greatest stress is to be received at the base of the columns, where a plastic hinge must necessarily form. But if the column bases are wholly deteriorated (Photograph 9-3) due, among others, to capillary damp from the soil, they will be unlikely to be able to resist such stress. Insufficient insulation of the structure from the soil caused damage in a sizeable share of buildings in Lorca.

The same type of damp-induced deterioration was systematically observed around both rain and foul water downpipes. Inappropriate or non-existent maintenance (note the condition of the beam reinforcement in contact with a "*repaired*" downpipe in Photograph 9-4) of these elements apparently caused widespread damage.

In other cases the subordination of the structure to functional demands contributed to its deterioration.

The uncontrolled placement of very heavy tanks on floor slabs was referred to in earlier chapters, but the standard practice of making openings in the slabs for building services, such as in Photograph 9-5, could also be cited in this context.

Changes in floor plans also affected building structure in some cases. Photograph 9-6 depicts a cableway drilled into the very core of a column, behind the tie bar[1].

The problem is not limited to structures. Much of the damage reported on façades (Photograph 9-7) would appear to have been attributable to the prior rheological behaviour of the masonry walls more than to seismic action. The deterioration of brick façades is alarmingly widespread in Spain.

Similarly, the detachment of façade cladding (Photograph 9-8) could probably be attributed more to its precarious attachment than to the earthquake. This procedure for securing cladding constitutes obvious building deterioration.

9.3. Code compliance

One of the causes of the aforementioned vulnerability is patchy compliance with the legislation. The implications of this issue are so great that it is believed to merit a separate item.

The preceding chapters reiteratively stressed that none of the severe effects of the Lorca earthquake was by any means unprecedented. Each and every one has been amply described in earthquake literature for many years now.

That cumulative knowledge is naturally mirrored in the applicable legislation, whose provisions explicitly aim to prevent

▲ Photograph 9-4

▲ Photograph 9-5

▲ Photograph 9-6

▲ **Photograph 9-7**

▲ **Photograph 9-8**

many of the types of failure observed at Lorca: parapet and façade collapse, captive or short columns, staircase collapse, pounding and so on.

Obviously, then, neither legal provisions nor the most basic experience were honoured in many cases, the reasons for which have also been discussed.

It is often contended that some buildings pre-dated the legislation and were consequently not subject to its provisions. That was only true in a small number of cases, however, for the vast majority of the buildings discussed here are less than 50 years old: i.e., mandatory earthquake-resistant legislation was in place when they were built. Moreover, some of the most severe problems, such as parapet collapse or pounding, affected the most recently erected buildings, subject to the legislation presently in effect.

Another argument frequently wielded is that since the loads induced by the quake, at least in terms of the basic design parameters, were greater than envisaged in the legislation, failure was not necessarily symptomatic of non-compliance. This is not wholly consistent either, because as shown in the preceding chapters, while the loads induced by the earthquake were greater than envisaged in the legislation, they were similar to or smaller than other types of loads provided for in the building code (wind or service loads, for parapets) that these elements should have been designed to resist. Moreover, respecting parapet collapse, the legislation (the Technical Code document on masonry walls in this case) explicitly prohibits the system itself (by precluding reliance on the bending strength in the bed joint direction), irrespective of stress values.

The joint clearance between buildings required in the legislation may have been insufficient to accommodate action such as induced by the earthquake, but that does not justify casting the concrete for one building directly against the columns of another.

In short, the legislation on building design and construction was obviously not applied in Lorca. Despite expectations, in light of the area's seismicity, construction practice there was no different from the standards applied in Spanish cities located in non-seismic zones, at least in the authors' experience. Most of the buildings did not appear to have been designed or built with the possibility of an earthquake in mind.

The overall impression is that, in the best of cases, seismic design was confined to ensuring that the structure would resist equivalent loads, an approach that the Lorca quake proved to be woefully insufficient.

Structural engineering is wholly useless if the basic rules of earthquake-resistant design are not followed, rules that impose such elementary requirements as some minimal order rather than the confusion which on occasion was observed to affect every single stage of construction in Lorca. For instance:

- The urban layout itself in much of the city (and not always the oldest quarters) is so complex that it was difficult at times to determine building boundaries, given the irregular shape of the property on which they were located. Neither analysis nor action is conceivable under such conditions. The basic unit for earthquake behaviour is the city block and the most severe actions affecting each building are exerted by the adjacent buildings.

- The configuration of many such buildings was completely lacking in any structural logic, which actually only calls for simplicity, regularity and symmetry.

- Design changes during construction introduced further uncertainties. Some of the short column configurations that caused severe damage were the result of such changes.

- Construction arrangements which are sometimes, and fallaciously, contended to be "*time-honoured*" actually are not attributable to any tradition whatsoever. This issue is particularly relevant in connection with façade-related problems, whose attachment to the structure is an issue that has not been satisfactorily solved in standard practice in Spain.

The existence of mandatory, clear and precise legislation does not appear to have sufficed to impose a minimum amount of order in building construction, a very important question because it affects not only earthquake-resistant factors, but construction as a whole.

Lorca, in INTEMAC's opinion, has revealed the shortcomings of the industry in general.

2. *Providing the column section is not overly tapered with height.*

9.4. Viability of construction systems

The findings would appear to reveal a need to question the actual suitability of the construction system most widely used in Mediterranean countries, in which buildings are the result of the sum of essentially incompatible elements such as a flexible portal frame (in the best of cases) attached to much stiffer and, worse than that, often stronger, enclosure and partition walls.

Unfortunately, the problem cannot be readily solved. The possible solutions are confined to reducing the stiffness and strength of the non-structural elements, increasing structural stiffness and strength, or separating the two.

- Reducing enclosure and partition strength and stiffness is not simple because it entails ruling out the use of brick masonry, for which a general and cost-effective alternative is not available, at least in conventional construction.

- Separating non-structural elements from the structure, a solution put forward many years ago (Dowrick [23]), poses a few very substantial problems with respect to enclosure and partition stability, water-tightness, thermal and acoustic insulation and so on. While it is the solution laid down in the most advanced legislations, such as in Japan and New Zealand, it would not appear to be readily adaptable to Spanish circumstances.

  In those countries social awareness of seismic hazards justifies the increase in cost associated with this type of measures. In contrast, where earthquakes are not perceived as a real threat, but rather a remote legislative imposition, the difference in cost is more difficult to justify.

  Moreover, the load levels induced by earthquakes in those countries are so high that no other alternative is feasible (or their architects and engineers have yet to find one).

- Since increasing structural stiffness would therefore, at least apparently, be the sole logical alternative in the Spanish context, it constitutes the focus of the following discussion.

The advocates of this solution pose the need to replace portal frames with shear wall-based systems, which provide a series of obvious advantages.

Shear walls afford not only stiffness, but induce a less aggressive type of building deformation on enclosures than portal frames. Further to the shear model traditionally used (Figure 6-5), in the typical mode shape for portal frames the greatest angular strain is on the lower storeys[2], contrary to the distribution observed in shear wall vibration modes[3] (Figure 9-1). The soft storey problems referred to earlier are irrelevant in shear wall systems.

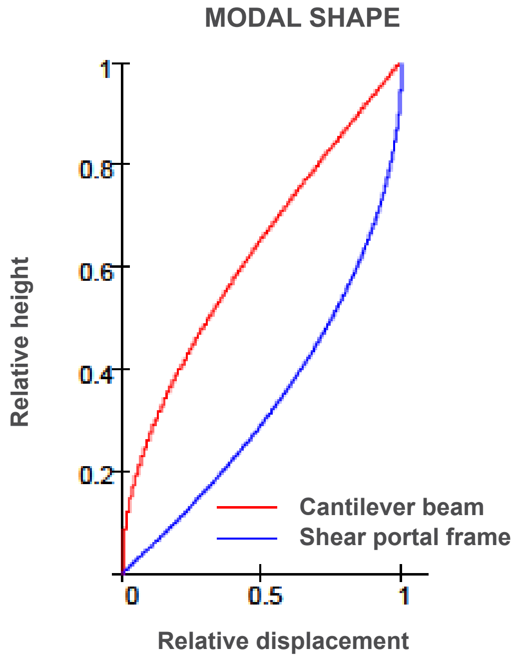
While the advantages of shear wall systems are conceptually clear, their practical implementation poses substantial problems. The most obvious of those problems are discussed in item 7.3.4, “Inappropriate use of shear walls”. The following addresses the one believed to be the most prominent, namely determination of the necessary stiffness.

Two basic criteria may be used to quantify the stiffness required by a structure. Stiffness should suffice to control the building’s response to an earthquake as well as to limit the damage to non-structural elements. These two criteria are analysed below.

9.4.1. Structural effectiveness

The idea behind the first criterion is simple: if the structure is not stiff enough, it does not participate in the building’s response to an earthquake, which depends, rather, on the behaviour of its non-structural elements, which are not designed for that purpose (although such an arrangement is not necessarily unsuitable). That is senseless enough, but even more absurd is designing an earthquake-resistant structure only to subsequently wrap it in a stiff box that prevents it from acting.

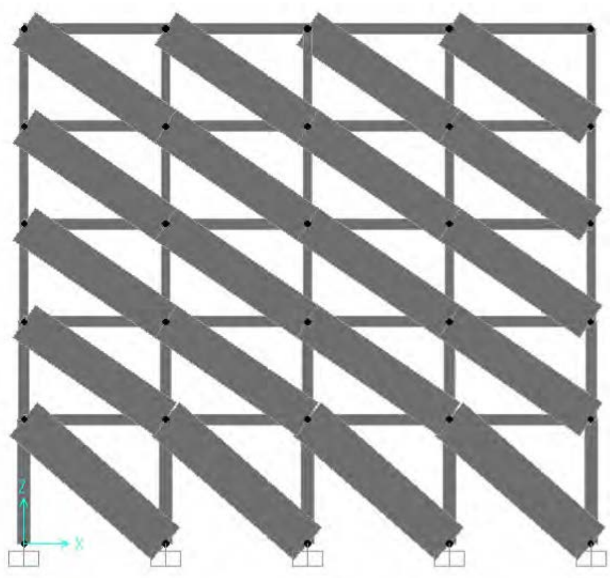
A first estimate of the stiffness needed for the structure to actually participate in countering horizontal action can be found by analysing the dimensions required of a shear wall that would afford a typical portal frame the same stiffness as provided by its outer enclosures (to at least share the loads).



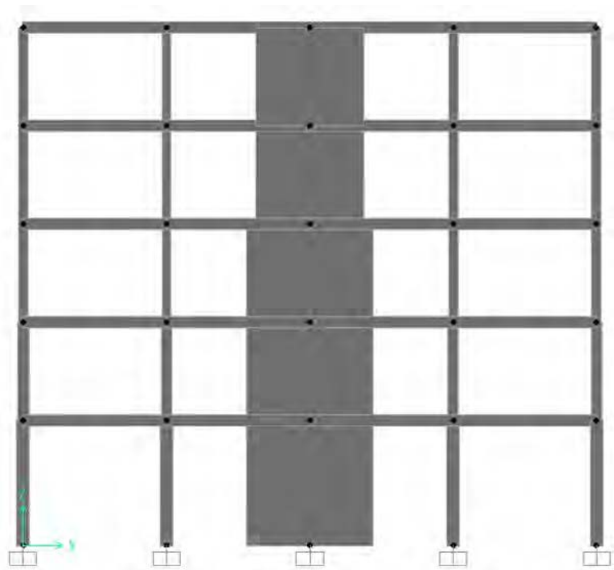
▲ Figure 9-1

The portal frame studied here is the five-storey, four-bay building introduced in Chapter 5, “Conventional buildings: wall behaviour” and used throughout the text. Here the structure is fitted with braces equivalent to the walls. Figure 9-2 reproduces the model, in which the bar dimensions are adapted to the true geometric scale. Although the width does not reflect the actual stiffness proportionally, because the modulus of elasticity of the material comprising the diagonals (masonry) is less than 20 % of the value of the (concrete) structure modulus, it does clearly show how large the braces are.

