Earthquake-resistant building in Cyprus

EARTHQUAKE - RESISTANT BUILDING IN CYPRUS

by

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ABSTRACT

This research was carried out to prove the hypothesis that existing low rise reinforced concrete framed structures in Cyprus are inadequate to sustain the probable seismic loads likely to occur in the next few years and that strengthening techniques can be identified that if carried out would reduce the likelihood of catastrophic collapse.

The objective was to include in this thesis, presented in the form of a manual of seismic design and practice, all the necessary information for a Cypriot Civil Engineer, to understand and face the problem.

In chapter 2 the Cyprus seismic risk is assessed going through historic documents and geological facts. The background theory is included in chapter 3. As the vast majority of buildings in Cyprus are of reinforced concrete framed constructions, it is mainly on this type that the background information focuses. Chapter 4 reviews the existing building practice both before and after 1984 when designers became aware of the seismic problems. Strengthening and repairing techniques, appropriate to the existing stock, are identified in chapter 5.

Using the information in all the above chapters two case studies are presented in chapter 6. A house built in 1984 and a house built in 1991 are analysed under seismic loads and generally assessed to identify deficient areas. Having done that, then specific solutions are suggested accompanied by a cost analysis.

Chapter 7 sums up the results of this research and it concludes that, indeed the existing structures are seismically inadequate and that strengthening measures, though expensive, should be taken to reduce the risk of a catastrophe.

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EARTHQUAKE – RESISTANT BUILDING IN CYPRUS.

1. INTRODUCTION TO THE PROBLEM

The last major earthquake in Cyprus occurred in 1953. According to seismologists, a new major earthquake should be expected soon.[54, 65] It is unknown whether it will occur tomorrow or in ten years, but it would be very strange if did not occur within the next twenty years.

The frequent seismic activity during – recent years, although weak, created a lot of worries and doubts about the strength of the buildings in Cyprus. A lot was written in the newspapers about the dangers due to the expected earthquake and the threat to the existing buildings. Headlines like, 'Most of the houses will collapse!' and 'All the houses are unsuitable for a seismic area!'[56] created panic among the people.

The problem in Cyprus is indeed serious, as it is concluded by studying the existing historical evidence and examining the common building practice. Nevertheless, panicking is not exactly the right way of facing the problem.

The objective was to include in this thesis all the information necessary for a Cypriot Civil Engineer to enable him to face this emerging problem. A good background basis is required if a good appreciation of the situation is to be achieved. It should be established what constitutes good design practice in seismic areas. As the vast majority of buildings in Cyprus are of reinforced concrete framed construction, it is on this type that the thesis will concentrate.

A sound knowledge of the background theory should be accompanied with a knowledge of the existing practice in Cyprus, the problems and generally to be familiar with the whole attitude of people. With these, the main target will be to minimize the effects of an earthquake. One thing to be done is to build new buildings in accordance with a seismic code. A seismic code for Cyprus now exists, however, the worries about the existing structures remain. There is another extremely important area to be investigated, namely to examine the existing structures and to strengthen them if necessary.

It should be made clear that designing from the beginning or strengthening afterwards a building does not guarantee that there will be no damage to it in the event of an earthquake. Compliance with a seismic code is intended to make the structure earthquake-resistant not earthquake-proof.

In the following chapters the sources of information were mainly American, British and Greek publications and a short course on earthquake-resistant design, organised by the Cyprus Joint Group of Civil and Mechanical Engineers. It must be mentioned, however, that for somebody who wants to study a certain scientific or engineering topic, Cyprus with its poor sources of information, is not the ideal place. There is a great demand by the Cypriot engineers for a clear-simple set of instructions for designing and strengthening earthquake-resistant structures. Background theory is, however, equally necessary.

The problem is therefore the threat of a major earthquake. The solution starts by understanding the problem and finishes by taking measures.

2. CYPRUS SEISMIC RISK

2.1 INTRODUCTION

Earthquake resistant building implies an increase in the cost and that will only be accepted if a good reason exists. Constructing earthquake resistant buildings is a must for countries with a high seismic risk. The first chapter will review the existing evidence, although limited, so that an assessment of the seismic risk rating of Cyprus will be possible.

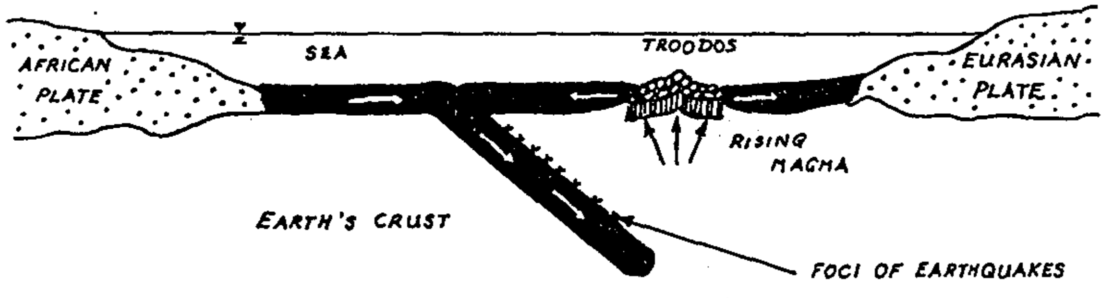
The seismicity of Cyprus will be assessed with respect to historic events and to the geology of the island.

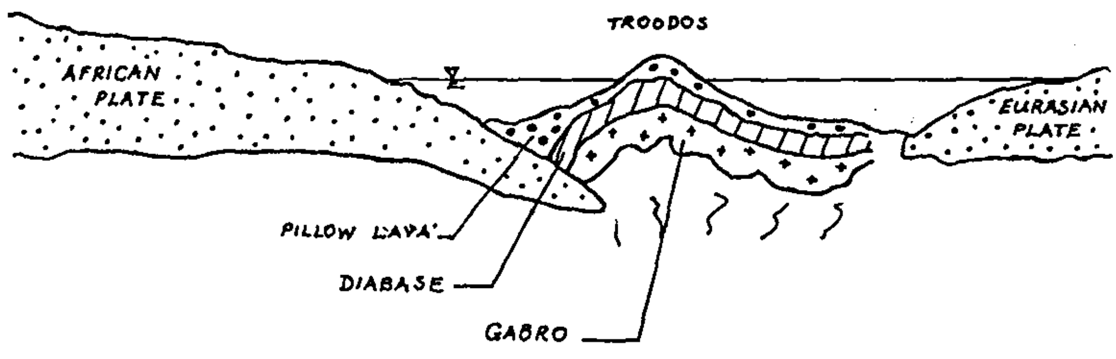
2.2 SEISMICITY OF CYPRUS

Usually prior to a strong earthquake, a number of weak earthquakes are noted. During the last two years 10 such earthquakes were noted by the seismographs in Cyprus. These earthquakes, although not exceeding 4.6 on the Richter scale, did created some panic among the Cypriots. The seismologists were alerted and made it clear that Cyprus should expect a strong earthquake (6 to 6.5 on the Richer scale - see Appendix I for general information on Earthquakes) quite soon according to probabilities. On the other hand the Cyprus Association of Civil Engineers and Architects started warning[45, 56] that the buildings in Cyprus were not earthquake-resistant and asked the Government to legislate a stricter control on the building industry.

Cyprus lies in the second largest seismic zone of the Earth. D. Dowrick in his book[1] 'Earthquake Resistant Design For Engineers and Architects' - Table 5.4, includes Cyprus in the countries with a high seismic risk rating.

Ninety million years ago the whole area around Cyprus was the bottom of an ocean known as the Tithis Sea. Igneous rocks were



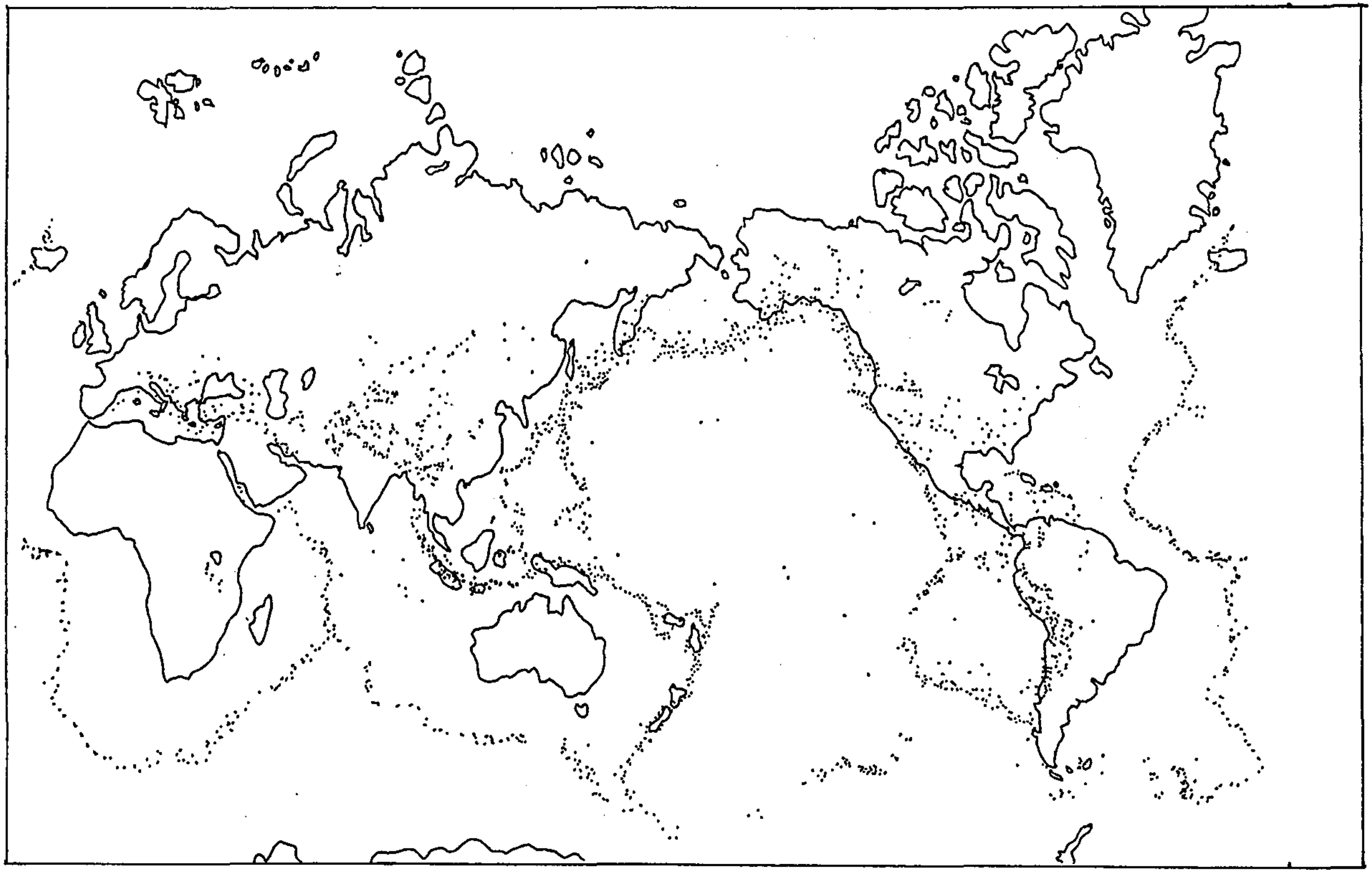


2.1. The emergence of Cyprus 90 million years ago deposited at the bottom of the sea. At that time, however, two of the major plates at the surface of the Earth, the Eurasian and the African, collided forcing a part of the sea to emerge; Mount Troodos emerged first (see figure[62] 2.1). The Eurasian plate continued moving towards Africa forcing the African plate, which was also moving towards Europe, to sink. Due to these movements a lot of seismic action occurred. The emergence of the area continued and 15 million years ago a tiny island appeared near to Troodos. That was Mount Pentadaktylos. The whole of Cyprus emerged 1.5 million years ago and the emergence of Cyprus continues even today since the two plates are still moving.[62]

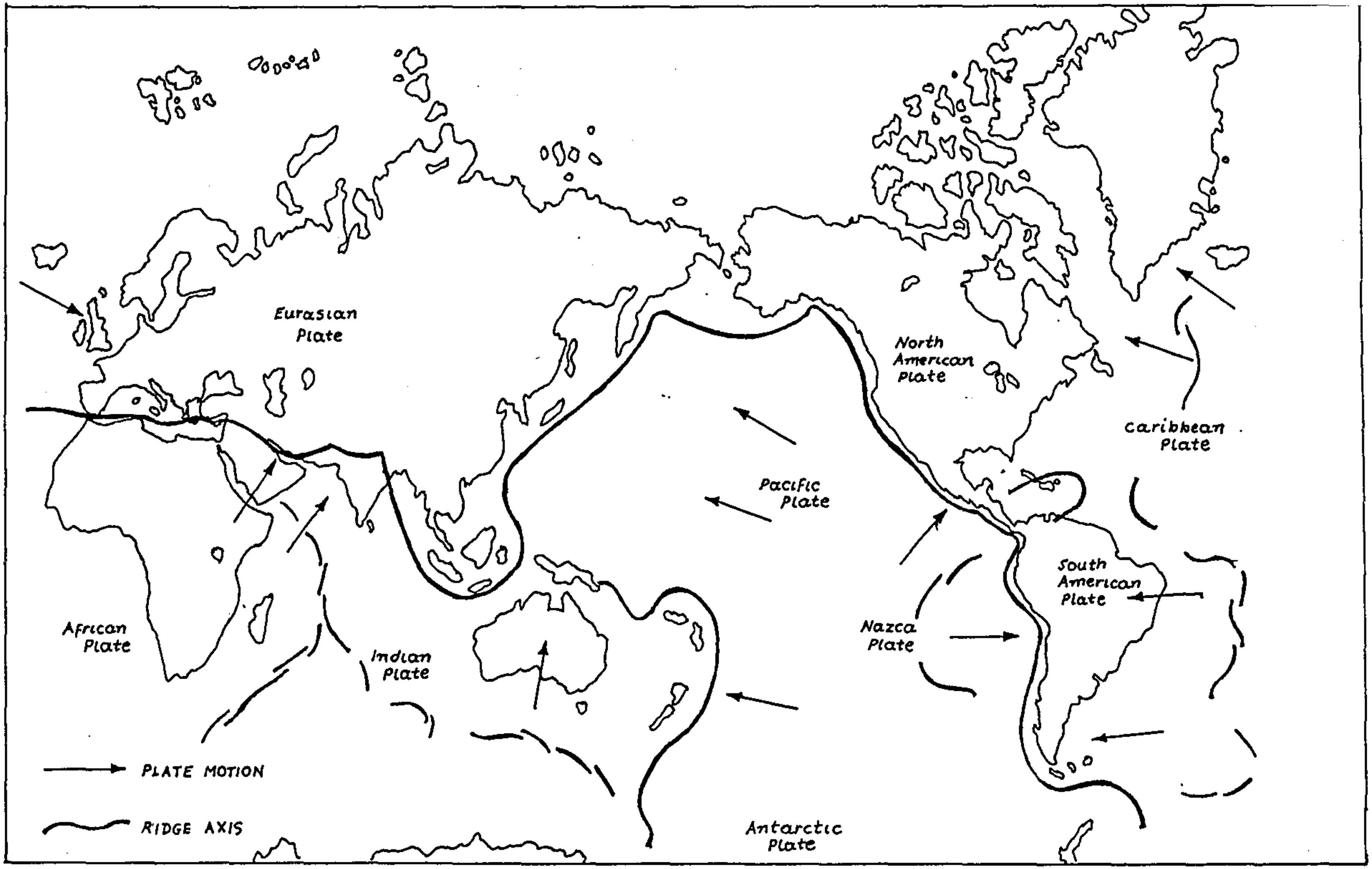
Cyprus therefore lies along the boundary of the two plates. A seismicity map[21] for the Earth shown in figure 2.2 and a map[59] showing the major plates in figure 2.3, demonstrates that seismicity is concentrated along plate boundaries. A fact that spells danger to Cyprus. The seismic action recently observed in the territory is mainly affecting the southern and south-eastern coasts of Cyprus (see map[38] in figure 2.4).

2.3 HISTORIC EVENTS[54, 57, 65, 33]

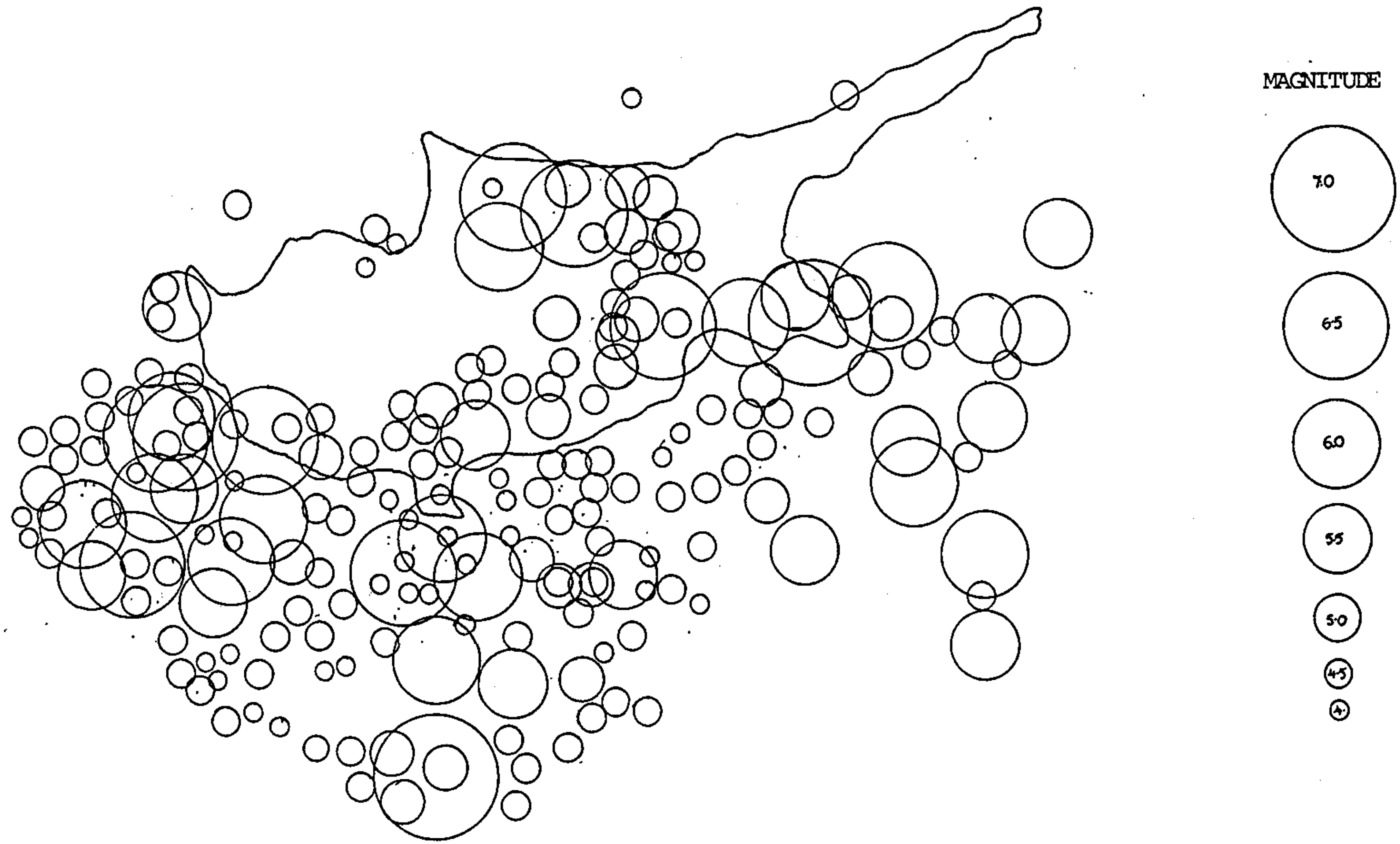
In the history of Cyprus quite a lot of destructive earthquakes are reported. Kourion, like other ancient coastal cities of the island, was ruined by the severe earthquakes of the late 4th century A.D. Nicosia, the capital of Cyprus, although in the centre of the island, also met a number of strong earthquakes. Such earthquakes were reported in 1222 A.D., in 1267 and in 1303, damaging important churches. No information exists about other buildings or even human life losses, religion being very important at that time and only the churches were considered noteworthy. In 1481 the reports talk about an 'almost destroyed Nicosia'. It is believed that the same earthquake struck the island of Rhodes. More destructive earthquakes were reported in the years 1491, 1546, 1575, 1718, where many human life losses are mentioned, and in 1900. The last earthquake that shook Cyprus was on the 10th of September 1953 with an intensity of 9 degrees on the Modified Mercalli scale (described as 'ruinous' – see



2.2 Seismicity map for Earth



2.3 Major Plates of Earth



2.4 Seismicity map of Cyprus

Appendix I). The town of Paphos was mostly affected. Sixty-three people were killed and more than two hundred were injured; whereas some villages, near Paphos, were completely destroyed (see photo[65] 2.1). Prior to that strong earthquake a number of weak shocks were noted. Unfortunately the sources of information are historical books which are not interested in the behaviour of the buildings during an earthquake. Little documentation exists on the character of the above earthquakes, nevertheless the evidence on the high seismic rate is there.

According to information collected by Cypriot seismologists there is a probability of 63% for a strong earthquake every thirty years approximately. It is believed that thirty-nine years after the last strong earthquake quite an amount of energy has been accumulated, so that the probability for a strong earthquake is now raised to 70%.

2.4 GEOLOGY OF CYPRUS[59-63]

In 1953 the damage caused in the district of Paphos was mostly due to the clayey soil of the area. The geology of a territory is very important since it influences the dynamic response of a structure to the earthquakes.

Cyprus, although small in size, has a big variety of rocks (shown in figure 2.5). The island is considered to be quite mountainous. All three main types of rocks (Igneous, Sedimentary and Metamorphic) can be found:

Igneous: These are found mostly on Mount Troodos. They are those first depositions formed from magma 90 million years ago at the bottom of the Tithis Sea as it was mentioned earlier. The igneous rocks are mostly pillow lavas, serpentities, gabrro, dounites, berlite. Around gabrro, diabase is formed in the form of multiple veins. Diabase has good resistance to erosion and degradation.

Photo 2.1. Ruins at the village of Stroumbi in Paphos after the earthquake of September 1953 where 63 people were killed and about 200 were ingured. The earthquake damaged many houses in 90 villages in the district of Paphos.

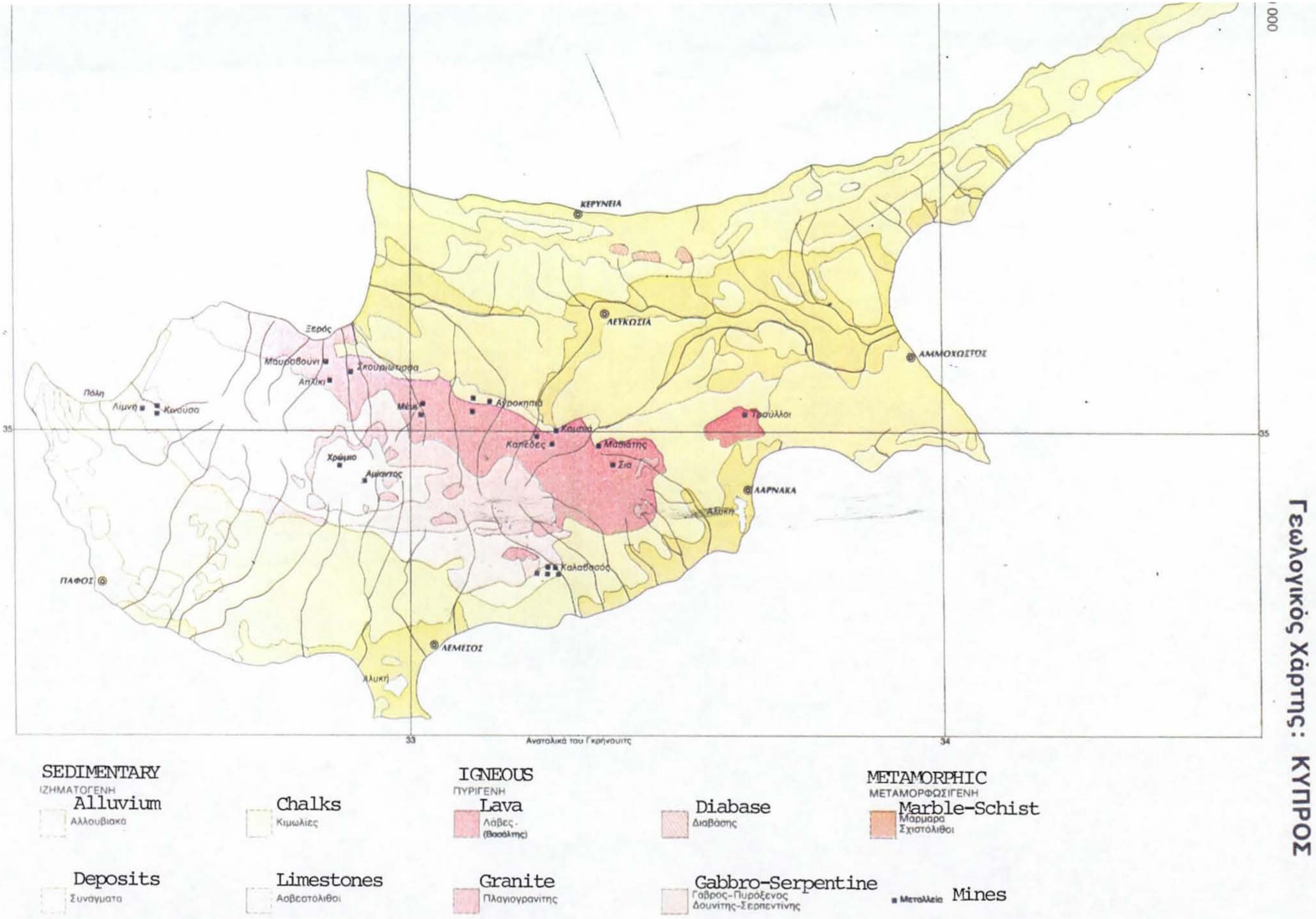


Figure 2.5. Geological Map of Cyprus

Sedimentary: They are found on Mount Pentadactylos in the district of Nicosia and Limassol and almost everywhere in Cyprus with the exception of Mount Troodos. The Sedimentary rocks are the most important for shallow excavations and thus building foundations. They are relatively modern depositions covering most of the solid rocks. The main examples of this type are limestones, asbestoliths, dolomites, fanglomerates and clay and aeolian (windburn) alluvium recently formed (made of clay, sand and gravel).

Metamorphic: These are mostly found on Mountain Pentadactylos and in the district of Paphos. Marble originated from limestone, schist are the main rocks of this type.

The rocks and the mediterranean climate are the main factors influencing the soil types in Cyprus. The main soils in Cyprus are the following:

'Kafkalla': Hard limestone found everywhere apart from Troodos and in the district of Famagusta. Areas where 'Kafkalla' is present are traditionally considered to be good for foundations. Excavations are difficult due to the hardness of soil. Usually shallow foundations of about 1.5 metres depth are adequate in such areas.

Terra Rozza: Red coloured soils created on limestone rocks or on Kafkalla.

Red-soils: Found on the hills around Mountain Troodos and in the central plains. They contain clay and they show volume changes with seasonal changes.

Limestone soils: White coloured soils consisting of a high gravel content and negligible organic material. Lime is usually 70% of the volume. The soils are found in the villages of Limassol and Paphos.

Stiff clays: Found mostly in the district of Nicosia. They show some variation in their strength with seasonal changes. Foundations need to go to such depth so that these seasonal changes do not affect them.

Aeolian (windborn): These are unstable sea sand depositions mixed with clay covering 1% of Cyprus. They are mostly around the coasts and also in the plains near to Famagusta. According to clay or sand content they create problems sometimes to the foundations of buildings.

2.4.1 Influence of soil conditions on building behaviour and problematic soils in Cyprus

Similar structures located near each other but standing on different soil will behave in a different way during the same earthquake. This conclusion was practically proved by the Tanankai earthquake of 1974 in Japan.[6] The degree of damage was higher for buildings erected on clay than those erected on sand and rock. The dynamic characteristics of soil layers overlying bedrock are of great importance. Stiff soils have smaller periods and soft soils have longer periods of natural vibrations. It may happen that the frequency of the vibration of the bedrock is similar to the frequence of the overlying soil. If it is, then resonance is possible, which will magnify the intensity of vibration of the overlying soil. Resonance is also possible between the structure and the soil (see chapter 3).

Japanese seismologists concluded that the intensity of earthquakes increases with increasing degree of soil saturation. During the earthquake in Chile in 1960 the main reason for structural collapses was the instability of saturated clay. In Niigata during the earthquake in 1964, big differential settlements occurred due to saturated sandy soil. Some of the buildings were tilted and overturned. The earthquake of Caracas in 1967 showed that flexible buildings founded on alluvial soils will be badly damaged whereas stiff buildings standing on the same soil were very lightly damaged.[35] Nevertheless, exactly the opposite results were observed for buildings founded on solid rock; the stiff buildings suffered greater damages. More recently (1985) in Mexico[12] City the soft soil (mostly clay) overlying the bedrock was the reason for the magnified intensity of the earthquake. The Tacubaya Clay which is found in the first 20 – 30 metres of the strata profile in Mexico City is an extremely compressible clay with high plasticity. The low damping ratio and the low stiffness led to upward propagation of shear waves from the bedrock.

It looks therefore that two different types of soil can create serious problems to buildings: Soft clay and saturated sand. Generally in saturated granular soils shaken over a period the pore–water–pressure increases and the soil strength drops sharply. The soil behaves like a liquid, a phenomenon called liquefaction.[31]

Such problematic soils are found in two different coastal areas. Soft clay can be met in the district of Paphos and was the main reason[48] for the damage in 1953. Saturated sand is found in the town of Larnaka, very near to the present main airport of Cyprus. Unfortunately both areas lie on the southern coasts where most of the seismic action has been observed. Buildings constructed on the aeolian soils performed well until now since they are stiff structures (usual case in Cyprus).

2.5 MEASURES AGAINST EARTHQUAKES

Having in mind the above factors Civil Engineers in Cyprus, quite rightly suggested a number of precautionary measures:[55]

a. People should be correctly informed and prepared to face an earthquake and its consequences.
b. Groups of technicians should be formed and trained so that they are prepared to proceed immediately in repairing and strengthening dangerously damaged buildings after an earthquake. Materials should be already stored, ready for this purpose.
c. Important services like hospitals should be safeguarded.
d. A seismic code should be prepared and established by law. At the same time seminars should be organised for Civil Engineers and Architects on the aseismic design of structures.

Since 1985 Civil Engineers have been trying to employ some earthquake-resistant design ideas. In 1989 the Association of the Civil Engineers and Architects were asked by the Government to prepare a seismic code for Cyprus. The committee appointed for this purpose presented in 1991 a draft of the code and managed to pass it on to the government of Cyprus. Finally, on July 1, 1992 a new law was imposed making the use of the seismic code compulsory for all the buildings.

2.6 CONCLUSIONS

There is little doubt that seismic risk is high enough in Cyprus to warrant serious measure being taken. The historic events presented in this chapter and the geological facts strengthen this view.

Among other measures suggested the most important perhaps is the creation of a seismic code for buildings in Cyprus, which in combination with a good knowledge of earthquake-resistant design practice will enable Cypriots not only to build earthquake-resistant buildings but also to strengthen existing buildings. The next chapter will deal therefore with the main requirements for seismic design.

3. DESIGN OF EARTHQUAKE-RESISTANT BUILDINGS

3.1 INTRODUCTION

The objective of this thesis is to examine methods of strengthening existing buildings in Cyprus to resist seismic loading. As most buildings in Cyprus in recent years have been of reinforced framed construction, it is on this type of construction that the thesis will concentrate. In chapter 2 the review of the seismic risk in Cyprus has shown that there is a 63% probability that a strong earthquake will occur every thirty years. The need for seismic design in buildings is clear. Before building practice in Cyprus is examined, it is necessary to establish what constitutes good design practice in areas subjected to earthquakes. This chapter reviews the main requirements for seismic design of buildings as established by several seismic codes, mainly the European and some American ones.

The general background theory presented here can be found in a number of standard texts such as references 1, 3, 4, 5, 6 and 13.

3.2 EARTHQUAKE AND THE AIMS OF THE ASEISMIC DESIGN

All existing structures stand on soil, which is supposed to provide a permanent support for them. Since all the structural loads are transmitted to the ground, it must be adequately strong, stable and reliable. Under normal conditions the soil takes all loads coming from the structure. During an earthquake, however, the situation is reversed:

Seismic loads are induced by the movement of the soil. Hence the soil is not stable and thus the support of the whole structure is unstable as well.

The only reliable information about seismic loads known to us are that they are predominantly horizontal and have dynamic character. Vertical seismic loads can only be significant in areas very near to the epicentre of the earthquake. The amount of energy released, the direction of the movement, the frequency and the duration of the vibration are not known to us. The response of a structure to an earthquake depends on a number of different conditions such as the period of vibration (T), damping (μ), ductility, and of course the soil type. The design of earthquake-resistant buildings is therefore not a simple task to treat with confidence.

It is essential, however, that structures constructed in a seismic zone must be able to withstand a sudden ground movement. The direction of this attack may be arbitrary and it is not possible to orientate the structure in accordance with the direction of seismic action. In spite of complexity and uncertainty of seismic loads some structures have sustained even the most extensive earthquakes. The long term accumulation of previous experience helped to work out some useful protective measures that may reduce the disastrous effects of earthquakes. These measures constitute the aseismic design which aims to provide the structure with some properties that will enable it to resist earthquakes as much as possible. The targets are, first to save human lives and then to reduce damage to buildings. The properties required are sometimes contradicting each other. A structure has to be stiff enough and at the same time it has to be flexible as well. A balance is needed between the two.

3.3 FLEXIBILITY, STIFFNESS AND DUCTILITY

Stiffness is needed in a structure so that displacements are acceptable. In this way interaction between structure, cladding, partitions and equipment can be controlled. Nevertheless a very stiff structure is not desired due to brittle failure. Problems in avoiding re sonance of the structure with the period of the site may also appear. For short period sites, like rocky sites, more flexible and taller buildings being long period structures are appropriate. On soft soils, short period structures (i.e. stiff and low buildings) should be designed.

A stiff reinforced concrete structure (a common case in Cyprus) has another advantage in comparison to a flexible structure. It is easier to reinforce it using shear walls; something useful in the case of strengthening an existing structure.

A very important property desired is ductility. Ductility is the ability of a structure to distort repeatedly without collapse. In other words it is the ability of structural element to undergo a considerable amount of plastic deformation before failing. This is essential to enable the structure to absorb as much energy as possible, generated by the earthquake, and thus to increase the resistance of the structure. In addition a ductile failure will give time to people to escape.

A measure of ductility is given by

$$\frac{\delta_u}{\delta_y} = \frac{\text{Curvature at ultimate capacity}}{\text{Curvature at first yield of tension steel}}$$

Now,

$$\delta_u = \frac{\varepsilon_{cu}}{x_u} \qquad \text{and} \qquad \delta y = \frac{f_y}{E_s \cdot (1-n) \cdot d}$$

where all symbols are explained in figure 3.1

Therefore,

$$\frac{\delta_u}{\delta_y} = \frac{\varepsilon_{cu} \cdot d(1-n)E_s}{x_u \cdot f_y}$$

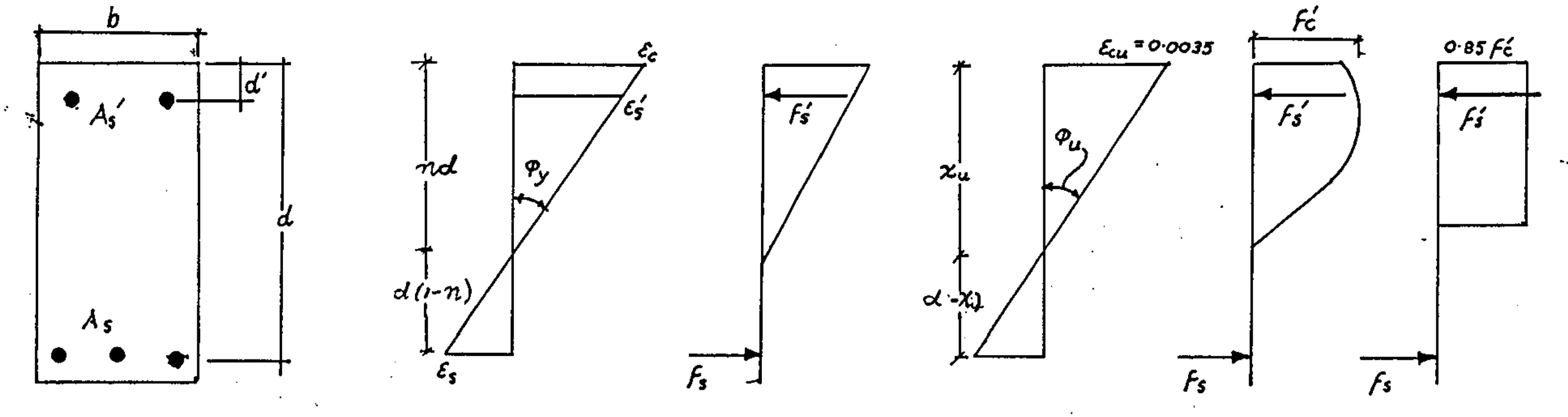
Having in mind that,

$$\varepsilon_{cu} = 0.0035 + 0.2\,\frac{b}{l_c} + \left(\frac{P_v \cdot f_{yv}}{138}\right)^2$$

$$x_u = \frac{A_s \cdot f_y}{0.68\ f_{cu} b}$$

where f_{yv} is the shear links yield stress

P_v is a value according to the dimensions of the links given



Elastic State

Ultimate State

3.1 Beam in the Elastic and Ultimate State

Then we can conclude that ductility increases by:

a. increasing the yield stress of the links, f_{yv}
b. increasing the amount of transverse reinforcement, A_{sv}
c. decreasing steel yield stress, f_y
d. decreasing tension steel content, A_s
e. increasing concrete compressive strength, f_{cu}
f. increasing the width of the section.

In figure 3.2[3] the effects of increasing the amount of transverse reinforcement on the stress-strain relationship are shown.

In figure 3.3[35] the effects of decreasing the yield strength are also shown.