3. *In the elastic phase at least. If the structure responds plastically to the earthquake (because the design envisaged a high ductility reduction factor) deformation should be similar and essentially linear.*



▲ Figure 9-2

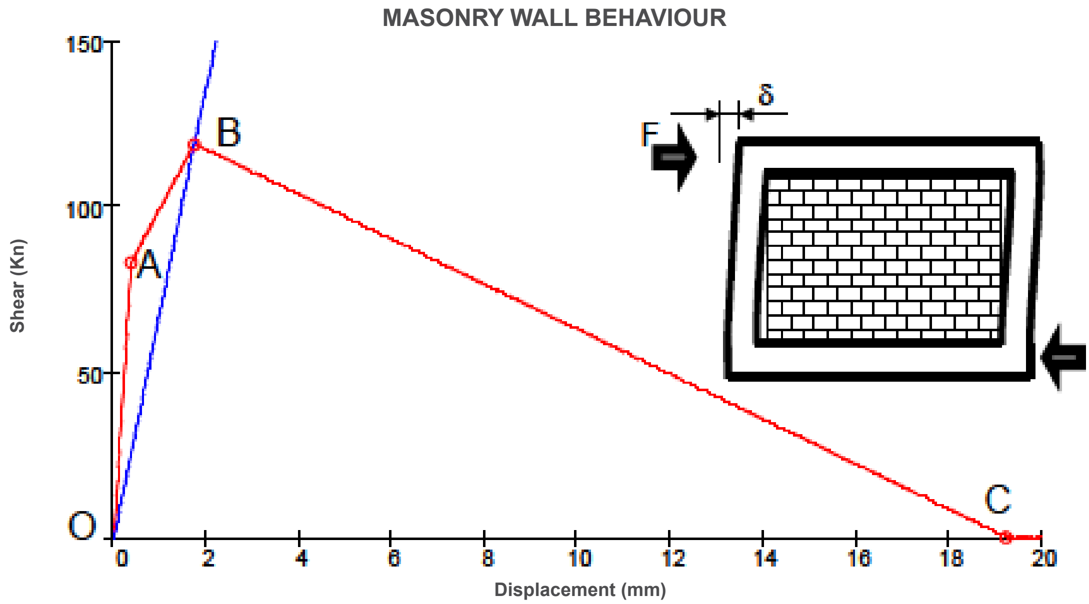


▲ Figure 9-3

Modal analysis yields a natural period of 0.31 seconds. A shear wall with a stiffness comparable to enclosure stiffness should reduce the period of the bare portal frame in the same proportion. Trials with different size shear walls led to the conclusion that to attain a similar period, the wall would have to measure... 3.5 m wide and 0.4 m thick! (as shown approximately to scale in Figure 9-3).

Consequently, shear walls are clearly not an all-purpose solution. Moreover, even in such systems, the possible structural effects of partition walls referred to in Chapter 6 must be borne in mind: increased loads, plan or elevation irregularities and structural damage.

This last item may prove to be a determinant, because columns in shear walled buildings are normally sized to bear only gravity loads. That results in very small cross-sections unable to bear the shear stress that may be generated by masonry walls.



▲ Figure 9-4

9.4.2. Damage limitation

If the objective is to avoid masonry wall rupture in an earthquake of some severity, the first thing a designer needs to know is when they break.

The stress-strain relationship of a wall adopts the shape shown in Figure 9-4 (particularised for the wall described in item 6.1, "Increased loads").

The first arm of the curve, "OA", represents wall shear stiffness before it separates from the structure. Its upper limit, point A, is located between 60 and 75 % of the peak strength of the infill-frame assembly. This part of the graph (both segments "OA" and "AB") is of scant practical interest because since the interfacial bond is assumed to break with the first shocks, during most of the duration of the event actual stiffness would vary as shown in blue line "OB", which represents the compressed brace. When failure is due to bed joint movement, "BC", the downward arm, has a slope around 10 times shallower than "OB".

While the specific values on the graph plotted may vary widely with the type of masonry (as well, although to a lesser extent, as with the masonry-structure stiffness ratio and the geometry of the assembly), the most significant value to be gleaned from the figure is the failure displacement of the wall, i.e., around 2 mm, which is equivalent to a strain of 0.06 % (2 mm / 3500 mm).

Verderame *et al.* [60] suggested higher values for this parameter (failure strain): 0.15 to 0.2 % (which, for the present example, would mean displacement of slightly over 5 mm). Note that the values are expressed in millimetres, i.e., one order of magnitude smaller than the unit used for the structure (centimetres: see item 3.4.2 "Displacements"). These findings are consistent with the results obtained in item 6.1, "Increased load", where the stress on the masonry was shown to be a full order of magnitude greater than its strength.

To put it another way, the structures needed would have to be ten times stiffer than the ones now standing. The question immediately posed is whether that is possible.

To find the answer, the response to the Lorca earthquake by the conventional portal frame building used as an example was re-integrated, adding shear walls of increasing size until the first storey displacement obtained was on the order of 5 mm. The result was that the section would have to measure 3 m wide by 40 cm thick to reduce the ground storey displacement to 4 mm.

Furthermore, since all the foregoing refers to a single portal frame, several of these shear walls would have to be positioned in each of the two plan directions...

This only proves the obvious, that such a building, whose characteristics are typical of many in the city, could not be expected to withstand a seismic shock such as recorded at Lorca and remain intact. In fact, performing the above calculations directly with the earthquake's acceleration values showed that to be overly demanding.

On the one hand, the Lorca quake was particularly hard on stiff buildings because, as explained in connection with its response spectrum, such buildings are subject to the greatest amplification. Stiff buildings are better suited to soft soils, where spectral amplification lies in a higher range of periods, than to rock.

On the other, it seems scantly reasonable to use the same criterion for damage as for safety. In some of the most recent codes, damage limitation is defined for less severe load levels than safety. The approach is similar to the one adopted for variable actions, where damage is calculated on the grounds of service limit state conditions rather than the more demanding ultimate limit state conditions used to calculate safety. The Eurocode on earthquake-resistant design does not explicitly adopt that approach, but for damage limitation it multiplies the displacement obtained with the design load (equivalent to the ultimate limit state) by a factor of 0.4 or 0.5 (depending on the size of the building), which yields the same result.

Spanish code NCSP-07 [11] on bridges, presently in effect, puts forward a similar scheme.

Under such legislative provisions, the use of shear walls to reduce damage is more sensible.

What should not be overlooked is that where wall rupture may cause injury to people, ultimate limit state criteria would have to be applied. The problem is defining where breakage may cause personal injury.

9.4.3. Design criteria

The foregoing is just one more, and very minor, chapter in the more general and traditional controversy around whether rigid or flexible systems are more suitable.

The introduction to Akiyama's [32] book contains a splendid review of the evolution of these premises since the early years of seismic analysis. The author defines up to four stages of the controversy around flexibility and stiffness, stages during which design tendencies leaned in one or the other direction. Far from regarding the issue as settled, he foresees a fifth controversy around flexibility and stiffness.

Local differences can also be identified. In Mexico ductile, fairly flexible portal frames are normally used, whereas in Chile standard practice is to build very stiff shear wall solutions.

The root of the controversy lies in the nature of the loads. Initially, structures stiff enough to limit deformation would appear to be desirable. Since in earthquakes, however, additional stiffness may mean additional loads, that approach would essentially mean spending money on elements whose main function would be to absorb the loads they themselves generate.

From another standpoint, the type of earthquake expected in a given area may determine the choice of one approach or the other. Stiff structures would be more applicable in regions with low seismicity. Flexible structures, on the contrary, would constitute a suitable model in areas where high intensity events with long return periods are envisaged.

9.5. Research needs

Many can be identified, all very simple and eminently practical. One would be to gain an understanding of existing buildings, the problems posed and how to solve them. Others arise during the design phase and refer to future buildings. Yet others would address the definition of seismic action itself.

Masonry walls and their interaction with the structure constitute a clear example of the first group of needs. The construction solutions that have been used in Spain must be identified and catalogued to at least single out the buildings where the hazard is greatest (which would very likely include the ones with façades supported only partially on steel shapes) and adopt any necessary decisions. Façade collapse is one of the most serious problems affecting Spanish housing. As this report was being written (autumn 2012) the media carried news of the partial collapse of a façade during a storm in Oliva, in the Spanish province of Valencia, that caused injuries to five people.

The solutions presently in use must also be identified. As noted, the Eurocode on earthquake-resistant design provides that columns must be designed for a shear strength no smaller than the strength of the masonry infill.

Nonetheless, likewise as discussed earlier, that calculation is impossible in practice due to the wide variety of masonry types. A text that lays down requirements impossible to meet forfeits some of its credibility.

It would be desirable to provide some minimal classification of clay-based solutions to narrow their variety and focus research on a smaller number of products. Alternatively, contact between the façades and the columns could be avoided by setting the latter back far enough to accommodate the wall.

The problems associated with short columns constitute another example. Their behaviour need not be researched, for it has been well established. What should be reviewed is the reason why they were originally built, for that is the sole manner to ascertain where they are and the action to adopt in each case.

Basic research, then, is not involved (although some of that would also be very helpful), but rather simply surveys to understand the effects of the Lorca earthquake to prevent their occurrence in Totana (a nearby city very similar to Lorca), Murcia, Granada or elsewhere.

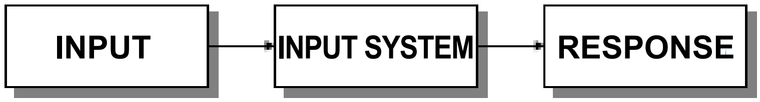
Annex I
Response spectra

10

10. Annex I
Response spectra

In its most basic definition, a response spectrum is no more than the (normally graphic) expression of the peak value of a simple linear oscillator's response to dynamic excitation versus its period. For the oscillator described in Chapter 3, a mass concentrated at the end of a cantilever, the excitation would be any seismic displacement at the base and the spectrum would be, for instance, the maximum displacement of the mass for each oscillator period (Figure 10-1).

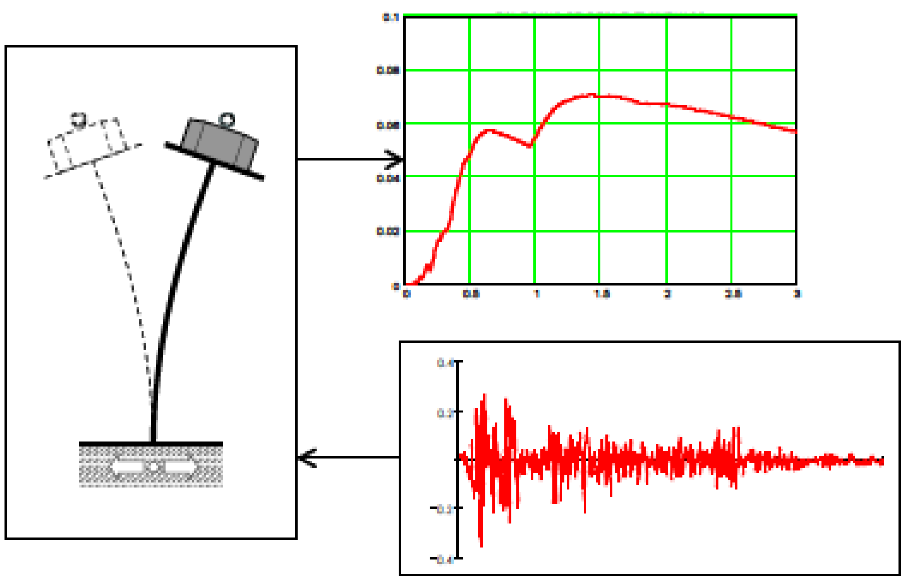
That idea can be generally applied to a system's response to any given input (Figure 10-2). From this perspective, the spectrum would be no more than a way to describe a signal from the response it generates in each system. In Chapter 3, this premise was used to characterise the accelerogram for the Lorca earthquake.



▲ Figure 10-2

It might be contended that as a method for characterising a signal it is artifactual and unnecessary, for the signal (the accelerogram itself) already exists. Nonetheless, it takes little reflection to realise that an accelerogram, in and of itself, provides rather sparse information on a seismic event.

If the contrary were true, all it would take to design a new building in the area would be to enter the 2011 accelerogram as the action value.



▲ Figure 10-1

That obviously would be no guarantee of anything at all, in light of the fact that a quake with an identical accelerogram, second by second, would never occur. The city's next earthquake will generate a very different acceleration history, compared instant by instant, to the one studied here. Nonetheless, that new accelerogram would very likely look familiar and be very similar to the one at hand.

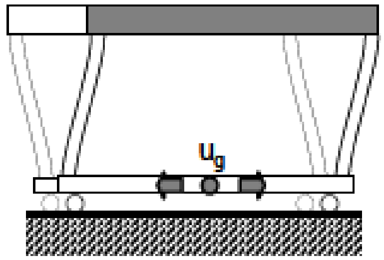
That is the idea that underlies the legal provisions in this respect. All legal codes envisage checking the safety of a structure by studying its response to real or simulated accelerograms. First, however, the spectrum for each must be calculated to determine whether it exceeds the code spectra, point by point.

Accelerograms are one of the types of records that can be studied as stochastic processes. From that standpoint, the information of interest in an accelerogram is less the simple list of points (time-acceleration) than the relationship between each point and its neighbours, its frequency content, the expectation of reaching a certain threshold and so on. In short, what is sought is a series of global parameters able to identify what all accelerograms may have in common.

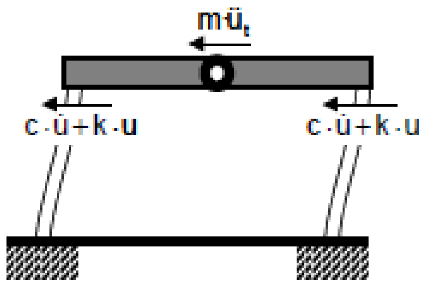
One such characterisation tool is the spectrum.

10.1. Basic formulation

Take the simplest and most representative structure, the elementary portal frame referred to throughout this book. The most intuitive way, at least apparently, to envision seismic stress would be to impose the displacement history on the base (moving the foundations the same way as the earthquake, Figure 10-3).



▲ Figure 10-3



▲ Figure 10-4

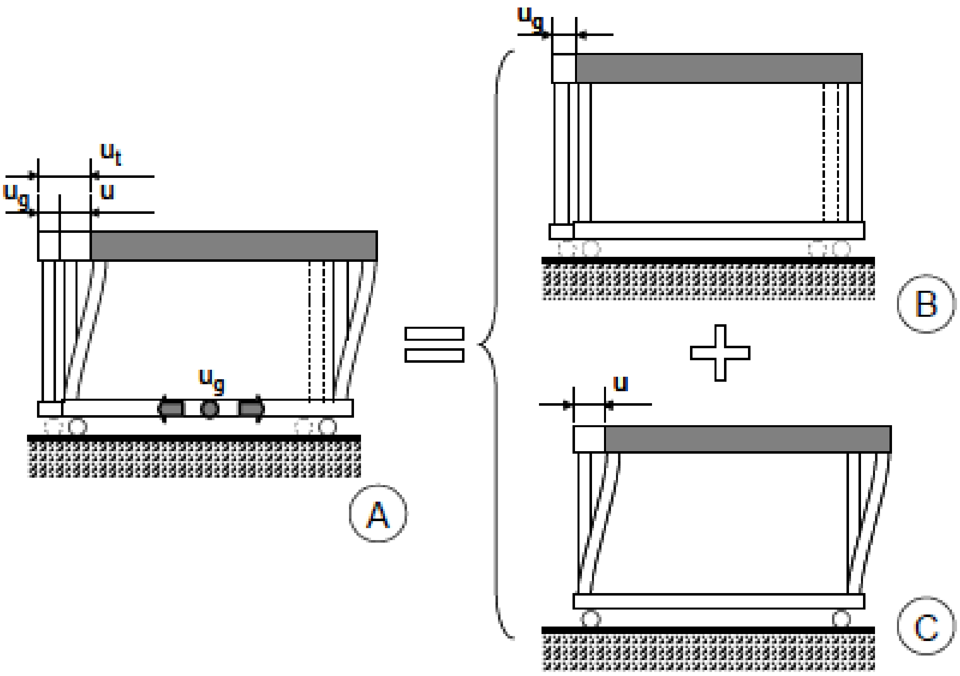
In practice, however, the balance of forces is more often applied directly to the mass (Figure 10-4). Equating the (elastic and damping) forces introduced by the columns to the inertia due to their mass yields the following equation:

$$m\cdot\ddot{u}_t+c\cdot\dot{u}+k\cdot u=0$$

The premise informing that equation is that the forces exerted on the mass by the columns are proportional to the differential drift of the former with respect to the base (Figure 10-5 C), whereas the inertial forces (in keeping with elementary mechanics) always refer to the total, i.e., the sum of the ground motion and relative displacement ($u_t=u_g+u$, as in Figure 10-5).

Performing the respective operations yields:

$$m\cdot\ddot{u}+c\cdot\dot{u}+k\cdot u=-m\cdot\ddot{u}_g$$

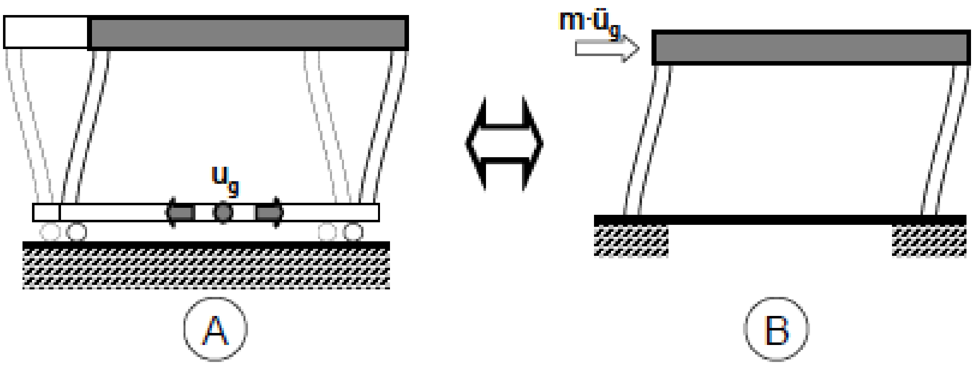


▲ Figure 10-5

While all this may appear to be a merely formal exercise with no practical implications, it helps understand the process followed.

Physically speaking, the base of the portal frame is exposed to variable motion, a more or less violent and repetitive shock (as Figure 10-6 A attempts to represent). In other words, no external force is applied to the system and the seismic action consists of variable motion at the base.

When modelled, however, this actual situation is depicted very differently. The base of the portal frame remains still and the seismic action is a time-variable force on the diaphragm proportional to its mass and ground acceleration (Figure 10-6 B).



▲ Figure 10-6

1. *Period "T" is related to angular frequency "ω" as follows: T=2·π/ω*

The problem formulated in terms of displacement is converted to an equivalent problem formulated in terms of forces, an approach more familiar to structural engineers. The change is also in keeping with the obvious fact that seismic motion has traditionally been recorded in terms of acceleration.

In any event, the equation obtained is typical of any branch of physics and is thus usually standardised as follows:

$$\ddot{u} + 2 \cdot \zeta \cdot \omega \cdot \dot{u} + \omega^2 \cdot u = \ddot{u}_g$$

Where:

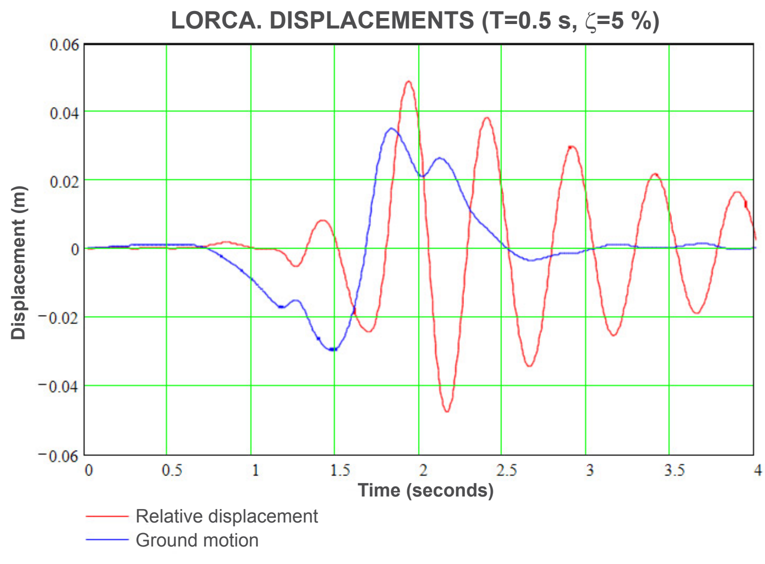
$$2 \cdot \zeta \cdot \omega = \frac{c}{m} \text{ and } \omega^2 = \frac{k}{m}$$

10.2. Characterisation of seismic action

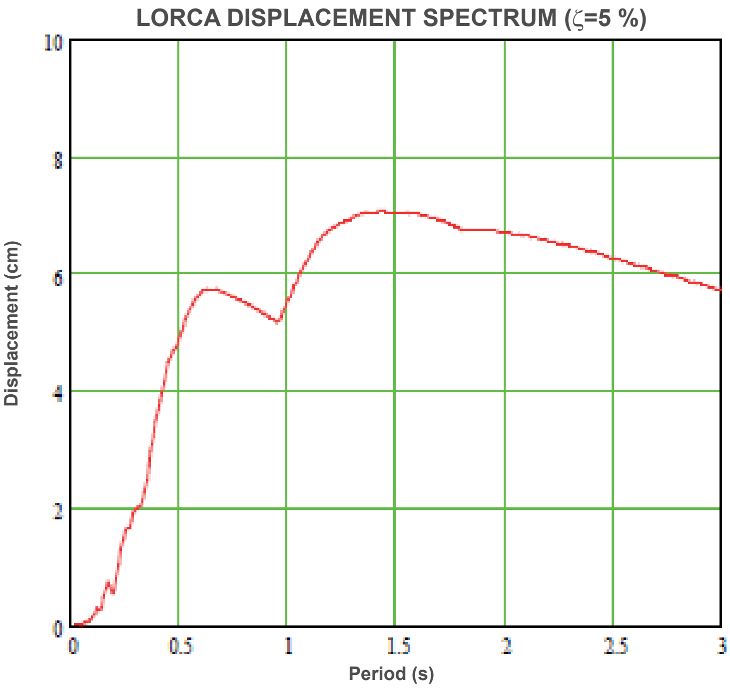
Solving the above equation for the accelerogram generated by a given earthquake is trivial. Figure 10-7 shows the result of numerically integrating the Lorca accelerogram for a system where T=0.5 s. The peak displacement obtained is 4.9 centimetres.[1]

Re-calculating the integral for systems with a different period yields the graph in Figure 10-8, called the "*displacement spectrum*", which simply represents the peak value of the displacements generated by the Lorca earthquake in systems with different periods. The ordinate for a 0.5-s period would be the 4.9 cm found above.

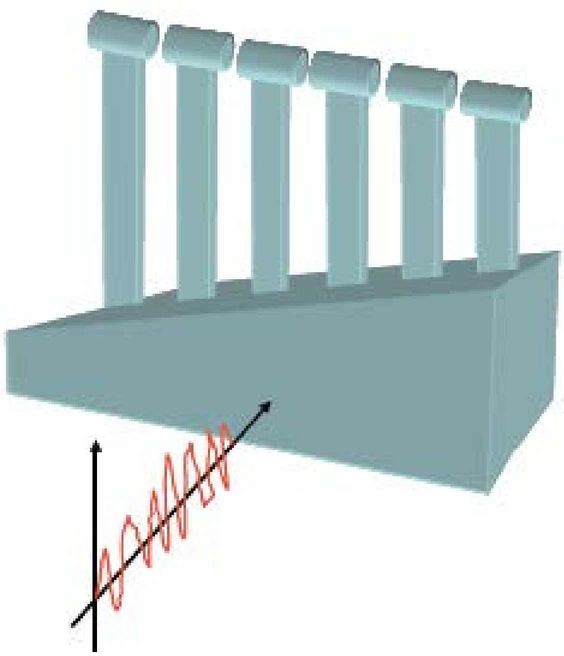
For readier comprehension of the spectrum, the reader might imagine a mechanical device as simple as depicted in Figure 10-9: a series of steel rods attached to a common base, with a small mass on the free ends and lengths adjusted to obtain a specific natural period (such as 0.1, 0.2, 0.3 s). When the earthquake acts on the base of the device, each mass moves a certain distance, giving the spectral value of its period.