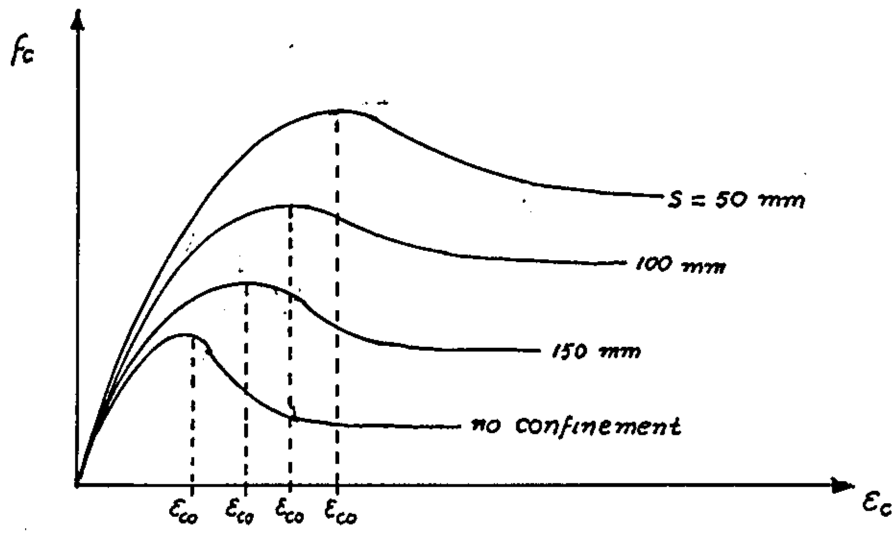
3.4 METHODS OF ANALYSIS

Several methods are employed worldwide for seismic analysis. These methods may be divided into two categories:

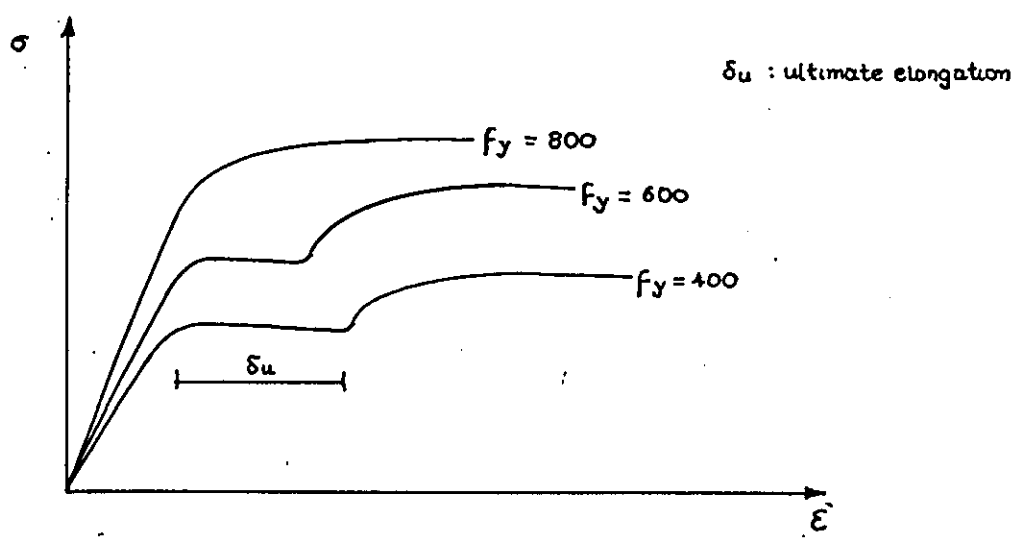
a. the equivalent static force analysis
b. the dynamic analysis

The majority of buildings are designed in accordance with the first method of analysis. In this method the seismic loads are visualised as static loads applied at the centres of lumped masses and causing the same maximum displacements which may be experienced when structures vibrate during an earthquake. Hence, the real dynamic loads are conventionally substituted by 'equivalent static loads'. The determination of seismic loads is carried out with respect to the dynamic nature of the earthquakes. The dynamic characteristics of structures are used while determining the seismic loads and while analysing the behaviour of the structural members.

The simplified method, however, can be only employed for regular structures having a height of less than 80 metres according to the European Code[2] (C.E.B.) and a fundamental period of



3.2 Effect of confinement on stress-strain relationship of concrete



3.3 Effect of yield strength to deformability of steel vibrations shorter than two seconds. The limitation of the building height is influenced by experience and common sense and is not yet scientifically proved. In other codes these limits may differ. Buildings are generally treated as regular ones if they comply with the following requirements:

a. A building should have an approximately symmetrical plan configuration with respect to two orthogonal axes.
b. Masses, rigidity, and flexibility should be uniformly distributed across the plan and over the height of the building.
c. At any storey level the eccentricity between the centres of mass and rigidity should be very small(see pg 29).

If structures do not comply with the above requirement or if they are exceptionally important (hospitals, airports, etc.) then dynamic methods should be used. The dynamic methods of analysis are based on the analysis of the differential equations of the building motions. The acceleration of the soil during an earthquake, measured by instruments are used for determining the seismic forces. Dynamic analysis is more accurate but difficult to employ. It requires powerful computers and expensive software and therefore is only used for major buildings.

3.5 FUNDAMENTAL ASEISMIC PLANNING

Decisions made at the conceptual stage are sometimes difficult to modify later. Care should be therefore taken at the early stages to avoid difficult situations. The behaviour of an irregular building can not be predicted with a required accuracy. Even sophisticated mathematical models[4] failed to provide reliable information about the response of irregular structures to seismic loads.

Excessive deformations may be experienced due to difference in rigidity along the height of the building. This may be also observed when there is a considerable change of plan dimensions or in cases when buildings have 'flexible' and 'rigid' storeys.

Before getting into real analysis some important factors should be considered.

3.5.1 Materials

The choice of the materials should be such as to enable the structure to achieve the desirable levels of ductility, stiffness and flexibility. Generally the materials to be used should have the following properties:

a. High strength to weight ratio. Light but strong materials are needed. Since increasing the natural frequency of the structure is something we want to achieve, to avoid resonance, then from

$$w = \sqrt{\frac{K}{m}}$$

It is clear that the higher is the value K (stiffness) and lower the value m (mass) then the better it is.

b. High deformability. Plastic deformation of structural elements are desirable for energy absorption and to avoid brittle failure.

c. Low degradation in strength and stiffness under the repeated loading.

d. High uniformity. This is to prevent separation of structural elements.

e. Reasonable cost.

The larger the structure the more important the above properties are. The suitability of the commonest structural materials is shown in Table 3.1.[4] Of course the order of suitability can not really be fixed as it will depend on the local availability of materials, the type of structure and the skill of the local labour in using them.

TYPE OF BUILDING	STRUCTURAL MATERIALS IN APPROXIMATE ORDER OF SUITABILITY
High-rise	1. Steel 2. In situ reinforced concrete
Medium-rise	1. Steel 2. In situ reinforced concrete 3. Precast concrete 4. Prestressed concrete 5. Reinforced masonry
Low-rise	1. Timber 2. In situ reinforced concrete 3. Steel 4. Prestressed concrete 5. Reinforced masonry

TABLE 3.1 Suitability of Structural Materials

Obviously steel is an excellent material for earthquake-resistance since it can have all the necessary properties except for one: It is very expensive. Hence it is not usually used for low or medium-rise buildings.

The most popular earthquake-resistant material, or rather combination of materials , is insitu reinforced concrete which can perform relatively well and is less expensive. It is inferior to steel, however, and some codes limit its use to buildings up to six storeys. Steel reinforcement can improve greatly the properties of concrete in a number of ways:

a. It provides stability
b. It increases shear strength
c. It confines concrete and provides ductility
d. It improves joint rigidity

An upper limit, however, should be imposed both on the reinforcement ratio and the yield strength of the steel. This is to reduce possibility of brittle failure as it was explained earlier in this chapter (see figure 3.3). An under-reinforced structure is still the target.

Timber is another excellent material for earthquake-resistant structures. It is perhaps the only one which can best satisfy all the desired properties. Nevertheless there is an indirect problem: Its low fire-resistance. Many cases[7,16] were reported of structures made of timber that after standing an earthquake they failed because of fire due to failing electric devices.

Precast (reinforced or prestressed) concrete are considered to be poor for earthquake-resistant structures and should be avoided. Their main drawback is that they lack uniformity and are not monolithic.

3.5.1.1 Masonry

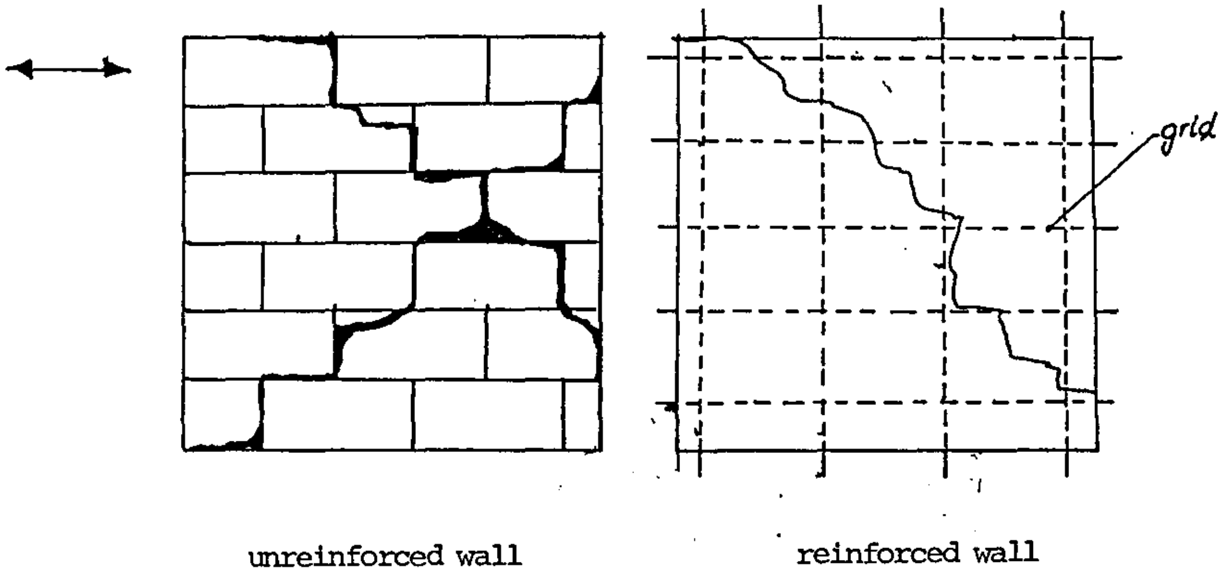
It is stressed by most of the design codes that unreinforced masonry is unsuitable in seismic areas. The bricks are brittle and have a large stiffness. Failure is normally due to shear rather than instability, and the shear cracks expected are as shown in figure 3.4 (a)[5].

Reinforcement is therefore essential. Especially horizontal reinforcement is very important for all structural members subjected to shear failure. The effectiveness of the reinforcement is shown in figure 3.4 (b). To reinforce the masonry, steel bars are used both vertically and horizontally. Hollow bricks may be used or a double wall is used (see figure 3.5).

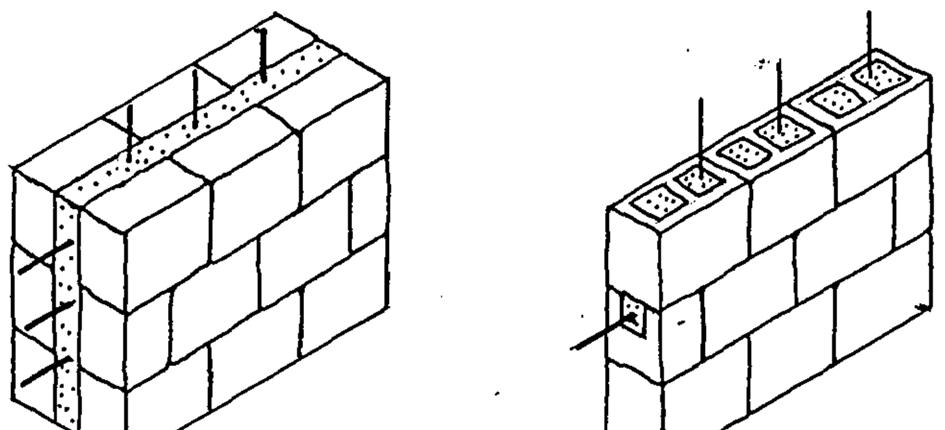
Of greater importance to the Cypriot Engineers is the use of infill masonry walls. The full implications of frame-infill masonry design are complex. As illustrated in figure 3.6, the interaction between a frame and infill masonry created problems.[4,6] The principal effects of infill walls on the overall seismic response of a structural frame are:

a. To increase the stiffness and hence increase the effective lateral force (base shear)
b. To increase the overall energy absorption capacity of the building.
c. To alter the shear distribution throughout the structure.

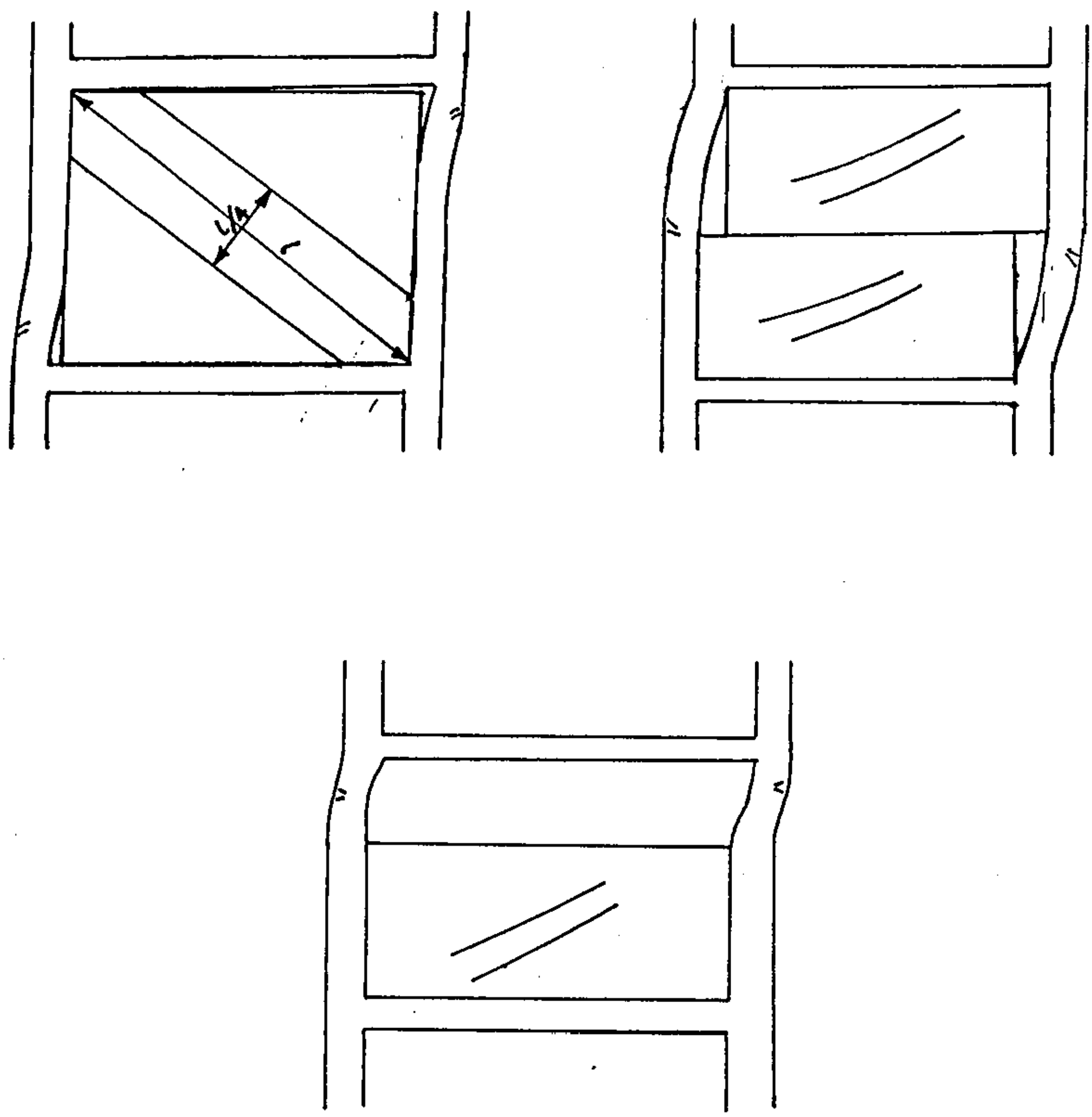
As the infill is made of a brittle material, the response of the whole building will be greatly influenced by the damage sustained by the infill and stiffness-degradation characteristics. The wall will act as a diagonal compression strut, stiffening for some time the frame. When this strut fails suddenly (a brittle failure should be expected), it can cause a sudden failure to the frame too.



3.4 Effectiveness of reinforcement in a masonry wall



3.5 Typical methods for reinforcing a masonry wall



3.6 Interaction between frame and infill masonry

Horizontal sliding can also occur causing failure of the panel. Once the panel has sheared (sliding) the effect of the diagonal compression strut is lost.

Infill walls show larger ductility than isolated walls.[3] Additionally, as it was mentioned, the strength and the energy-dissipation capacity of the frame is increased by the infill walls since they are acting as shear walls, up to a certain point of course. Thus a frame with a reinforced infill panel will remain effective against earthquakes despite the problems described above.

3.5.2 Building configuration

World-wide experience has shown that the architectural engineering of buildings is an extremely important factor in resisting seismic loads. The structural response of a building heavily depends on its arrangements in both plan and elevation.

General principles of architectural planning are presented in a form of empirical recommendations. A general concept for small to medium-size buildings in seismic areas is to have a clearly defined simple plan and elevation. Buildings should preferably have rectangular or circular plan configuration, symmetrical with respect to at least two orthogonal axes. The distribution of masses and rigidity should be uniform and symmetrical both in plan and elevation. The centres of mass and rigidity at each floor level should coincide. The location of the centre of mass, C_m may be determined by,

$$x_m = \frac{\Sigma\, Q_u \bullet x_u}{\Sigma\, Q_u}$$

where x_m is the distance to the centre of mass from a certain point in the x-direction

Q_u are the gravitational loads concentrated at specified points.

x_u is the distance to the loads from a certain point.

(see figure 3.7 for more clarification)

The location of the centre of rigidity, C_r may be determined by,

$$x_r = \frac{\Sigma K_{ml} \cdot x_l}{\Sigma K_{ml}}$$

where x_r is the distance to the centre of rigidity in the x–direction

K_{ml} is the flexural stiffness of member l and at the level m (relative stiffness will be used).

x_l is the distance of member l.

The same procedure should be followed for y_m and y_r in the y–direction. Since it is almost impossible for the two centres to coincide perfectly some eccentricity is expected. The eccentricities, e_{ox} (x–direction) and e_{oy} (y–direction) are limited by the European Code as follows:

$$e_{oy}/r_x < 0.15$$

$$e_{ox}/r_y < 0.15$$

where

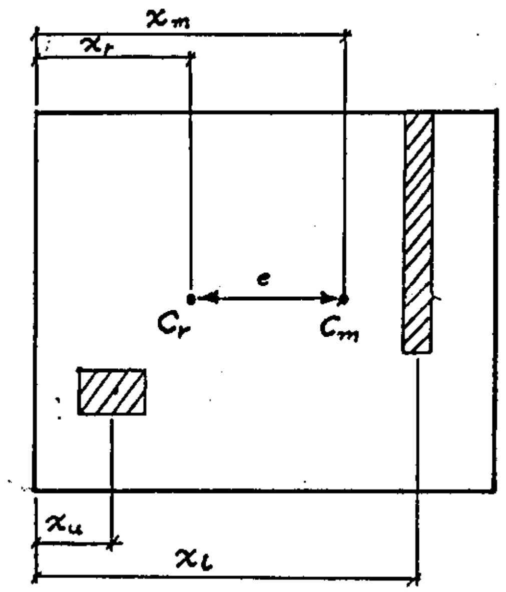
$$r_x = \frac{K_t}{K_x} \qquad \text{and} \qquad r_y = \frac{K_t}{K_y}$$

where K_t is the torsional stiffness (K_t = GJ)

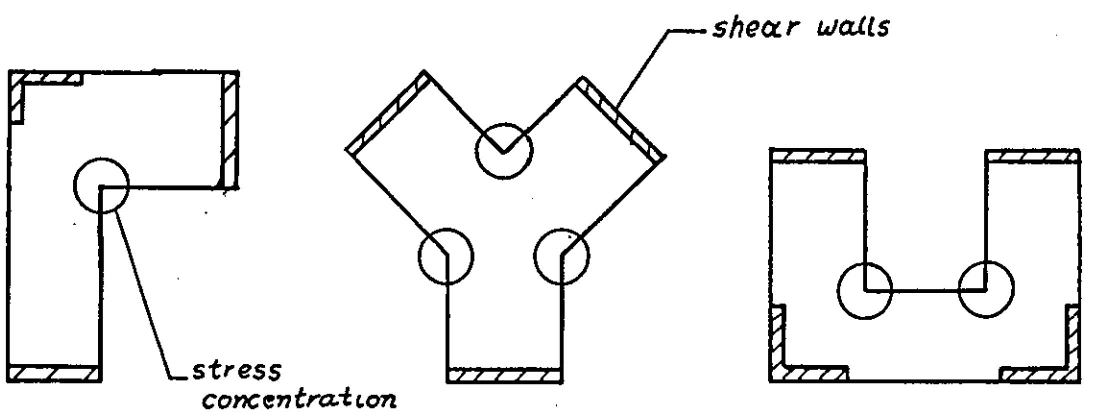
K_x is the flexural stiffness in the x–direction.

If these limits are exceeded then the structure is an irregular one and dynamic analysis is necessary. On the other hand, if the eccentricities are within the limits then almost all the codes suggest a minimum eccentricity so that torsional forces are still taken into account.

The length to width ratio of buildings should be less than 2. In this way the effects of torsional motion will be small and



3.7 Centres of mass and rigidity



3.8 The use of shear wall in buildings with an irregular plan additional displacements due to torsion will not considerably influence the level of stresses.

Corners of buildings create specific problems. Re-entering corners should not exceed 25 per cent of the building external dimensions. Shapes of the form of L, H, T, or Y should be avoided, because they introduce complexities into the analysis. Points of stress concentration are created and it was observed that such buildings were often damaged in earthquakes.

There are two alternative possibilities to tackle the problem if such shapes cannot be avoided: Either structural division of buildings into simple parts by means of aseismic joints, or by strengthening of some structural members using shear walls (see figure 3.8).

Aseismic joints divide long or irregular buildings over their whole height, leaving a gap of width greater than the sum of the possible displacements of the two adjacent blocks during an earthquake. This gap is essential to prevent the so-called 'hammering effect': collision between the two separated parts. According to USA codes[1] displacement of structures should not exceed 0.5% of the floor height and therefore gap width, A is given as:

$$A = 0.005\ h_1 + 0.005\ h_2 + 2 \quad \text{(cm)}$$

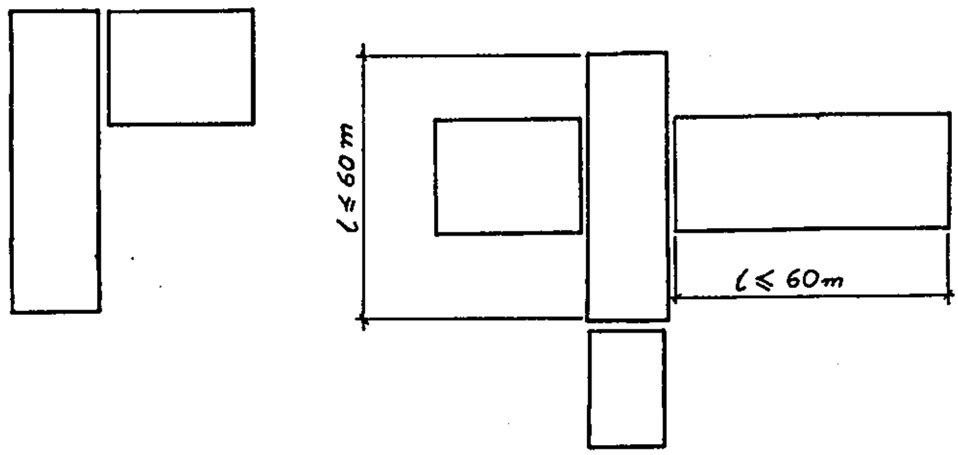
where h_1 and h_2 are the height of floors in blocks 1 and 2 respectively in centimetres.

The USSR code[35] suggests a different formula for the gap width:

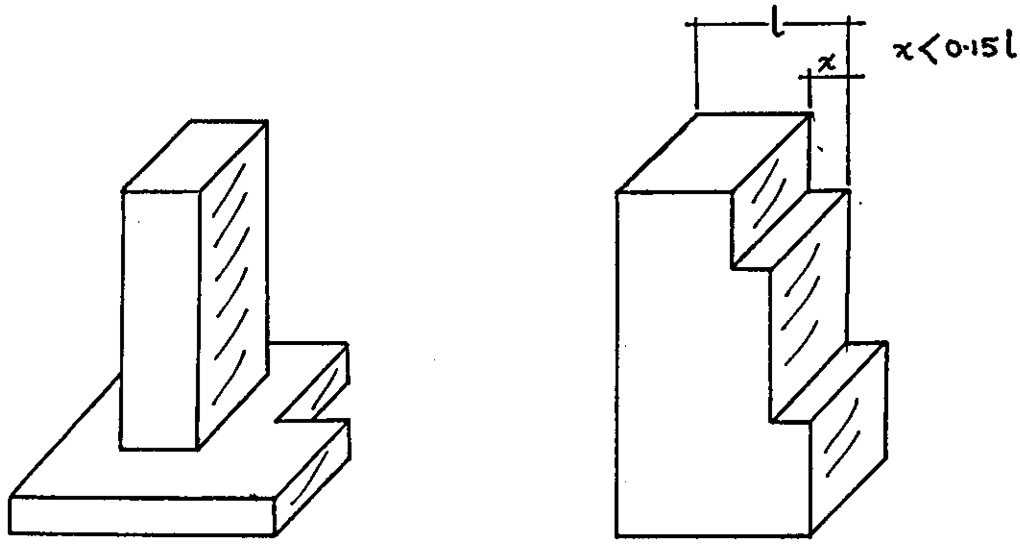
$$A = 3\ \text{cm} + \left(\frac{h}{5} \cdot 2 \right)$$

where h is the increment of height above the first 5 metres.

The minimum width allowed is 3 centimetres.



3.9 The use of seismic joints



3.10 Setbacks to be avoided

Generally seismic joints should be provided in the following cases:

a. Buildings with complex shape in plan.
b. Buildings with large dimensions in plan, length or width. The length of each block should not exceed 60 meters (see figure 3.9).
c. When there is a difference between heights of adjacent parts of a building exceeding 5 metres.
d. When different parts of a building have floors located at different levels.
e. When foundations are located on non homogeneous soil having different strain and strength capacities under different parts of the building.

The vertical dimensions of the building should be also carefully designed, as to provide uniformity, continuity and proportionality. Drastic changes should be avoided as shown in figure 3.10. The height of the building is sometimes limited. Some codes prefer restricting height to width ratio. It is commonly accepted that buildings having a height to width ratio less than 4 may be classified as relatively rigid, with adequate proportionality preventing overturning moment effects.

Sudden changes should also be avoided in the vertical distribution of stiffness and strength. Soft stories (a common case in Cyprus!) tend to collapse because of concentration of plastic deformation. This may cause the entire building to collapse.

Care is also needed in the horizontal distribution of stiffness and strength. Short columns (which are very stiff) should be avoided. Especially the combined use of short and long–slender columns creates differential problems. Note here, that non–structural members like masonry walls and parapets can alter the properties of a column. Such members should be separated from the columns leaving a gap in between.

To complete this topic some additional recommendations are given in figure 3.11[2,3,4,5,6,13,38]

3.6 LOAD COMBINATION

All structures to be constructed in seismic regions should be analysed twice. First for the normal load combination and then for the seismic load combination. After the analysis of both cases, the most unfavourable stress–strain conditions should be considered as the design criterion for the detailing of the structural members.

To analyse the action of seismic loads a particular combination of external loads should be adopted. It will include all permanent and reduced variable and all seismic loads. Dead and permanent live loads are usually taken with their characteristic values (i.e. safety factors are equal to 1.0). The variable live loads, whose duration of application is long enough for the probability to occur simultaneously with seismic action, are included in the load combination reduced by a factor ψ. Factor ψ varies from country to country.

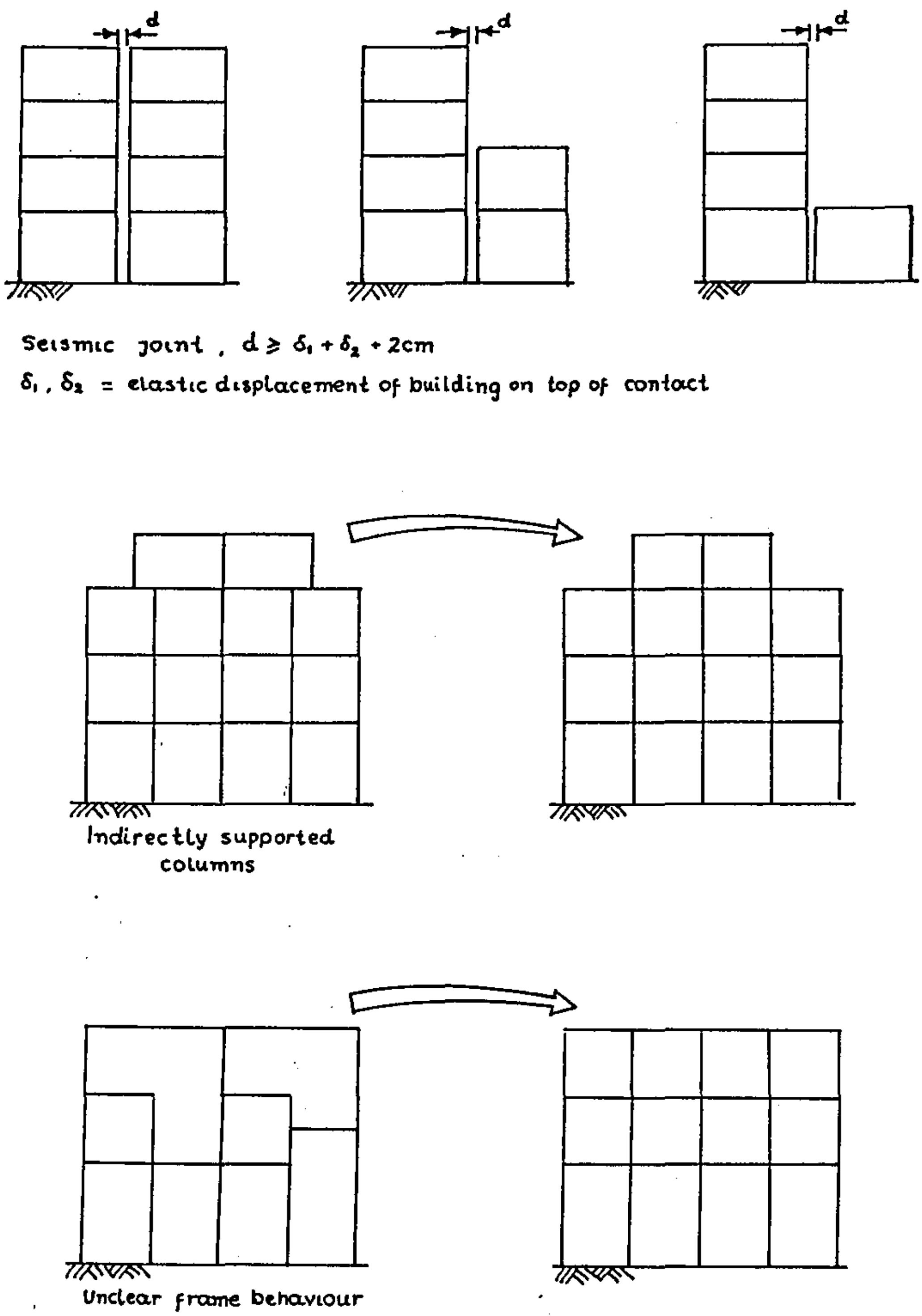
Generally load combination including seismic loads apparently vary in the different codes. A brief reference is made here to some of them.

CEB (European model code):

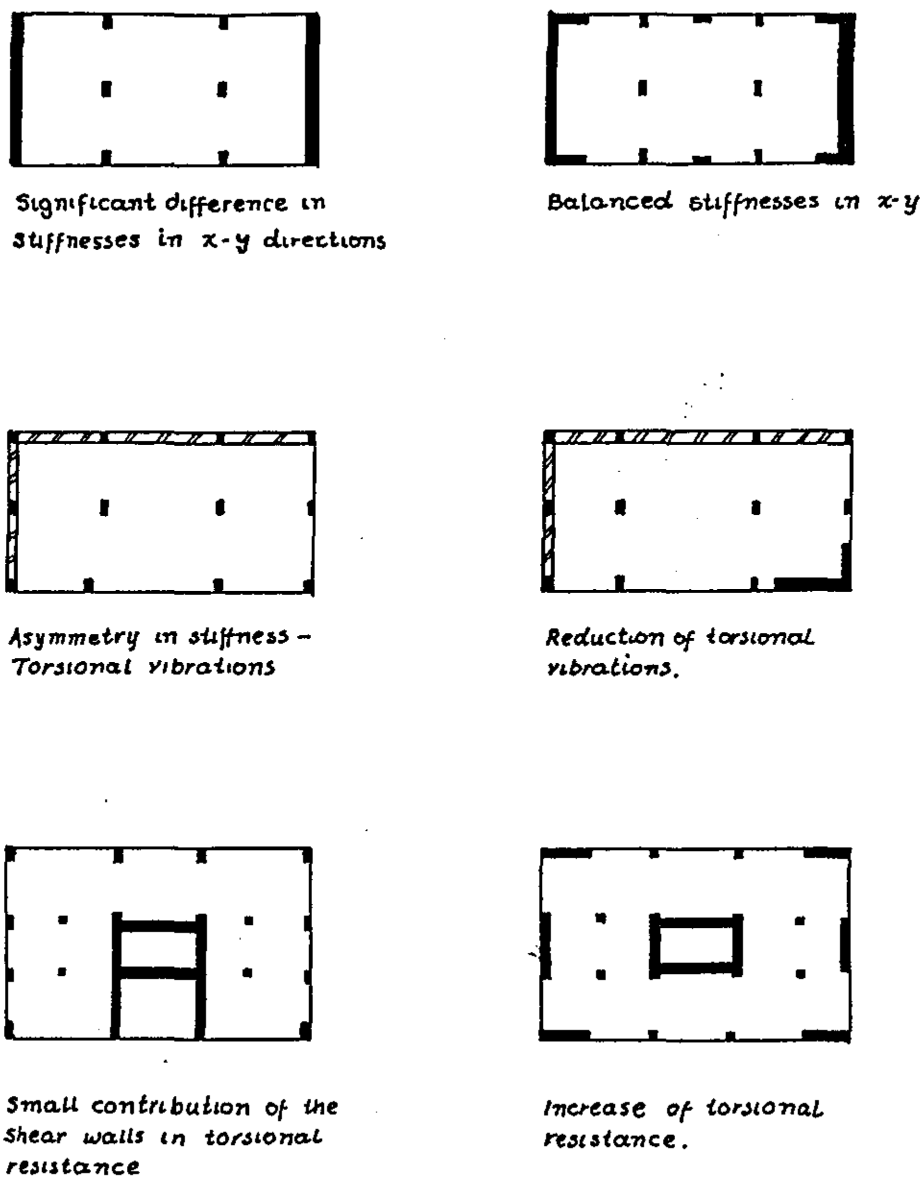
$$S_d = S\ (G + P + E + \Sigma\Psi_L Q_{LK})$$

where
- S_d is the seismic load combination
- S is a soil factor
- G is the nominal value of all permanent loads
- P is the prestressing force (for prestressed concrete structures)
- E is the design value of the seismic loads given as equal to $G + \Sigma\Psi_L Q_{LK}$
- Q_{LK} are the fractile values of all the variable loads.

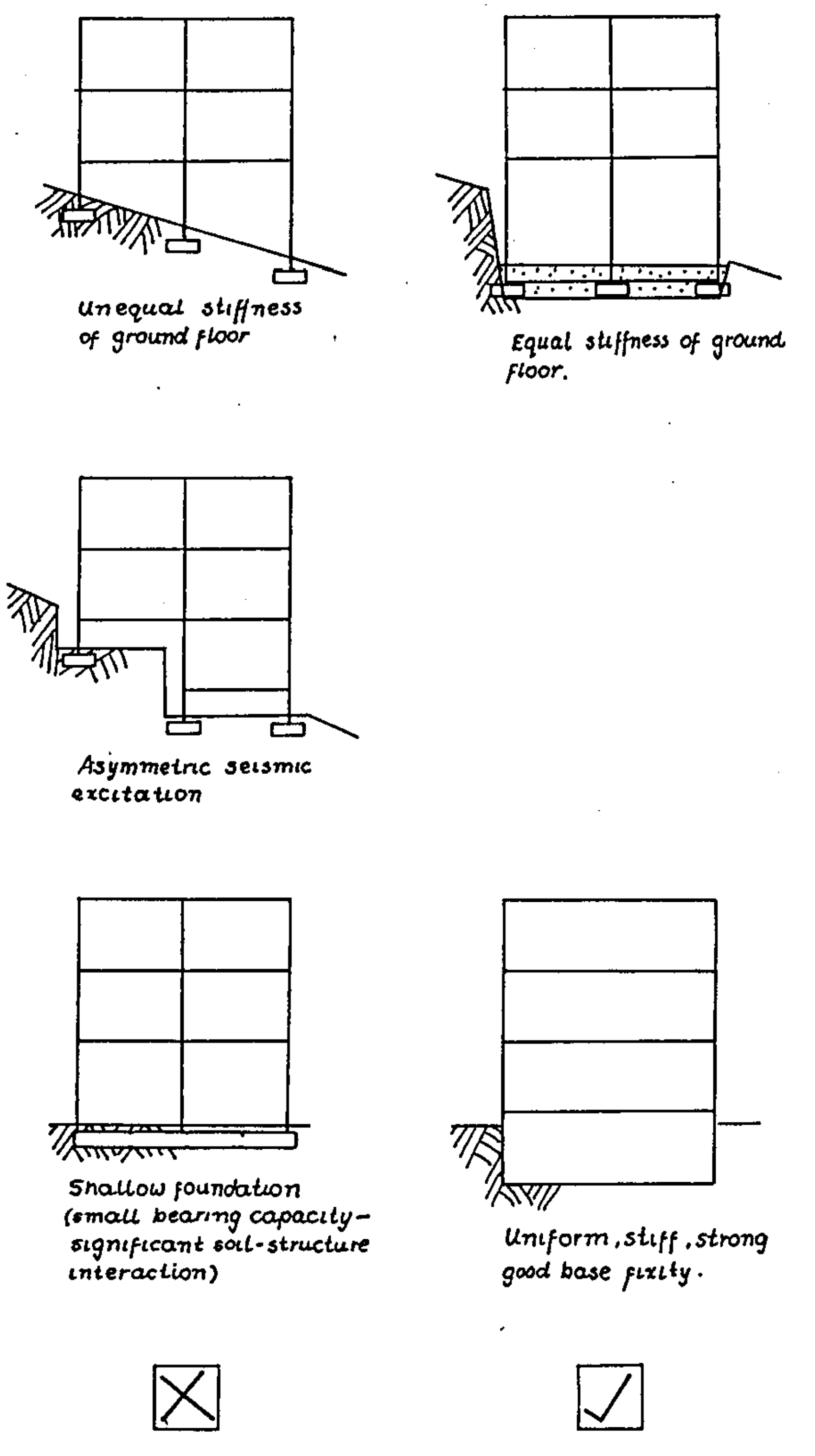
*



3.11 (a) Additional recommendation for configuration - elevation



3.11 (b) Additional recommendations for configuration - plan



3.11 (c) Additional recommendations for configuration - foundations

The Greek Code[35]:

$$S_d = 1.0\ G + 1.75\ E + 1.0\ \Psi Q$$

where $E = C_d\ (G + \Psi Q)$, C_d being a seismic coefficient

The Indian Code[35]:

$$S_d = S\ (1.4\ G + 1.4\ E + 1.4\ \Psi Q)$$

where $E = C_d\ (1.35\ G + \Psi Q)$

The U.S.S.R. Code[24]:

$$S_d = 0.9 + 1.0\ E + \Sigma\Psi Q$$

where $E = G + \Sigma\Psi Q$

Note that in this expression G and Q are design values and not characteristic. Ψ is given as equal to 0.8 for long-term live loads and equal to 0.5 for short-term.