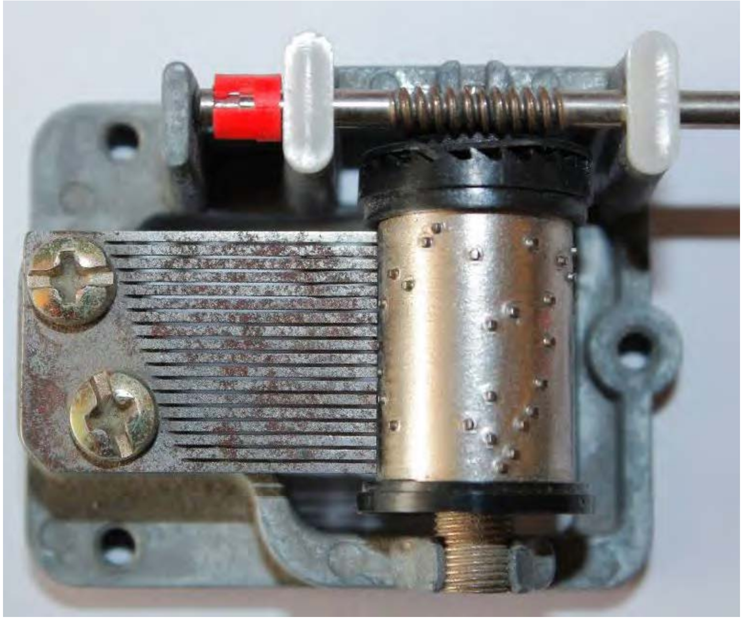
▲ Figure 10-7



▲ Figure 10-8



▲ Figure 10-9



▲ Photograph 10-1

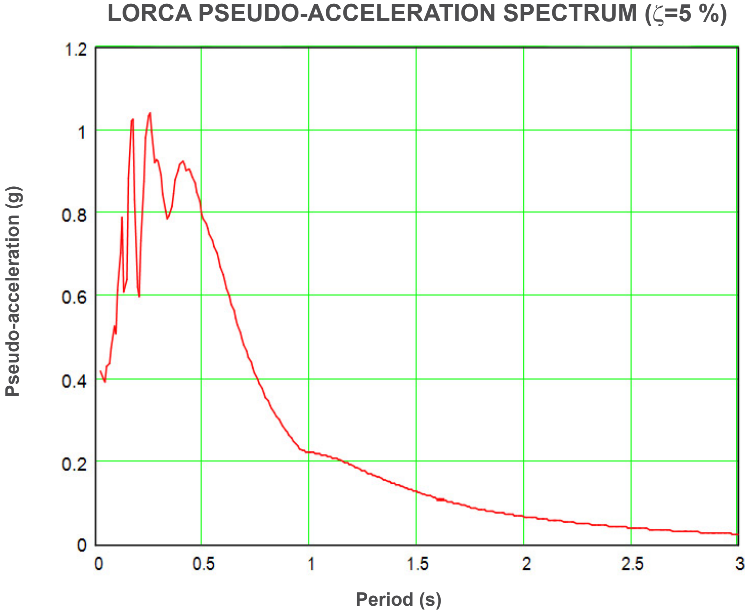
If a logger of whatever type is positioned on each mass, the result is a spectrum[2].

Be it said that the idea of adjusting the periods by increasing the length of the steel rods is not very original: it is the basic component of any music box (Photograph 10-1).

Interestingly, all displacement spectra have the same shape, regardless of the accelerogram integrated to plot them.

- They all begin at 0, for when the period is zero it is because the system either has infinite stiffness irrespective of the mass (in which case no relative displacement is possible) or no mass regardless of the stiffness (in which case no inertial forces are present to generate any displacement whatsoever).

2. *A 1948 article by Walsh and Blake, cited by Hacar and Alarcón [29] contained a photograph of such a device.*



▲ Figure 10-10

- For high period values, all such spectra tend to a relative displacement of the same value as the ground motion during the earthquake. Logically, $T \rightarrow \infty$ infers either that mass is infinite or that stiffness is nil, in which case the diaphragm does not move no matter how much the ground does.

Since structural engineering has traditionally been based on calculating systems to withstand forces rather than displacements, they are usually calculated not for the latter directly, but for the force that such displacement would induce, i.e.:

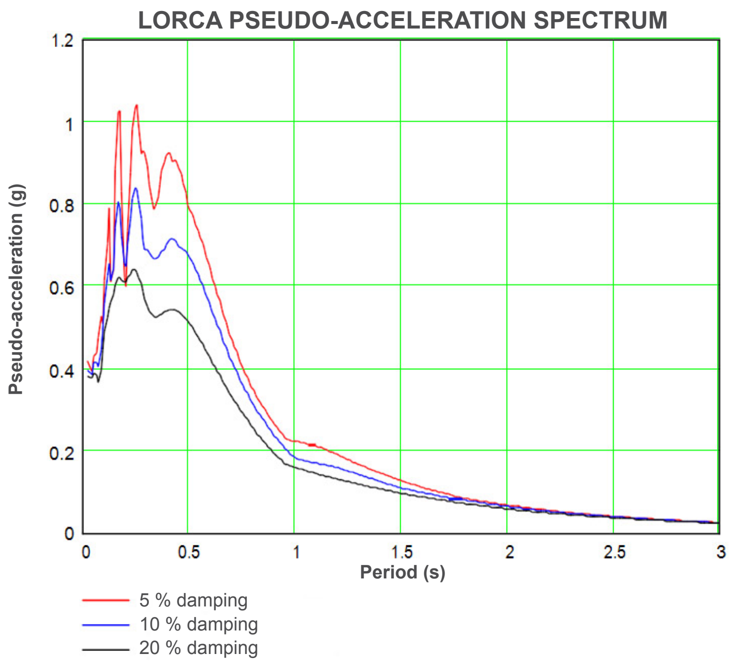
$$F_{equi}=k \cdot u_{max}$$

Since $k=\omega^2 \cdot m$, however, the expression more frequently used is:

$$F_{equi}=m \cdot \omega^2 \cdot u_{max}=m \cdot S_a$$

The term $\omega^2 \cdot u_{max}$ is usually called pseudo-acceleration (and expressed in the same unit as acceleration, m/s^2), while the graph obtained by plotting the products of multiplying the displacement spectrum ordinates by this term is called the "*pseudo-acceleration spectrum*" (although that term is often abbreviated to "*spectrum*").

Figure 10-10 shows the spectrum for the Lorca accelerogram. For T=0.5 its value would be:



▲ Figure 10-11

All of the foregoing can be generalised to systems with different damping values. Figure 10-11 shows the spectrum for 5 to 20 % damping. The greater the damping, the smaller the response, logically.

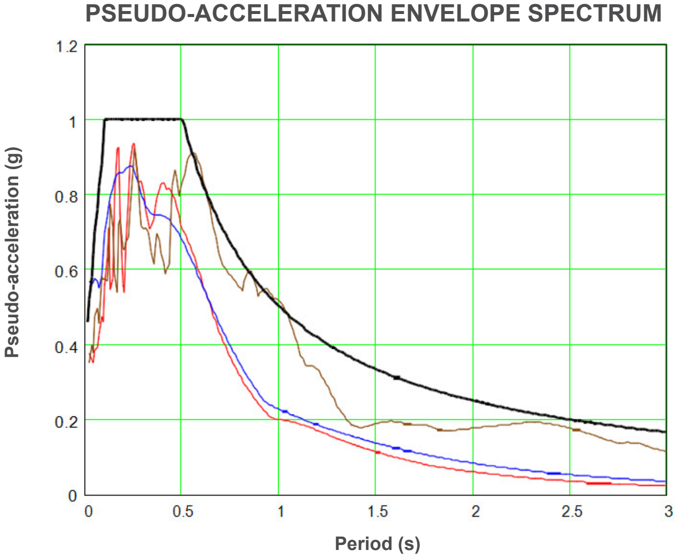
Like the displacement spectrum, the pseudo-acceleration spectrum always has the same shape.

- All the spectra start at an acceleration that concurs with the peak ground motion, for, as

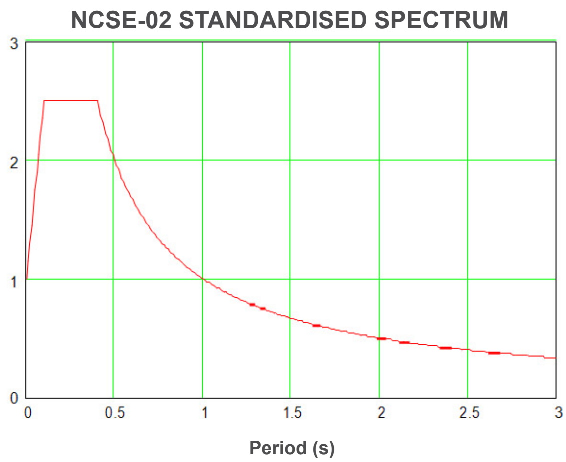
$$S_a(0.5s) = \omega^2 \cdot u_{max}(0.5s) = \left(\frac{2 \cdot \pi}{T}\right)^2 \cdot 4.875 \text{ cm} = \left(\frac{2 \cdot \pi}{0.5s}\right)^2 \cdot 4.875 \text{ cm} = 769.8 \frac{cm}{s^2} = 0.79 \text{ g}$$

explained, when the period is zero the system motion is the ground motion.

- For high period values, they all tend to nil acceleration, because then the mass does not move (relative displacement is equal to ground motion with the opposite sign).



▲ Figure 10-12



▲ Figure 10-13

In light of the above, the inherent appeal in characterising seismic loads with spectra can be readily understood. All it entails is plotting the spectra for all the available accelerograms obtained for similar sites and seeking a simple expression that envelops them all.

An example is depicted in Figure 10-12. The envelope's three branches are:

- a linear upslope from the peak ground acceleration expected at the site to the maximum amplification plateau

- a period-independent, constant amplification plateau

- a final downslope, with pseudo-acceleration inversely proportional to the period value.

10.3. Spectra in the legislation

The above reasoning explains the approach adopted by seismic legislation, which defines action on the grounds of a design spectrum. The idea is simple.

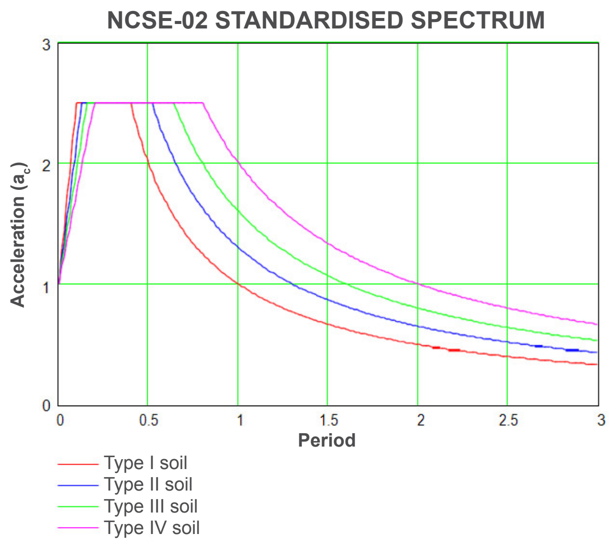
- The point of departure is a spectrum normally based on one derived from an accelerogram logged in rock with 5 % damping. Figure 10-13 reproduces the standard spectrum in effect in Spain, which is similar to any that can be found in other countries' codes (the maximum amplification value, 2.5, is found in a fair number of texts). Note that it is nearly exactly the same as the envelope spectrum described in the preceding item.

- Successive modifications are applied to the basic form described.

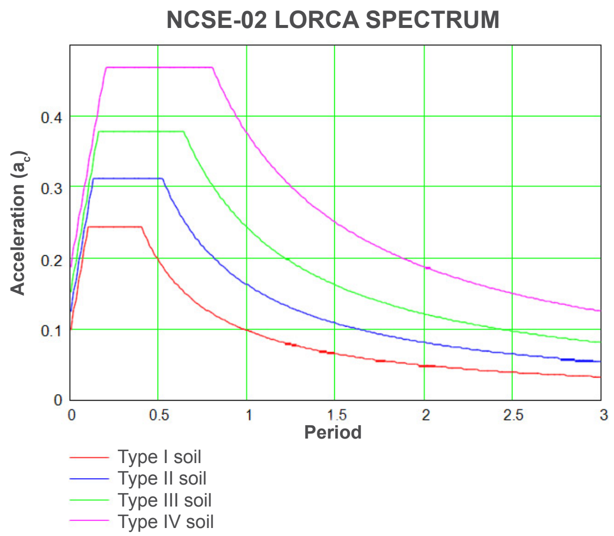
  The most significant refers to the local effect of the ground where the object of the design is to be sited. As explained in item 3.2, "Soil effects", surface layers of soft soil filter the highest frequencies, shifting the spectrum to the right (toward the longer periods). This is the effect depicted in Figure 10-14 for the standard spectrum defined in Spanish legislation. Another effect envisaged in the NCSE-02 is the proximity of the site to the Azores fault (contribution coefficient "K").

- Lastly, the ordinates are scaled by multiplying the spectrum by a constant (the design acceleration) that encompasses area seismicity (basic acceleration), importance of the building ("ρ" factor), soil type and other correction factors. Although the result would be expressed as acceleration, it is frequently normalised to gravitational acceleration, as in Figure 10-15, which shows the spectrum for the Lorca earthquake.

The end result of all the foregoing is a very appealing method for characterising earthquakes as well as a procedure for structural analysis. Given the spectrum for an earthquake, the safety of any structure in such a quake can be readily calculated. The mere static calculation involved is performed by applying an equivalent load whose value is the product of the mass by an acceleration term. The approach is, moreover, intuitive because the force apparently exerted on the diaphragm is no more than the product of its mass by the peak acceleration reached. That is actually not the case (except where damping is nil) and in any event the acceleration involved would be the total, i.e., the sum of the ground and relative acceleration.



▲ Figure 10-14



▲ Figure 10-15

10.4. The role of ductility

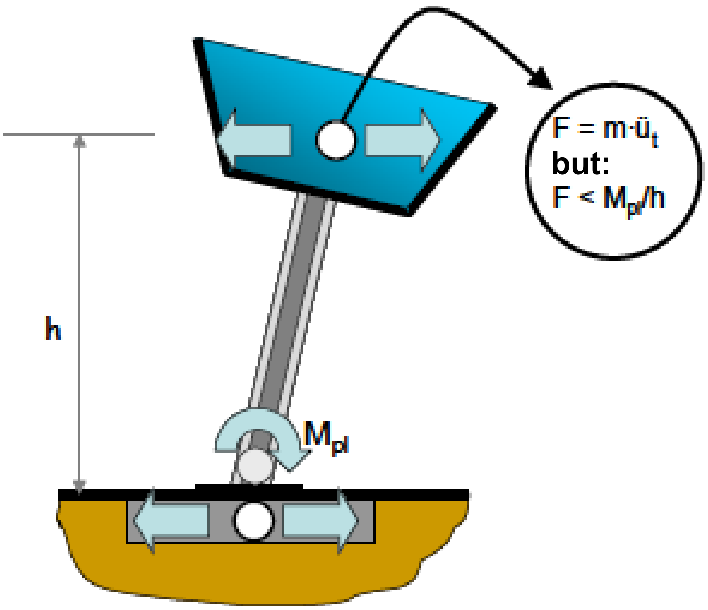
The equivalent loads deduced from elastic spectra are so high that their direct application to structures would call for extraordinarily robust designs. The cost of the structures needed to resist such forces elastically would be very high, even in medium seismicity areas.

The alternative put forward in the nineteen fifties consisted of allowing structures to plasticise. Take the simplest possible case, the water tower depicted in Figure 10-16. If the tank is likened to a point mass and damping is disregarded, the force acting on it is the product of its mass times the total acceleration.

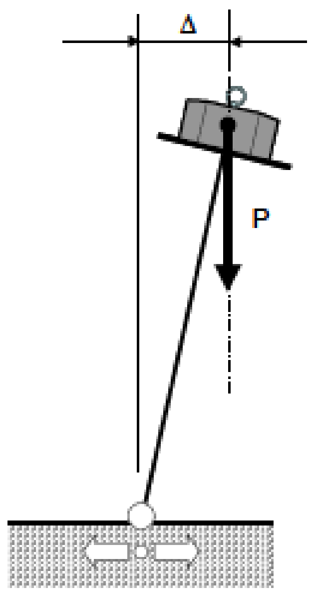
Logically, that force should not be greater than the force that induces plasticisation at the base of the shaft, because otherwise the system would not be in equilibrium. To put it another way, the force imposed on the tank by the hinge cannot be greater than the plastic moment divided by the height.

From this standpoint, it would appear to be reasonable to reduce the bending strength at the base of the shaft and thereby limit the seismic loads that reach the tank at the top. Such a reduction would naturally have to be modest because it could affect other types of horizontal loads (wind, imperfections) or even vertical loads (through the P-Δ effect, which consists of an increase in the moment at the base as the result of tank drift). That, in turn, would induce additional bending, quantified as the product of the weight times the drift (Figure 10-17).

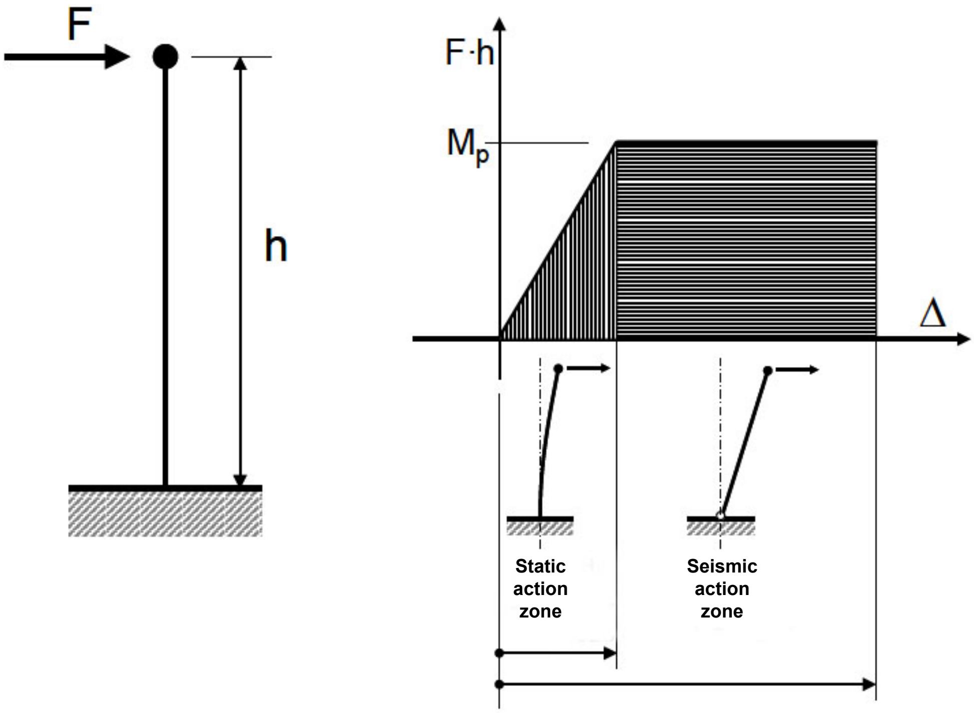
Such a simple idea clashes with the traditional, force-based approach to the problem.



▲ Figure 10-16



▲ Figure 10-17

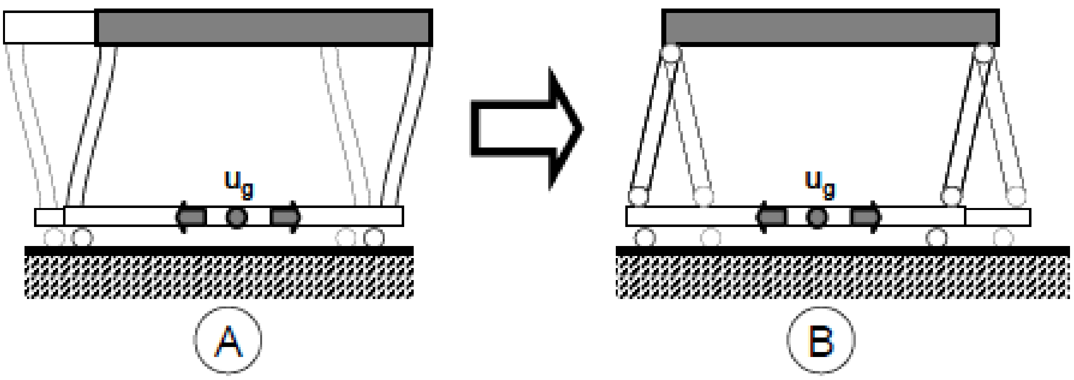


▲ Figure 10-18

Structural engineers, used to static calculations in which failure consists of the formation of a mechanism that structures are dimensioned to avoid, find it difficult to assimilate the idea that in seismic design the criterion is the opposite: the mechanism should not only form but it should even be designed to move (i.e., the hinge should rotate, Figure 10-18).

In this regard, the change referred to in the first item to this annex, from a formulation based on displacement to one based on forces, is of little help.

Indeed, the idea is easier to explain from the perspective of displacements. If, in the portal frame depicted in Figure 10-3 (reproduced, for the reader's



▲ Figure 10-19

3. *Such displacement is not very significant, on the order of 3 cm per side at Lorca.*

convenience, in Figure 10-19 A) column bending strength is reduced until hinges form (plain hinges, not even plastic hinges, with some bending capacity), the structure is converted into a mechanism.

When the base of the mechanism is shaken, what occurs no longer looks like the drawing in Figure 10-19 A. The diaphragm remains immobile in its initial position (Figure 10-19 B), while the value of its relative displacement at any given time is equal in value but opposite in direction to displacement at the base (the ground[3]). The total is consequently nil.

In short, while the logical way to absorb forces is to build strong, stiff structures, the reasonable way to absorb displacements is with mechanisms.

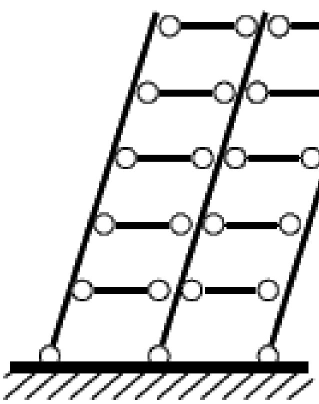
In fact both schemes are needed, because in earthquakes static loads (at least gravity) and displacements (the earthquake itself) act simultaneously. For that reason, columns cannot be totally hinged, for they must conserve the necessary plastic moment strength to at least absorb the bending moment induced by the P-Δ effect.

That simple idea constitutes the basis of modern seismic analysis, at whatsoever level.

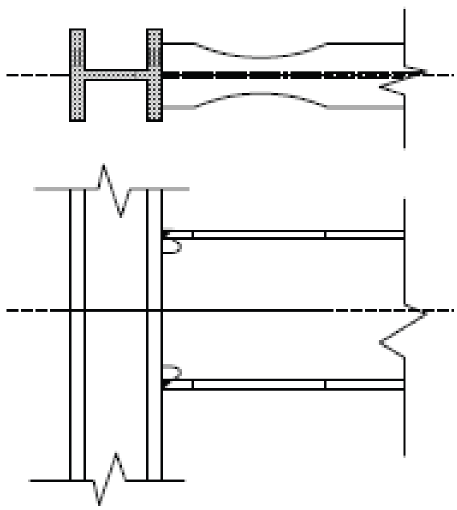
At the most general level, the basic aim of conventional earthquake-resistant design is to formulate a plastic mechanism. Real buildings are obviously somewhat more complex than the simple portal frame used in this example, and consequently the associated plastic mechanisms are likewise more elaborate (Figure 10-20).

Locally, the idea of reducing section strength to induce its plasticisation and thereby control loads at the base is known as "*calculation for capacity*". An example follows.

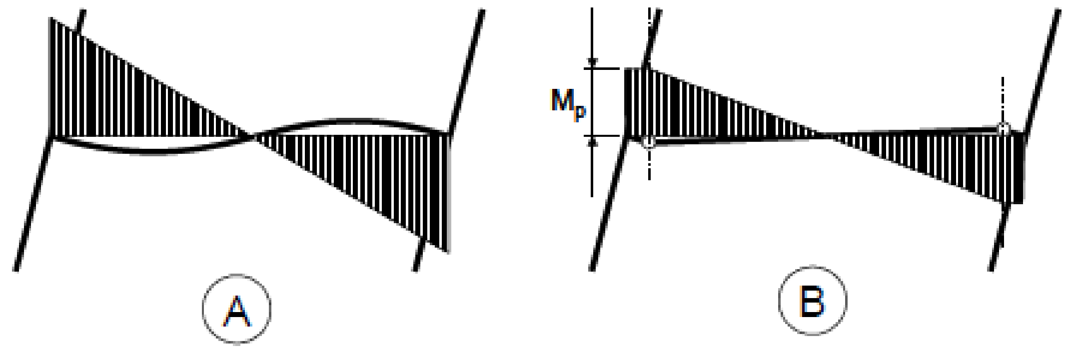
The 1994 Northridge earthquake is often cited as one of the turning points in seismic engineering, especially as regards steel structures. Many beam-column connections were observed to have failed during the quake due to rupture in the flange welds.