The U.S.A. Code (ATC-3)[5]:

$$S_d = 1.2\ D + 1.0\ L + 1.0\ S + 1.0\ E$$

or $$S_d = 0.8\ D + 1.0\ E$$

where D is the dead load
L is the live load
S is the snow load

and the ACI Code[5]

$$S_d = 0.75\ (1.4D + 1.7\ L + 1.87\ E)$$

or $$S_d = 0.90\ D + 1.43\ E$$

The design loads for a structure will vary from code to code. Not only because the seismic codes use different load combinations but also because the same variables are given different values from country to country. Of course nobody can say which code is more accurate. Each country (each area) should follow its own seismic code since each country has its own particularities.

3.7 THE EQUIVALENT STATIC FORCE ANALYSIS

When the equivalent static analysis is adopted, the design value of horizontal seismic force, which is to be applied at each floor level, is usually defined as follows:

$$F_i = C_d \bullet y_i \bullet W_i$$

where C_d is the design seismic coefficient

y_i is the distributed lateral force at floor i

W_i is the total weight of floor i determined with the load combination factors given for seismic analysis.

3.7.1 The seismic coefficient, C_d

The seismic coefficient, C_d, depends on five main factors:

a. seismicity of the region being considered
b. response of building to vibrations
c. soil conditions
d. occupancy importance of the building
e. ductility level desired.

The CEB code recommends a comprehensive equation for determining the seismic coefficient,

$$C_d = I \bullet A \bullet S \bullet a \bullet \frac{1}{K}$$

where:

I: The importance factor depending on the class of the building. The class of a building depends on the use, on its content and on the class of the importance of its function. Up to five classes may exist, in some codes, with class I being the most important and class V being insignificant for seismic design. Values vary from 0.5 to 1.5.

A: The relative value of the maximum soil acceleration, given by $A = \frac{\gamma_{cmax}}{g}$.
The seismic codes assign a different value for each seismic zone according to the intensity of the earthquakes expected. Values vary from 0.01 to 0.4.

S: The soil conditions factor. Usually three soil types are considered: Rock, stiff soils and soft soils. Values vary from 1 to 2.

a: The special amplification factor, depending on the frequency of the motion, and the natural period of vibration of the building considered. Different codes give different expressions for determining factor a.

(USA: $\alpha = \frac{a}{3\sqrt{T^2}}$, where a = 1.20, 1.44, 1.80 for soil types I, II and III, USSR: $\alpha = \frac{a}{T}$, where a = 1.00, 1.10 and 1.5.

The maximum value of α should not exceed 2.5.

K: The behaviour factor according to the ductility level desired. Usually three levels are considered, I being for the less important classes of buildings and class III for the very important ones. Values vary from 2.0 to 5.0.

Other codes recommend similar expressions for calculating the seismic coefficient. The same factors are taken into account.

Nevertheless they sometimes give different results. Most of the codes of the American countries give the following formula:

$$C_d = Z \cdot I \cdot K \cdot C \cdot S$$

where Z is the zoning factor (corresponding to a in the CEB formula)

I is again the importance factor

K is the behaviour factor (corresponding to $\frac{1}{K}$ in the CEB formula)

C given as $\frac{1}{15}\sqrt{T}$

S is the soil conditions factor.

A comparison of the different methods for calculating, C_d, specified by codes of several countries is shown in Table 3.2 (reproduced from Wakabayashi M.[3] 'Design of earthquake-resistant buildings' with the addition of the method suggested by C.E.B.).

3.7.2 The distribution factor, y_i

According to CEB the distribution factor, y_i standing for the i-th floor is given as,

$$y_1 = h_i \cdot \frac{\Sigma W_i}{\Sigma(W_i \cdot h_i)}$$

It is believed, however, that this expression will not be accurate enough for buildings with more than five floors. Some codes[13] suggest that for buildings with more than five floors, 85% of the lateral seismic force should be distributed according to the above formula and 15% of the force should be included as a concentrated force at the top of the building.

Countries and codes	C_s = ZIKCS					Examples of C_s	Remarks
	Z	I	K	C	S		
Canada	A 0.02–0.08	I 1.0–1.3	K 0.7–3.0	S $0.5/T^{1/2}=1$	F 1.0–1.5	0.04	FS ≦ 1.0
Chile	1	0.3–1.2	0.8–1.2	C $0.10IT_s/(T^2+T_s^2)≦1$			T_s=0.20–0.90
China	α max 0.23–0.90		C 0.25–0.50	α/α max 0.2≦0.2/T–0.7/T≦1		0.11	
Germany West	$α_o$ 0.025–0.10	α 0.5–1.0		β $0.528/T^{0.8}$=1.0	K 1.0–1.4	0.07	
India	$α_o$ 0.01–0.08	I 1.0–1.5		C 0.2–1.0	β 1.0–1.5	0.06	
Italy	0.01(S–2)		β 1.0,1.2	R $0.362/T^{2/3}$=1	ε 1.0,1.3	0.10	S=2
Japan	C_oZ 0.2 0.7–1.0 1.0	I 1 1	D 1 0.25–0.55 0.25–0.55	R_t 1 $1-0.2(T/T_a-1)^2$ $1.6T_s/T$	 for T < T for T_s≦T 2T_s for 2T_s≦T	0.20 0.30	
New Zealand	C	I 1.0–1.6	SM 0.8–2.5	 0.8–1.2		1.2	
Romania	K_s 0.07–0.32		ψ 0.15–0.35	β 0.75≦3/T ≦2.0		0.08	
United States UBC	Z 3/16–1.0	I 1.0–1.15	K 0.67–2.5	C $1/15T^{1/2}$≦0.12	S(≧1.0) $1+T/T_s-0.5(T/T_s)^2$ for T/T_s≦1.0 $1.2+0.6T/T_s-0.3(T/T_s)^2$ for T/T_s 1.0	0.09	CS≦0.14
ATC–3	A_u 0.05–0.40		1/R 0.125–0.8	$1.2/T^{2/3}$	S 1.0–1.5	0.11	
U.S.S.R	K_c 0.025–0.10	0.5–2.0		β = $1/T_1$ 0.3–3.0	0.5–2.0	0.10	
Yugoslavia	K_s 0.025–0.10	K_o 0.75–2.0	K_p 1.0–2.0	K_d 0.5/T–0.8/T		0.10	
CEB	A 0.4	I 1.0	1/K 1/35	α 2.5	S 1.5	0.43	

It is assumed that Z (or A) is maximum, I=1, reinforced concrete moment resisting frame, hard ground and T = 0.5 S

TABLE 3.2 Calculation of the Seismic Coefficient using Codes of Several Countries

Even more accurately this expression is given[20] as follows:

$$y_i = \frac{h_i{}^k \bullet W_i}{\Sigma(h_i{}^k \bullet W_i)}$$

where $K = 1$ for buildings with $T < 0.55$ secs.
$K = 2$ for buildings with $T < 2.55$ secs.

Interpolation should be used for values of T between the above limits.

Newmark & Hall[18] (1982) suggest a useful method for checking whether the lateral force distribution is in agreement with the structure as designed:

1. Calculate lateral forces, f_i
2. Select member sizes
3. Calculate lateral displacements, x_i
4. Recalculate f_i replacing $h_i{}^k$ by x_i in the above equation.
5. If the two values calculated for f_i differ more than 30% then dynamic analysis is needed.

3.8 DYNAMIC ANALYSIS

The total lateral force, F_n (or base shear) is calculated taking into account the vibrational modes. There exist as many vibrational modes as the number of storeys. So the base shear for the n^{th} mode (storey) is given by:

$$F_n = \beta \bullet w_n \qquad \beta = \frac{A_p}{A_g}$$

where A_g is the acceleration of gravity

A_p is the peak response acceleration, the 'ordinate corresponding to the n^{th} natural period of the pseudo-acceleration response spectrum and damping ratio' as quoted from Wakabayaski M. 'Design of Earthquake-Resistant Buildings'.

b can be obtained from the design spectrum. An example of a design spectrum is given in figure 3.12.[3]

W_n is given by,

$$\frac{(\Sigma w_i \bullet \Phi_{in})^2}{w_i \bullet \Phi_{in}^2}$$

where w_i is the weight lumped at the i-th level and Φ_{LV} the displacement at the i-th level produced by the n^{th} vibrational mode.

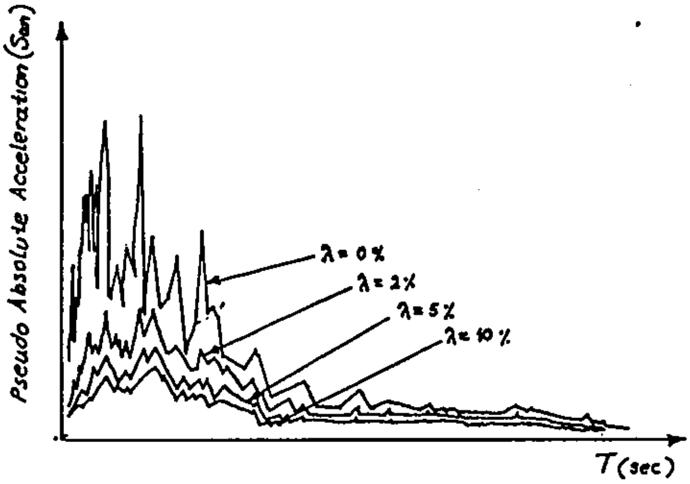
For most cases only the first mode (fundamental) need to be considered. This is known as the simplified dynamic analysis and is only allowed for regular buildings.

Response spectra, developed specifically for a given site are sometimes used for determining lateral forces. The curves of a response spectrum are presented on a 'four-way log plot' as shown in figure 3.13.[4] Damping factors are also taken into account when constructing the graph. Damping is induced not only by structural members but mostly by architectural and non-structural features like interior partitions or window walls.

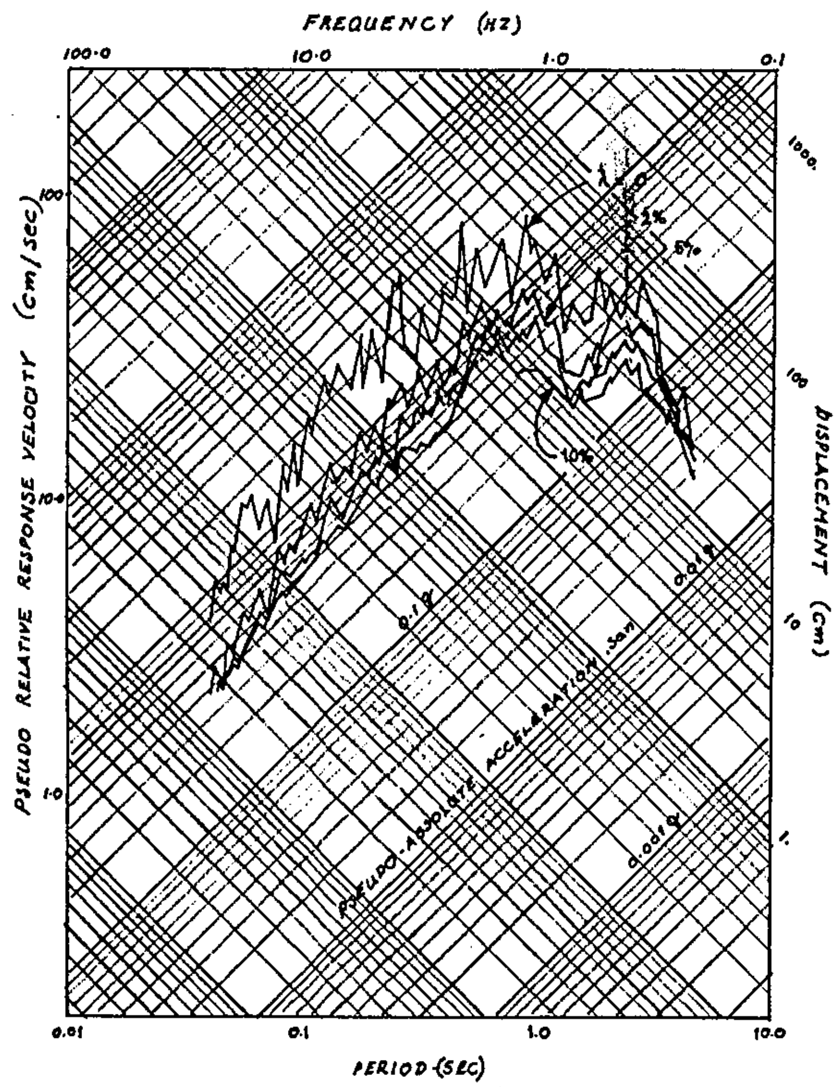
The lateral force can be distributed to each floor, i, by the following expression:

$$F_{in} = F_n \frac{w_i \bullet \Phi_{in}}{\Sigma(w_i \bullet \Phi_{in})}$$

For reasons that will be explained later, Dynamic Analysis is not necessary for buildings in Cyprus and therefore only a brief reference was attempted here. The aid of computers is necessary to proceed into employing a dynamic analysis method, especially for a multi-mode one.



3.12 Response spectrum



3.13 Design Response Sectrum

3.8.1 Periods of natural vibrations of buildings

The period of natural vibration (T) of a building is needed not only for dynamic analysis but is usually also required for the equivalent static analysis. The fundamental period is not just a design information required. It is essential to make sure that the fundamental frequency does not lie within the range at which resonance is likely. If the frequency of the forcing vibration becomes equal to the natural frequency of the structure resonance will occur and very high deflections and stresses are possible. In such cases, damping is of great importance because it can reduce the maximum response at resonance. Damping, that is loss of energy due to internal thermodynamic or fictional effects, may be provided by energy-absorbent finishes or partitions.

Every building has its own periods of natural vibrations, depending on several factors but mostly on the stiffness of the structure, the magnitude and the structural system and its dimensions. The first mode (fundamental) of vibration is the most important. To calculate the fundamental period accurately is not a simple task. Most of the seismic codes,[35] however, based on experimental results suggest simplified expressions for determining the period, T, like:

$$T_1 = 0.09 \, H / B \qquad \text{(France, Iran, Poland)}$$

where H is the height and B the width of the building.

$$T_1 = 0.1 \, N \qquad \text{(New Zealand, Canada)}$$

where N is the number of storeys.

$$T_1 = N/12 \qquad \text{(European Code – CEB)}$$

3.9 DIMENSIONING AND DETAILING OF REINFORCED CONCRETE

To achieve the desired level of ductility, reinforced concrete should be properly designed and dimensioned. The development of plastic hinges in columns should be avoided, since a column mechanism will create the total collapse of the building. The location of the hinges should be shifted, by proper detailing, towards the mid-span of beams. Generally beams should be designed to have lower rigidity and lower strength capacity than columns, using for example a different grade of concrete mix.

3.9.1 Dimensioning of beams and columns

Dimensioning involves consideration of geometrical constraints for beams and columns. Different seismic codes recommend similar constraints. Some general comments can be make looking at these codes:

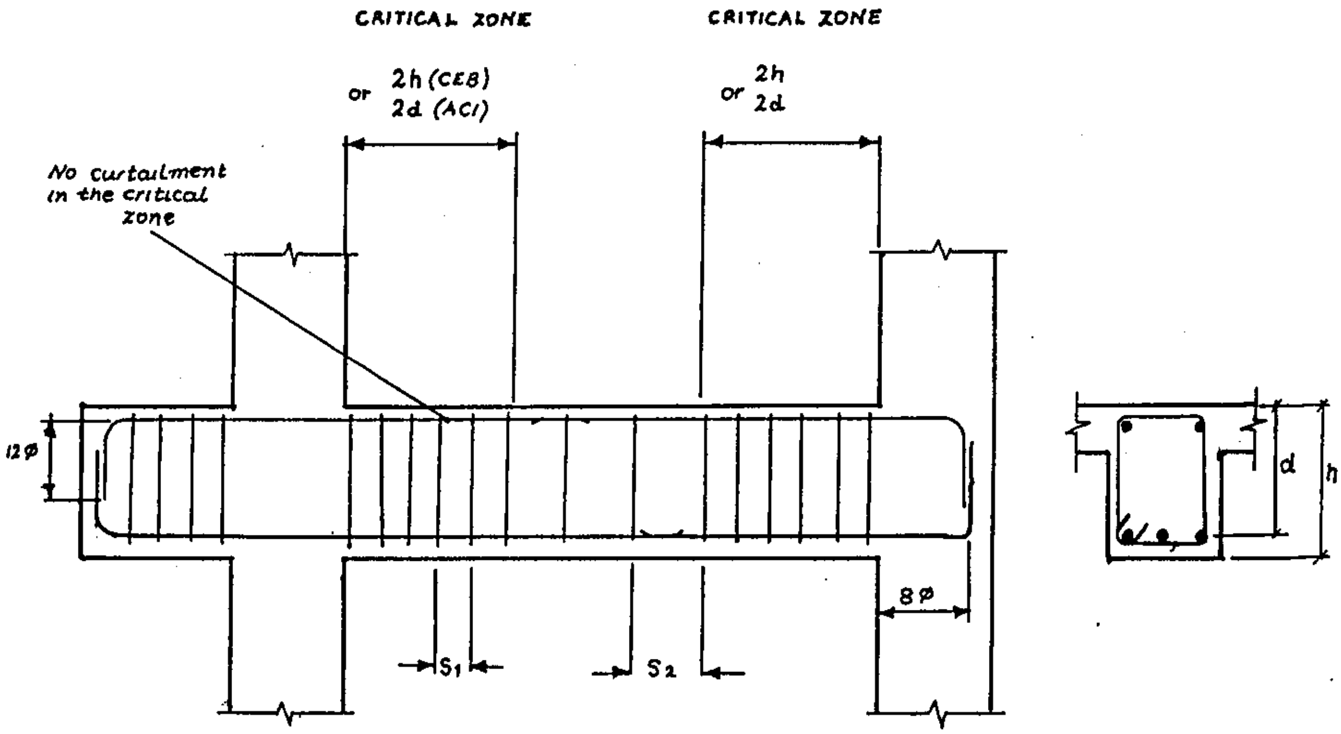
Beams: Width is constrained in respect to the depth of the beam and to the width of the column. It is usually limited to a minimum value of 200 millimetres. As it was seen previously, ductility is improved by increasing the width of the beam, up to the point of course that the beam is still a 'light but strong element'.

Columns: Again width is constrained in respect to the depth of the column. It is also constrained in respect to the ductility level desired. A minimum value of 250 millimeters is sometimes given. Obviously increasing the dimension of the columns will not only improve ductility but it will also decrease the shear stresses as shown by,

$$v = \frac{V}{bd}$$

The minimum values of width given for both beams and columns are based on observations and not on any theory.

Detailing of both beams and columns according to some codes (CEB, ACI, ARC) is shown in figures 3.14, 3.15 and 3.16. Some additional, very important, information will be given here:



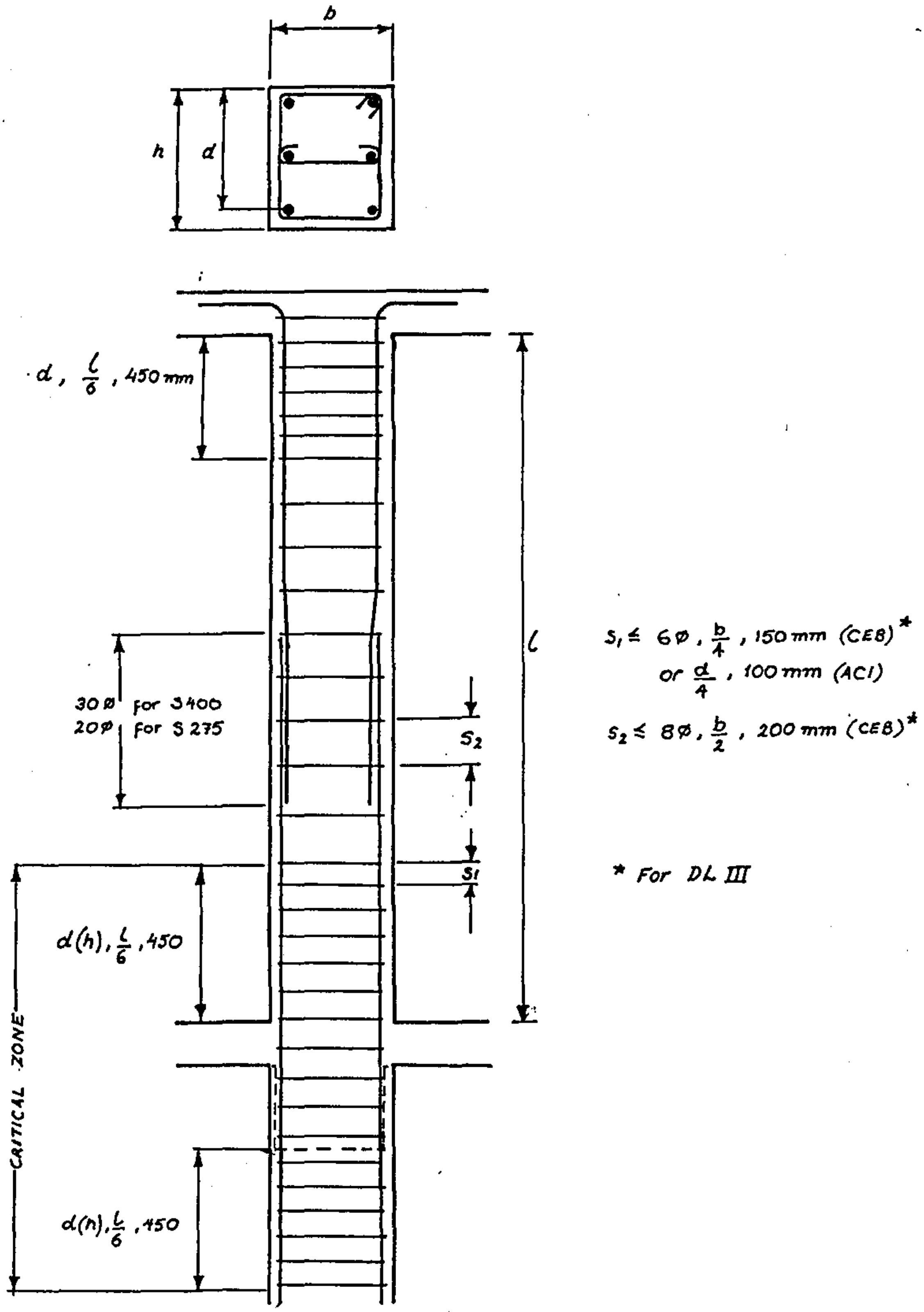
$S_1 \leq \frac{d}{4}$, 6Ø, 150 mm (ACI) or $\frac{h}{4}$, 6Ø, 150 mm (CEB)*

$S_2 \leq \frac{d}{2}$ or $\frac{h}{2}$, 600 mm.

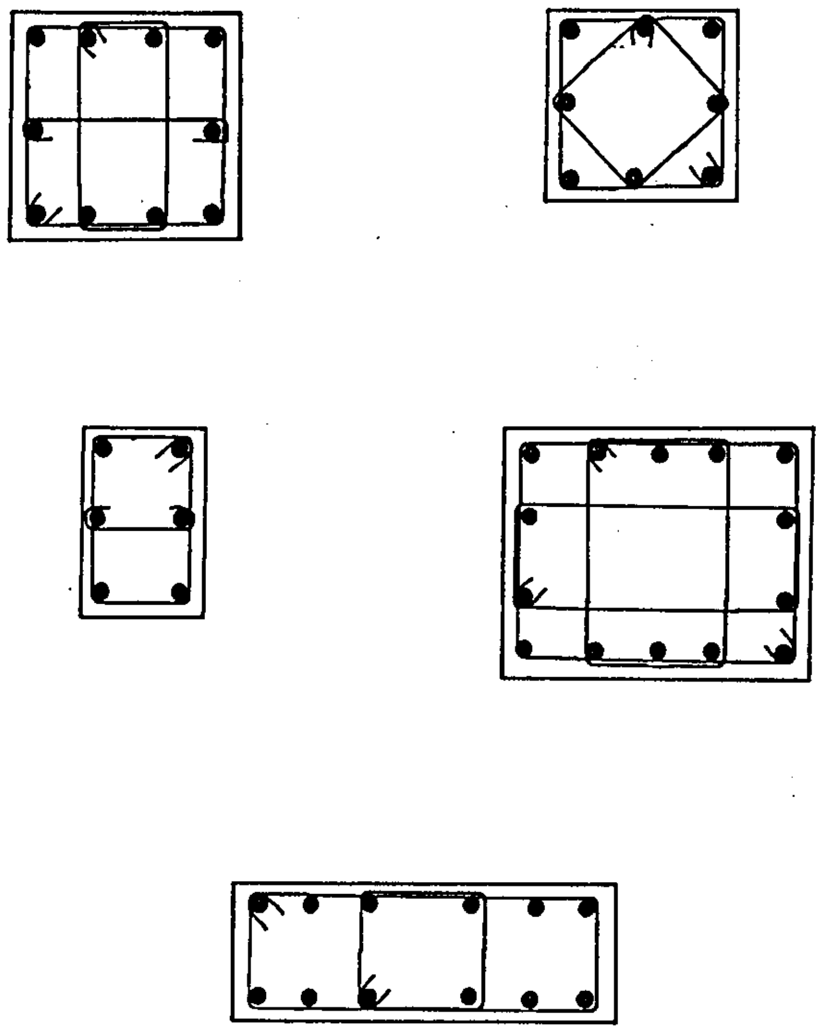
Ø is the diameter of the longitudinal bar (top or bottom - whichever is the smaller).

* For D.L. III

3.14 Detailing of beams



3.15 Detailing of columns



3.16 Typical arrangements of transverse reinforcement

Hinge zone: The plastic hinge zone is a critical zone where additional reinforcement is needed. The plastic hinge itself is shifted away from the column preventing its collapse. Within the hinge zone no splices of steel reinforcement is allowed.

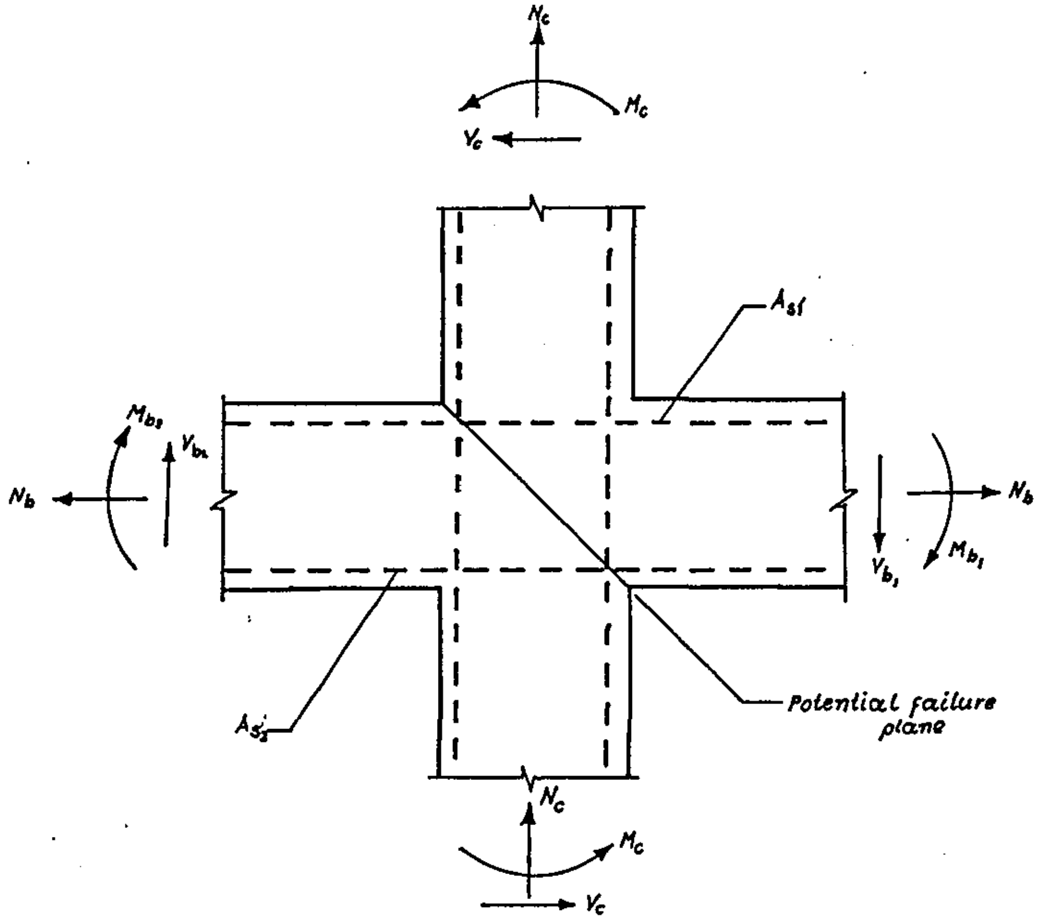
Beam-column joint: The joints should not fail before failure of the members framing into joints. In order to prevent brittle failure of the joint its ultimate bearing capacity should be 25% higher than that of the members. Figure 3.17 shows the forces acting on a joint. The most common mechanisms of failure are:

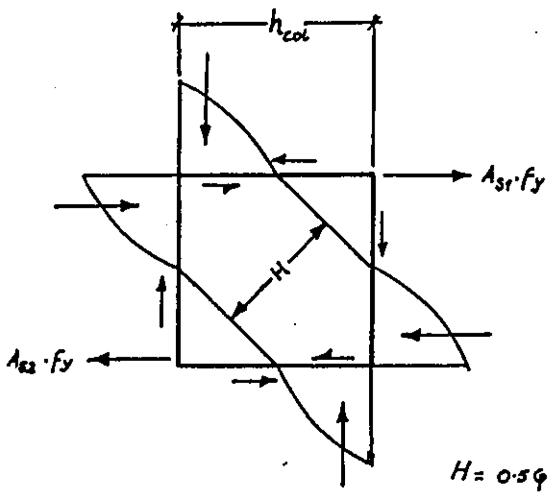
a. shear with the joint
b. anchorage failure of longitudinal bars
c. bond failure in longitudinal bars passing through the joint

Resistance to failure is applied by a compressive strut formed in the concrete (figure 3.17 c). This strut can be strengthened by steel reinforcement. Properly designed reinforcement will increase the rigidity of the joint thus preventing cracking tangentially to the failure plane. Therefore the longitudinal bars coming both from the column and the beams should continue across the joint. The transverse reinforcement (links) of the column must also continue through the joint.

Transverse reinforcement: The purpose of the transverse reinforcement are:

a. to provide shear resistance
b. to confine concrete in order to increase its ductility as shown in figure 3.18[35]
c. to restrain laterally the longitudinal bars subjected to severe plastic deformations and buckling
d. to improve bond between steel and concrete.



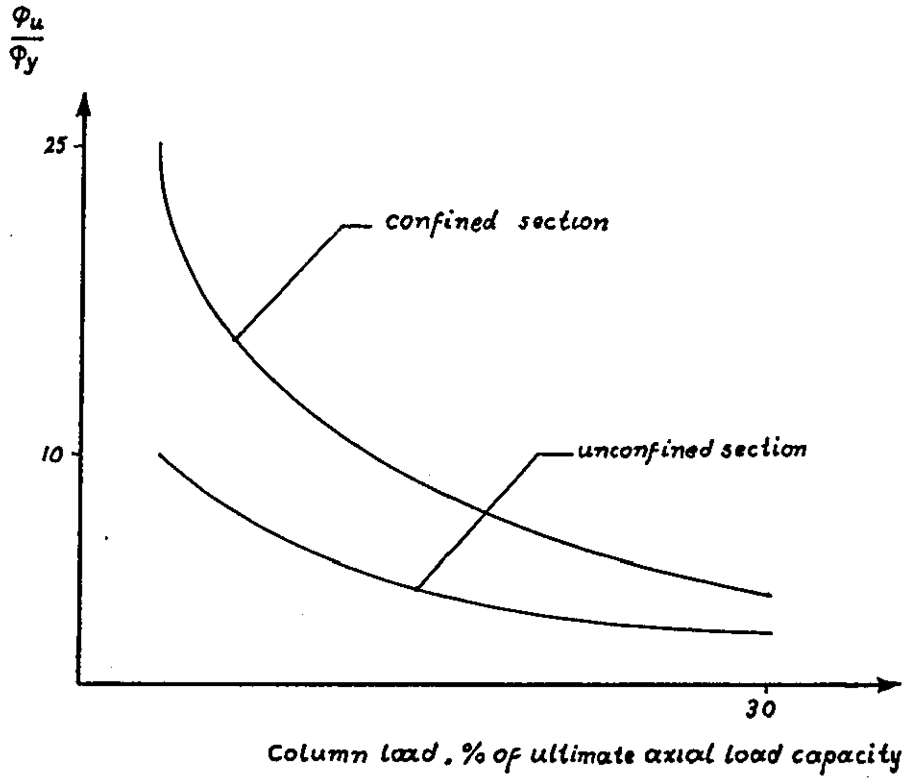


$$H = 0.5\varphi\sqrt{Z_{col}^2 + Z_{beam}^2} \quad \text{(metres)}$$

$$\varphi = 2 - \frac{e_o}{e} \qquad e_o = 0.33\,h_{col}$$

$$e = M/N$$

3.17 Forces acting on a beam-column joint



3.18 Ductility for columns

The transverse reinforcement should be denser at the critical regions and their spacing is specified by the seismic codes, according to ductility level.

3.10 SHEAR WALLS

A shear wall is an essential element in tall reinforced concrete structures and a valuable element for other structures. In a frame system a shear wall resists lateral seismic force while the structure behaves elastically. The combination of a frame system with shear walls has two main advantages: First it is more economic than using a frame system on its own. And second the combined system is more ductile[38] (see figure 3.19). Ductility of course also depends on the steel reinforcement ratio given as:

$0.0025 < p < 0.04$ for mild steel

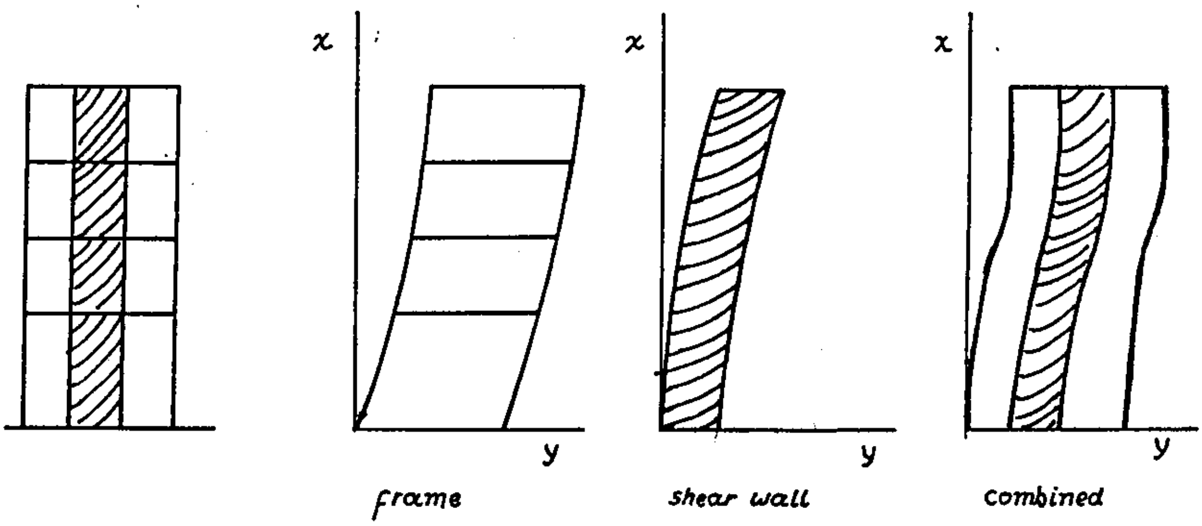
$0.0017 < p < 0.04$ for high yield steel

Reinforcement is usually provided in the form of a grid and it is needed in both faces of the wall. The reinforcement ratio requirements should be satisfied by both horizontal and vertical reinforcement. The two edges of the wall are considered to be critical since strain is maximum there. At a distance, L_c from the edge therefore extra reinforcement is needed according to seismic codes. The critical length, L_c is given by EC8 as:

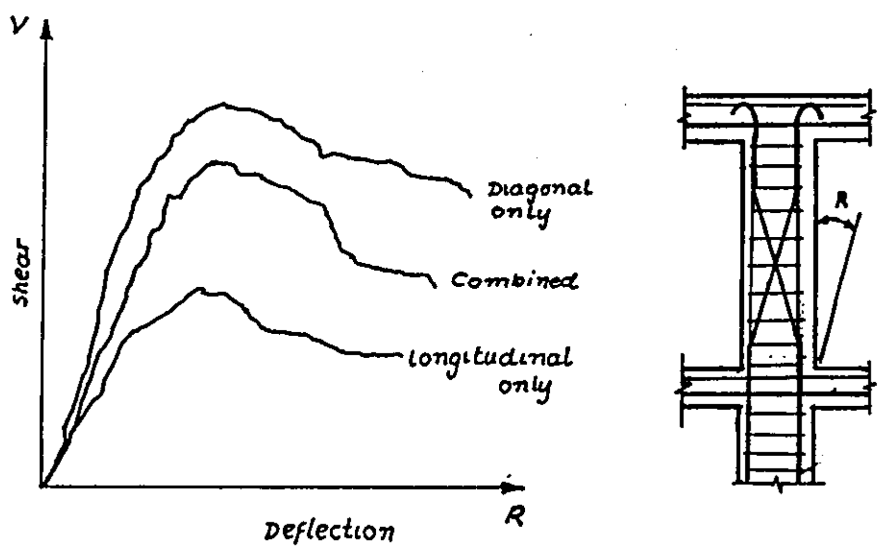
$L_c = 2\ b_w$ where b_w is the thickness of the wall

or $= 0.2\ L_w$ where L_w is the length of the wall

Openings in a shear wall should preferably be avoided, otherwise additional strengthening is needed around the openings, since a smaller length of the wall will have to resist the same amount of strain. It is recommendable in such cases to use diagonal reinforcement. Diagonal reinforcement (figure 3.20) can provide larger shear strength, larger ductility and it can slow down degradation.[4]



3.19 Behaviour of a combined system



3.20 Diagonal reinforcement

3.11 FOUNDATION

Horizontal shear is imported to the building by the movement of the supporting ground. Although shear failure between foundation and ground is uncommon, some measures are recommended for improving resistance against earthquakes.