▲ Figure 10-20



▲ Figure 10-21



▲ Figure 10-22

The solution proposed, and generally applied today, consists not of strengthening the welds but of "*weakening*" the ends of the beams by narrowing the width of the flanges (to form "dogbones" Figure 10-21).

That arrangement controls the moment reaching the connection, which is the plastic moment in the reduced section (Figure 10-22 B).

As noted in earlier chapters, in seismic zones elements exhibiting ductile failure should not be strengthened.

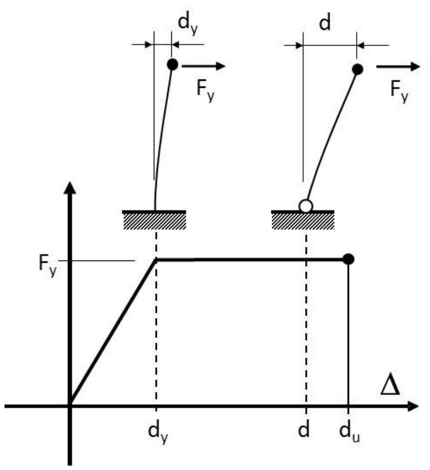
One last remark in this regard refers to the failure criterion. When action is defined as force, the limit is very easy to define. When it is defined in terms of displacements, however, such a definition is more complex. Coming back to the simple cantilever in Figure 10-18, some criterion would be needed to calculate the maximum rotation of the plastic hinge at its base, but that is no easy task. As a rule, displacement is not used directly, but normalised to yield strength values.

Ductility is then defined as:

$$\mu = \frac{d}{d_y}$$

Where "d" and "d_y" are the total and the elastic displacements, respectively.

According to this new point of view the design of a structure would not be based on the conventional comparison between the internal forces acting on each section the displacements induced by the earthquake and the displacements that can be accommodated by the structure. That comparison is usually posed in terms of ductility, as the difference between "ductility demand" and "ductility capacity".



▲ Figure 10-23

10.5. Ductility reduction factor

All of this poses a problem.

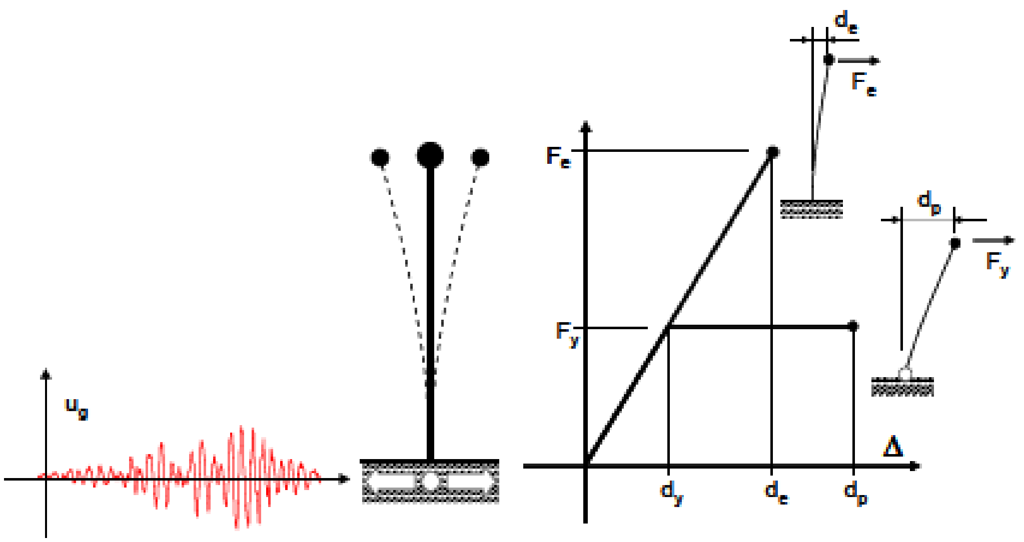
On the one hand, the extraordinarily simple and effective analytical method proposed, based on response spectra, assumes that the structure behaves elastically. That leads to stresses much too high to be used in the design, for they would call for an overly robust and costly building. On the other, an acceptable solution is at hand, but based on structural plasticisation and therefore incompatible with the use of response spectra. Plasticisation assumes obvious non-linearity, which invalidates all the assumptions on which the method is based.

For want of better alternatives, the procedure adopted is to apply the equivalent forces resulting from the elastic spectrum in a likewise elastic calculation. The ploy used is to reduce such forces by a factor known as the "*ductility factor*" in Spanish code NCSE-02 and "*behaviour factor*" in the Eurocode on earthquake-resistant design.

This can be explained with the elementary example used above: a simple oscillator consisting of a shaft with a mass concentrated at the top (the overhead deposit or the simple portal frame in earlier figures), whose base is affected by seismic displacement.

If the shaft does not plasticise at any time during the earthquake, the situation generated is as described in detail above: mass displacement reaches a peak value, "d_e", which gives rise to an equivalent force "F_e" (the subscript refers to the elastic regime, Figure 10-24).

If the capacity of the shaft is limited so that it plasticises under a force "F_y" and its displacement in the same earthquake is re-calculated, the value obtained, "d_p" differs as a rule from the elastic regime value.



▲ Figure 10-24

In this context the relationship $\mu = d_p/d_y$ is known as "*ductility demand*" and the quotient F_e/F_y as the force reduction factor[4].

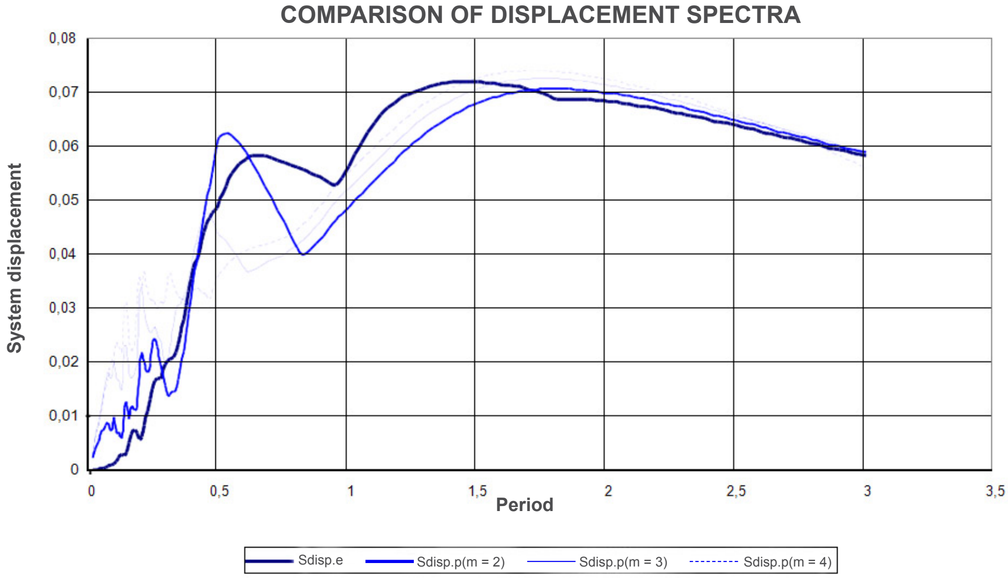
Ductility demand is equivalent, in displacement terms, to the design forces used to calculate structures in terms of forces. Similarly, the term "*calculation for ductility*" is used to refer to peak relative displacements.

No relationship exists between ductility and the force reduction factor. The displacement spectra for the Lorca accelerogram reproduced in Figure 10-25[5] that refer to systems with different force reduction factors prove the inexistence of any real correlation.

An analysis of a large number of spectra for real accelerograms integrated in the elastic and plastic regimes revealed that the spectra in Figure 10-25 fit a general pattern. In flexible systems, i.e., for high period values, displacements in the elastic and plastic systems are similar (Figure 10-26 A), whereas in stiffer systems, which lie on the amplification plateau on the spectrum, displacements are generally greater in the plastic than in the elastic regime (Figure 10-26 B).

4. *Although with some minor differences, it concurs with the behaviour or ductility factor.*

5. *This figure was prepared by José Ramón Arroyo, industrial engineer working out of INTEMAC.*



▲ Figure 10-25

In this case plastic displacement generates the same energy as elastic displacement, i.e., in Figure 10-26 B:

$$\frac{1}{2} f_e \cdot d_e = f_y \cdot \frac{d_p + (d_p - d_y)}{2}$$

Performing the respective operations yields:

$$\frac{f_e}{f_y} \cdot d_y \cdot \frac{f_e}{f_y} = 2 \cdot d_p - d_y$$

from which it follows that:

$$\frac{f_e}{f_y} = \sqrt{2 \cdot \mu - 1}$$

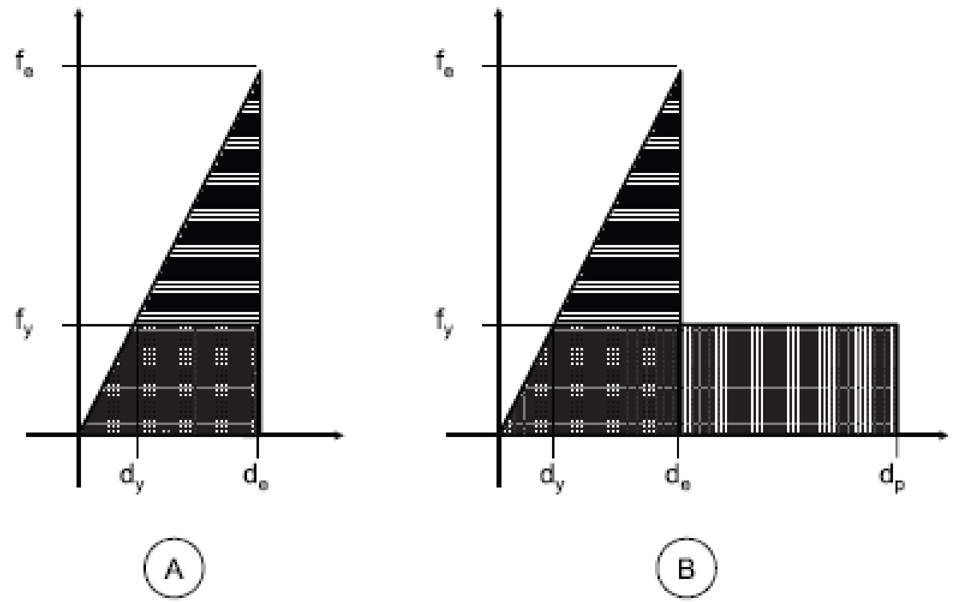
This type of behaviour is normally known as the "*equal rule*".

The practical implication of all this is that to reduce forces by a certain amount, a variable proportion of system ductility must be ensured, which is greater in stiff systems. Hence, if the system is flexible and the aim is to reduce the equivalent force to one-fourth of the elastic force, the calculations must ensure a displacement ductility factor of 4. The same reduction in a stiff building, however, would require a ductility factor of 8.5.

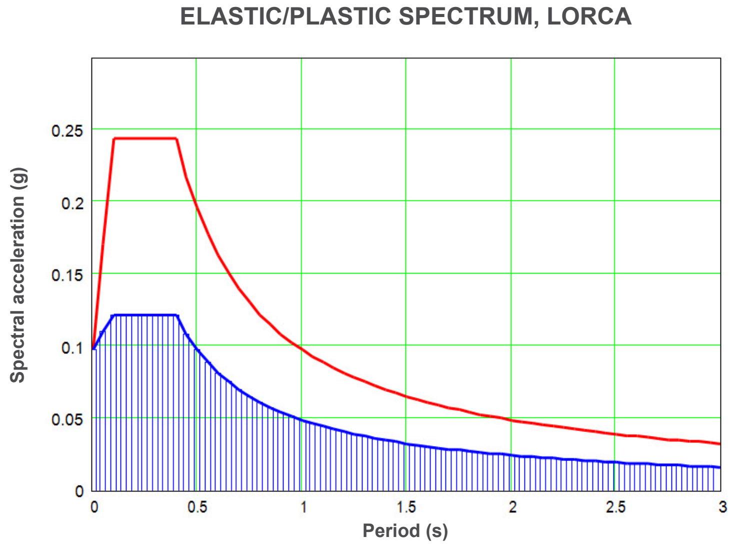
At the lower values of the spectrum ductility requirements climb dramatically. In the extreme case, i.e., when the period tends to zero, the entire displacement would be plastic (elastic displacement would be nil) and ductility would tend to infinity.

The ultimate reading of the foregoing is that when buildings are very stiff it is difficult to attain sufficient ductility to reduce force to any perceptible extent.