The supporting soil should be solid and rigid and homogeneous under the whole base of the building. Two types of soil may create problems:

a Dry sands, because of great possibilities for excession settlement. As a result of the shaking the sand will be consolidated.

b. Saturated sand, because of the danger of liquefaction. A sharp drop in shear strength will occur when the soil is shaken over a period of time.

If of course there is no chance of avoiding such conditions then the properties of the soil may be improved by a number of methods like vibro—compaction, drainage increasing the grain structure stability .[31]

However, even bedrock can create problems. It can modify seismic excitation. It is recommended that tall structures built on rock should be flexible to avoid resonance. For the same reason low—rise structures built on soft clay should be stiff as it was explained earlier in this chapter (3.3).

Provision of ductility in the foundations is unusual. It is good practice to tie together the individual pads using tie—beams at the bottom of the foundations. Stepped foundations should be avoided.

3.12 NON-STRUCTURAL ELEMENTS

A large part of the damage by earthquakes is non-structural. Failure of non-structural elements, like windows, ceilings, mechanical equipment, parapets can destroy humansafety systems blocking evacuation routes. They represent usually 70% of the value of a building.

Nevertheless few codes take them into account and little basic research has been done. They are, however, equally important with the structural members to the maintenance of the integrity of the structure.

For Equivalent Static Force acting at the centre of gravity of an architectural element was determined by the Uniform Building Code (U.S.A.) as:

$$F_p = Z \cdot I \cdot C_p \cdot W_p$$

where Z is the zoning coefficient
I is the importance of the building
C_p is the seismic coefficient given Table 3.3 (a)[6]
W_p is the weight of the element

The building Seismic Safety Council (USA) suggests an alternative approach:

$$F_p = A_v \; C_c \; P \; W_p$$

where A_v is the ground acceleration
C_c is a seismic coefficient given in Table 3.3 (b)[6]
P is a performance criterion factor (values from 0.5 to 1.5)

Some more practical measures are also recommended by these two codes:

PART OF BUILDING	DIRECTION OF FORCE	C_p
Non-structural walls	Normal to plane	0.3
Chimneys	Any direction	0.8
Parapets	Normal to plane	0.8
Appendages	Any direction	0.8
Tanks connected to a building	Any direction	0.3
Suspended ceilings	Any direction	0.3

TABLE 3.3 (a) The seismic coefficient, C_p

ARCHITECTURAL COMPONENTS	C_c
Exterior non-load-bearing walls	0.9
Wall attachements	3.0
Veneers	3.0
Roofing units	0.6
Stairs	1.5
Elevators	1.5
Corridors	0.9
Partitions	0.9
Ceilings	0.9
Architectural equipment mounted on walls or floors	0.9
Electrical, Fire, Lighting systems	2.0

TABLE 3.3 (b) The seismic coefficient, C_c

a. All non-structural members should be adequately anchored.
b. Mechanical equipment should be preferably located at the lower floor.
c. Non-structural infill panels should be strong enough and flexible to absorb deformation. These panels should be separated from the structure especially when flexible frames exist. The gap between the frame and the panel should be at least 40 mm. This of course will create problem with sound and fire-resistance, although special materials can be used to fill the gap.
d. Ducts for services should not be tied to the non-structural partitions.
e. Brittle finishes should be avoided or specially detailed since it is difficult to avoid them. Heavy finishes like marble or stones should be limited.
f. A grid should be used at the windows to hold the glass.

Non-structural elements, although not extremely important, sometimes can affect indirectly or even directly the earthquake resistance of a building. They can also put in danger the safety of human lives thus preventing us from achieving the most important aim of the aseismic design: to save lives. It is therefore essential to give more attention to the non-structural elements and more research should be done in this area.

3.13 CONCLUSION

In this chapter a review of the main requirements for the seismic design of buildings has been presented. A number of important parameters have been established:

a. The need for ductility to allow large deformation to occur and absorb energy.
b. Buildings should be regular with their centres of mass and rigidity being close enough, to avoid complication in analysis and torsional problems.

c. Stiffness should be distributed in all stories equally whereas 'soft stories' should be eliminated.

d. Beams should be designed to fail before columns to create a failure mechanism that will not lead to a total collapse of the building (as far as possible).

e. Beam-column joints should be adequately reinforced and strengthened zones should be provided in adjacent beams and columns to shift plastic hinge formation far enough away from the joint.

f. Shear walls greatly improve the earthquake resistance of buildings.

g. Non-structural elements should also be designed to resist seismic loads.

Having established the major points required for good performance under seismic loads, it is now possible to examine any building practice to see how that matches the requirements for earthquake-resistant design.

4. BUILDING PRACTICE IN CYPRUS

4.1 INTRODUCTION

In the previous chapters the seismic risk for Cyprus has been established and a summary of good design techniques for structures to withstand seismic loading has been given. In this chapter a review of established building practice in Cyprus is given. Prior to 1984 little attention was given to aseismic design but after that date new design rules were gradually introduced. In 1987 the Cyprus Association of Civil Engineers published recommendations to improve seismic design and in 1991 a (new) draft code for seismic design was published.

4.2 BUILDING IN CYPRUS UNTIL 1984

During the last forty years or so, almost hundred per cent of the buildings in Cyprus have been constructed in reinforced concrete. Concrete has replaced the old traditional materials used, namely the adobe block reinforced with hay for the walls and the timber and tiles for the roofs. Everywhere in Cyprus now, reinforced-concrete framed structures are erected. The infill panels between the framework are made of brickwork.

It is quite useful to mention something about the Cypriot attitude when building their houses. The appearance of the house not only from outside but inside as well, is the leading factor influencing its architecture. For the owners (clients), aesthetics are much more important than anything else. Usually the bigger room in the house is the 'sala' the guests' room. It is the show room of the house with the best possible furniture and decoration. Although the 'sala' covers usually a large area, no columns should appear in the middle of the room. It is also very common in Cyprus to see an 'open' ground floor (see photo 4.1) where the house is actually raised from the ground and built on columns. Generally the effort to make an interesting-looking

Photo 4.1. An "open" ground floor
a common case in Cyprus.

house led to buildings of irregular shapes, with non–uniform stiffness and flexibility. This is considered as a very bad practice when considering earthquake–resistant structures, as it was explained in chapter 3.

Until 1984 there was a lot of confusion in the rules governing the design of the structures and even today there is still not an official code of practice, uniformly applied in the whole of Cyprus. Since there was not a university, the Cypriot Civil Engineers are graduates of Greek, English and American universities and also of some eastern European ones. Therefore the designers were employing methods recommended by the codes of these countries to produce their static analysis and design the structures. This fact created a non–uniformity in the building industry and it was difficult to control or check the design. Additionally site–supervision by a professional Engineer was not forced by law and this led to structures being constructed differently from the design specified.

4.3 QUALITY OF WORKMANSHIP, MATERIALS AND DESIGN

In Cyprus, education is considered to be very important, and although there are no universities yet, ten thousand Cypriot students are travelling every year to other countries to study at universities. Some more are studying at the local colleges and institutes. The majority of young people – and not only the elite – seek to acquire a higher standard of education. This has obviously created a problem to the building industry. Where the old–experienced skillful craftsmen are becoming extinct. The new craftsmen, being a selection of second class, cannot really reach the high standards of the old craftsmen. As a result the quality of workmanship is lowered and professional supervision is therefore becoming more necessary.

Until 1980 or so, concrete was prepared by labourers at the site using a small mixer. Now, ready–mixed concrete is used. The concrete is prepared according to standards (which are similar to British standards). Concrete of Grade 20 (1:2:4) was usually used

for slabs and beams and Grade 25 (1:1 1/2:3) for columns. Floor slabs, they are generally constructed with a thickness of around 150 to 200 mm (in agreement with British standards).
This thickness is considered to be considerable and the slab is heavy as far as earthquakes-resistant buildings are concerned.
The reinforcement used consists of steel S 400 (yield strength = 400 MPa).

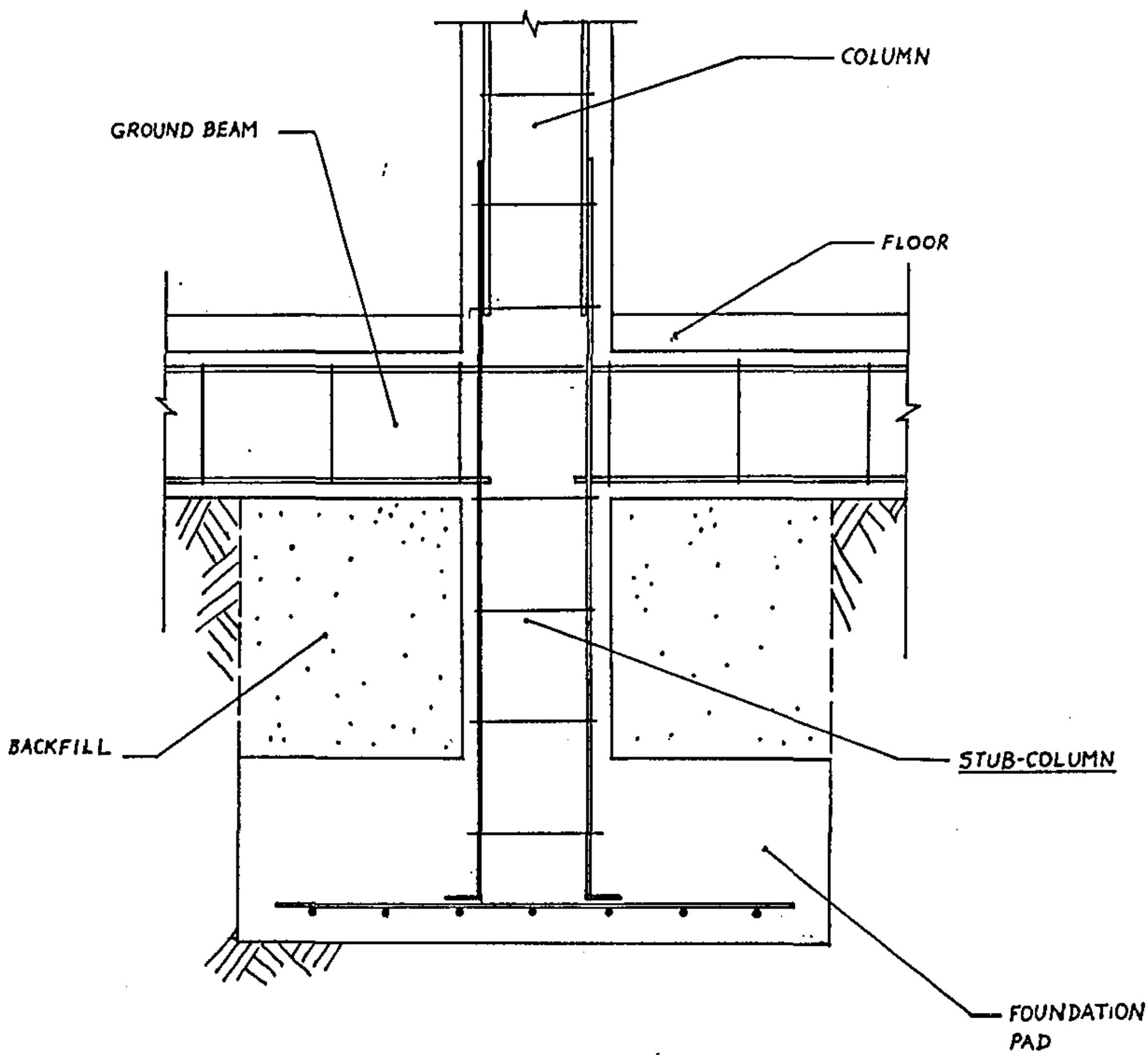
Until 1984 no aseismic measures were taken. The common practice in fixing the reinforcing bars was in some cases very unfavourable to aseismic measures. It was not only the common practice, where these problem arose, as the detailing drawings made by the designer often specified bad aseismic details. Some examples of bad practice (as far as aseismic design is concerned) can be given here:

Foundations-Stub-Columns

The individual pad bases were not tied together as is generally recognised in aseismic design. Additionally a particularly bad practice was employed by some contractors to economise on the concrete in the foundations. They did not fill the pads up to the level of the bottoms of the ground beams and thereby created the so-called stub-columns. These stub-columns (see figure 4.1) were not taken into account in the analysis or design and created a very undesirable 'stiff-storey' contrary to the need to maintain an even stiffness distribution between the stories as explained in chapter 3.

Joints

Until 1984 the only reinforcement of the joints were the main longitudinal bars of the columns. No links were passing through the columns and of course no extra reinforcement was included. No critical zones were identified around the joints. So plastic hinges were likely to form at the joints leading to catastrophic collapse mechanism under seismic loading.



4.1 Stub columns - A bad practice for the earthquake resistance

Walls

The walls, now, are everywhere made of bricks. The bricks are made of clay and are manufactured in the local factories.[58] They exclusively have dimensions of 100 X 200 X 300 mm, with either circular or squared holes (see photo 4.2). There is not much difference between the two types, structurally. According to CYPRUS STANDARDS[19]: 1983 the crushing strength of these bricks should not be less than 1 N/mm^2. Unreinforced masonry walls should be avoided for earthquake-resistant buildings. The ductility of a masonry wall can be improved by reinforcing it with steel. However no reinforcement at all is employed in Cyprus even today. The thickness of the external walls is usually 200 mm and that of the internal walls 100 mm. Bad practice is also observed when building the top layers of bricks as shown in photo 4.3 Such walls are not uniform and their behaviour during an earthquake is unpredictable. Concrete shear walls were very rarely employed. The only case where some sort of shear walls were used, was when constructing the core of the lift in multistorey buildings.

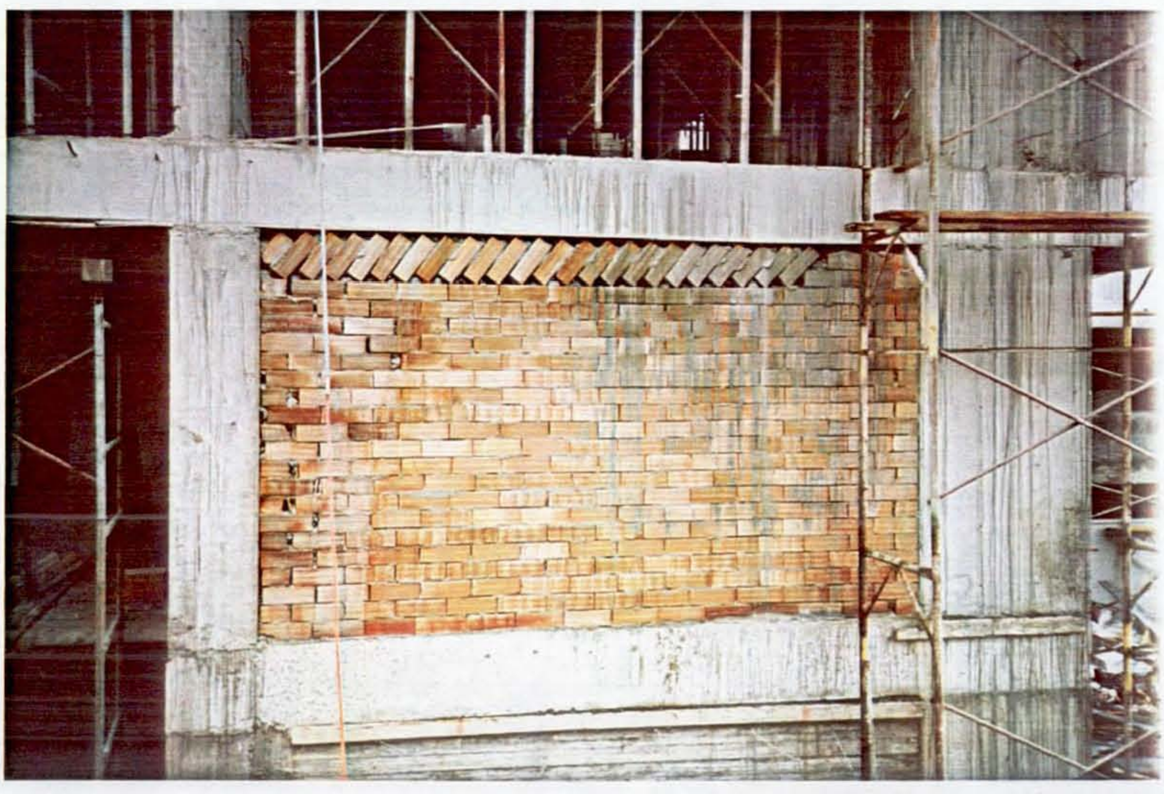
4.4 BUILDING IN CYPRUS SINCE 1984

In 1987 the Cyprus Association of Civil Engineers and Architects published a booklet called 'Measures For Protection From Earthquakes'. It was a set of recommendations to Civil Engineers to help them improve the earthquake resistance of their structures. The publication of this booklet came as a demand by many Engineers who had already started applying some ideas since 1984. This set of recommendations was far from being a code, nevertheless, it was a first step towards designing earthquake-resistant structures. However Civil Engineers had to refer to codes of other countries for the design, but the booklet was setting a number of conditions to be satisfied.

According to the recommendations using a recognised method (seismic code) a seismic analysis should be produced. Structures lower than 3.5 metres and not longer than 6 metres needed not to be analysed for seismic forces, as long as shear walls were

Photo 4.2. Typical clay bricks used in Cyprus.

Photo 4.3. Bad practice seen on the top layer of bricks.



constructed in the framed structure. Generally the booklet gave a lot of emphasis to the construction of shear walls. A very interesting formula was also given as a means for determining the length shear walls needed. This formula has not been encountered in other literature:

$$Wa\Pi > \frac{H^2 F \varepsilon}{5500}$$

where $Wa\Pi$ = section modulus of the shear wall in m^3

H = Total height of the building in m

F = Area of each floor in m^2

ε = seismic acceleration = $\frac{As}{g}$

To prove the above expression it is assumed that for a building of total height, H and floor to floor height equal to 3 metres,

mass of each floor = 1000 F (Kg) (= 10 KN/m^2)
lateral force per floor = 1000 $F\varepsilon g$ (Kg)

then,

$$\text{force per metre height} = \frac{1000 \cdot F\varepsilon g}{3} \text{ (N/m)}$$

$$\text{moment } M = \frac{WH^2}{2}$$

$$= \frac{1000 \cdot F\varepsilon \cdot H^2 \cdot g}{6}$$

Assuming that,

$$\sigma = 85 \times 10^4 \times g \text{ N/m}^2$$

and Waπ $= \dfrac{\text{Moment}}{\text{stress}} = \dfrac{M}{\sigma}$

$$= \frac{1000 \cdot F\varepsilon \cdot H^2 \cdot g}{6 \times 85 \times 10^4 \times g}$$

$$= \frac{H^2 F\varepsilon}{5100} \quad m^3$$

which is very near to the suggested formula.

Now, Wall $= \dfrac{bd^2}{6}$ for a wall $(= \dfrac{bd^3}{12}$ for a core) with a length of d metres and a width of b metres. Therefore using.

$$\frac{bd^2}{6} = \frac{H^2\ F\ \varepsilon}{5500}$$

and assuming that the width of the wall is 200 mm (minimum width according to the conditions and most common case in Cyprus), then the length of the shear wall can be calculated. Note here that the minimum length recommended is 1.2 metres.

As far as the seismic acceleration, A is concerned, it was given as,

Zone I	A = negligible
Zone II	A = 0.06 g (cm/sec2)
Zone III	A = 0.10 g (cm/sec^2)

The three zones are shown in a Map in figure 4.2 which is also included in the booklet.
Otherwise the booklet (consisted of 7 pages) is dominated by instructions on constructing shear walls.

Following the publication of the booklet a number of seminars and short courses were organised informing engineers on how to produce an aseismic design and asking them to start designing and constructing earthquake-resistant structures.

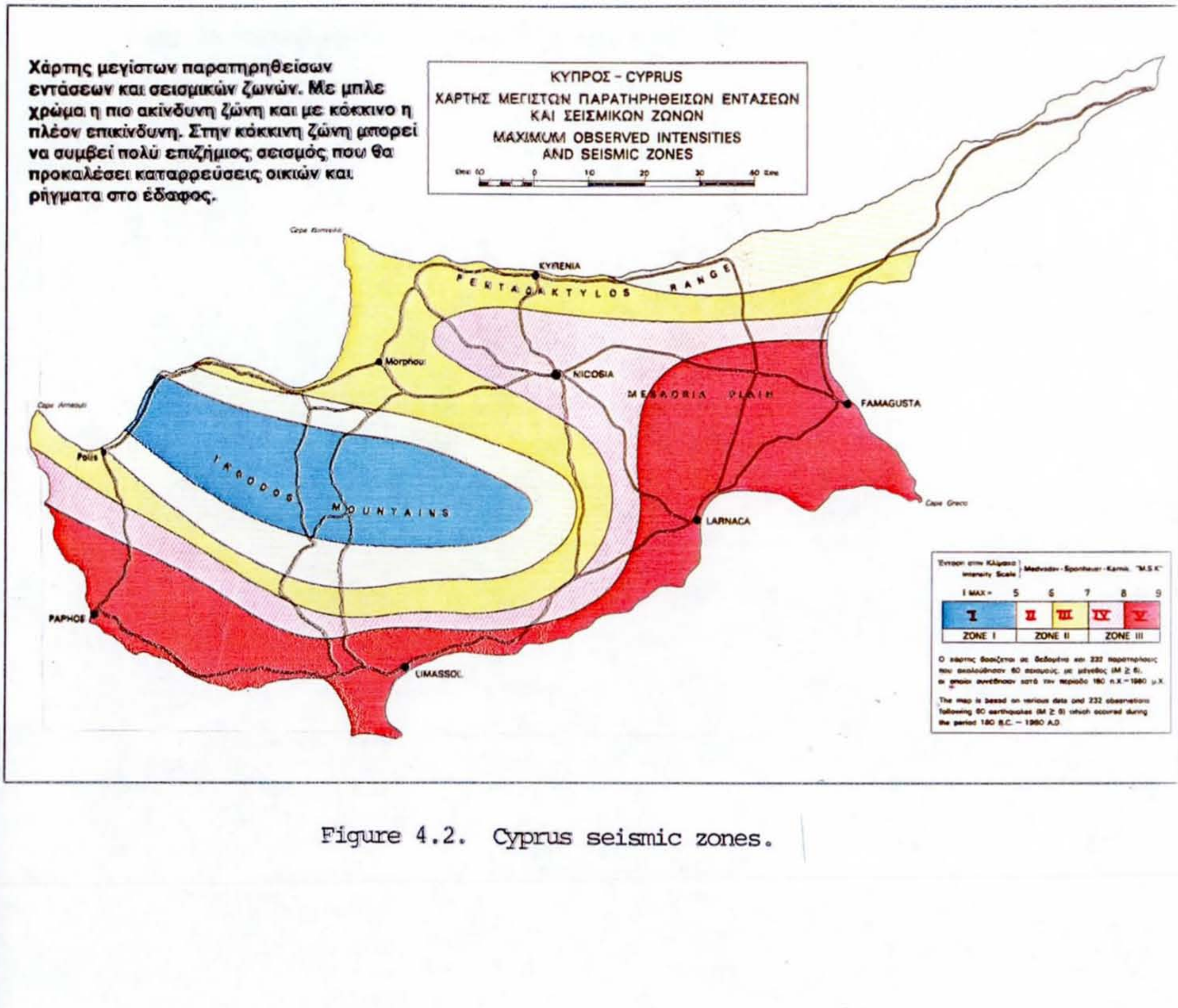


Figure 4.2. Cyprus seismic zones.

Even without a Law forcing them to do it, Cypriot engineers have since 1984 started applying the new 'ideas' bringing a number of changes. These, of course went as far as their own knowledge, on one side, and building tradition on the other, allowed them to go.

4.5 RESULTS

The changes brought into building as a result of the growing awareness for the need for aseismic design, were mostly structural. Architecturally rather irregular shapes are still produced and existing practice was only partially improved.

Masonry is still not reinforced, whereas the so-called 'stub columns' are limited but still re-appear sometimes, (see photo 4.4 (a) and (b)) especially in sites where no professional supervision exist. The absence of a law forcing professional supervision has created many problems allowing bad practice to continue. However, a new law has now been imposed which makes supervision compulsory.

Most of the changes have been observed in the construction of the frame. First the size of columns and beams was generally increased which has caused a lot of argument and disagreement with Architects. Civil Engineers wanted the columns to have a minimum width of 250 mm, in accordance with the recommendations given by several seismic codes. The Architects did not like the idea because it created architectural problems. The clay bricks used in Cyprus can match in walls of either 100 mm or 200 mm width since they have dimensions of 100 x 200 x 300. Therefore they did not want either the columns or the beams to create any projections and they supported the principle that it makes no difference if the width is 200 or 250 mm as long as it is aseismically reinforced with steel.

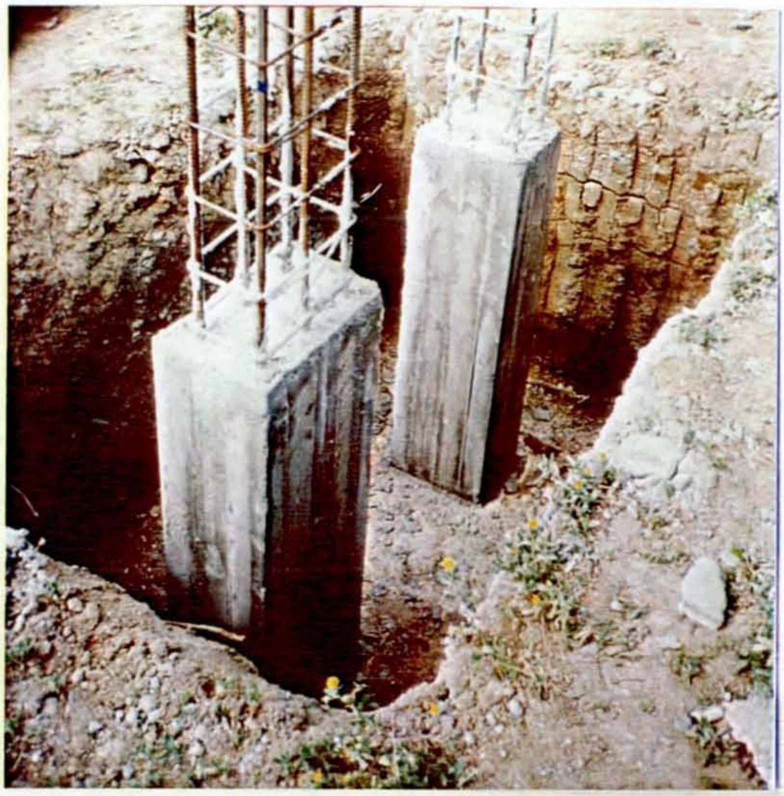


Photo 4.4(a). Columns filled with concrete up to the surface of the ground.

Photo 4.4.(b). Stub-Columns are thus created. The construction of ground beams will follow as shown in figure 4.1.

Nevertheless, although it cannot be proved, an increase in the width of the column will increase the ductility and the shear strength of the concrete preventing it collapsing in a britle way as it was explained in chapter 3. At the moment both columns and beams have an increased depth whereas the width is usually kept to 200 mm.

Shear walls are now well introduced (see photo 4.5) and especially at the corners of the building (photo 4.6). The problem is that they are sometimes not positioned correctly. It is not enough to introduce the right amount (length) of shear walls but some experience is needed to position them in a way that they will serve their aims. Sometimes they are positioned in such a way that they create torsional problems since the centre of rigidity and the centre of mass are well apart.

Seismic Joints are sometimes employed especially in buildings with a large plan area or a non-symmetrical shape in plan.

Where most of the improvement was made, however, was in the steel reinforcement detailing (see photo 4.7 and 4.8). The methods employed for detailing the concrete varies from building to building, according to which seismic code is followed. Generally it can be said that there is an increased amount of steel reinforcement mainly in the columns and the beams. Critical zones are now detailed with care in most of the cases. The links spacing is denser and extra longitudinal bars are added. The shear links are continuing throughout the column-beam joints. Finally the length of the splices is increased to 40 - to 50 Ø, where Ø is the diameter of the bar with the larger diameter. This increase is rather a lot since a 30 Ø splice would be adequate according to most of the seismic codes. Once more site supervision is necessary due to the reluctance shown by the steel-fixers to follow instructions given in the drawings considering the changes and the increase of steel as 'exaggerations!'

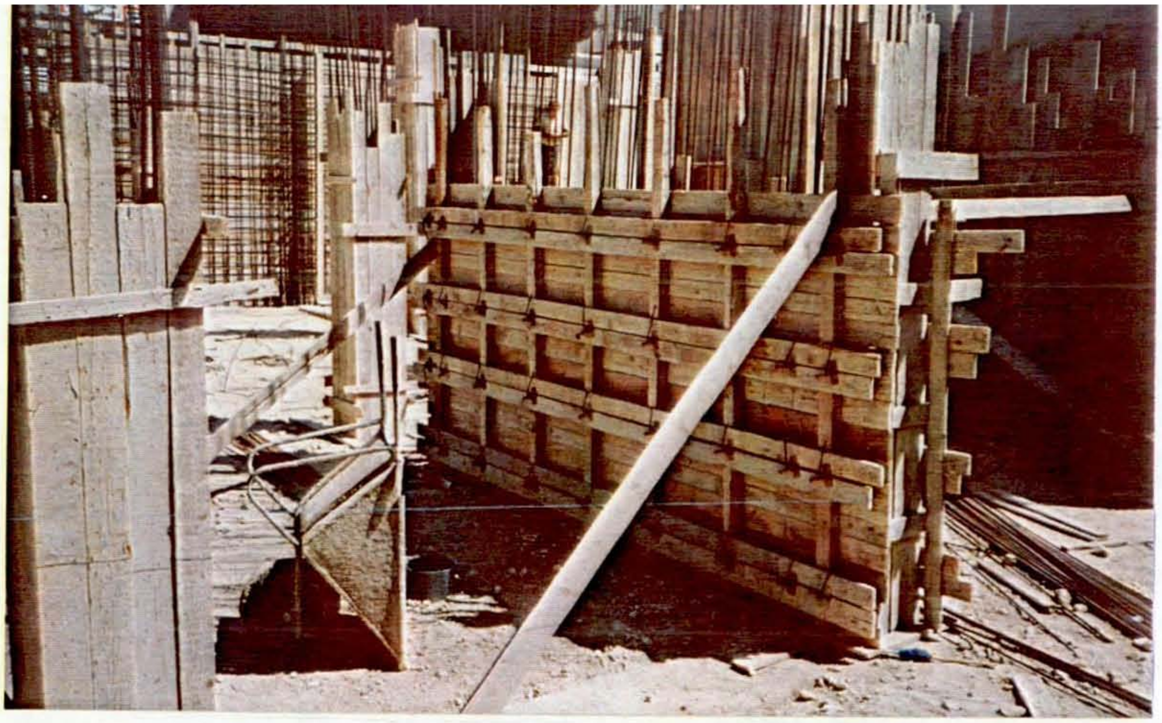
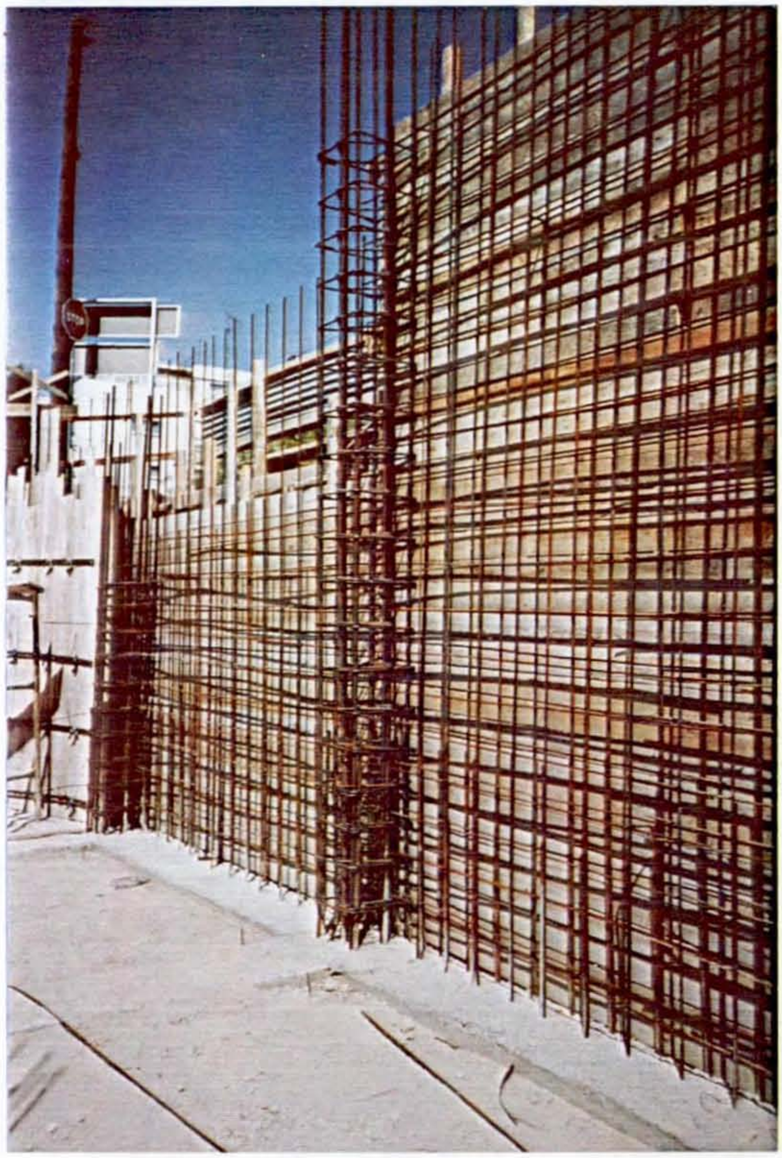
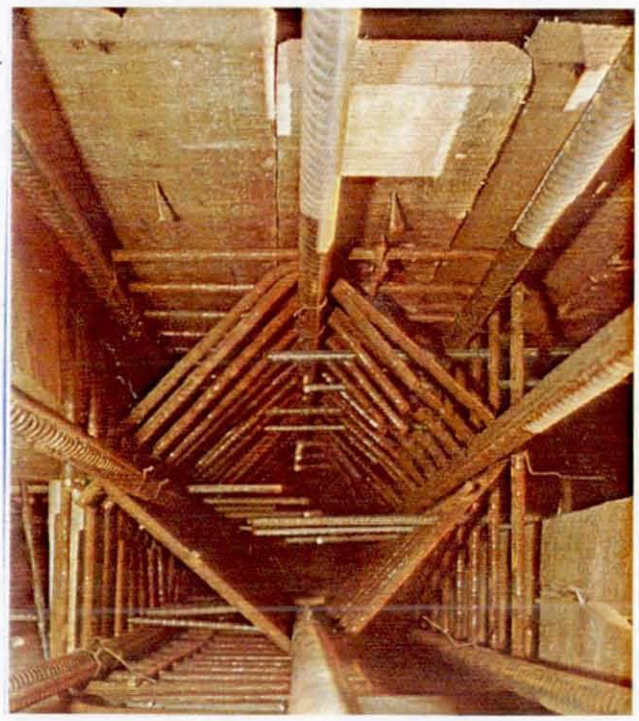


Photo 4.5. Internal shear wall for earthquake resistance.

Photo 4.6. Corner shear walls. Note however that in this case the shear wall does not continue into the foundations leaving the ground floor unprotected.



Photos 4.7 & 4.8. Steel reinforcement in columns is now denser and the diagonal links added.



There is, however, still a lot to be done. For example the splices are still occurring in the critical zones. Some of the most common cases of bad practice as far as earthquake-resistance is concerned are shown in figure 4.3.

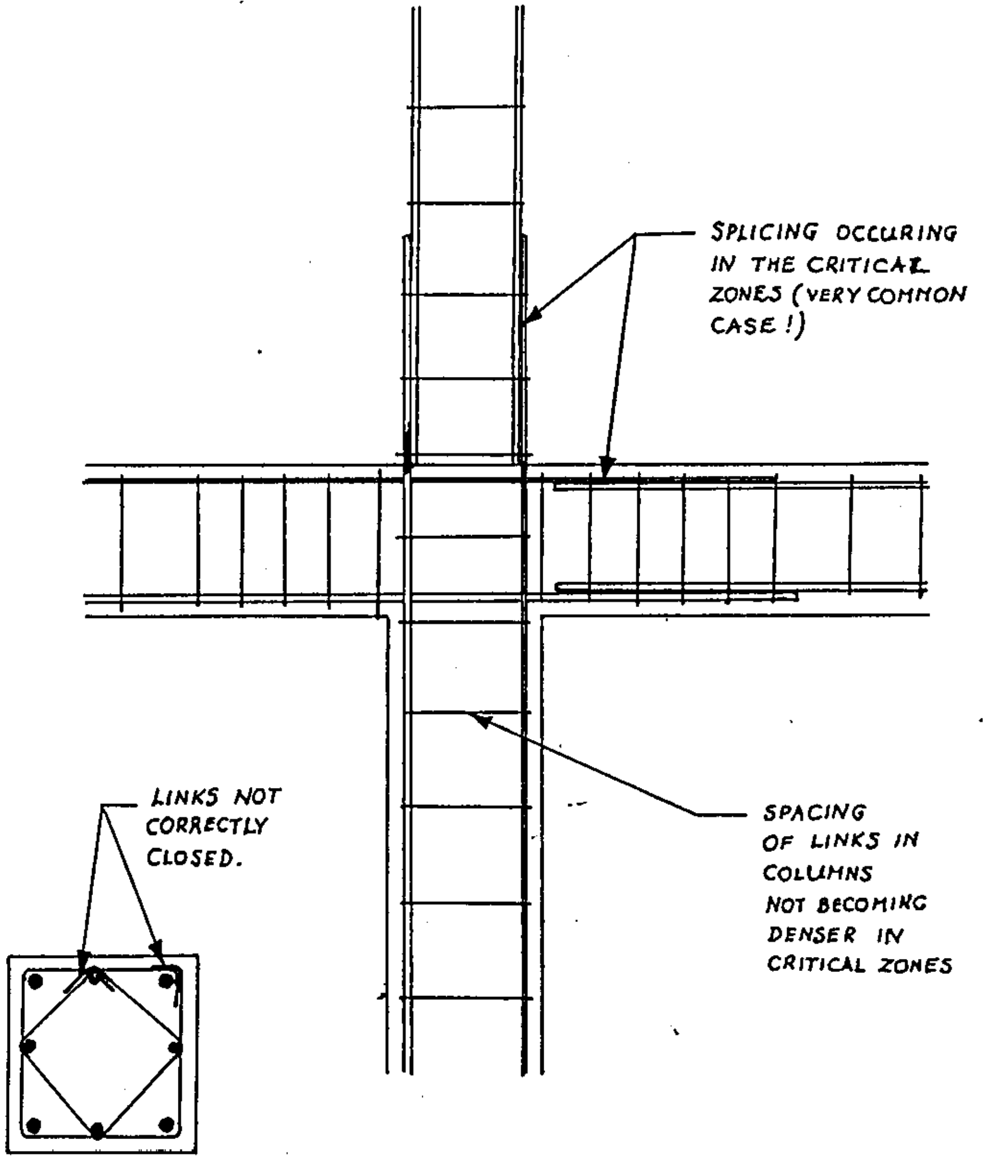
4.6 A SEISMIC CODE FOR CYPRUS

Together with the decision to apply the aseismic measures recommended, a committee consisted of Civil Engineers was appointed to work out a seismic code for Cyprus. The draft[37] of the Code was ready in 1989 and after a lot of discussion it was handed in to the government in 1991. In July 1992 the Code was officially accepted and became a law. Extracts from the official Code are presented in Appendix II translated into English.

The committee undertaking this task faced some serious problems making things really difficult for them. There was no possibility of direct investigation, and therefore they had to use as a basis other codes and works by engineers in other countries. There was no adequate information about the seismology of Cyprus. The Seismology Centre of Cyprus, however, helped them a lot giving them as much information as possible. Finally, there was no financial assistance and the whole work was done voluntarily.

The model code of CEB (Euro-International Committee for Concrete) bulletin D' information - was used as the base for the code for Cyprus. The effort of the committee was to follow the code CEB as closely as possible. The main idea is that two separate analysis should be produced: First the normal static analysis and then the seismic force analysis. The main difference between the two analysis is the load combination taken into consideration. The seismic load combination is given by the draft of the Code for Cyprus as,

$$S_d = S\ (G + E + \Psi Q)$$



4.3 Some bad practice still observed in Cyprus where S is a soil factor given in the code according to soil characteristics

G is the gravitational loads (nominal value)

E is the seismic load

Q is the imposed loads

Ψ is a factor given in the code according to the part of the structure which shows what percentage of the imposed load should be taken into account. Values of Ψ given by Cyprus code are different from the typical ones given by CEB.

Comparing the above expression with the expression given by the code of CEB (chapter 3) it can be seen that prestressed concrete is not covered by the Cyprus code since such structures are very rare.

The concept of the ductility levels is introduced. In Cyprus structures are now categorised as a ductility level I (DLI - ordinary structure without seismic design, where limited plastic deformation will develop). Ductility level III is still out of the scope as far as Cyprus is concerned.

Another concept introduced is that of the reliability. Buildings are classified according to their importance to one of the reliability levels. CEB is classifying all buildings to two levels only. According to the draft of the Cyprus Code five reliability levels exist (see Appendix - clause 3.2). Thus a more accurate differentiation is made. The five classes are as follows:

Class I: Buildings where collapse may have catastrophic consequences (nuclear stations etc), buildings with more than 15 floors.

This class does not apply to Cyprus since such buildings do not exist.

Class II: Buildings with large number of occupants (cinemas, halls, etc.) or important communal buildings (hospitals, schools, airports, etc.).

Class III: Multistory buildings, houses, restaurants and other buildings not included in classes II and III;

Class IV: Auxiliary buildings and farms.

Class V: Temporary structures where collapse will not create any danger to people.

For this class no seismic analysis is necessary.

Two other things must be pointed out: First the seismic code for Cyprus limits the height of a 'regular' building to 50 metres, instead of 80 metres given by CEB and other codes. The vast majority of the buildings in Cyprus do not exceed this height anyway. The height of a building is also restricted by other authorities like the Town Planning Authority.

Secondly, the seismic acceleration symbolised now by the code as A_{max} is increased to the following values:

Zone I, II, III	$A_{max} = 0.075g$	cm/sec^2
Zone IV:	$A_{max} = 0.10\ g$	cm/sec^2
Zone V:	$A_{max} = 0.15\ g$	cm/sec^2

The increase was found necessary since from the existing information seismic acceleration has reached such values (i.e. 0.15 g cm/sec^2) in the past. As it is obvious Cyprus is now divided into five seismic zones instead of three for more accuracy. Zones I, II and III correspond actually to grade 7 in the Modified Merchali Scale, whereas Zone V to grade 8. The five zones are also shown in the Map in figure 4.2 with red letters.

A word of criticism, perhaps, is that the code (and the same applies to the CEB code) does not include anything about the masonry walls. Other codes - the Japanese for example and some American ones - ask for reinforced masonry walls considering unreinforced walls as totally unacceptable for seismic areas.

4.7 CONCLUSION

From the brief review of building practice presented in this chapter it can be seen that in recent years major steps have been made towards making seismic design an integral part of the structural design process. This should ensure that all structures built should be better equipped to resist the expected seismic load. However it has been established that in the forty years prior to 1984 a substantial stock of reinforced concrete framed buildings have been constructed that are potentially very poorly equipped to withstand seismic shock. They are often buildings of irregular shape, often contain soft stories, invariably rely on unreinforced masonry and often include non designed stub columns. All of these attributes have been identified as potential areas of failure under seismic load in chapter 3. This means that if an earthquake occurred in the near future the majority of buildings have a high risk of suffering major damage with the consequent loss of life that this implies.

It is therefore important that as the need for seismic design has been accepted then the existing building stock should ideally be brought up to the same standard. To do this for all buildings would of course be prohibitively expensive, nevertheless consideration should be given to strengthening those that are of major importance (eg hospitals). In chapter 5 a review of possible strengthening techniques is given.

5. STRENGTHENING OF EXISTING BUILDINGS AND REPAIR OF EARTHQUAKE DAMAGE

5.1 INTRODUCTION

It is safe to say that all the buildings designed without a seismic code need strengthening. And in Cyprus all the existing buildings at present are either designed without a seismic code or designed using a code of another country which perhaps is not suitable for Cyprus.

Following a review of the structural design and an inspection of the building we should be able to determine whether a building should be strengthened and to what extent. The result of the ideal earthquake - strengthening procedure is a building that has the same earthquake resistance as a new building. Practice, however, shows that this is never possible and there is nothing better than designing a structure properly from the beginning. Strengthening procedures may be very expensive sometimes. A compromise is therefore sought so that it will not be too great a hardship on the property owner and not too great against the earthquake resistance and safety of the people.

Earthquake strengthening of existing buildings is a topic on which much research is being done at present but not so much published. Some methods are suggested in this chapter, based mostly on earthquake damage results and on ideas given by engineers investigating this topic. Having no means of testing their effectiveness, these methods could be considered as a list of ideas for further research.

Similar methods to those for strengthening may be employed for repairing the damages after an earthquake. Additionally, methods for repairing cracks and effecting patch repairs are suggested. These methods may be useful for concrete repairs not only after an earthquake but also before, so that no weak points are left to the seismic resistance of the structure.

In the following pages 'Seismic Code' refers to the Cyprus Code[66].

5.2 STRENGTHENING OF REINFORCED CONCRETE ELEMENTS

Perhaps the most important topic for Cypriot Engineers to know is how to strengthen a reinforced concrete frame. This includes increasing either the stiffness of the structure or the ductility or both. In considering stiffness an engineer should have in mind the comments made in chapter 3 about stiffness and flexibility.

It was explained in chapter 3 that ductility is a very important property for earthquake resistance. Reinforced concrete elements shall become more ductile by,

a. arranging additional shear reinforcement
b. using high yield shear reinforcement
c. enlarging the sectional areas
d. using high strength concrete (high compressive strength)
e. employing special methods which have been shown to improve ductility

On the other hand care should be taken so that the increase of the main reinforcement remains within the acceptable limits as imposed by the seismic code. The upper bound of the reinforcement ratio ensures a sufficient curvature ductility (Clause C 5.1.2) and that the element is not over - reinforced. Even so, any increase to the longitudinal reinforcement will decrease ductility. Special attention during strengthening should be drawn on the joints and the critical zones for reasons already explained in chapter 3.

5.2.1 The use of polymers in concrete repair and strengthening[23,36]

Over the past 30 years polymers have been used in a range of applications in the repair of structures. The increasing demand for the polymer systems was due to their unique properties and the savings in money and time.

When talking about polymers in concrete repair we are usually referring to two types of materials:

a. Polymers used to modify cementitious systems
b. Reactive thermosetting resins - epoxy.

a. Polymers used to modify cementitious systems:
These are polymers added to cementitious mortars and renders to help overcome many of the problems of using unmodified mortars as repair and strengthening materials. They are normally supplied as milky white dispersions (latex) in water. The latex acts in several ways; water-reducing plasticizer, improving workability, lowering shrinkage improving bond between old and new concrete. It also reduces permeability and increases resistance to some chemicals. Although using the latexes have proved to give very satisfactory results, there is still the possibility of an unsuccessful mortar due to mixing errors or sand and cement quality. To eliminate such problems, factory pre-blended cementitious mixes requiring only the addition of clean water were recently employed. Polymer modified cementitious mortars are successful for repairs with minimum thickness of 12 mm, according to the producers.

b. Reactive thermosetting resins
These include mainly epoxy, but also polyester resins and acrylic resin systems. Epoxy resins can be formulated to produce high strength materials with excellent adhesion, and resistance to a wide range of chemicals. Both epoxy and polyester resins are classed as thermosetting materials because when cured the molecular chains are locked permanently together and do not melt or flow when heated but become more rubbery, and gradually lose strength. They are generally supplied as two components: resin and hardener. Epoxy resin mortars can be applied to a minimum thickness of 5 mm. It is, however, not suggested to use for more than 30 mm in a single layer. Although epoxy mortars are stronger than polymer modified cementitious mortars their high cost makes them less popular.

	EPOXY Resin, grout mortar, concrete	POLYMER modified cementitious system	NORMAL cementitious grout, mortar concrete
Compressive strength, N/mm^2	55–110	20–80	20–70
Flexural strength, N/mm^2	25–50	6–15	2–5
Tensile strength, N/mm^2	9–20	2–8	1.5–3.5
Elongation at break, %	0–15	0–5	0
Linear coefficient of thermal expansion per °C x 10^{-6}	25–30	8–20	7–12

TABLE 5.1 COMPARISON OF PRODUCTS USED IN CONCRETE REPAIRS

A comparison of normal products to polymers used for repairing and strengthening are shown in Table 5.1.

5.2.2 Beams and columns

All the action planned should be in accordance with the seismic code, and they should follow a proper investigation and analysis of the structure using seismic loads.

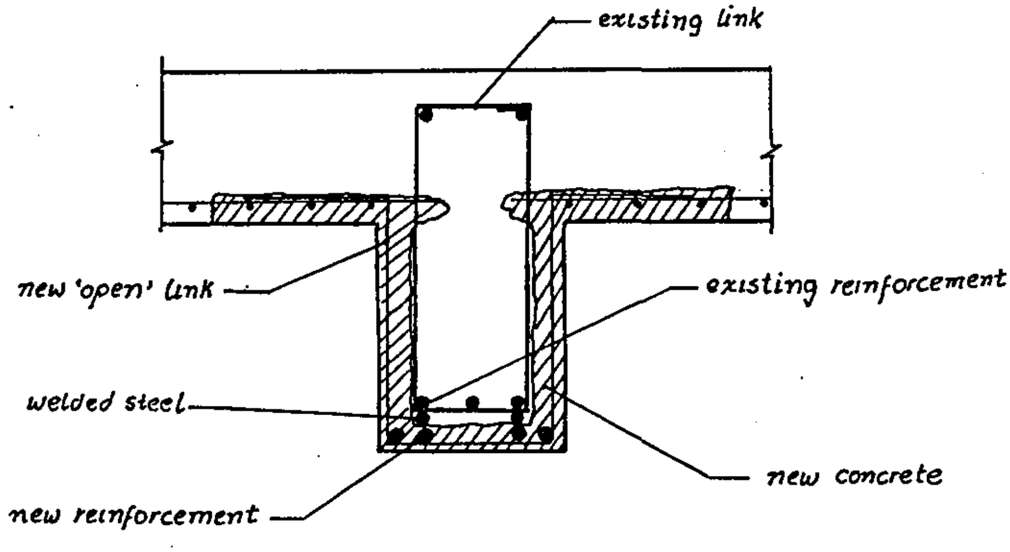
A possible procedure for strengthening a beam is suggested below:

a. The slabs adjacent to the beams should be first properly supported. Propping is necessary both for structural and safety reasons.

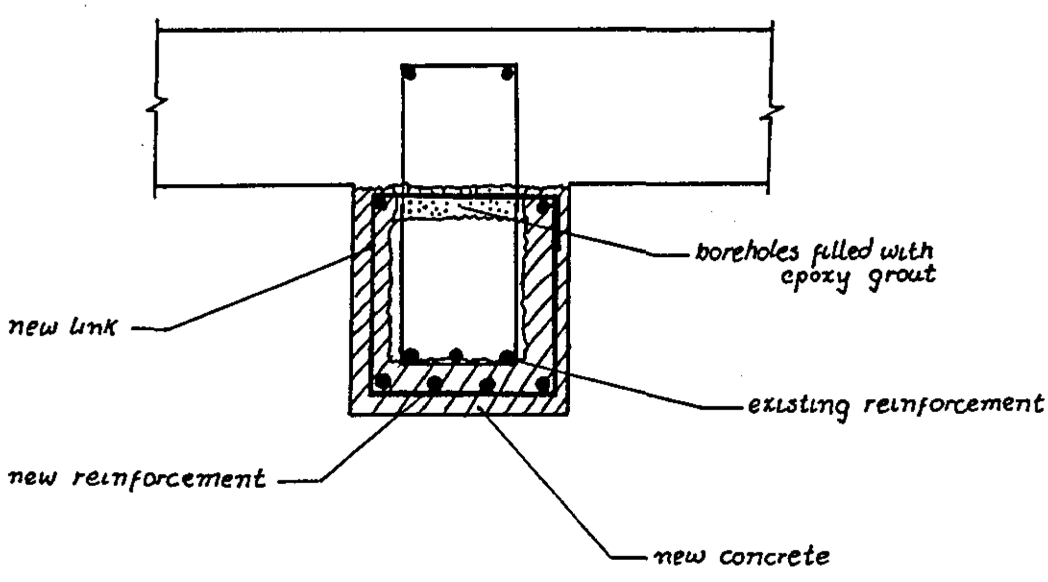
b. The concrete cover should be removed until the existing reinforcement is found. The whole surface should be roughened. The existing steel may need special treatment in case of corrosion. This will include cleaning thoroughly the steel by sandblasting or wire-brushing and applying a coat of epoxy paint for protection. However, if excessive steel has been corroded away, the reinforcement must be reinstated.

c. The additional reinforcement may be then placed. Two different methods are suggested here: In figure 5.1 (a) pieces of steel bar are welded to the existing bars. The new reinforcement is tied on the welded pieces and thus on the old reinforcement. Then new open links are placed tied on the slab bottom reinforcement should be partially uncovered. This method, however has some drawbacks. Welding may affect the yield strength and the open links will not provide the best possible confinement.

   In figure 5.1 (b) the second method is shown. Boreholes are drilled just under the slab making possible the use of closed links. The boreholes must be filled with special epoxy grout for high strength anchoring. Since the links are closed the longitudinal reinforcement bars need not to be welded.



(a)



(b)

5.1 Strengthening of a beam

d. A bonding agent (either epoxy or a pre–blended polymer cementitious mix) is then necessary to bond new to old concrete. It is applied immediately to the prepared concrete surface which must have been thoroughly dampened. Using a brush the agent is applied both to the concrete and the reinforcement.

e. The formwork may be then fixed in such a way to facilitate application of concrete. It must be stressed here that the bonding agent should be still tacky when the new concrete is placed. Therefore the formwork should be fixed in place within 1 hour maximum (in hot weather the agent will dry quickly).

f. Concrete may be then placed. According to the consistency of the mix the necessary formwork should be constructed. Polymer cementitious mixes or normal concrete with suitable admixtures can be used. A mix with specially selected lightweight fillers is used directly without any formwork. Watertight formwork is necessary in case of a pourable mix (see photo 5.1). Such mixes are again polymer modified cementitious requiring simply the addition of clean water to produce a high strength flowing concrete. Some special flowing grouts do not need a bonding agent since they have bonding properties. This will give more time for the erection of formwork. In case of normal concrete being used then the formwork is needed on the soffit of the beam. The admixtures used should be able to improve compressive strength, provide adhesion for better bond to the old concrete and reduce shrinkage (and shrinkage cracking).

The result of the above steps should be a homogeneous element, having composite action, giving the desired level of ductility.

Supposing that a beam was found adequately reinforced and dimensioned but the critical zones need to be strengthened then

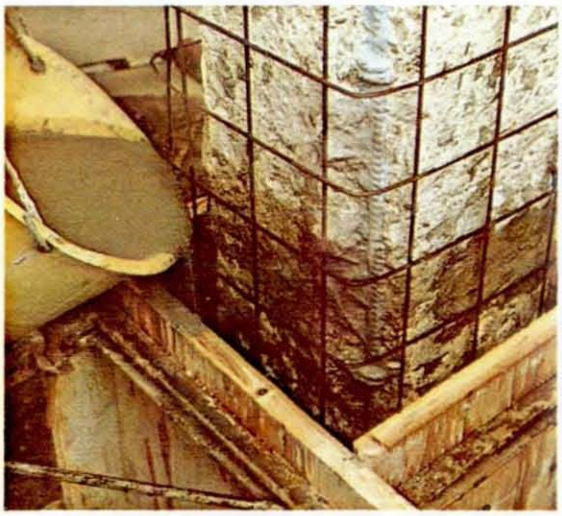
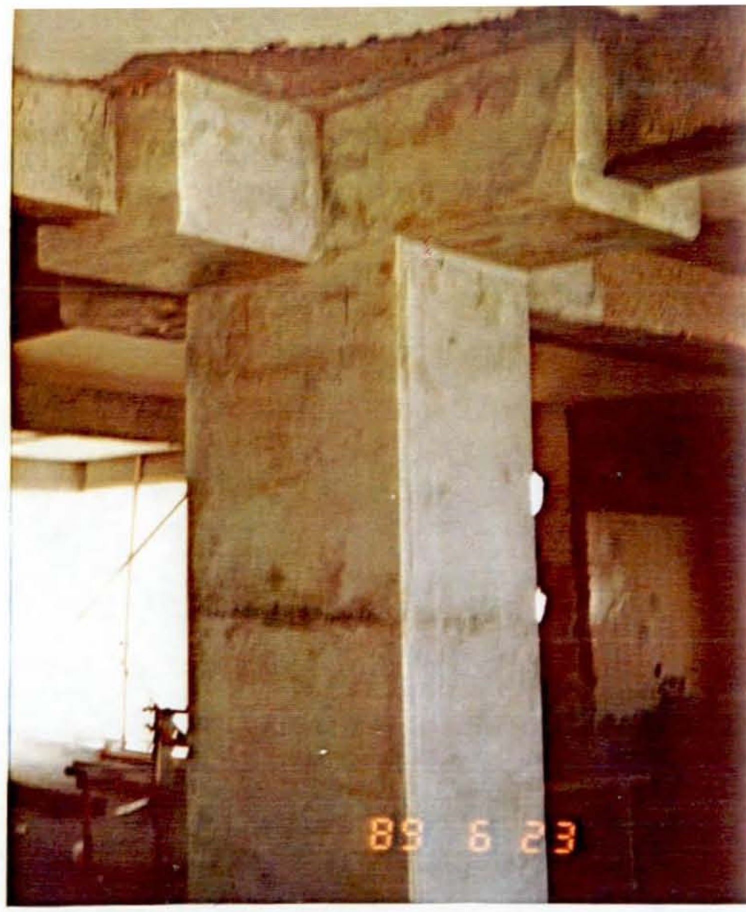
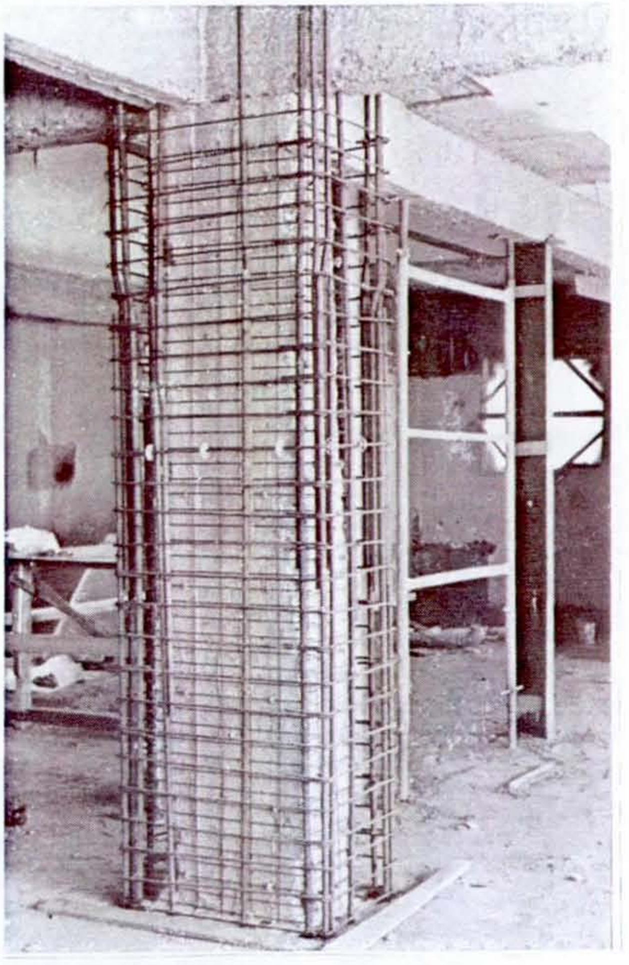


Photo 5.1. Pourable mix used for strengthening a column.



Photos 5.2 & 5.3. Partial Strengthening of a beam-column joint.



Photos 5.4 & 5.5. Strengthening of columns by additional reinforcement and enlargement.

only a partial strengthening is necessary as shown in photo 5.2 and 5.3. This will lower the costs but it may affect aesthetics.

The procedure for strengthening a column is very similar (see photo 5.4 and 5.5).

a. Adjacent beams should be propped

b. The preparation of the existing concrete and reinforcement should be done in the same was as for the beams. It should include removal of concrete cover, treatment of steel and roughening of the surface.

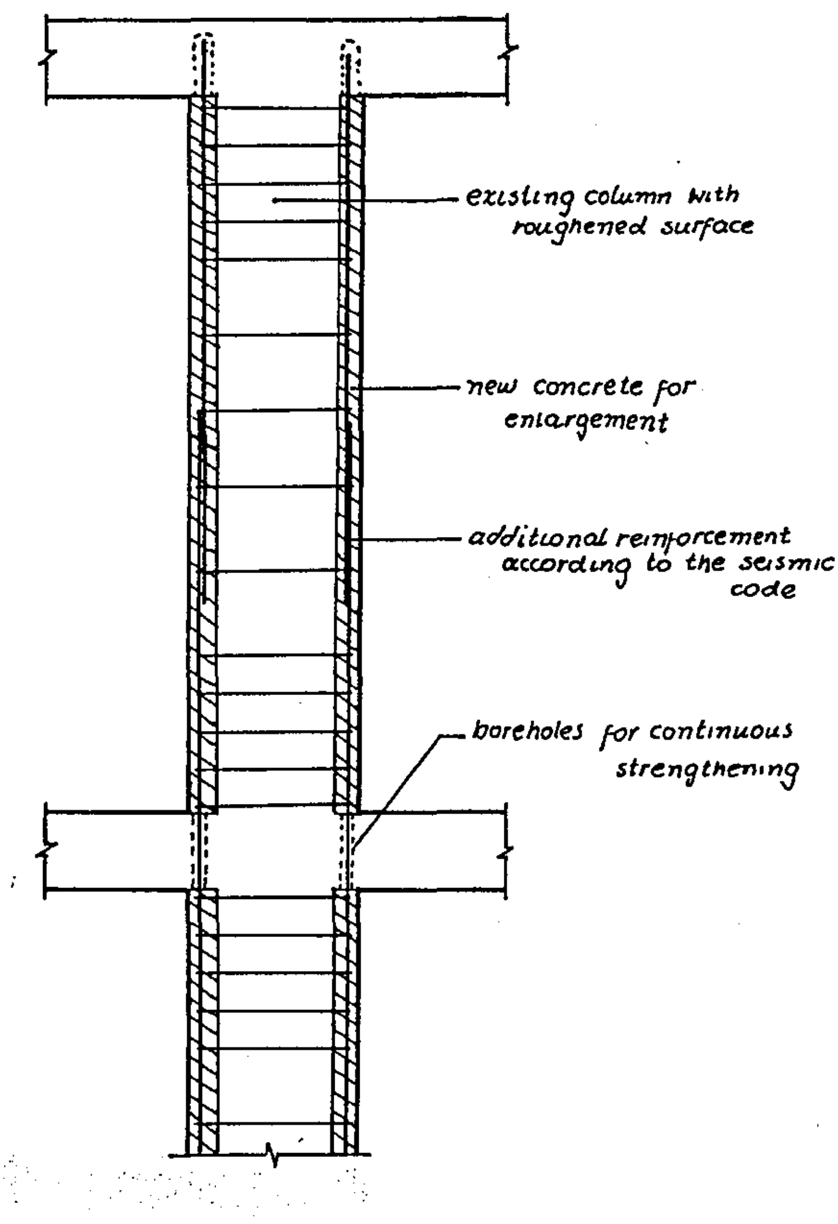
c. The new links can be placed without a problem and the longitudinal reinforcement need not to be welded on the old one (see figure 5.2).

d. Bonding agent is applied on wetted surface.

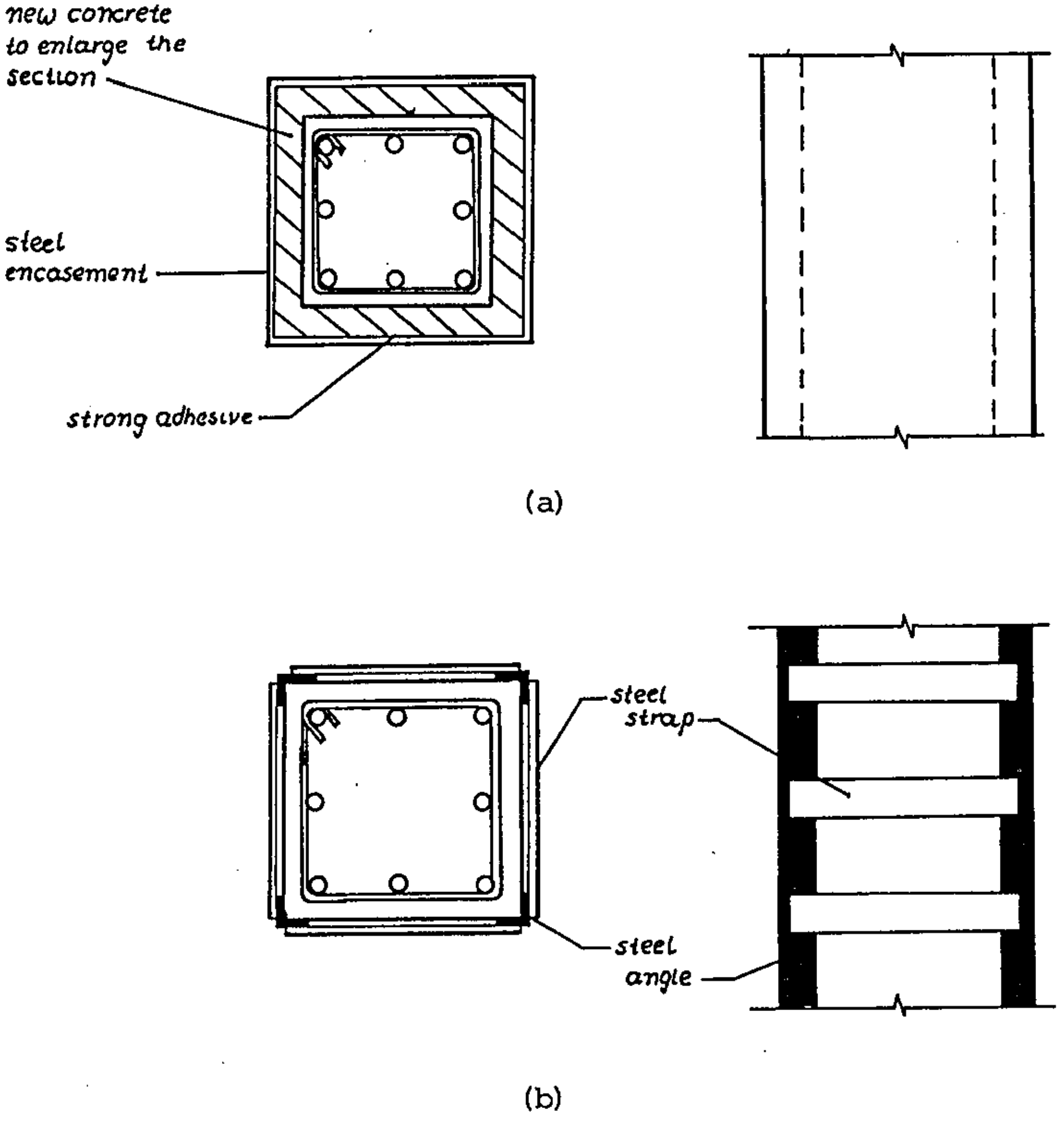
e. The forms in the case of columns should have an opening on the top so that pourable concrete may be placed. Polymers may be used again to form cementitious mixes either pourable or trowellable. A slurry can be placed by a trowel or even by hand without formwork. The material used should be of course especially selected for vertical surfaces.

The columns should be strengthened continuously from the foundations to the roof penetrating all the floors. In this way the column will be homogeneous and uniformly strengthened whereas the column-beam joints are strengthened too.

Steel elements[4] can be used for increasing the ductility. A brittle column is surrounded by steel plates bonded by epoxy resin based adhesive with tensile and flexural strength (see figure 5.3 a). The metal plates on the column surfaces, however, may not be easily accepted on aesthetic grounds. A similar method is to use a welded metal sheet attached to the existing column



5.2 Strengthening of a column



5.3 Column strengthening by steel elements and then covered by a bonding agent followed by a polymer mix. Hence the metal will not be visible, whereas some enlargement of the column is possible. Another method is to place steel angles at the corners of the column connected by the plates (see figure 5.3 b). A strong bonding is needed in all three methods so that composite action is achieved.

5.2.3 Shear Walls

Reinforced concrete walls already serving as shear walls may need strengthening for two reasons: Either their dimensions are not suitable or their reinforcement is inadequate.

In chapter 3 the following expression was given:

$$\frac{bd^2}{6} > \frac{H^2F\varepsilon}{5500}$$

This expression although not included in the seismic code may be most useful for checking the dimensions of the wall. An example follows for more clarification:

Assume, a 2-storey building of total height, $H = 6$ m
area of each floor, $F = 300$ m^2
the building is in Limassol (zone V, $\varepsilon = 0.20$)
and the width of the wall, $b = 0.20$ m

The above are typical values for Cyprus.

$$\frac{bd^2}{6} > \frac{H^2F\varepsilon}{5500}$$

$$d^2 > \frac{6H^2F\varepsilon}{5500\ b}$$

$$\therefore \quad d^2 > 11.78$$

Assuming that two walls were constructed in each direction then,

$$d^2 > \frac{11.78}{2}$$

$$d > 2.43 \text{ m}$$

With three walls in each direction,

$$d^2 > \frac{11.78}{3}$$

$$d > 1.98 \text{ m}$$

Hence the length of the existing wall can be checked.

If the length of the wall is not adequate then lengthening is a possible way of strengthening the existing wall. This might involve demolition of brickwork adjacent to the shear wall, so that its replacement by reinforced concrete is possible.

In such cases two things are very important: First the bonding of the new concrete to the existing one must be strong to ensure composite action. And then the reinforcement of the new part must be tied on the existing one. The enlargement of the wall will follow a similar procedure with that of a column. Sufficient preparation of the surface is essential. To ensure that the bond will not form the weak point, concrete surfaces should be sound, properly prepared and wetted (necessary for hydration of the cement). A bonding agent should be applied and then the appropriate formwork can be constructed.

Not only the length but also the width may not be adequate. Thickening, therefore, may be necessary to strengthen an existing wall. The section modulus, Wall, and thus the moment resistance of a wall is increasing by lengthening and thickening the wall, since,

$$Wall = \frac{bd^2}{6}$$

(as given in chapter 3).

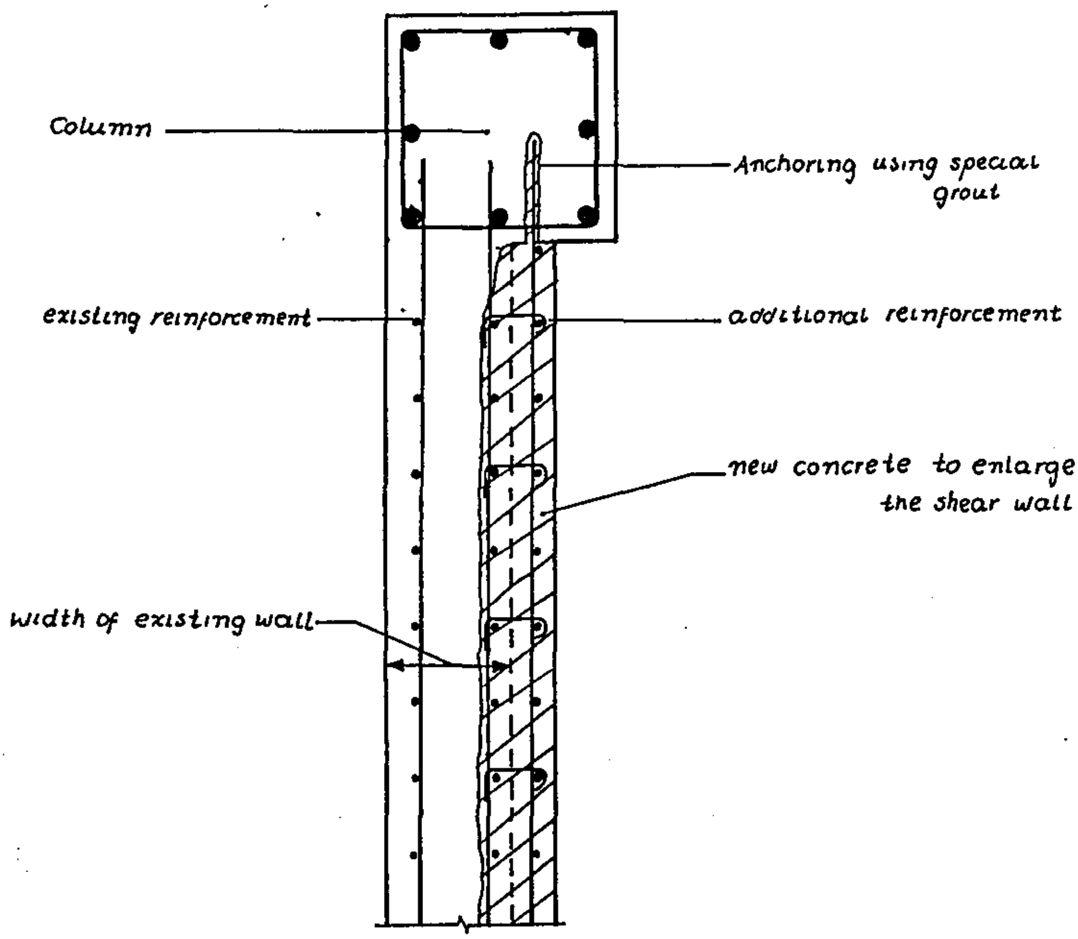
The seismic code limits the minimum thickness of a shear wall to 150 millimeters. (Clause 5.4.1)

To thicken an existing shear wall a layer of reinforced concrete is added to either one or both surfaces. Similar methods to those for beams and columns are appropriate to walls too. Pourable or trowellable concrete may be used whereas a bonding agent is necessary. Guniting[6] may be employed for the construction of the additional layer. Gunite is pneumatically applied concrete. There are two methods of application, one known as gunite and the other as shotcrete. Shotcrete is a wet concrete mix pumped through a hose to a nozzle at the point of application. Using compressed air the concrete is directed against the surface. Gunite on the other hand is applied in the same way but dry, water being injected near the nozzle forming a spray. The amount of water can be adjusted during the application.

The advantages of gunite in compare to normal concrete are:

a. The increased bonding strength
b. The ability to be placed without formwork

Additional thickness may be also necessary when additional reinforcement is needed. This additional reinforcement should be of course in accordance with the seismic code, and it may be in form of a grid covering the whole surface or only at the critical areas (see figure 5.4). The concrete should be cut back to 15 millimeters[26] behind existing reinforcement steel. This will enable us to tie the new reinforcement onto the old and it will give additional mechanical bonding as well. It must be stressed here that the strengthening will be more effective if it is applied continuously from the foundations to the roof.