The legislation incorporates this idea by reducing the elastic spectrum by a variable factor.



▲ Figure 10-26



▲ Figure 10-27

Figure 10-26 shows the elastic (red curve) and plastic (blue curve) spectra for a ductility factor of 2 in Lorca. Note that the required ductility factor is only met for medium and high periods. For the lower periods, linear interpolation is performed until nil reduction is reached with T→0.

10.6. Conclusions

While response spectra constitute an excellent procedure for characterising earthquakes, their utility in structural analysis is clearly debatable. Design spectra, based on reducing forces in keeping with the ductility of the structure, actually lack any sound theoretical justification. Even elastic spectra, which might be more readily justified, are clearly a simplification of the seismic action in which such essential elements as quake duration are omitted.

Moreover, application of the method to more complex structures than the simple oscillators taken as examples in the foregoing entails resorting to highly questionable procedures such as modal combination criteria. Obviously, then, the response spectrum method is not ideal. And yet it has been the benchmark procedure for over 50 years, despite the advent in the interim of many other approaches that are at least more rigorous. The reason for its longevity lies in its ease of use, an advantage that offsets its many shortcomings.

Annex II
Proposed repair procedure

11

11. Annex II
Proposed repair procedure

The procedure for repairing locally fractured columns with exposed buckled reinforcement is discussed in this annex.

In such cases the damaged concrete must be removed and the initial section restored.

11.1. Preparatory operations

No action may be taken in the area until the respective safety calculations are performed, which may be based on the use of temporary braces or calculation of section residual capacity during repair.

Where the top of a column needs repair, all necessary access facilities (scaffolding) must be in place before action is initiated.

11.2. Procedure

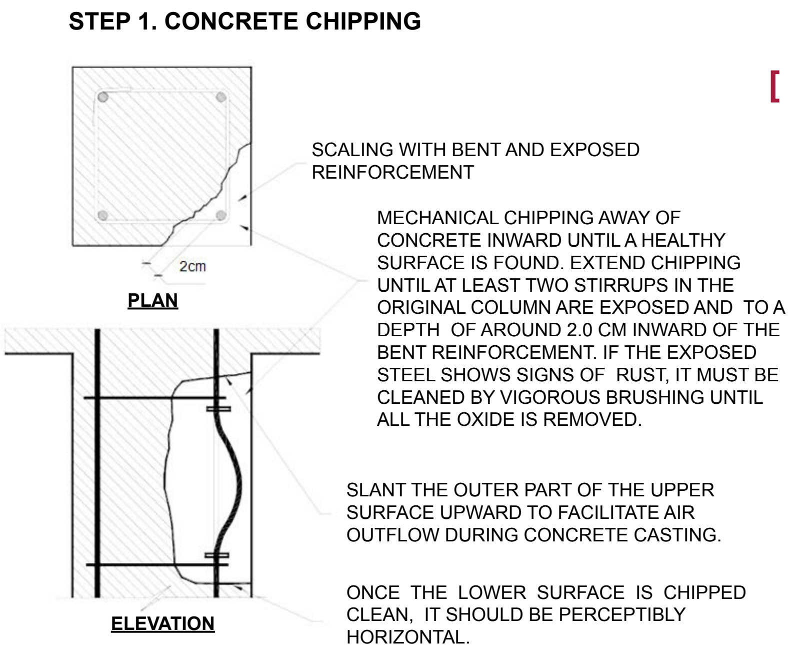
The procedure for repairing partially fractured columns is described below.

The concrete in the area affected must be removed with a light chipping hammer, carving out a space bounded by vertical and perceptibly horizontal surfaces (the latter should be slightly slanted as shown in Figure 11-1 to facilitate outward air flow during concrete casting).

Any reinforcement bars so deformed that they have separated off the concrete core must be cut away. See Figure 11-2.

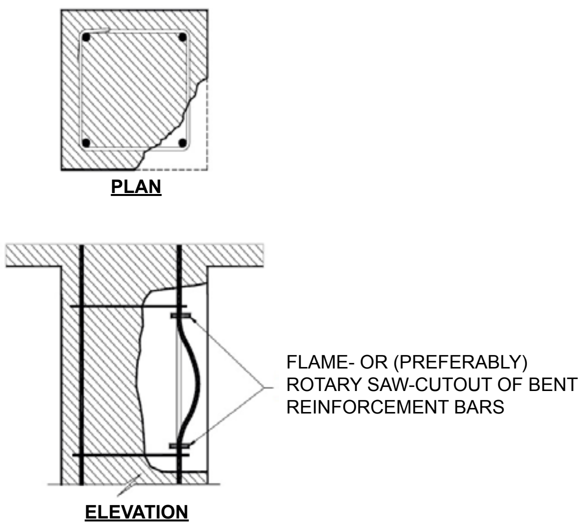
A rotary saw should preferably be used, although flame cutting is admissible in bars measuring 14 mm in diameter or less.

Two bars of the same diameter as the bar removed are positioned on each side of the two butt ends. One of the new bars is then welded to the butt ends with a weld no shorter than 5 diameters if both sides of the bar can be welded (which a priori would appear to be fairly difficult) or 10 diameters otherwise, which is more likely to be the case. Once the slag is removed and the weld cools, the same procedure may be repeated with the other bar.



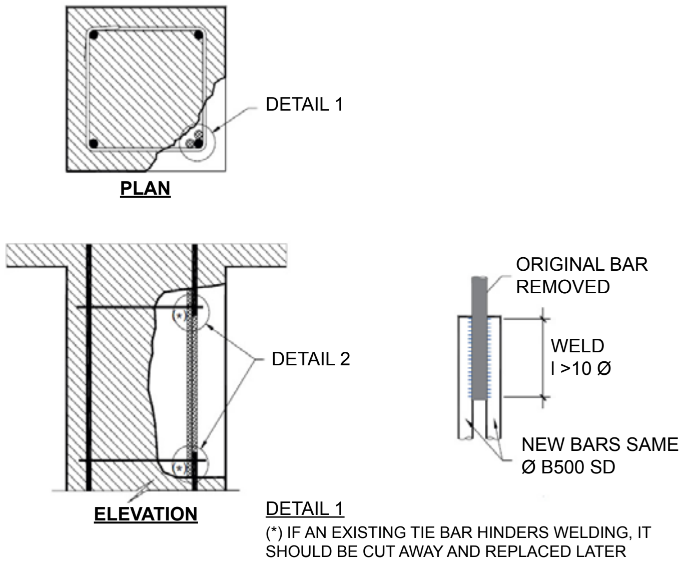
▲ Figure 11-1

STEP 2. CUTTING BENT REINFORCEMENT



▲ Figure 11-2

STEP 3. CUTTING BENT REINFORCEMENT



▲ Figure 11-3

Care should be taken to ensure that the weld beads for the second bar join it to the butt ends of the bar that was removed, and not to the first new bar. See Figure 11-3.

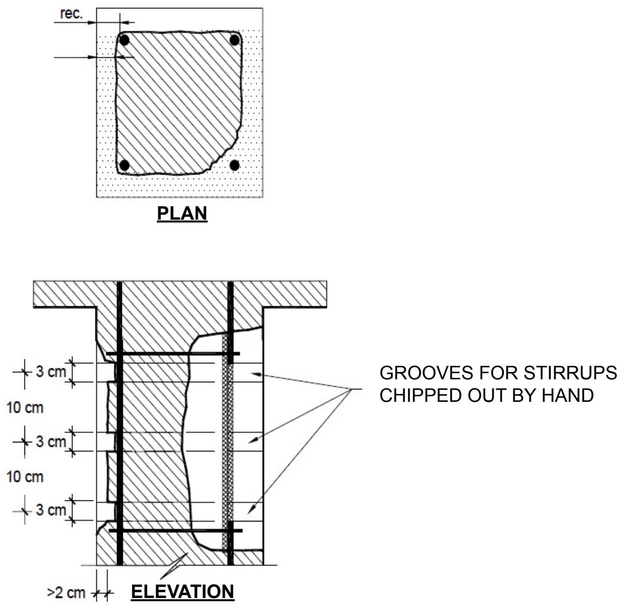
Where the original tie bars hinder welding, those bars are cut away and replaced in a subsequent step.

If the distance between the top cut and the bottom of the beam-column node is insufficient to lap-weld the bars, they can be welded continuously, in which case the ends of the two bars must be duly prepared. This procedure should be used only exceptionally, however.

Welding should be performed by qualified welders only and preferably by welders with experience with that type of steel. Special care must be taken to comply with the original specifications to minimise the rise in bar temperature.

Grooves are then drilled into the accessible sides of the column, approximately three centimetres high and to a depth equal to the depth of the concrete cover. They are spaced equally across the repair area at no more than 10 cm from one another. The concrete between the grooves is removed to a depth of at least 2 cm. See Figure 11-4.

STEP 4. PREPARATION OF SURFACE TO POSITION TIE BARS



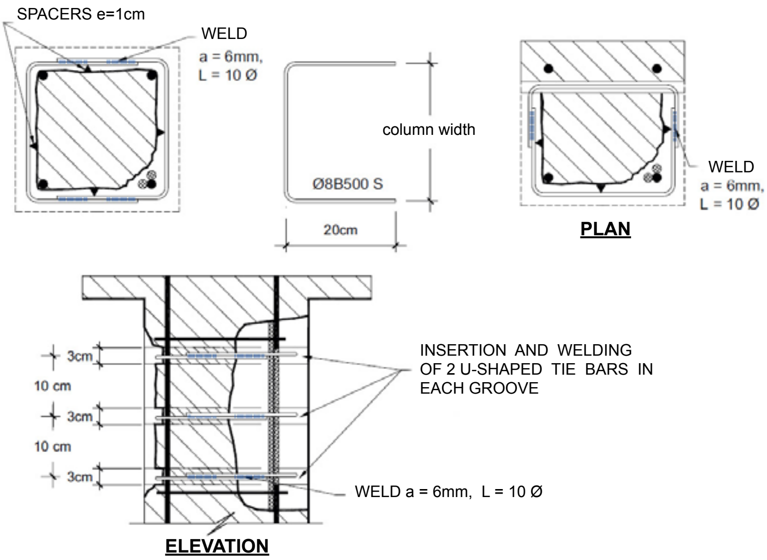
▲ Figure 11-4

Where expansion joint columns are involved, horizontal holes 12 mm in diameter are drilled at the same height as the grooves, parallel to the joint and at a sufficient distance from it to be inward of the main reinforcement. These holes are cleaned out with compressed air and filled with injected thixotropic epoxy resin immediately before proceeding to the next operation.

Two U-shaped, B 500 SD steel stirrups 6 mm in diameter are then inserted into each groove from opposite ends and lap-welded at the open ends. In expansion joint columns, one of the two components, which should be L-shaped, is threaded through the holes made in the column before the specific anchorage resin hardens and subsequently bent to overlap with the other component. See Figure 11-5.

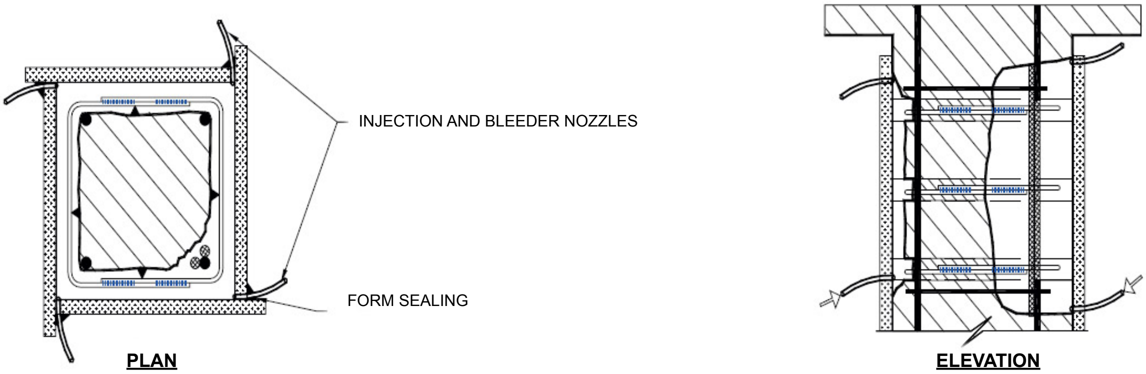
After careful preparation and cleaning of the concrete surfaces and positioning of the stirrups in the grooves, a water-tight form fitted with injection and bleeder nozzles is placed around the repair area to inject fill into the void.