5.4 Strengthening of shear wall

Nevertheless, this is not always possible. However, the reinforcement should be bonded into floor and adjacent columns (if any), as it is necessary to provide adequate structural connection between the wall and the structure. Anchoring of the bars can be done using special epoxy grouts.[26] Holes, of 4 – 8 mm diameter larger than the steel bar diameter, are drilled. The depth of the hole is limited to 100 mm by the product producers. The grout is then pumped into the back of the hole using the correct gun. The grout is left just short of the face of the concrete and then the bar is inserted. A high strength anchoring can be obtained assuming that the concrete was sound, the hole was properly prepared (dry and free from dust) and curing followed.

5.2.4 Foundations

It is obvious that strengthening of existing structures involves enlargement of beams, columns and shear walls, which means extra weight in the frame and thus the foundations. It may be found therefore that strengthening of foundations may be required for a number of reasons:

a. Extra weight due to enlargement of sections as it is already mentioned.

b. Inadequate dimensions of the foundation itself after being analysed using the seismic loads.

c. Inadequate amount of steel reinforcement

d. To 'cure' the problem of stub–columns arbitrary constructed without being properly designed.

The following calculations will enable us to see the implications of altering the dimensions of a foundation:

Assume a foundation pad of width b, length l, and effective depth d. The maximum pressure of the foundation on soil, p is given as

$$p = \frac{N}{bl} + \frac{6M}{bl^2} \qquad \text{when } e < \frac{l}{6}$$

$$p = \frac{2N}{3b\left(\frac{1}{2} - e\right)} \qquad \text{when } e > \frac{1}{6}$$

where N is the vertical load

M is the moment

e is the eccentricity usually taken $\frac{M}{N}$

The maximum pressure must not exceed the permissible bearing pressure for the soil as given by BS 8004 (see Table 5.2).

From Bs 8110, the shear stress v is given as,

$$v = \frac{V}{bd}$$

which must be less than 0.8 f_{cu} or 5 N/mm^2 (whichever is smaller)

It is therefore clear that by any reasonable increase to the dimensions of the pad (since that will increase self weight) the result would be positive for our purposes.

That is, if strengthening procedures result in extra weight being transferred to the foundations then by enlarging the pad the problem will be solved. Similarly in the case of inadequate dimensions or steel reinforcement or in the case of Stub-columns again enlargement will not create any problems. Especially in the case of a stub-column the height of the pad can be increased so that the top reaches the bottom of the ground-beams. The stub-column will thus disappear.

The strengthening of existing foundation is not an easy task. It has been done in Cyprus, however, although proved to be very expensive. Recently a number of buildings in Nicosia had problems

CATEGORY	TYPE OF ROCK/SOIL	ALLOWABLE BEARING VALUE KN/m^2
ROCKS	- Strong igneous and greesic rock in sound conditions	10000
	- Strong limestone and sandstone	4000
	- Schist and Slate	3000
	- Stong shale, mudstone and siltsone	2000
NON-COHESIVE SOILS	- Dense gravel with (or without) sand	$>$ 600
	- Medium dense gravel with (or without) sand	200-600
	- Loose gravelwith (or without) sand	$<$ 200
	- Compact sand	$>$ 300
	- Medium dense sand	100-300
	- Loose sand	$<$ 100
COHESIVE SOILS	- very stiff and hard clay	300-600
	- Stiff clay	150-300
	- Firm clay	75-150
	- Soft clay and silt	$<$ 75
	- very soft clays and silts	NOT APPLICABLE
PEAT, ORANGE SOILS, MADE GROUND, FILL		NOT APPLICABLE

Table 5.2 Allowable bearing values under static loading as given by BS 8004

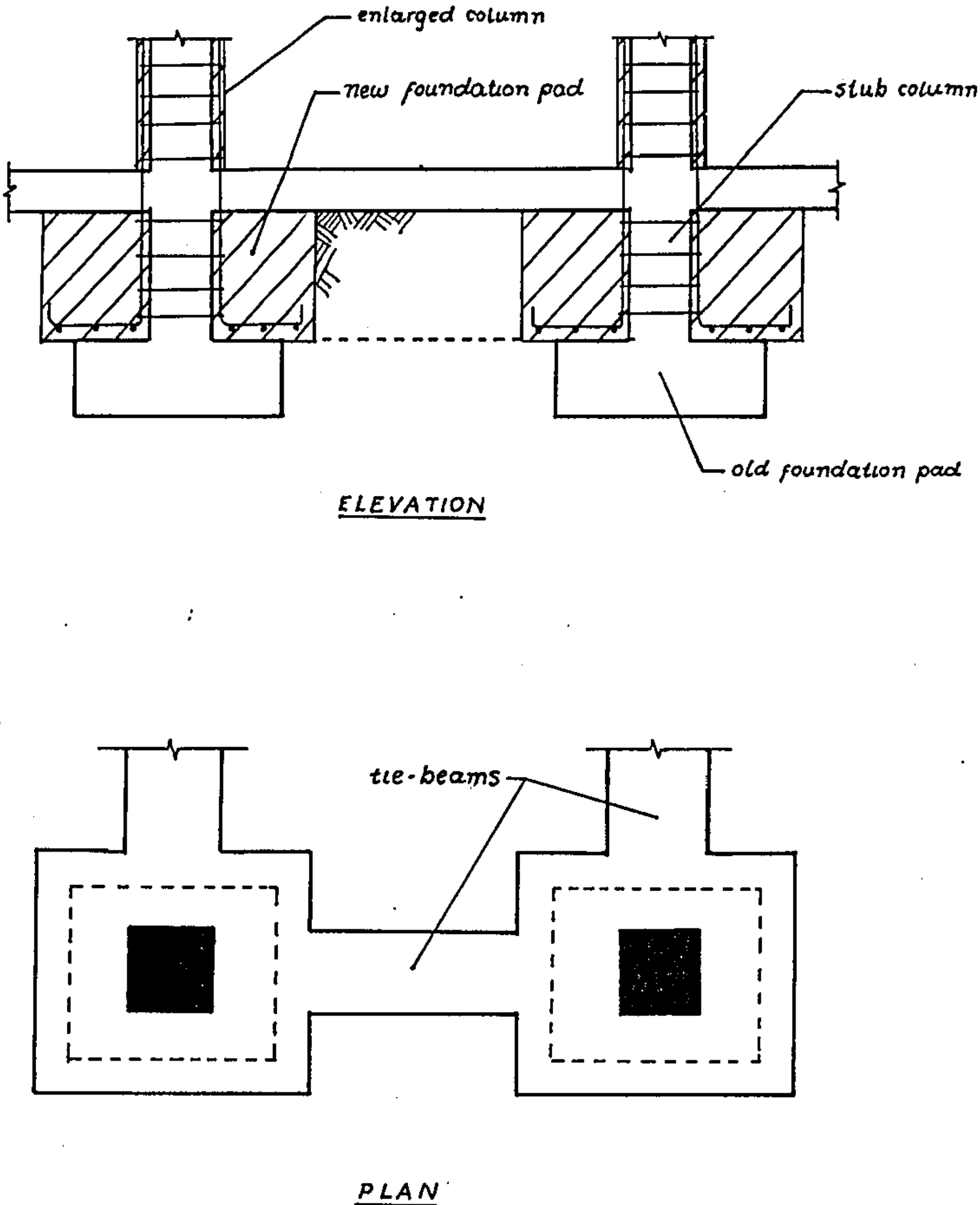
due to the clayey soils to dry out as a result of the construction of the sewage system. The construction of the system let the existing pits empty and thus the wet clay started drying out . The clay shrinks when it drys and this led to considerable settlements with consequential cracking. The methods used for the strengthening of such foundations varied a lot. Usually quite an amount of digging was necessary. Foundation pads were sometimes enlarged (photo 5.6). When strip foundations existed digging was progressing under the foundations and reinforcement was inserted. The most convenient method seen, however was as shown in figure 5.5. A totally new foundation was constructed on top of the existing one. There is no bonding between the two and their reinforcement is completely separated. It does, however, serve its aims which is to provide the structure with larger foundation pads. Strip foundations or tie–beams were also constructed. Tie–beams are recommended (see chapter 3). Assuming there is enough depth this method looks to be most suitable. And in most of the cases there is enough depth since the typical depth for foundations in Cyprus is 1.50 metres under the surface of the ground.

5.3 INTRODUCTION OF REINFORCED CONCRETE WALLS

In order to increase the strength of a building, shear walls can be constructed. Such additional shear walls must be very carefully designed. The effects on the structures must be considered. First they will increase the dead load on the structure. Then they may create torsional problems if not positioned properly. Finally they will increase stiffness which as it was explained in Chapter 3 may not be desired.

On the other hand for the majority of buildings in Cyprus increased stiffness is desired. Torsional problems created by the irregular plans and by non–uniform distribution of masses and stiffnesses may be solved by careful design and positioning of additional shear walls. Generally a shear wall can increase the lateral load resistance capacity of a structure. Once more the formula,

Photo 5.6. Strengthening of the foundation pads by converting them into strip foundations.



5.5 A method for strengthening the foundations

$$\text{Wall} > \frac{H^2 F\varepsilon}{5500}$$

is very useful when designing a shear wall. It should be used together with the directions given in Chapter 3 and the Seismic Code.

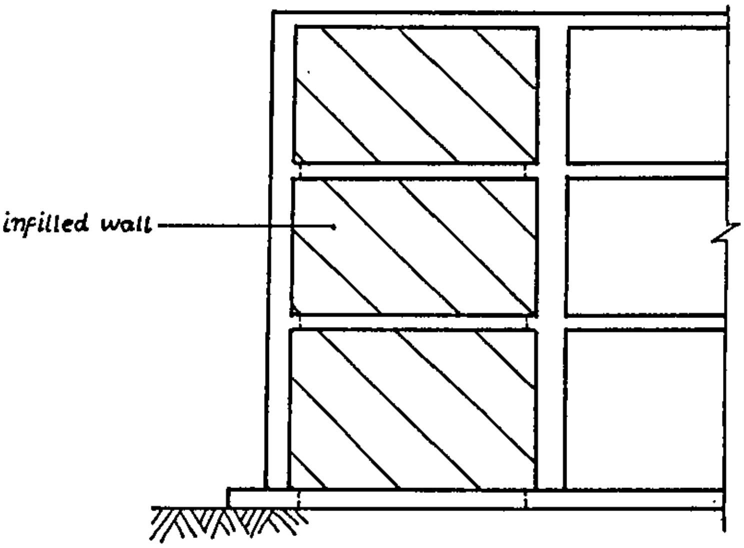
A shear wall may be constructed by reinforced concrete, reinforced brickwork, gypsum boards or plywood. More appropriate to the reinforced concrete-framed buildings of Cyprus is reinforced concrete, because they are quite stiff laterally and they will match with the stiff frames. Hence cracking will be prevented.

Wing walls can be constructed or walls from column to column as shown in figure 5.6 (a) and (b). In either case the connection with the existing columns must be effective. A bonding agent (epoxy) must be used for joining new to old concrete. The steel reinforcement of the wall must be anchored on the columns using the method suggested earlier. Strengthening must be done continuously from foundations to the roof so that the additional weight is taken by the strengthened foundations. The walls should penetrate all floors for better connection. In order to be continuous vertically they can be placed just off the column centre lines missing the floor and roof beams. The existing floor reinforcement should pass through the new walls.

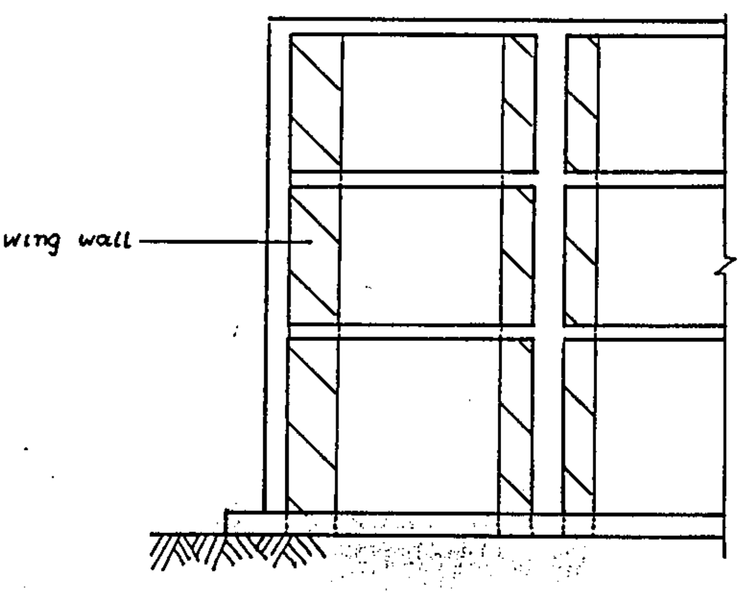
External shear walls in the form of concrete buttresses are also suitable for strengthening, when space is available outside the building. They have however, to be aesthetically acceptable.

5.4 STRENGTHENING OF BRICKWORK

As it was stressed in the previous chapters unreinforced brickwalls are not suitable in regions subject to earthquakes. They are, however, widely used in Cyprus and this fact will



(a)



(b)

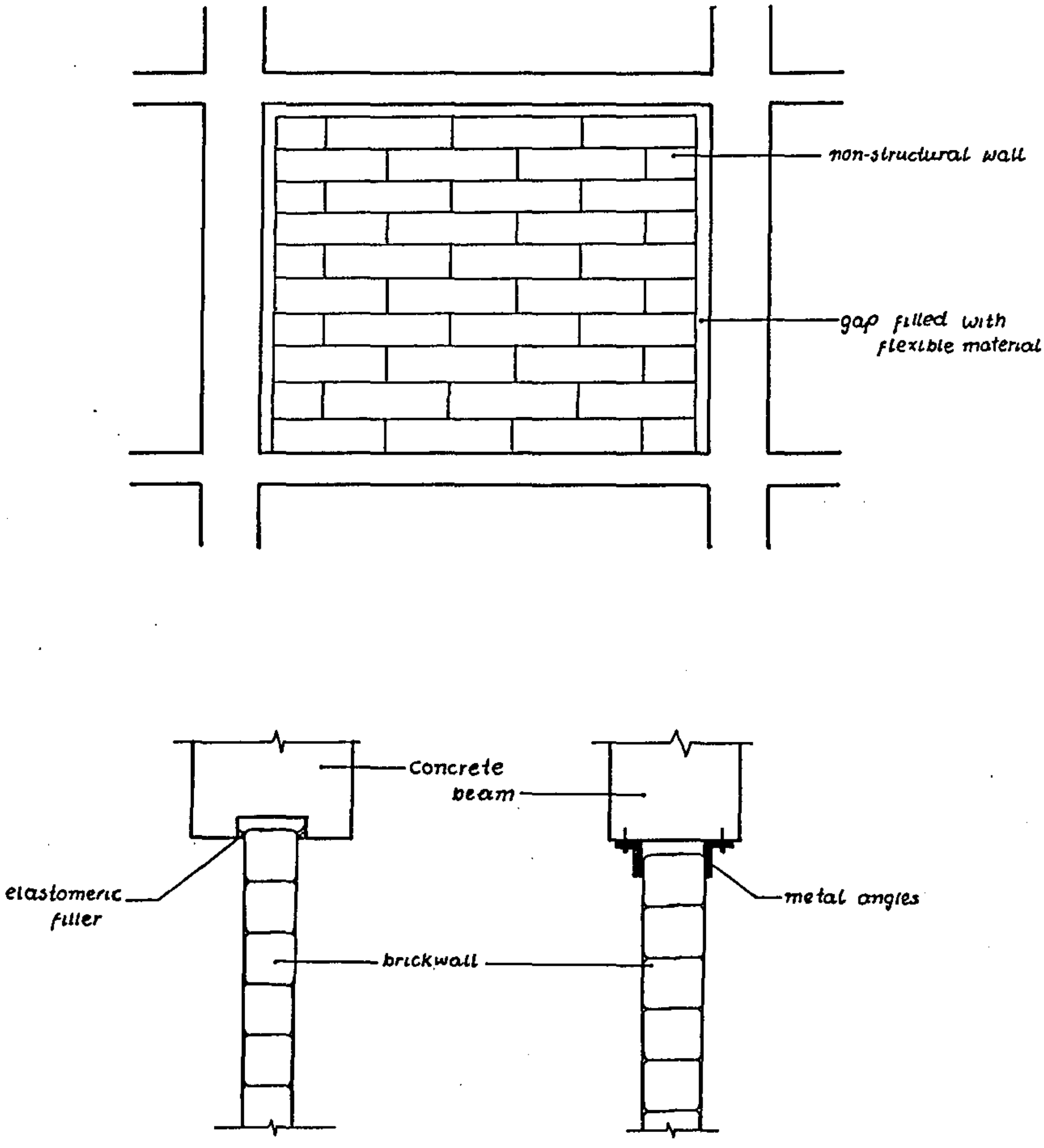
5.6 Introduction of shear walls

possibly present Cypriot engineers with a real problem during an earthquake. The type of the brickwork (see chapter 4) employed nowadays in Cyprus has never been tested by an earthquake.

Failure in masonry is usually due to shear, but also due to instability and sliding. As it was explained in chapter 3 the interaction between a frame and infill masonry creates complex problems. Stiffness is increased but only up to a point. A brittle failure should be expected. So before strengthening a brickwall a decision is to be taken as to the function of the wall: Is it going to act as a non-structural partition or as a structural shear wall?

Assuming that a non-structural wall is desired. Then it is essential to separate the wall from the frame (see figure 5.7). The problem is the width of the gap that should be left. Some American codes suggest 50 mm. Displacements of the frame may be calculated of course. This gap must be filled with an insulating material like polystyrene or styrofoam. Styrofoam can accept rendering successfully. The failure expected in a non-structural brickwall is due to instability. Therefore the strengthening actions taken should prevent the wall from falling out. A fine wire mesh may be added on both sides of the wall. The existing finishes should be first removed and the surface roughened. The mesh must be riveted to the adjacent columns, beams or slabs. Then rendering follows with a mortar strengthened by a suitable admixture to give some plasticity. Fine synthetic fibres may be added into the render for the same reason.

When the brickwall is required to act as a shear wall the strengthening procedure must include proper analysis and consideration of torsional problems. Strengthening can be achieved by enlarging the section of the wall by thickening (as it was explained earlier for reinforced concrete walls) and by additional reinforcement for ductility. The existing finishes and some plaster must be removed and the surface roughened. Steel reinforcement on both sides preferably, in the form of a grid

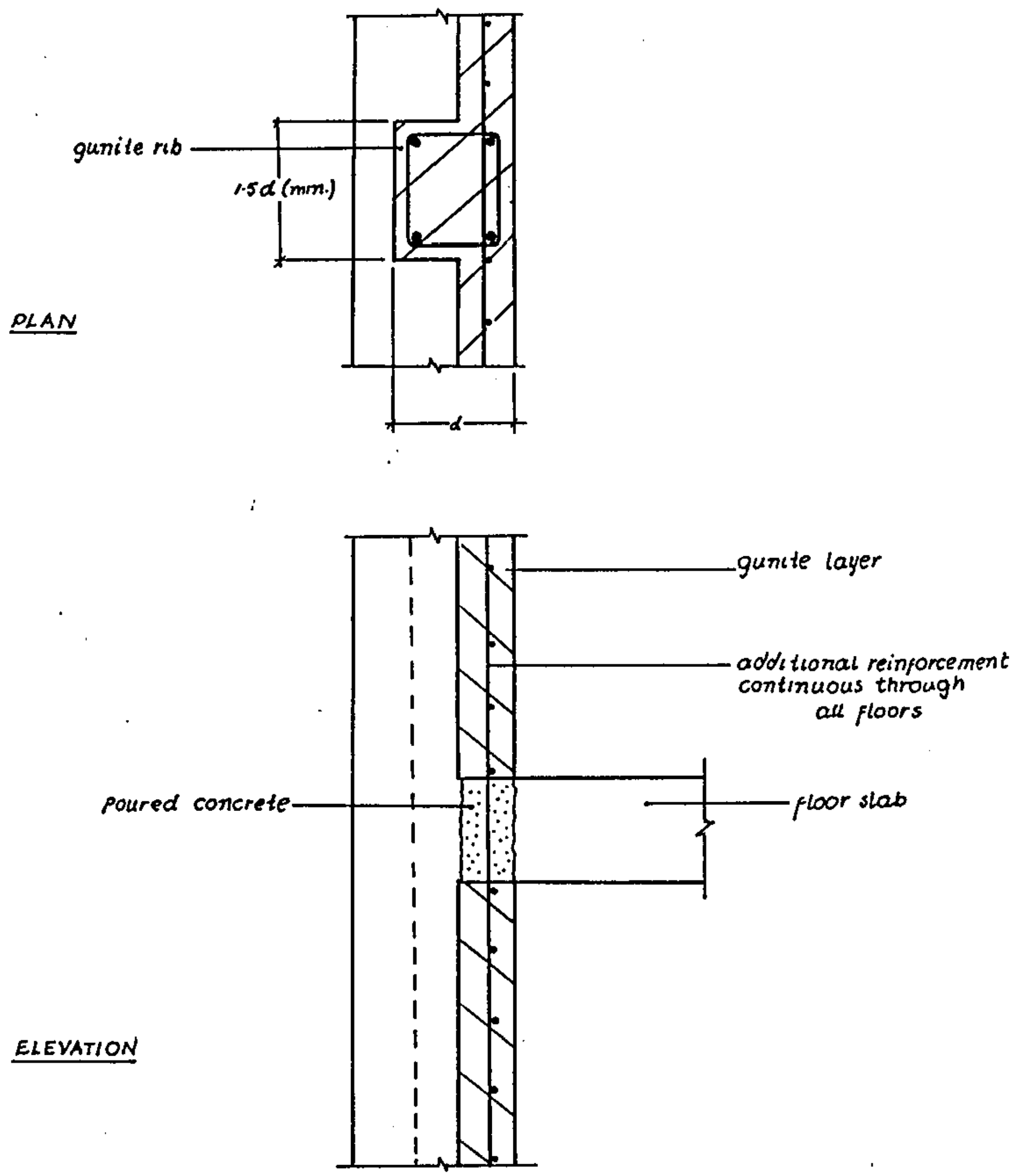


5.7 Separation of non-structural walls from the frame should be placed, anchored on the adjacent columns, beams and slab. The anchoring can be done by drilling and grouting as it was explained earlier in this chapter. The concrete can be placed using the appropriate formwork. Pourable grouts are necessary if the wall is not to be thickened a lot. In this case the formwork should be watertight. The most cost effective method, however, is guniting [6]. Gunite is normally applied to 40 mm thick and well compacted material can be achieved. Layers of up to 200 mm have been applied [6], using lightweight materials. A skilled operative is needed to ensure that there is a good compaction behind the reinforcement grid. A more sophisticated method for strengthening a brickwall follows:

The gunite reinforcement on the wall consists of a continuous layer 70 to 100 mm thick and vertical ribs cut into the wall every 2 or 3 metres. The ribs are reinforced with vertical bars and ties, like small columns, whereas the layer is reinforced by a grid (see figure 5.8). In case of a continuous strengthening the vertical ribs should be continuous from the foundations to the roof penetrating all slabs. The portion of the rib that passes through the slabs will not be gunited of course. Pourable concrete should be used for this. Such problems would be avoided if guniting was done on the external surface of the walls. If, however, there is no access to the outer face of the wall or if the wall to be strengthened is an interior one, then wall guniting must be done from inside the building.

5.4.1 Improvement of openings

Most of the damage in the walls whether brickwork or concrete, is usually concentrated around openings, either made of brickwork or concrete. The Seismic Code following closely the European Code is recommending additional reinforcement around openings (Cl 5.4.1) in reinforced concrete walls, so that the strength of the missing portion, that is the opening, is compensated. Nevertheless no directions are given for openings in brickwalls which is the most common case in Cyprus.


gunite rib
1·5d (mm.)
PLAN
d
gunite layer
additional reinforcement continuous through all floors
poured concrete
floor slab
ELEVATION

5.8 Guniting

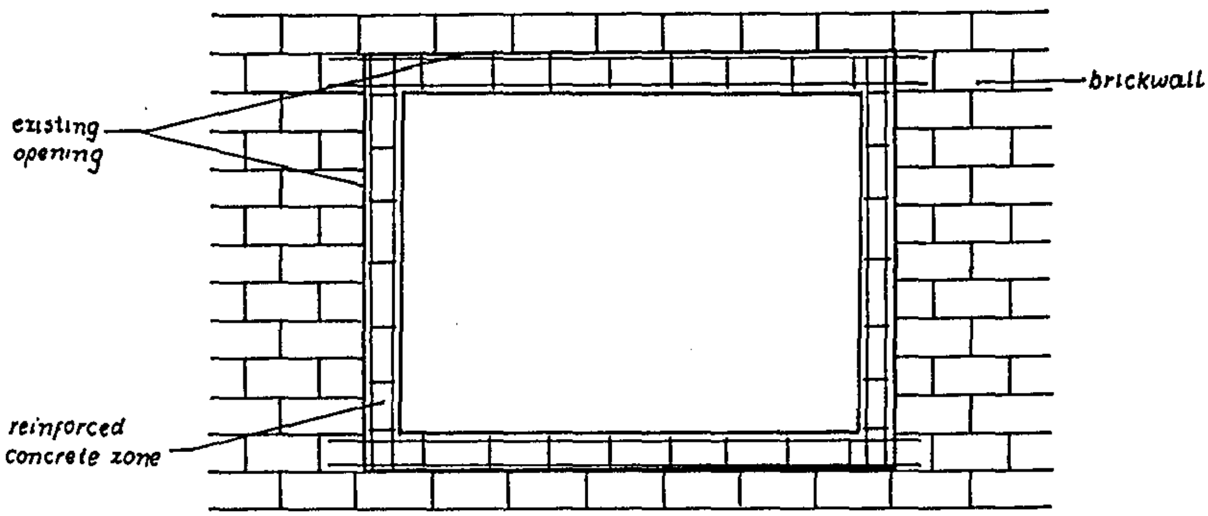
Obviously openings in brickwalls serving as shear walls or not separated from the frame, must be treated with additional measures. Two actions can be taken; first to make the opening as small as possible and second to reinforce it with steel. A large opening will give some flexibility to our methods of strengthening. A reinforced zone around the opening made of steel reinforcement and concrete may be constructed with aid of the appropriate formwork (see figure 5.9). Pourable concrete or lightweight mortars should be used. A polymer modified cementitious mix should be used to ensure better bonding between bricks and concrete and eliminate shrinkage and cracks. This procedure, however, may prove to be very expensive since the cost for constructing the reinforced zones around the openings will be added to the cost for replacing windows and shutters due to the new dimensions.

When no alteration of the dimensions of the openings is possible then some demolition is necessary. It will be rather impossible, however, to demolish part of the soffit of the opening. In this case prefabricated steel lintels must be used (see figure 5.10). This means that the top part of the reinforced zone will not be in accordance with the Seismic Code, although the steel lintel will improve the opening. During the whole work the soffit must be well propped. The problem of the soffit will be presented only to some cases. Usually the top part of the openings stops at a beam of the above floor.

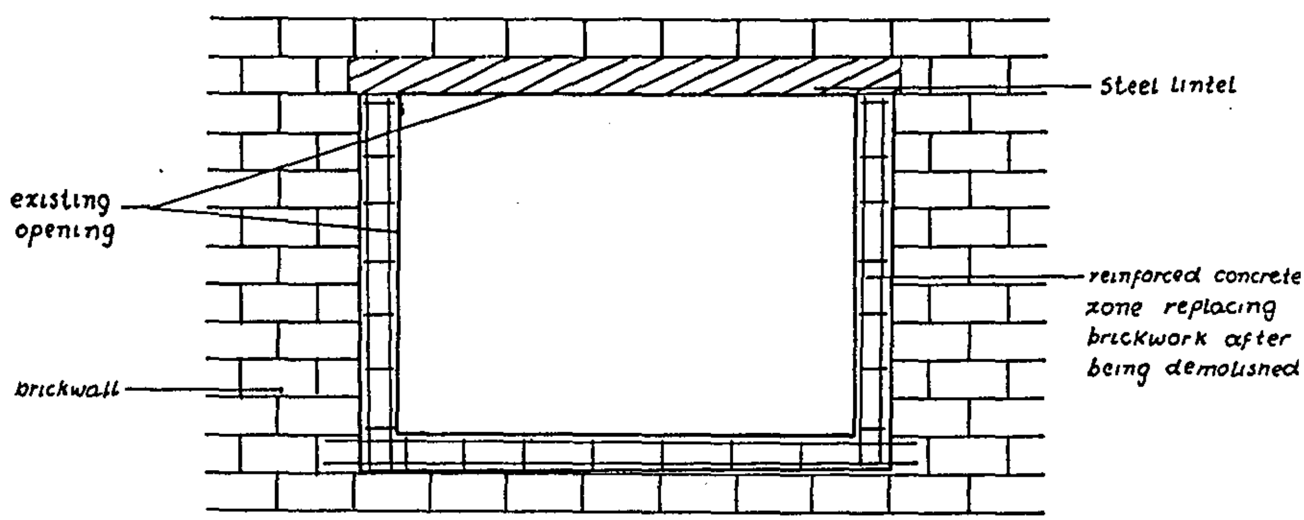
It will be perhaps too great a hardship on the owner to proceed to such strengthening. The above ideas, however, may be useful when a whole brickwall is strengthened and an opening exists.

5.5 PREVENTION OF NON-STRUCTURAL DAMAGE AND OTHER HAZARD

Although not much research has been done on the subject of the non-structural damage, it is now very obvious that such damage can create serious problems.



5.9 Strengthening of a large opening



5.10 Strengthening of an opening using a steel lintel

The first danger can come from all equipment or decorative elements hung from the ceiling. It includes light fixtures – sometimes huge and heavy ones – heating units, air conditioners and pipes for services. All these items should be braced so that they cannot swing.

Equipment that is not fastened properly to the floor again create problems during an earthquake due to dynamic amplification increasing the forces applied. Such things are the boilers, furnaces, air conditioning units and especially water tanks. This equipment before anchoring securely to the floor or the walls should if possible, be isolated using vibration isolation supports. Grouts can be used to strengthen the base of heavy equipment and for fixing bolts. Such grouts are based on specially selected portland cements graded aggregated and admixtures.

Finishes that are of bad quality or heavy may separate and fall during an earthquake. Items like granite, marble or stone used for decoration should be removed and replaced by lighter material, or re–fixed using a strong adhesive, either cement–based or organic–based. Other finishes can be repaired and strengthened.

Non–structural walls may be separated from the frame, especially in the case of a partial wall attached to a column, creating a critical situation (Seismic Code Cl 5.2.3.1).

A gap should be created at the two sides and the top of the wall. The gap being 40 – 50 millimetres can be filled with a proper isolating material for sound and heat as it was mentioned earlier in this chapter.

5.6 USE OF TIMBER

In figure 5.11 a foundation (a) and a wall (b) are shown dated about 2000 B.C. They are both reinforced with a wooden framework. Such constructions were found in Asia Minor and in the Greek

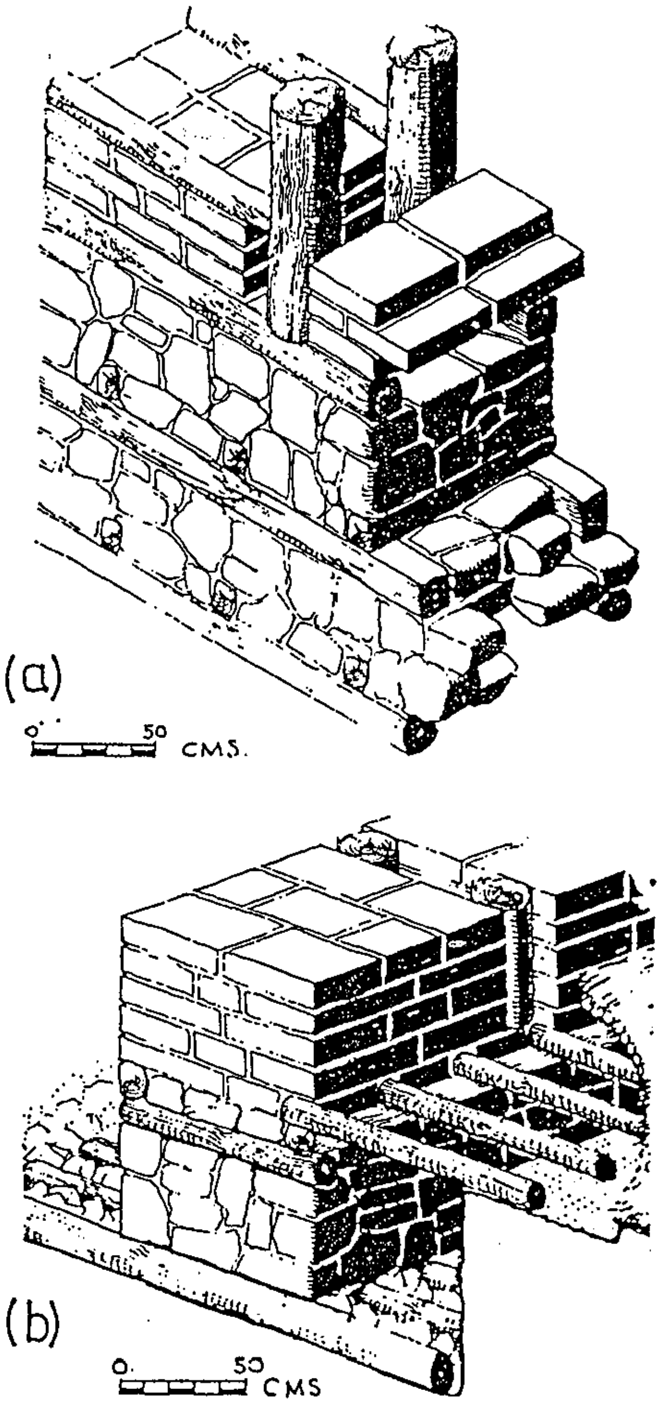


Figure 5.11 Antiseismic foundation and walling found in Asia minor.

island of Crete.[34] In Crete wooden frameworks were especially used in multistorey buildings like the palaces of Minoas at Knossos. Archaeological excavations gave evidence of more or less complete houses preserved under many metres of soil after earthquakes destroyed whole cities. These houses had wooden-framed walls and foundations.

It is believed, although not proved, that this wooden framework was an aseismic technique employed by the ancient Greeks.

Timber, like steel, is in itself an excellent material for earthquake resistance. It is strong enough in both flexure and shear. Plywood shear walls have already been mentioned as a means of earthquake resistance, not appropriate, however, to the reinforced concrete structures. Using the ancient technique as an example, wooden frameworks could perhaps increase the strength of unreinforced masonry walls.

The damping coefficient of a wall, or any other element, can increase by the use of wood. A sheet of wood applied on the surfaces of walls and floor slabs can serve as a vibration absorber.

It appears that timber can solve some earthquake problems relating to strengthening. It is surely an area for much more investigation.

5.7 REPAIR OF EARTHQUAKE DAMAGE

The immediate actions after an earthquake will aim at restoring a building to a reasonably safe and functionable condition. The restoration should be followed by earthquake-resistant strengthening. It will take time, of course until people – authorities, owner, engineers – are in a position to proceed with the strengthening methods, after a major earthquake and its consequences. Repairing of the earthquake damage is therefore an

important task and it should be properly designed and performed immediately.
It has been proven in practice that a reinforced concrete building damaged by an earthquake can be restored, to its original condition, easier than any other types of buildings. This assumes of course, that the building has not collapsed or tilted or overturned. Damage in reinforced concrete buildings is usually in the form of cracks, structural or non-structural. Excessive cracking may lead to deterioration of concrete.

Concrete repair methods are given in Appendix IV as a general guide.

The above methods have been applied already in Cyprus for general repair works, although not widely yet. Polymer modified cementitious material and epoxy resins and mortar are available, either imported from United Kingdom, United States of America, Germany and Italy or even manufactured locally. Up to now the results of the concrete repair works were good, although it is still very early for conclusions to be drawn. There is an increasing demand for such works especially in the coastal area of Cyprus.

It must be emphasized once more that when bonding to concrete it is the surface strength of the concrete which plays the vital part. It is often possible to have a concrete which on the basis of compressive strength is satisfactory but which has a very low surface strength and is unacceptable.
The proper surface preparation is therefore essential.

5.8 CONCLUSIONS

Earthquake-resistant strengthening methods and concrete repair methods for repairing earthquake damage were included in this chapter. Some methods were specified step by step. These methods are feasible and the materials to be used are available on the Cyprus market. Some other methods or rather suggestions, were mentioned more generally and they need further investigation.

Concrete repair and strengthening has been aided by the development of polymers and their use for modifying cementitious mortars. Cementitious and epoxy mortars are mostly used for such jobs rather than normal concrete due to their properties with which they can offer:

a. good bonding of old to new concrete

b. elimination of on-site mixing errors since they can be factory pre-blended products

c. no problems of quality, availability and grading of local cements and aggregates

d. high strengths

e. reduced shrinkage and shrinkage cracking

f. high performance even when used in a flowing condition

g. penetrating properties, essential for filling cracks and voids

h. easy application

Nevertheless, no matter, how good a material is, it will not operate successfully unless the repair area is carefully prepared. Additionally the right material should be used for each particular case.

In this chapter it has been shown that a number of techniques exist that permit the strengthening of existing buildings to resist seismic loads and for effective repairs to be carried out on structures damages by earthquakes. The cost of strengthening works need to be investigated to convince building owners of the cost-effectiveness of carrying out strengthening works. The costs involved will be considered in the next chapter where two case studies are presented.

6. CASE STUDIES

6.1 INTRODUCTION

In this chapter two cases will be studied. These two cases were selected in such a way to represent the houses built before and after 1984. It is important to assess the quality and the earthquake resistance of a building constructed prior to 1984 when no aseismic measures were taken. It is equally important to assess the earthquake resistance of a building constructed after 1984 when basic concepts of the aseismic design were known.

Having assessed the quality of the construction, then strengthening steps will be suggested, if necessary. The costs involved will be estimated by analysis.

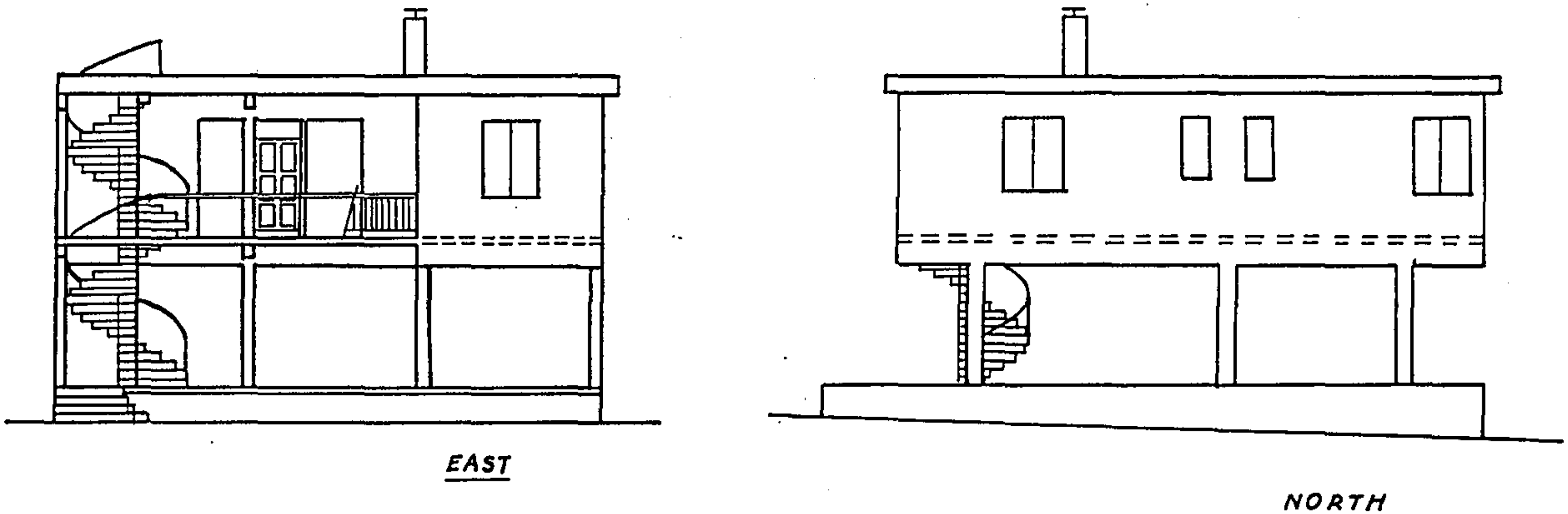
The design of both structures was checked as far as the normal loads are concerned and it was found adequate.

6.2 A HOUSE BUILT IN 1984

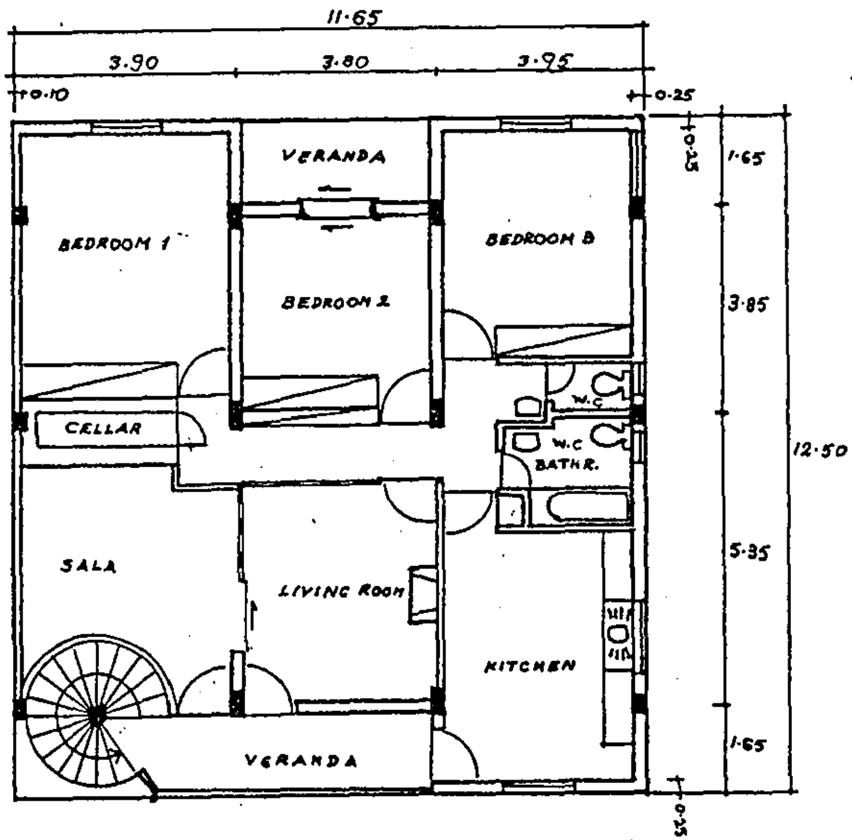
6.2.1 Description

A house designed and built in Nicosia during 1983-84 is the first case to be examined. Until 1984 the concept of earthquake resistance was still unknown and no measures were taken during the design stage or later. The house following the fashion of those years is raised from the ground being built on columns. It is a reinforced concrete framed structure with infill brickwalls. A similar house to be built will cost nowadays around 60,000 pounds (Cyprus pounds).

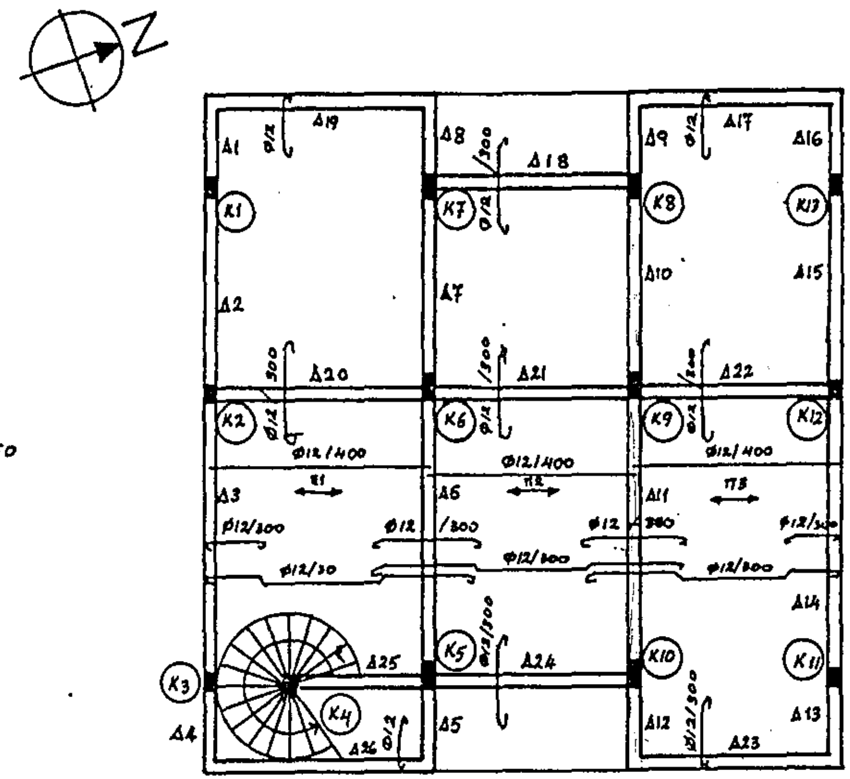
Drawings of the building are shown in figures 6.1 (a),(b),(c),(d). To assess the resistance of the building to earthquakes the suggested code for Cyprus (given in Appendix II) and the instructions given in chapter 4 were followed. A ductility level II was assumed.



6.1 (a) Drawings of the house built in 1984

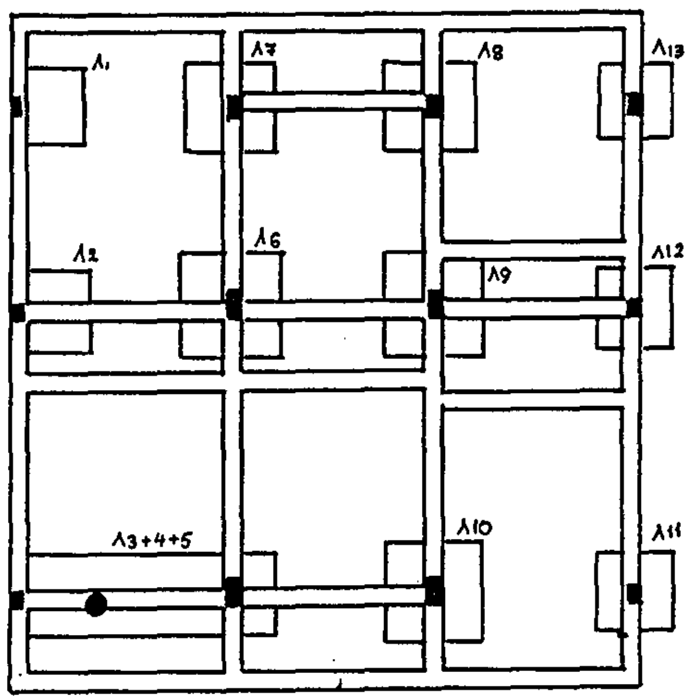


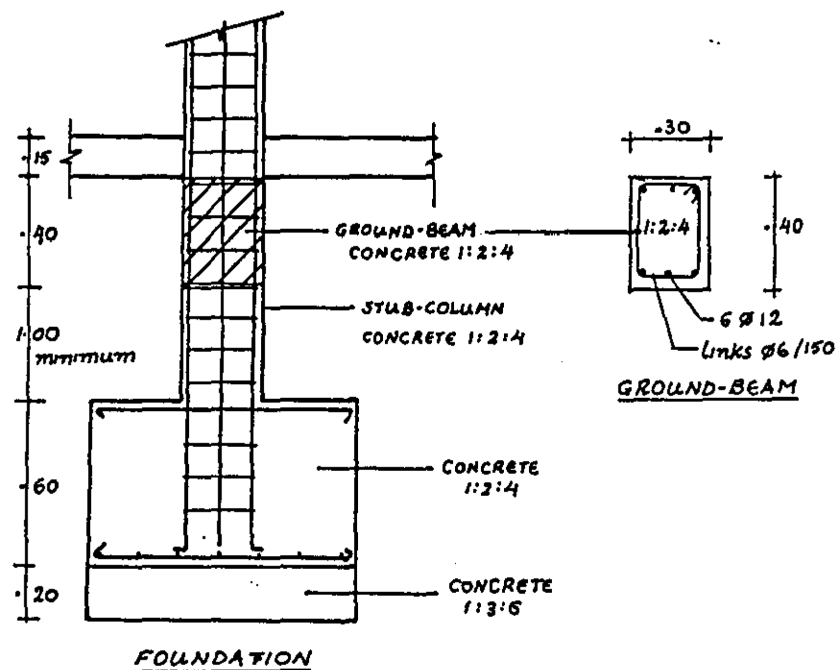
FIRST FLOOR



DETAILING OF FIRST & TOP FLOOR SLABS

6.1 (b) Drawings of the house built in 1984





FOUNDATION & GROUND-BEAMS

6.1 (c) Drawings of the house built in 1984

BEAMS

FIRST FLOOR & TOP FLOOR SLABS

BEAM	DIMENSION	MAIN REINFORC.	·	TOP REINF.	LINKS	REMARKS
A1	25 × 60	6Ø16	+2Ø16		Ø6 /150	Contil.
A2	25 × 60	4Ø16	2Ø16	2Ø10	"	
A3	25 × 60	5Ø18	2Ø18	"	"	
A4	25 × 40	6Ø18	+2Ø16		"	Contil.
A5	25 × 40	5Ø18	+2Ø16		"	"
A6	25 × 60	5Ø18	2Ø18	2Ø10	"	
A7	25 × 60	4Ø18	"	"	"	
A8	25 × 60	5Ø18	2Ø16		"	Contil.
A9	25 × 60	5Ø18	2Ø16		"	"
A10	25 × 60	4Ø18	2Ø18	2Ø10	"	
A11	25 × 60	5Ø18	"	"	"	
A12	25 × 60	5Ø18	2Ø16		"	Contil.
A13	25 × 60	4Ø18	2Ø16		"	"
A14	25 × 60	4Ø18	2Ø18	2Ø10	"	
A15	25 × 60	4Ø18	"	"	"	
A16	25 × 60	4Ø18	2Ø16		"	Contil.
A17	25 × 60	4Ø12	2Ø12	2Ø10	"	
A18	25 × 60	4Ø12	"	"	"	
A19	25 × 60	5Ø12	"	"	"	
A20	25 × 60	4Ø12	"	"	"	
A21	25 × 60	4Ø12	"	"	"	
A22	25 × 60	4Ø12	"	"	"	
A23	25 × 60	4Ø12	"	"	"	
A24	25 × 60	4Ø12	"	"	"	
A25	25 × 60	4Ø12	"	"	"	
A26	20 × 15	6Ø16	"	"	"	slab-beam

All the bars should pass through the columns.

PADS		COLUMNS				
PAD	DIMENSIONS	COLUMN	DIMENSIONS	REINFORC.	LINKS	CONCRETE
A1	140×140	K1	25×30	6Ø12	Ø6/125	C25
A2	150×150	K2	25×30	6Ø12	"	"
A3,4,5	500×150	K3	25×30	6Ø12	"	"
·	·	K4	Ø40	10Ø16	"	"
·	·	K5	25×50	6Ø16	"	"
A6	190×190	K6	25×50	6Ø16	"	"
A7	170×170	K7	25×40	6Ø16	"	"
A8	170×170	K8	25×40	6Ø16	"	"
A9	190×190	K9	25×50	6Ø16	"	"
A10	180×180	K10	25×50	6Ø16	"	"
A11	140×140	K11	25×30	6Ø12	"	"
A12	150×150	K12	25×30	6Ø12	"	"
A13	140×140	K13	25×30	6Ø12	"	"

6.1 (d) Drawings of the house built in 1984

6.2.2 Assessment of the building's earthquake resistance

Structural analysis of the building should be done first using the seismic loading. Such an analysis is nowadays done easily with the aid of a computer programme. However some analysis by hand of selected elements follows. It is to be assumed of course, that during the stage of assessing the earthquake resistance a complete analysis was employed, as suggested by the seismic code.

After analysis the structure, an assessment of the architecture of the building should be done. All the characteristics of the house including non–structural elements should be examined.

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ANALYSIS OF FRAME Δ12-K10-Δ11-K9-Δ10-K8-Δ9

LOADING.

Floor Slab	$= 0.15 \times 24 \times g$	$= 3.5$	
Beam	$= \frac{1}{3.8}(0.5 \times 0.25 \times 2.4 \times g)$	$= 0.8$	
Floors	$=$	$= 1.0$	
Walls	$=$	$= 0.9$	
Finishes	$=$	$= 0.5$	KN/m^2
		6.7 KN/m^2	$G_K = 6.7$

NOTE: $Q_K = 2.0\ KN/m^2$ is typically used in Cyprus. $Q_K = 2.0$

SEISMIC DESIGN LOAD

$$S_d = S\,(G_K + E + \psi Q_K)$$

$$E = G_K + \psi Q_K \qquad \psi = 0.25 \text{ (Table 4.1.4)} \qquad E = 7.2$$
$$= 7.2\ KN/m^2$$

EQUIVALENT STATIC ANALYSIS

$$F_i = C_d \cdot \gamma_i \cdot W_i$$

A_{max}	$=$	0.10	Nicosia-Zone II
S	$=$	1.25	Class II
K	$=$	3.0	D.L. II
I	$=$	1.0	Class III
α	$=$	2.5	Cl. 6.4.1.

$$W_i = E \times \text{Floor Area}$$
$$= 7.2 \times 146$$
$$= 1051 \quad KN$$

$$C_d = I \cdot A \cdot S \cdot \alpha \cdot \frac{1}{K}$$
$$= 0.10$$

$$\gamma_i = \frac{\Sigma W_i}{\Sigma W_i \cdot h_i}\, h_i$$

$$= 0.072\ h_i$$

$\therefore$ F_i = 21 KN GROUND FLOOR
= 49 KN FIRST FLOOR
= 77 KN ROOF

LOADS ON FRAME

$F = F_i \times \frac{3.8}{11.65}$ = 7 KN GROUND FLOOR
= 16 KN FIRST FLOOR
= 25 KN ROOF

ANALYSIS OF THE FRAME

The frame was analysed using a computer programme and the results are attached.

DESIGN OF BEAMS.

$\frac{1.4}{250} < \rho < \frac{7}{250}$ $\qquad \rho = \frac{A_s}{A_c}$

$\therefore \; 910 < A\rho < 4550 \; (mm^2)$

a. Roof beams

At K_8 :

$K = \frac{M}{f_{cu} \cdot b \cdot d^2}$

$= 0.026 \; (K < K')$

$A_s = \frac{M}{0.87 \cdot f_y \cdot z}$

$= 395 \; mm^2$

$\Rightarrow \underline{\underline{910}} \; mm^2$ (MINIMUM)

M = 47 KNm
K = 0.156
f_{cu} = 20 N/mm²
d = 600 mm
b = 250 mm
f_y = 250 N/mm²
z = 0.91 d

At K9 : K = 0.0341

A_s = 572 mm²
$\Rightarrow \underline{\underline{910}}$ mm²

M = 66 KNm
z = 0.87 d

At K10 : K = 0.039

A_s = 730 mm²
$\Rightarrow \underline{\underline{910}}$ mm²

M = 79 KNm
z = 0.89 d

Midspan Δ10 : $K = 0.015$ $M = 14$ KNm

$A_s = 110\ mm^2$ $z = 0.95d$

$\Rightarrow \underline{\underline{910}}\ mm^2$

Midspan Δ11 : $K = 0.032$ $M = 51$ KNm

$A_s = 440\ mm^2$ $z = 0.94d$

$\Rightarrow \underline{\underline{910}}\ mm^2$

b. First-floor beams

At K8 :

$$K = \frac{M}{f_{cu} \cdot b \cdot d^2} = 0.026$$

$$A_s = \frac{M}{0.87 \cdot f_y \cdot z} = 395\ mm^2 \Rightarrow \underline{\underline{910}}\ mm^2$$

$M = 47.2$ KNm
$f_{cu} = 20\ N/mm^2$
$f_y = 250\ N/mm^2$
$d = 600$ mm
$b = 250$ mm
$K' = 0.156$

$z = 0.91d$

At K9 : $K = 0.041$ $M = 73$ KNm

$A_s = 630\ mm^2$ $z = 0.91d$

$\Rightarrow \underline{\underline{910}}\ mm^2$

At K10 : $K = 0.058$ $M = 102$ KNm

$A_s = 875\ mm^2$ $z = 0.91d$

$\Rightarrow \underline{\underline{910}}\ mm^2$

Midspans Δ10 & Δ11 : Moments are low and therefore the lower limit on As should be used i.e. $A_s = \underline{\underline{910}}\ mm^2$

c. Ground beams

At K8 : $K = 0.026$ | $M = 47.2$ KNm, $f_{cu} = 20$ N/mm², $f_y = 250$ N/mm², $b = 250$ mm, $d = 600$ mm, $K = 0.156$

$$A_s = \frac{M}{0.87 \cdot f_y \cdot z}$$

$= 395$ mm²

$\Rightarrow \underline{\underline{910}}$ mm²

$z = 0.91d$

At K9 : $K = 0.040$ | $M = 72$ KNm

$A_s = 625$ mm² | $z = 0.91d$

$\Rightarrow \underline{\underline{910}}$ mm²

At K10 $K = 0.056$ | $M = 98$ KNm

$A_s = 835$ mm²

$\Rightarrow \underline{\underline{910}}$ mm²

Midspans Δ10 & Δ11 : Moments are low. Lower limit on As should be used i.e. $A_s = \underline{\underline{910}}$ mm²

DESIGN OF COLUMNS

Unbraced columns, $l_e = \beta l_o$ | $\beta = 1.2$, $l_o = 3.15$ | BS 8110 Table 3.22

$= 3.78$

$$l_e/b = \frac{3780}{250}$$

$= 15.3 > 10$ ∴ Slender

$\beta_a = 0.11$, $K = 1$ | Table 3.23

∴ $a_u = \beta_a \cdot K \cdot h$

$= 0.11 \times 1 \times 500$, $= 0.11 \times 1 \times 400$

$= 55$ mm (K9-K10), $= 45$ mm (K8)

$M_{add} = a_u \cdot N$

$1.0\,\% < p < 6.0\,\%$ | Cyprus Code

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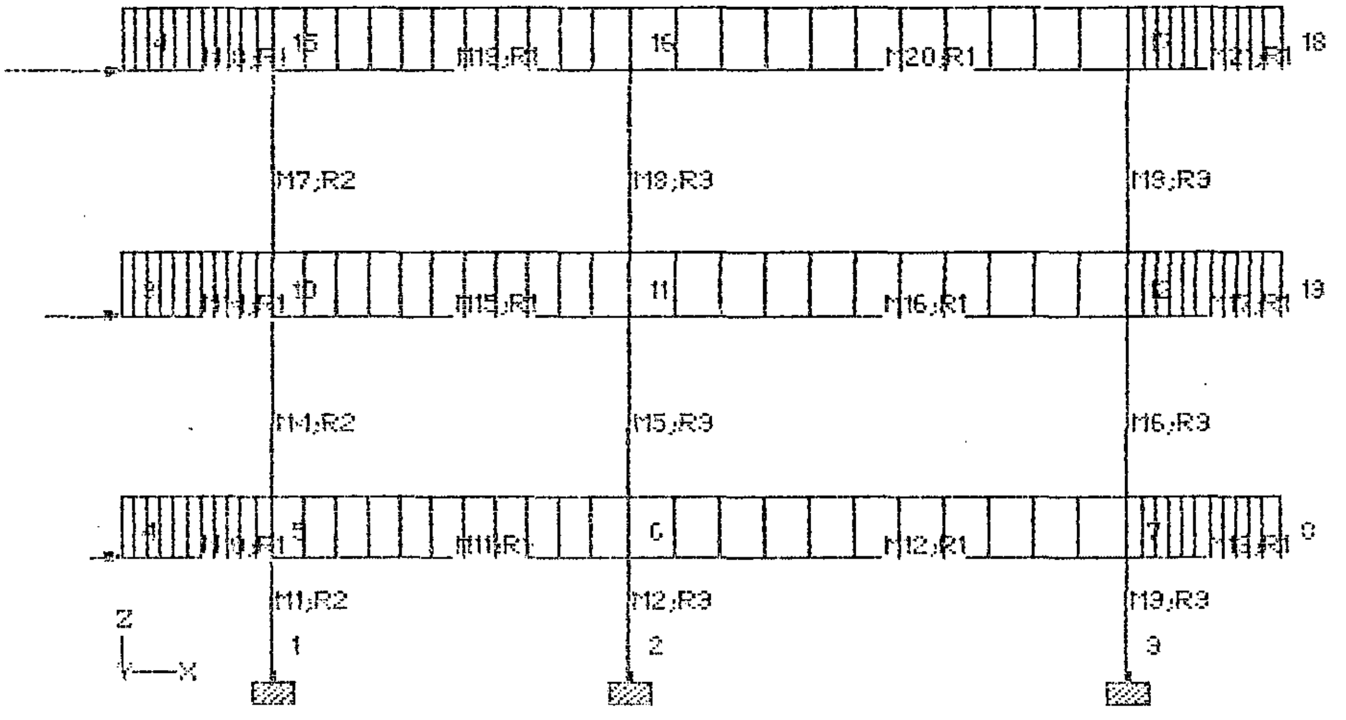
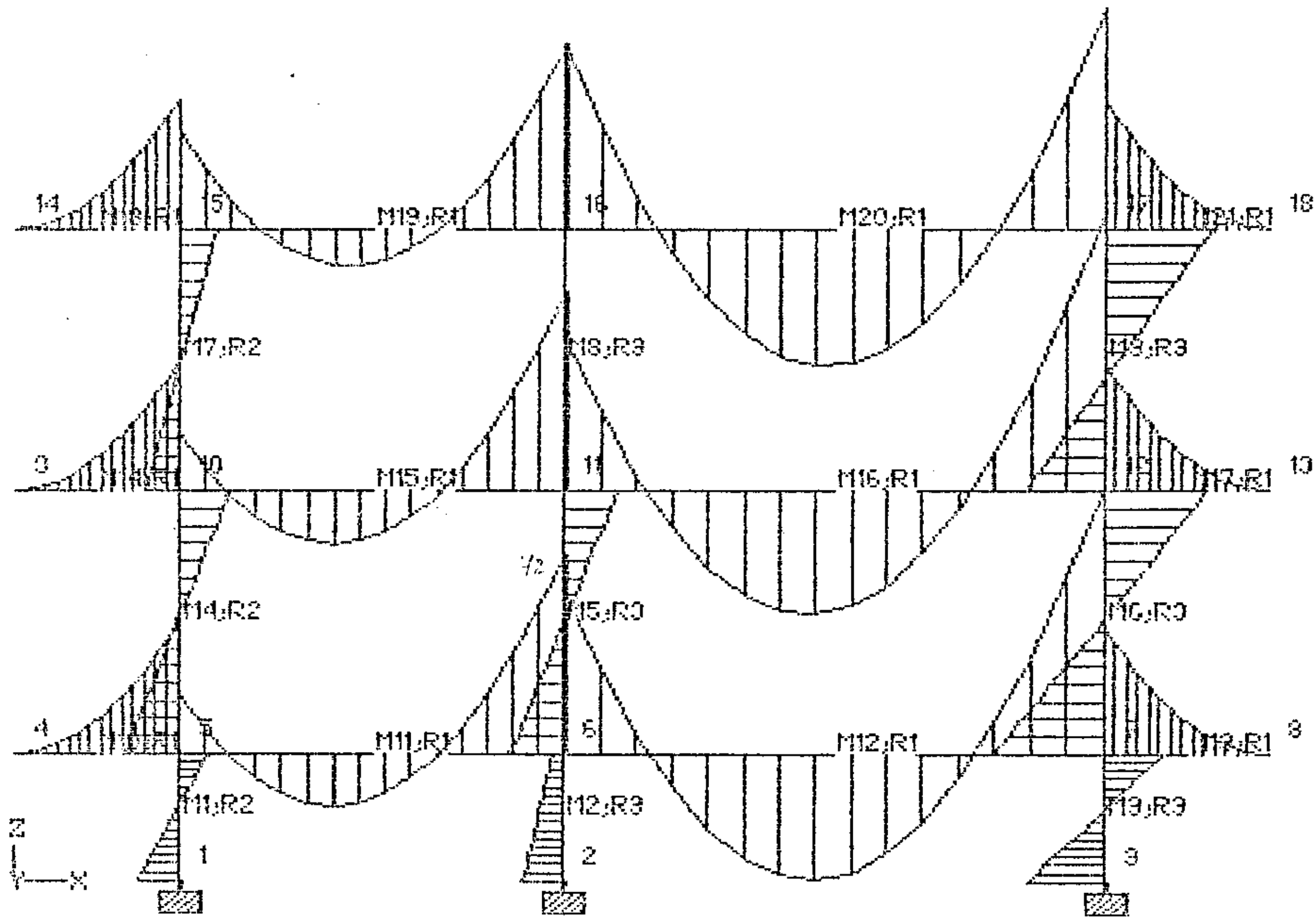
Using chart 29 (BS8110) and with the limitations above :

Column		N(KN)	M(KNm)	M/bh^2	N/bh	ρ(%)	Asc (mm^2)
K8	1ST Floor	125	30	0·75	1·25	1	1000
	Gr. Floor	250	44	1·1	2·50	1	1000
	Stub-C	375	41	1·0	3·75	1	1000
K9	1ST Floor	160	11	0·2	1·28	1	1250
	Gr. Floor	320	53	0·9	2·56	1	1250
	Stub-C	480	43	0·7	3·84	1	1250
K10	1ST Floor	150	74	1·2	1·20	1	1250
	Gr. Floor	300	83	1·3	2·40	1	1250
	Stub-C	450	72	1·2	3·60	1	1250

Loading Diagram.



Bending Moment Diagram.



Nodes

No.	X(m)	Z(m)	Supports
1	1.650	0.000	FB
2	5.500	0.000	FB
3	10.850	0.000	FB
4	0.000	1.500	NULL
5	1.650	1.500	NULL
6	5.500	1.500	NULL
7	10.850	1.500	NULL
8	12.500	1.500	NULL
9	0.000	4.650	NULL
10	1.650	4.650	NULL
11	5.500	4.650	NULL
12	10.850	4.650	NULL
13	12.500	4.650	NULL
14	0.000	7.800	NULL
15	1.650	7.800	NULL
16	5.500	7.800	NULL
17	10.850	7.800	NULL
18	12.500	7.800	NULL

Members

No.	Node A	Node B	Ref	Pins- A/ y	; B/ y	Length
1	1	5	2	f	f	1.500m
2	2	6	3	f	f	1.500m
3	3	7	3	f	f	1.500m
4	5	10	2	f	f	3.150m
5	6	11	3	f	f	3.150m
6	7	12	3	f	f	3.150m
7	10	15	2	f	f	3.150m
8	11	16	3	f	f	3.150m
9	12	17	3	f	f	3.150m
10	4	5	1	f	f	1.650m
11	5	6	1	f	f	3.850m
12	6	7	1	f	f	5.350m
13	7	8	1	f	f	1.650m
14	9	10	1	f	f	1.650m
15	10	11	1	f	f	3.850m
16	11	12	1	f	f	5.350m
17	12	13	1	f	f	1.650m
18	14	15	1	f	f	1.650m
19	15	16	1	f	f	3.850m
20	16	17	1	f	f	5.350m
21	17	18	1	f	f	1.650m

Properties

No.	A $\times 10^3 mm^2$	I $\times 10^6 mm^4$	E kN/mm^2
1	162.50	5700.00	28.00
2	100.00	1333.30	28.00

3	125.00	2600.00	28.00

Properties

No.	A $\times 10^3 mm^2$	I $\times 10^6 mm^4$	E kN/mm^2

Loadings

Node	Load No.	Type	Dir.	Mag(kN).
4	1	POINT	X	7.000
9	1	POINT	X	16.000
14	1	POINT	X	25.000

Member	Load No.	Type	Glob/Loc		Mag. A (kN/m)	Mag. B (kN/m)
10	1	DISTRIBUTED	GLOBAL	Z	-34.65	-34.65
11	1	DISTRIBUTED	GLOBAL	Z	-34.65	-34.65
12	1	DISTRIBUTED	GLOBAL	Z	-34.65	-34.65
13	1	DISTRIBUTED	GLOBAL	Z	-34.65	-34.65
14	1	DISTRIBUTED	GLOBAL	Z	-34.65	-34.65
15	1	DISTRIBUTED	GLOBAL	Z	-34.65	-34.65
16	1	DISTRIBUTED	GLOBAL	Z	-34.65	-34.65
17	1	DISTRIBUTED	GLOBAL	Z	-34.65	-34.65
18	1	DISTRIBUTED	GLOBAL	Z	-34.65	-34.65
19	1	DISTRIBUTED	GLOBAL	Z	-34.65	-34.65
20	1	DISTRIBUTED	GLOBAL	Z	-34.65	-34.65
21	1	DISTRIBUTED	GLOBAL	Z	-34.65	-34.65

Analysed Member Forces (Local Axes)

No.	Node	x-Force Axial (kN)	z-Force Shear (kN)	y-Force Shear (kN)	x-Moment Torsion (kNm)	z-Moment B.M. (kNm)	y-Moment B.M. (kNm)
1	1	336.680	14.299	--------	--------	--------	-12.153
	5	-336.680	-14.299	--------	--------	--------	-9.295
2	2	494.846	6.240	--------	--------	--------	-12.717
	6	-494.846	-6.240	--------	--------	--------	3.357
3	3	467.848	27.461	--------	--------	--------	-22.792
	7	-467.848	-27.461	--------	--------	--------	-18.400
4	5	226.150	10.232	--------	--------	--------	-16.273
	10	-226.150	-10.232	--------	--------	--------	-15.956
5	6	329.312	10.347	--------	--------	--------	-16.527

	11	-329.312	-10.347	--------	--------	--------	-16.066
6	7	310.788	20.421	--------	--------	--------	-32.747
	12	-310.788	-20.421	--------	--------	--------	-31.581
7	10	115.720	6.935	--------	--------	--------	-9.947
	15	-115.720	-6.935,	--------	--------	--------	-11.898

Analysed Member Forces (Local Axes)

No.	Node	x-Force Axial (kN)	z-Force Shear (kN)	y-Force Shear (kN)	x-Moment Torsion (kNm)	z-Moment B.M. (kNm)	y-Moment B.M. (kNm)
8	11	165.063	0.113	--------	--------	--------	-0.408
	16	-165.063	-0.113	--------	--------	--------	0.051
9	12	152.342	17.952	--------	--------	--------	-23.729
	17	-152.342	-17.952	--------	--------	--------	-32.819
10	4	7.000	-0.000	--------	--------	--------	0.000
	5	-7.000	57.173	--------	--------	--------	47.167
11	5	2.933	53.358	--------	--------	--------	-21.599
	6	-2.933	80.045	--------	--------	--------	72.970
12	6	7.040	85.490	--------	--------	--------	-59.801
	7	-7.040	99.887	--------	--------	--------	98.314
13	7	0.000	57.172	--------	--------	--------	-47.167
	8	-0.000	-0.000	--------	--------	--------	-0.000
14	9	16.000	0.000	--------	--------	--------	0.000
	10	-16.000	57.172	--------	--------	--------	47.167
15	10	12.703	53.257	--------	--------	--------	-21.264
	11	-12.703	80.145	--------	--------	--------	73.022
16	11	2.470	84.104	--------	--------	--------	-56.548
	12	-2.470	101.274	--------	--------	--------	102.477
17	12	0.000	57.172	--------	--------	--------	-47.167
	13	0.000	0.000	--------	--------	--------	-0.000
18	14	25.000	0.000	--------	--------	--------	-0.000
	15	-25.000	57.172	--------	--------	--------	47.167
19	15	18.065	58.548	--------	--------	--------	-35.270
	16	-18.065	74.855	--------	--------	--------	66.662
20	16	17.952	90.208	--------	--------	--------	-66.713
	17	-17.952	95.170	--------	--------	--------	79.986
21	17	0.000	57.172	--------	--------	--------	-47.167
	18	-0.000	0.000	--------	--------	--------	0.000

Maximum Member Forces (Local Axes)

No.	Node	x-Force Axial (kN)	z-Force Shear (kN)	y-Force Shear (kN)	x-Moment Torsion (kNm)	z-Moment B.M. (kNm)	y-Moment B.M. (kNm)
2	2	494.846					
16	12		101.274				
16	12						102.477

Minimum Member Forces (Local Axes)

No.	Node	x-Force Axial (kN)	z-Force Shear (kN)	y-Force Shear (kN)	x-Moment Torsion (kNm)	z-Moment B.M. (kNm)	y-Moment B.M. (kNm)
17	12	0.000					
14	9		0.000				
14	9						0.000

Reactions (Global Axes)

Node	X (kN)	Z (kN)	Y (kN)	X Moment (kNm)	Z Moment (kNm)	Y Moment (kNm)
1	-14.299	336.680	-------	-------	-------	-12.153
2	-6.240	494.846	-------	-------	-------	-12.717
3	-27.461	467.848	-------	-------	-------	-22.792

Maximum Reactions (Global Axes)

Node	X (kN)	Z (kN)	Y (kN)	X Moment (kNm)	Z Moment (kNm)	Y Moment (kNm)
3	-27.461					
2		494.846				
3						-22.792

Minimum Reactions (Global Axes)

Node	X (kN)	Z (kN)	Y (kN)	X Moment (kNm)	Z Moment (kNm)	Y Moment (kNm)
2	-6.240					
1		336.680				
1						-12.153

Detail for Member number 1

Distance (m)	B.M. (kNm)	S.F. (kN)	Disp X (mm)	Disp Z (mm)

6.2.3 Comments

Following the analysis using seismic loading and the other observations mentioned above, a number of points may be brought up:

Stiffness Variations

a. The 'open' ground floor being very flexible creates a big stiffness difference between the ground floor and the first floor. The stiffness variation along the height of the building is such that the structure may be classified as irregular.

Ductility Requirements

b. The yield stress of the shear links is 250 N/mm^2 since mild steel is used. A higher yield stress would increase ductility. High yield steel should be used.

c. The transverse reinforcement, which is increasing ductility, is generally enough, as it would be explained later under Detailing. However, no critical zones are recognised and there is still the danger of a plastic hinge to be formed at the column-beam joints.

d. The use of mild steel as main reinforcement is good as far as ductility is concerned as it was shown in figure 3.3 (chapter 3).

e. The concrete compressive strength, f_{cu} used, 20 N/mm^2 for beams and 25 N/mm^2 for columns is adequate for ductility level II. It must be stressed, however, that C 20 concrete was used at the stub-columns and not C 25.

Masonry

f. Double walls are used in some cases (250 mm) or single walls of 200 mm and 100 mm. The infill walls are not separated in any way from the frame. During an earthquake they may act as shear walls and there is the possibility of torsion problems since all the double walls are mostly concentrated in the northern side of the house.

g. No reinforcement appears on the walls.

Building Configuration

h. The plan of the house and the arrangement of the columns are very symmetrical which is ideal for earthquake resistance. There are some doubts about positioning of walls due to their thickness variation as already mentioned.

i. The elevation is also simple without setbacks. The soft ground floor however, may cause total collapse of the building due to concentration of plastic deformation.

j. The presence of stub-columns createsa lot of doubts.

Shear Walls

k. No shear walls (assuming that the infill walls were considered as non-structural) were employed.

Using,

$$\frac{bd^2}{6} = \frac{H^2Fe}{5500}$$

where
$b = 0.15$ m
$H = 7.00$ m
$F = 150$ m^2
$e = 0.15$

Therefore,

$$d^2 = 8.02$$

Assuming four walls in each direction,

$$d^2 = \frac{8.02}{4}$$

$$= 1.45 \text{ m}$$

Detailing (clause 5.1 Cyprus Code)

l. **Beams**

Geometrical constraints: The width being 250 mm is adequate. Adequate are also the ratios b/h and l/h.

Longitudinal reinforcement: The results of the analysis show some problems with the amount of the reinforcement as shown in figure 6.2. More serious are the problem of inadequate reinforcement at column K 9 (all floors) and the ground beams.