STEP 5. INSERTING STIRRUPS



▲ Figure 11-5

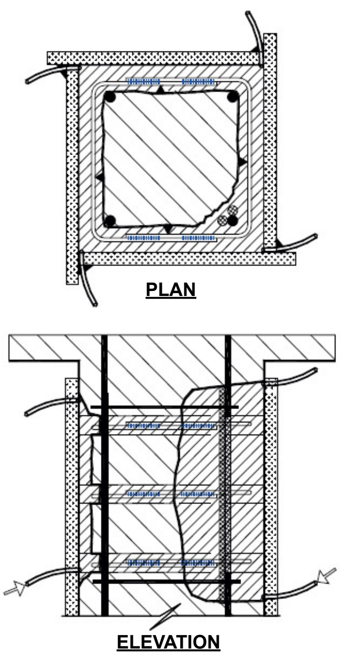
STEP 6. POSITIONING FORMS AND INJECTION NOZZLES



PLACEMENT OF FORMS GREASED ON THE COLUMN SIDE, AND FITTED WITH INJECTION AND BLEEDER NOZZLES FOR SUBSEQUENT FILL

▲ Figure 11-6

STEP 7. INJECTION



▲ Figure 11-7

Water-tightness can be ensured by sealing the form joints with a material compatible with the product to be injected. See Figure 11-6.

The grooves and voids are then filled with a high bond strength injection mortar at a pressure of 2.0 to 4.0 N/mm², depending on the formula and pursuant to supplier specifications. The material is injected through the nozzle in the lowest position. See Figure 11-7.

Lastly, the forms are stripped and any finishing operations needed are performed. See Figure 11-8.

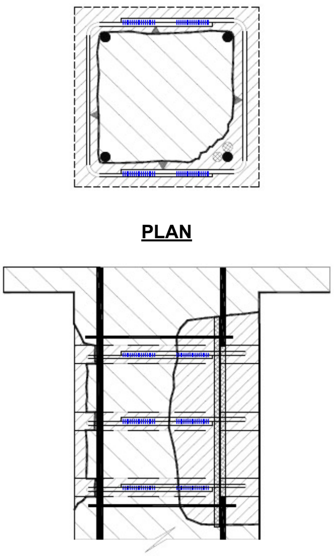
11.3. Control

A written record should be made of the products to be used, their suitability for the intended purpose and the presence of the CE quality mark.

Where no safety bracing is installed, the various stages of the repair must be conducted in the presence of authorised site management engineers or architects and special care must be taken to ensure that the depths specified for the grooves described above are honoured.

Workmanship control includes direct inspection of the first repair performed and at least 25 % of all the remaining repairs. In addition, strengthening quality must be controlled in all the expansion joint columns.

STEP 8. FINISH



PLAN

ELEVATION

▲ Figure 11-8

Annex III
The *Consorcio de Compensación de Seguros* and insurance cover for earthquake damage

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12. Annex III
The Consorcio de Compensación de Seguros and insurance cover for earthquake damage

By Alfonso Nájera Ibáñez
Consorcio de Compensación de Seguros

In Spain fortunately, destructive earthquakes are fairly uncommon and neither as frequent nor as intense as the seismic events that afflict other areas of the world. In practice, flooding and wind storm are the risks that account for the country's largest volume of natural catastrophe-induced casualties. Nonetheless, judging from Spain's history of high intensity quakes, which may well recur, the greatest potential loss in this regard may be attributed to earthquakes.

The historical series need not be taken back overly far: the quake that razed Lisbon in 1755 originated a tsunami that battered the south coast of the Iberian peninsula, impacting the Spanish provinces of Huelva and Cadiz in particular, whose death toll was over one thousand. In 1884, an earthquake with a magnitude of 6.7 and intensity IX caused substantial damage in Arenas del Rey, a town in the Spanish province of Granada, and smaller nearby villages, killing around 900 people. Less severe tremors were recorded then and later, and within our own recent memory, on 11 May 2011, the earthquake that shook Lorca (a city in the Spanish province of Murcia) with a magnitude of 5.1 (Mw) and a maximum intensity of VII (Mercalli scale) left nine fatalities in its wake, as well as over 900 injured and considerable damage to homes, shops and historic buildings. The Consorcio de Compensación de Seguros (CCS) has received 32 500 claims in connection with that event, on which it has paid nearly 462 million euros. That was the highest earthquake-induced burden of claims that the CCS, whose origins date back to the immediate post-Spanish Civil War (1936-1939) period, has ever had to shoulder. It was followed in scale by the 2 February 1999 Mula earthquake (also in the province of Murcia), when CCS honoured 6 852 claims totalling 15.9 million euros (adjusted to 31-12-2011).

Spain has an internationally reputed system for insuring against natural catastrophes, its Extraordinary Risk Insurance scheme. It is handled by CCS, a State-managed fund under the aegis of the Ministry of the Economy and Competitiveness, through its Directorate General of Insurance and Pension Funds.

Spain's Extraordinary Risk Insurance covers damage caused by natural events or acts of violence. The former include flooding, earthquakes and tidal waves, atypical cyclonic storms (winds of 120 km/h or greater), volcanic eruptions and the impact and falling of meteorites and astral bodies. The latter include, among others, terrorism and civil commotion.

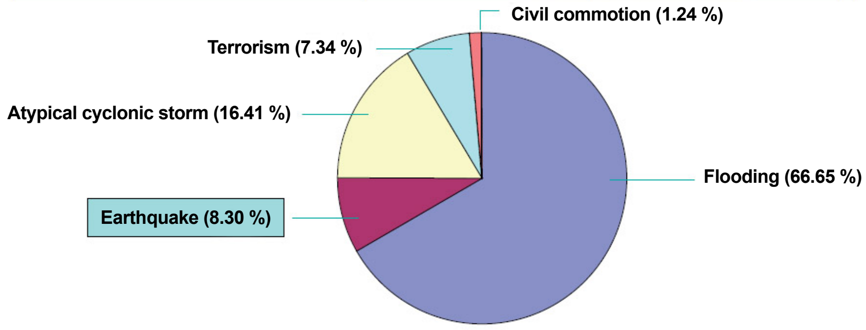
In Spain, Extraordinary Risk cover is mandatory in most classes of property insurance[1], as well as in personal accident and life insurance policies. In other words, if the company underwriting a policy does not explicitly assume such risks, any loss affecting the persons or property[2] protected is automatically assumed by the Consortium up to the amounts insured, providing the casualty is occasioned by one of the aforementioned events. To that end, when such events occur and CCS receives claims from the insured concerned, it proceeds to appraise the damage and pay the respective indemnity[3].

That has two implications. First, CCS only pays indemnity for loss affecting persons with a policy in those classes of insurance; and second, CCS writes no policies itself. Rather, it bases its payments on the policies issued by private companies, which collect a surcharge with their premiums for CCS cover. Be it said, however, that protection against extraordinary risk is wholly independent of the other risks covered by the policy, although it refers to the same property, persons and sums insured.

Property damage, lost profit and personal casualties
Claims and indemnities by cause, 1987- 2011

Amounts in euros (at constant 31-12-2011 value)

Cause	No. claims	Payments	%
Flooding	416.237	3.946.715.136	66,65
Earthquake	42.683	491.232.017	8,30
Atypical cyclonic storm	417.074	971.361.480	16,41
Impact and falling of astral bodies	3	98.429	0,00
Terrorism	22.174	434.883.578	7,34
Insurrection	152	1.096.493	0,02
Civil commotion	5.951	73.127.999	1,24
Acts of armed forces in times of peace	1.086	2.613.164	0,04
Other	-	-	-
	905.360	5.921.128.296	100,00



Source: Consorcio de Compensación de Seguros

1. Property damage includes both direct material loss, i.e., destruction or deterioration of the property insured as a direct result of the catastrophe in question, and business interruption stemming from direct damage, subject to its inclusion in the policy. No other types of derived or consequential losses are covered.

2. Property damage includes both direct material loss, i.e., destruction or deterioration of the property insured as a direct result of the catastrophe in question, and lucrum cessans stemming from direct damage, subject to its inclusion in the policy. No other types of derived or consequential losses are covered.

3. Except for automobiles, housing and freeholds, indemnities for direct property damage are subject to a 7 % deductible. In personal insurance (accident, life) no deductible whatsoever is applied, and in the event of lost profit, the applicable deduction is as laid down in the policy.

Further to the Extraordinary Risk Insurance Regulations[4], an earthquake, for the intents and purposes of CCS cover, is characterised by “abrupt shaking of the soil propagated in all directions and caused by movement in the earth’s crust or deeper”. Unlike the provisions of earthquake cover systems in some other countries, that broad definition establishes no minimum magnitude or intensity values.

As the table below shows, in 1987-2011, the Consortium paid out a total of 491.23 million euros (at constant 31-12-2011 value) for earthquake damage, distributed across 42 683 claims. That sum, which includes indemnity for property damage, business interruption and personal casualties, accounts for 8.3 % of all the indemnities paid by CCS in the period as a result of extraordinary risk-induced losses. From 1987 to 2011, earthquakes constituted the third most common cause of claims, after flooding (66.7 %) and storms (16.4 %) and ahead of terrorism (7.3 %).

Other insurance systems specifically designed to cover natural catastrophe-induced damage are in place in other areas. The solutions involved are very diverse, for they are meant to respond to situations that vary enormously from country to country in many respects: types of natural hazards to which they are exposed, degree of economic development, insurance market structure, insurance culture and so on.

In light of the potential damage attributable to natural catastrophes and the financial wherewithal and management acumen required to guarantee indemnities, a fair share of these systems entails some degree of public participation. This is the case of the systems operating in Belgium, California (that state only), the Caribbean, Denmark, France, Iceland, Japan, New Zealand, Romania, Switzerland, Taiwan, Turkey, United States (federal law) ... as well as Spain[5].

The cover provided by such systems is mandatory in most cases. That requisite is associated either with home ownership (Switzerland, Iceland, Romania, Turkey) or taking an ordinary policy (Belgium, Denmark, France, Japan, New Zealand, Spain, Taiwan). The natural risks covered by the guarantee likewise vary over a wide spectrum. Some systems provide single coverage: Denmark, (sea floods), Japan (earthquake for homes), Taiwan (earthquake for homes), Turkey (earthquake for homes), United States (flooding anywhere in the country), California (earthquake within the state boundaries) and Florida (hurricane within the state boundaries). The French system affords cover for all risks regarded by the market to be uninsurable, while Spain has a multi-cover system that includes the risks listed above.

Insurance schemes differ depending on the object (in Japan, New Zealand, Romania and Turkey, for instance, they cover housing only), as well as on the type of damage covered.

4. *Enacted under Royal Decree 300/2004 of 20 February and amended by Royal Decrees 1265/2006 of 8 November and 1386/2011 of 14 October.*

5. *See htpp:// www.wfcatprogrammes.com.*

Some include direct property damage only (Iceland, Japan, New Zealand, Turkey, United States), while others, in addition to direct damage, indemnify for business interruption (Denmark, France, Norway). Only the Spanish system covers direct damage, business interruption and personal casualties.

Moreover, most schemes (except France and Spain) have an indemnity ceiling and many benefit from State backing, unlimited in some cases (France, New Zealand, Spain).

While many other differences can be identified among the various schemes, the Spanish system, managed by the Consorcio de Compensación de Seguros, may be concluded to be among the most comprehensive from the standpoint of number of risks, diversity of interests and variety of damage covered.

In certain (normally the most advanced) countries and markets, the private sector offers insurance arrangements. Such arrangements provide a greater or lesser breadth of cover (in terms of property protected, damage guaranteed, indemnity ceilings, types of natural hazards covered and so on) and normally resort to reinsurance protection or other money market instruments (catastrophe bonds, for instance). Such financial risk management resources are also used by a significant share of the systems with State participation.

CLAIM SUBMISSION

In the event of injury to people or damage to property resulting from any of the risks covered by CCS, the insured, policy holder or beneficiary or their respective legal representatives should submit their claims as soon as possible in one of the following manners:

By telephone, on 902 222 665, Monday through Friday, from 9.00 AM to 6:00 PM.

On line, through the CCS website (www.consorseguros.es). Use of this option is subject to having a digital signature or electronic ID.

If the claim cannot be submitted by telephone or on line, however, a **damage notification sheet** (downloadable from the aforementioned website) may be filled in and submitted or posted to the **Regional CCS Office** directly or through the insurer with which the policy was taken or the agent or broker involved in the transaction. It may not, however, be submitted by fax or e-mail.

More information at: **www.consorseguros.es**

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