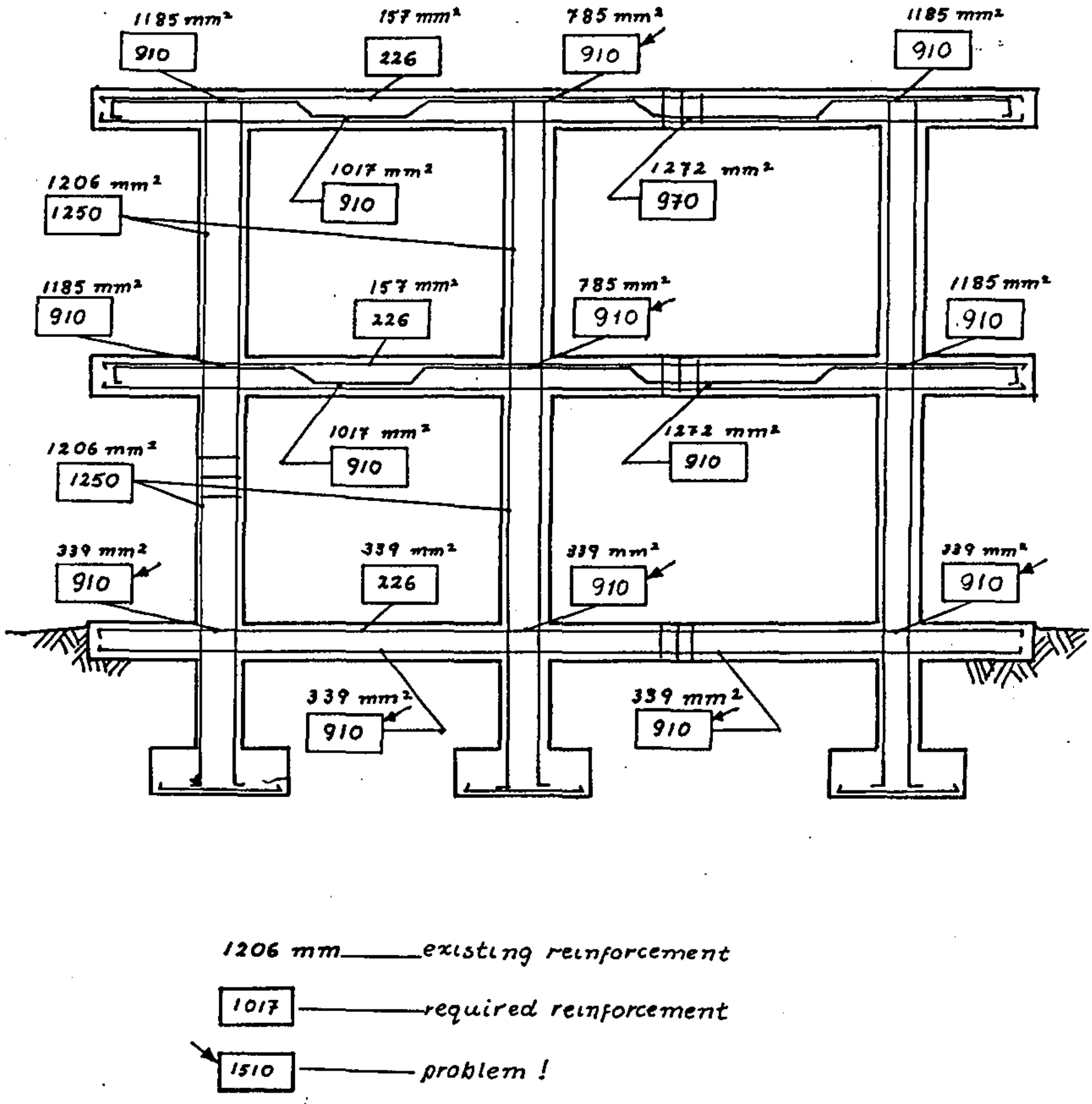
Minimum bar diameter is specified as 2 Y12 whereas 2 Y10 are used. Transverse reinforcement: No critical zones are distinguished. The spacing should be in the critical zones maximum 145 mm (80). Generally the transverse reinforcement is fixed at a spacing of 150 mm. Although this is roughly adequate the beam—column joint is not strengthened in relation to the beam and the plastic hinge is not shifted. The joint should be stronger than the beam. Splicing is done within the critical zones.

m. **Columns**

Geometrical Constraints: Minimum dimension requirements (250 mm) and the ratio l/b are in accordance with the code.

Longitudinal reinforcement: The results of the analysis show some problems with the amount of steel reinforcement used. (A comparison is shown in figure 6.2). Generally the amount of reinforcement is by 4% less than required (with exception of column K 8 on all floors). Transverse reinforcement: Again critical zones are not recognised. Generally the spacing of the links is satisfactory.

Splicing is done within the critical zones.



6.2 Comparison of existing steel reinforcement to required reinforcement after analysis with the seismic loads

6.2.4 Strengthening

Although no strengthening methods can provide the building with 100 per cent earthquake resistance the following measures will improve it considerably.

a. Elimination of the stub–columns (and thus the ground beams as well) by constructing new foundation pads on top of the old ones.
b. Construction of shear walls to stiffen the ground floor and to strengthen the first floor.
c. Strengthening of columns K 6 and K 9 and their joints on both floors.
d. Strengthening of the brickwall on the north side of the house.

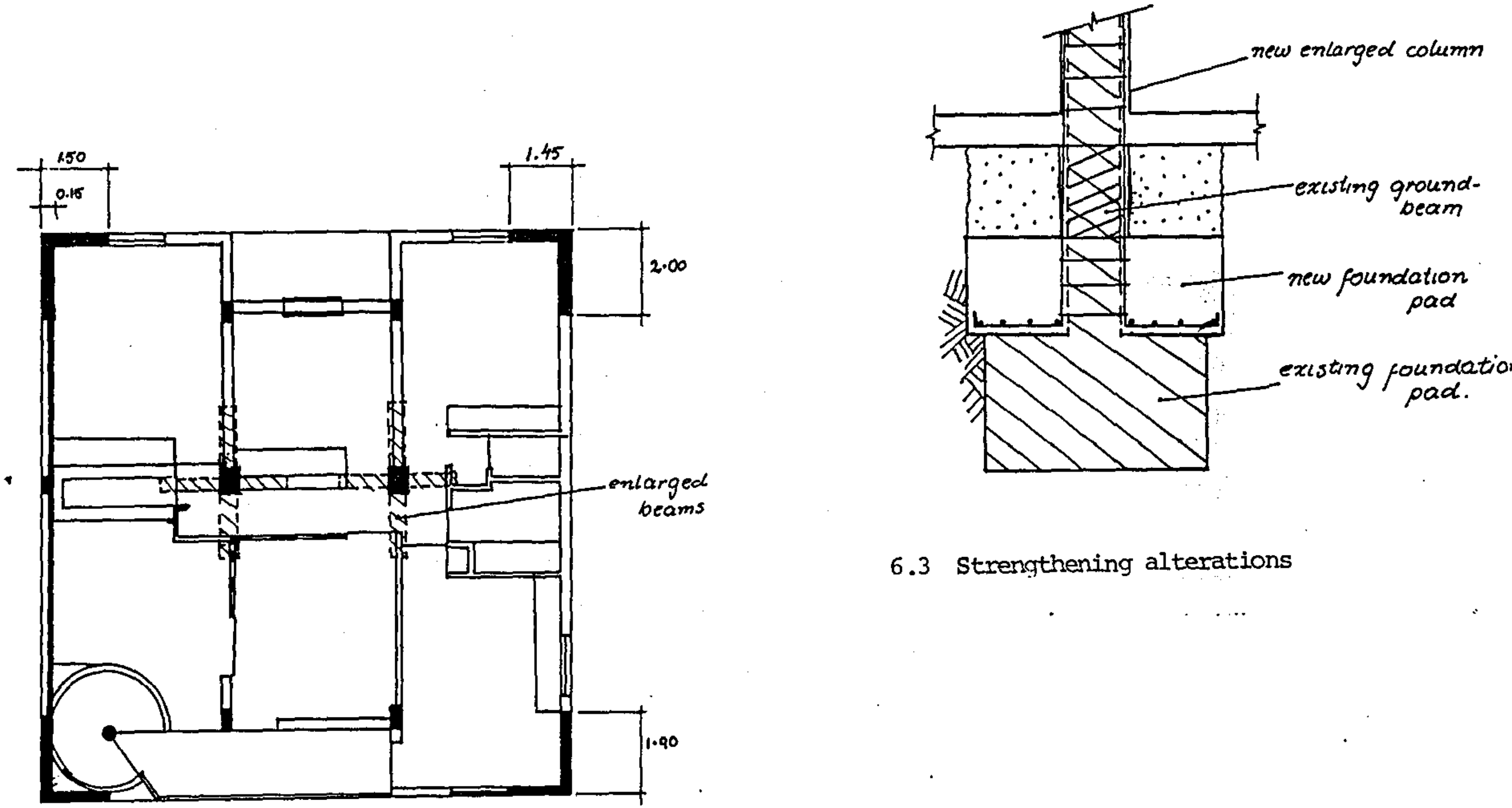
The above methods have been already discussed in chapter 5 and certain methods were suggested. Further specifications are given below:

6.2.4.1 Construction of foundation pads on top of the existing ones (see drawing 6.3 a)

- Existing pads are already adequate to hold additional weight due to strengthening procedures (shear walls, enlarged columns, beams and brickwalls). Therefore minimum possible reinforcement is necessary. Use Y12.250 for both directions.
- Starters for new shear walls and strengthened columns should be left, tied well on the foundations.

6.2.4.2 Introduction of shear walls

- Four walls in each direction are to be constructed as shown in figure 6.3 (b). Thickness of walls will be 150 mm and length 1.50 metres minimum.
- Steel reinforcement according to code Y12/200 horizontal and vertical. Vertical bars should continue through slabs whereas the horizontal ones should be anchored (every second will be enough) on the columns.



6.3 Strengthening alterations

- Concrete mix to be used (should achieve 30 – 35N/mm^2) consists of:

Cement	325 Kg	
Sand	850 kg	
10 mm Aggregates	720 kg	
Water	130 ltrs	
Polymer Latex	65 ltrs	(Yields 1 cubic metre)

- Internal faces should be fair–face whereas external should be rough. No rendering or painting is necessary at the ground floor.

<u>6.2.4.3 Columns and Joints K 6, K 9</u>

- New dimensions:

Columns K6, K9:	32 x 58
Beams, adjacent to K 6, K 9:	32 x 65

- Strengthening and enlargement will be done along the whole length of the columns from foundation to the roof. For the beams only the critical zones, i.e. 1.30 metres from the face of the columns will be strengthened.
- Additional steel reinforcement:

  Columns: 4 Y12 (corners)
  Y10/150 - 300 for links

  Beams (critical zone only):

Top	2 Y12
Bottom	2 Y12
Middle	2 Y10

- Concrete mix should be pourable (achieving 30 N/mm^2) and consists of:

Polymer grout	25 Kg	
Aggregates 5 mm	20 kg	
Water	5 ltrs	(Yields 25 ltrs)

- Internal faces should be fair–face whereas at the ground floor all faces should be rough.

6.2.4.4 Strengthening of brickwalls

- New thickness 150 mm (therefore 50 mm extra are necessary)
- The 50 mm layer should be produced by guniting using special grout. A cementitious bonding agent should be applied first.
- A reinforcing grid Y8/200 should be fixed. It must be tied on both the top beam and floor-slab.

A complete cost analysis follows:

The total cost of 11,681 Cyprus pound is considered to be quite high. It will, however, improve greatly the earthquake resistance of the building, and it will pay back. The total cost could be decreased by leaving out several 'improving factors' like the polypropylene fibres and employing less anchoring using the expensive epoxy grout. Needless to say that earthquake resistance decreases with cost as well...

COST ANALYSIS

No.	Item description	Unit	Quantity	Rate	Amount £	c.
A	FOUNDATIONS					
A1	Remove existing concrete floor 150 mm thick only on top of foundations pads	m^2	35	20.-	700	
A2	Remove existing backfill	m^3	55	20.-	1100	
A3	Steel reinforcement - High yield steel bars to BS 4449: Y12/250 #	kg	300	0.40	120	
A4	Place concrete of grade C 25 up to the level of the bottom of the existing ground floors	m^3	44	35.-	1540	
A5	Backfill 400 mm thick	m^3	15	8.-	120	
A6	Re-construct concrete floor 150 mm thick (including a steel grid Y8/20 #)	m^2	35	12.-	420	
B	SHEAR WALLS					
B1	Remove plaster, cut back to reinforcement and roughen sides of both columns	m^2	38	8.-	304	
	and beams affected.	m^2	17	15.-	255	
B3	Drill through beams for steel bars to pass	m	24	12.-	288	
B4	Steel reinforcement - High yield steel bars to BS 4449: Y12/200 #	kg	1700	0.40	680	
B5	Drill columns 100 mm deep at 400 mm intervals and anchor horizontal bars using an epoxy grout.	cm^3	2880	0.40	1152	
B6	Erect watertight formwork 'letter box' type.					
	fair - face	m^2	57	6.-	342	
	rough - face	m^2	58	3.50	203	
B7	Place concrete (as specified)	m^3	10	150.-	1500	
B8	Repair damages				200	
B9	Render external faces adding polypropylene fibres in the mortar	m^2	30	6.-	180	
B10	Paint internal affected areas	m^2	57	5.-	285	

COST ANALYSIS

No.	Item description	Unit	Quantity	Rate	Amount £	c.
C	STRENGTHENING OF COLUMNS K6,K9 AND JOINTS					
C1	Remove plaster, cut back to reinforcement and roughen affected beams and columns	m^2	32	15.-	480	
C2	Demolish brickwork	m^2	2	8.-	16	
C3	Drill through slabs for column bars to pass	m^2	0.5	40.-	20	
C4	Drill through beams for new links to pass	m	19	14.-	266	
C5	Steel reinforcement on columns:					
	Y12	kg	72	0.40	29	
	Y10	kg	140	0.40	56	
C6	Steel reinforcement on beams					
	Y12	kg	72	0.40	29	
	Y10	kg	245	0.40	98	
C7	Erect watertight formwork - letterbox type					
	fair - face	m^2	22	6.-	132	
	rough - face	m^2	22	3.50	77	
C8	Place pourable concrete (as specified)	m^3	1	400.-	400	
C9	Repair damage	-			150	
C10	Paint affected areas	m^2	16	5.-	80	
D	STRENGTHENING OF BRICKWALL					
D1	Remove plaster - finishes	m^2	21	6.-	126	
D2	Cut back to beam reinforcement on top floor and first floor	m	8	10.-	80	
D3	Fix steel reinforcement - grid					
	Y8	kg	90	0.40	36	
D4	Use bonding agent (cementitous) 1kg/m^2	m^2	21	8.-	168	
D4	Guniting with special grout 50 mm thick	m^2	21	40.-	840	
D5	Render affected areas	m^2	21	4.-	84	
D6	Paint	m^2	21	5.-	105	
					11681	

6.3 A HOUSE BUILT IN 1991

6.3.1 Introduction

The second case is one of a building designed in 1990 and built in 1991. It is a two-storey house in the area of Dassoupolis in Nicosia. It is considered to be a 'luxury' house costing about 80,000 pounds (Cyprus pounds).

It is a reinforced concrete structure with infill masonry walls. The owner of the house had actually asked for an earthquake-resistant building and it would be therefore interesting to study this case.

Once more the suggested seismic code for Cyprus (Appendix II) and the information given in chapter 4 were used for the assessment of the earthquake resistance of the building. A ductility level II was assumed. Drawings of the building are shown in figures 6.4 (a),(b),(c),)d),(e),(f) and (g).

6.3.2 Assessment of the building's earthquake resistance

In the following pages some structural elements were selected for analysis using the seismic loads according to the code. The analysis is done by hand and it is to be assumed that prior to any diagnosis a complete analysis of the whole structure is made using a computer programme.

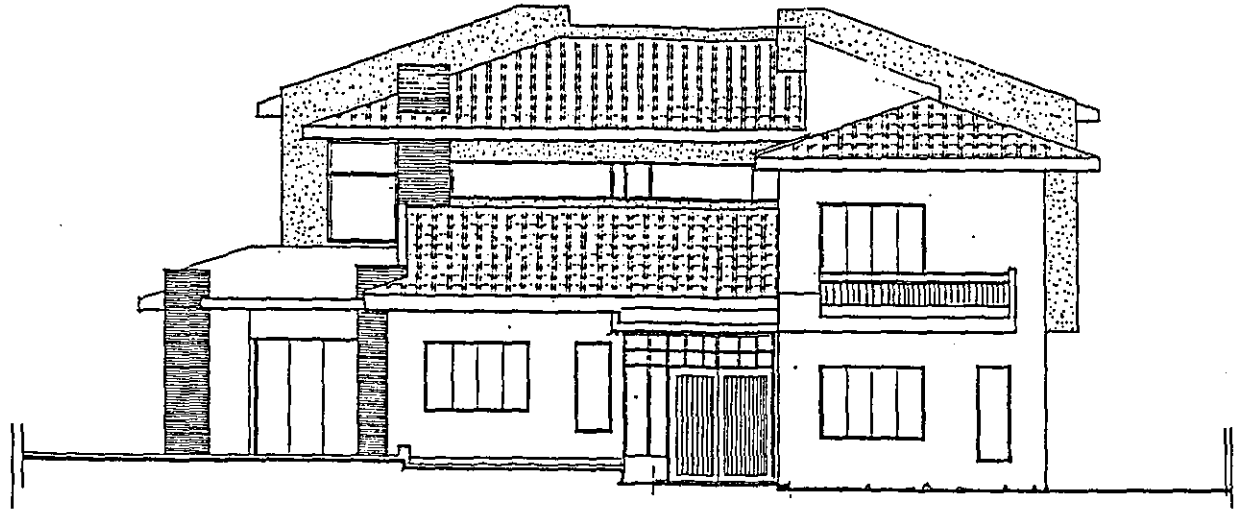
Following the analysis the assessment will proceed by examining the architecture of the building and all its characteristics.

6.3.3 COMMENTS

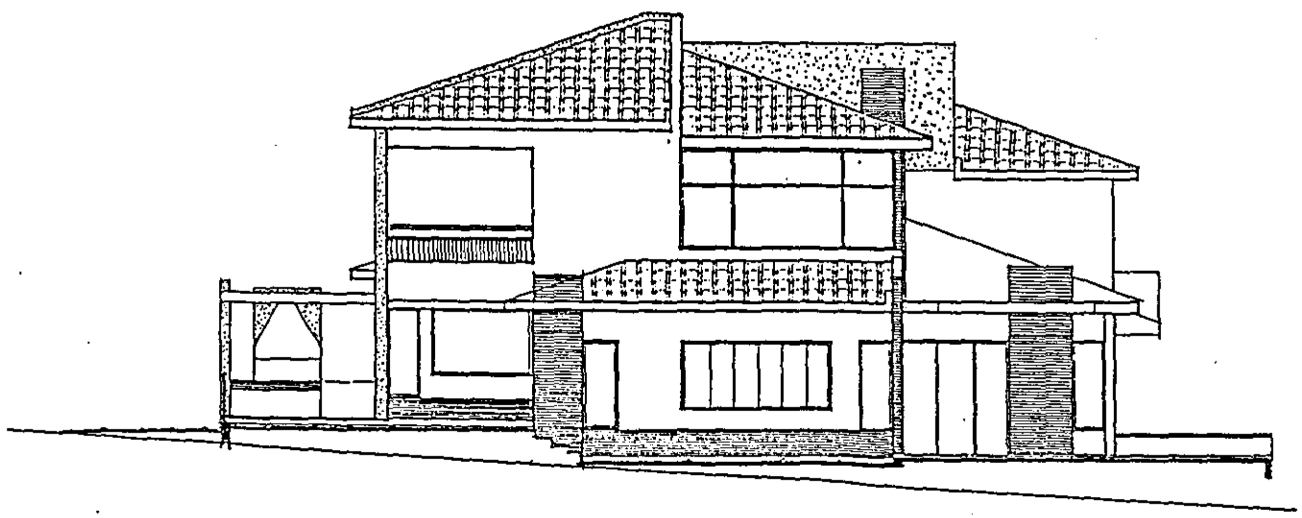
Having analysed the structure and examining the whole building, the following points may be brought up:

Ductility Requirements

a. Mild steel is used for shear links (f_y = 250 N/mm^2). A higher yield steel should be used.



ΒΟΡΙΟΔΥΤΙΚΗ ΟΨΗ



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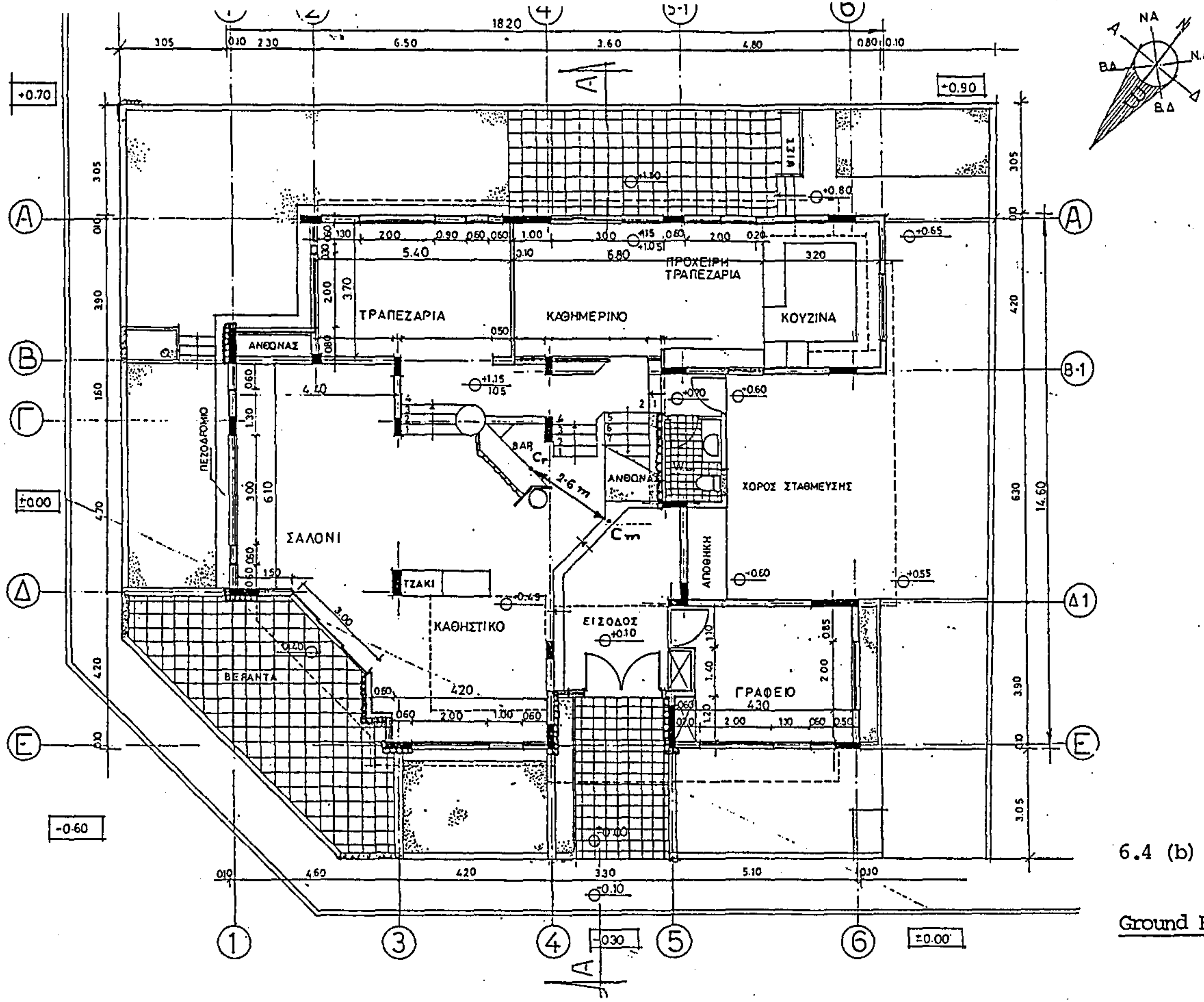
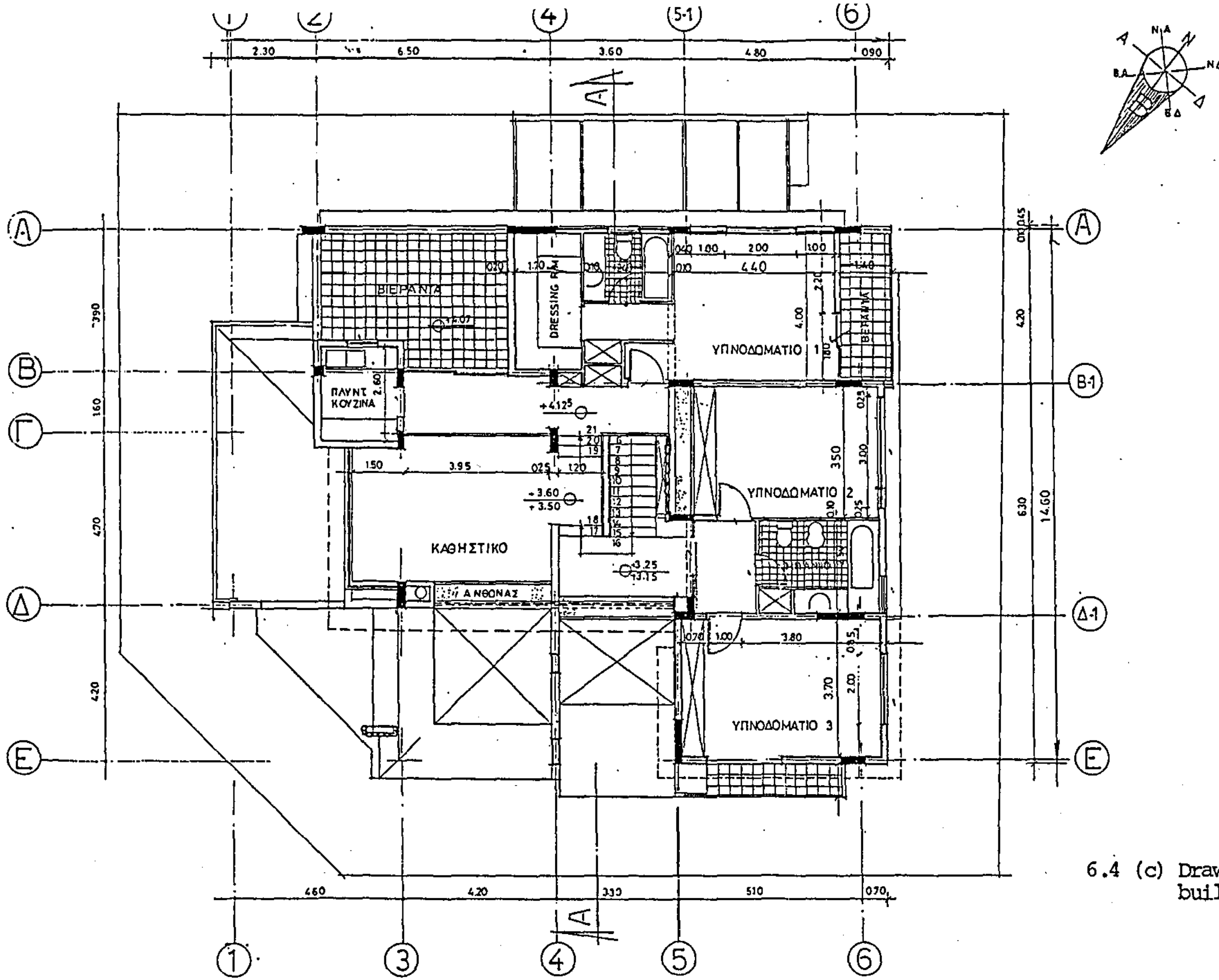
6.4 (a) Drawings of the house built in 1991

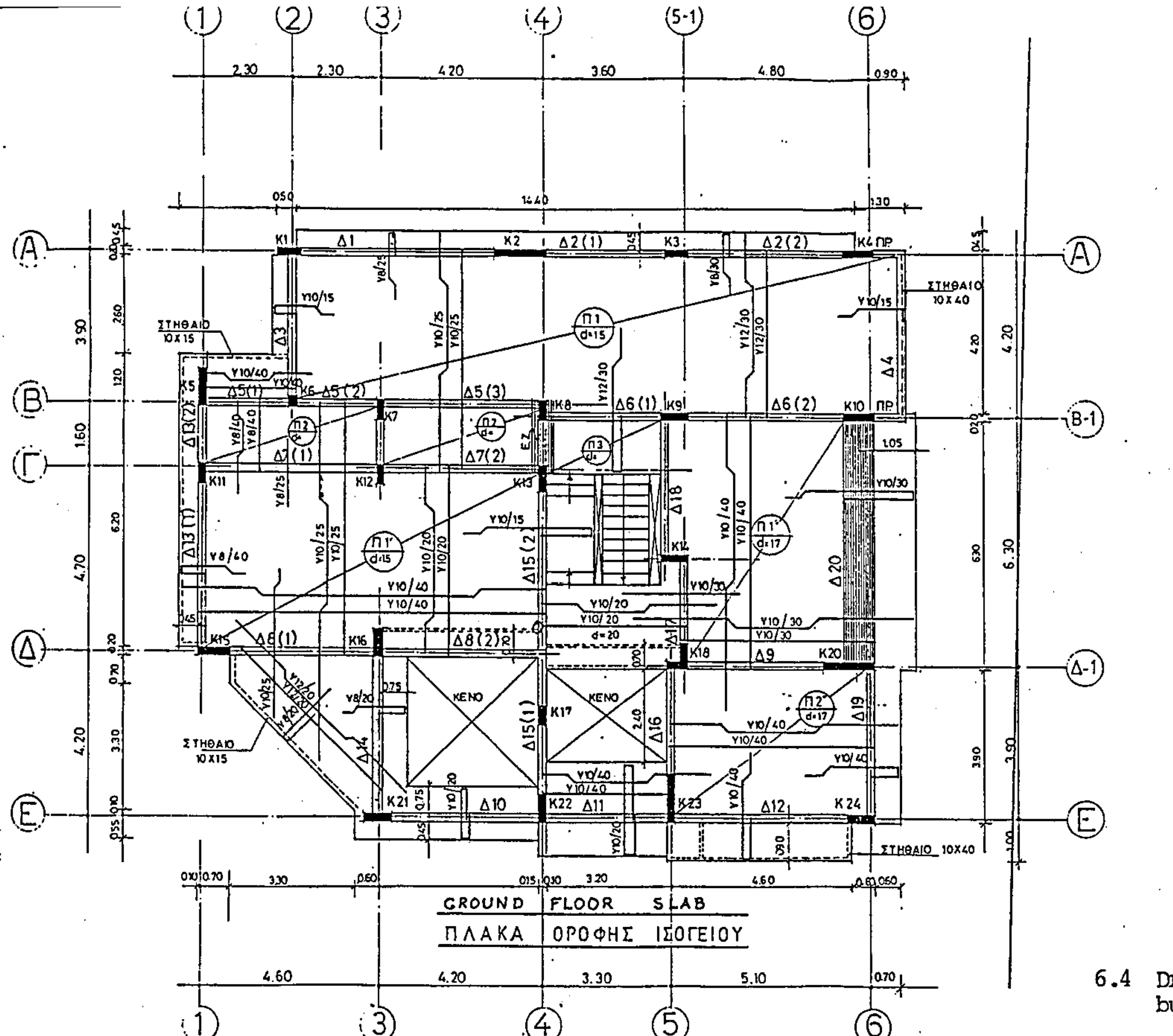


6.4 (b) Drawings of the hous built in 1991,

Ground Floor



6.4 (c) Drawings of the house built in 1991



GROUND FLOOR SLAB

ΠΛΑΚΑ ΟΡΟΦΗΣ ΙΣΟΓΕΙΟΥ

BEAM	DIMENS.	REINFORCEMENT TOP	BOTT.	B-UP	LINKS	SUPPORTS	REMARKS
Δ1	20x60	2Y12	2Y14	2Y14	Φ8/20		
Δ2 (1)	20x60	2Y12	2Y14	2Y14	Φ8/20		
Δ2 (2)	20x60	2Y12	2Y14	2Y14	Φ8/20		
ΠΡ.	20x60	2Y12	2Y12	----	Φ8/10		
Δ3	20x60	2Y12	2Y14	2Y12	Φ8/20		
Δ4	20x60	2Y12	2Y12	2Y12	Φ8/20		
Δ5 (1)	20x100	4Y12	4Y12	----	Φ8/15		
Δ5 (2)	20x70	4Y12	4Y12	----	Φ8/15		
Δ5 (3)	20x60	2Y12	2Y14	2Y14	Φ8/20		
Δ6 (1)	20x60	2Y12	2Y16	2Y14	Φ8/20		
Δ6 (2)	20x105	2Y12	2Y16	2Y16	Φ8/20		
ΠΡ.	20x105	2Y12	2Y12	----	Φ8/10		
Δ7 (1)	20x70	2Y12	2Y14	2Y14	Φ8/20		ΑΝΤ
Δ7 (2)	20x70	2Y12	2Y16	2Y14	Φ8/20		ΑΝΤ
Δ8 (1)	20x60	2Y12	2Y14	2Y14	Φ8/20		ΑΝΤ
Δ8 (2)	20x60	2Y12	2Y14	2Y14	Φ8/20		ΑΝΤ
Δ9	20x60	2Y12	2Y14	2Y14	Φ8/20		
Δ10	20x60	2Y12	2Y14	2Y14	Φ8/20		
Δ11	20x60	2Y12	2Y14	2Y14	Φ8/20		ΑΝΤ
Δ12	20x60	2Y12	2Y16	2Y16	Φ8/20		
Δ13 (1)	20x60	2Y12	2Y14	2Y14	Φ8/20		
Δ13 (2)	20x60	2Y12	4Y12	----	Φ8/10		
Δ14	20x60	2Y12	2Y16	3Y16	Φ8/15		ΑΝΤ
Δ15 (1)	25x55	2Y14	2Y16	3Y16	Φ8/15	2Y14	ΑΝΤ
Δ15 (2)						2Y14	ΑΝΤ.
Δ16	20x60	2Y12	2Y12	2Y12	Φ8/20		
Δ17	20x60	2Y12	2Y14	2Y14	Φ8/15		
Δ18	20x60	2Y12	2Y14	2Y14	Φ8/20		ΑΝΤ
Δ19	20x60	2Y12	2Y14	2Y14	Φ8/20		
Δ20	50x60	10Y16	5Y20	3Y20	2Φ8/12.5		

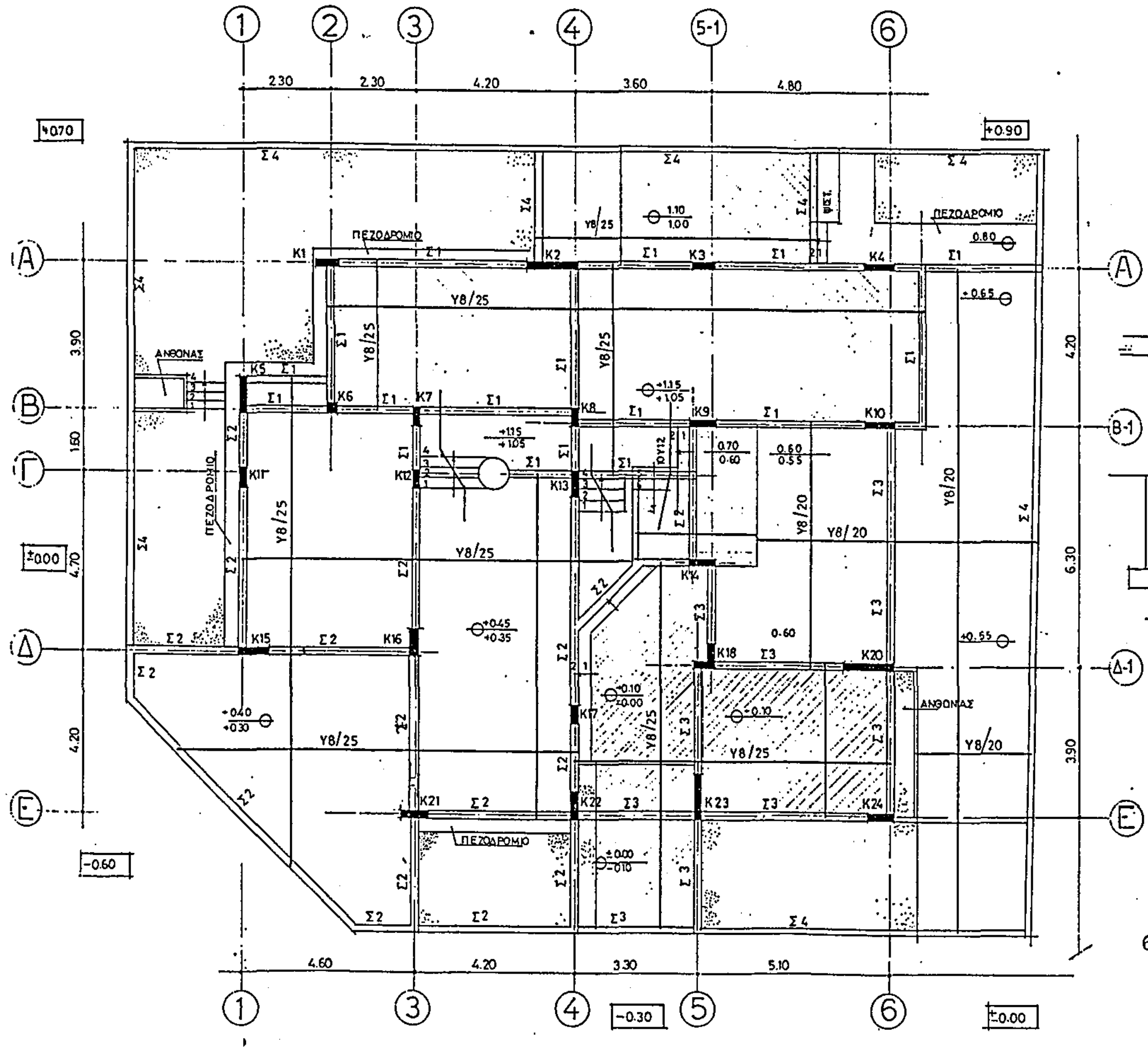
ΥΛΙΚΑ ΚΑΤΑΣΚΕΥΗΣ

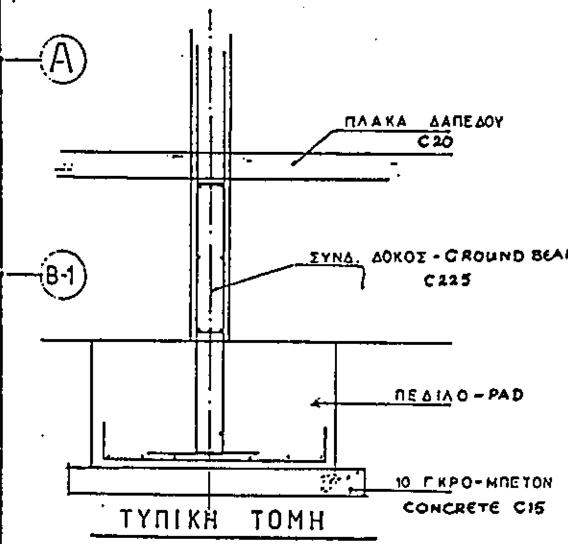
- ΣΚΥΡΟΔΕΜΑ B 225
- ΟΠΛΙΣΜΟΣ Y STIII (ΣΤΡΙΦΤΟ)
  Φ STI (ΜΑΛΑΚΟ)

6.4 Drawings of the house built in 1991

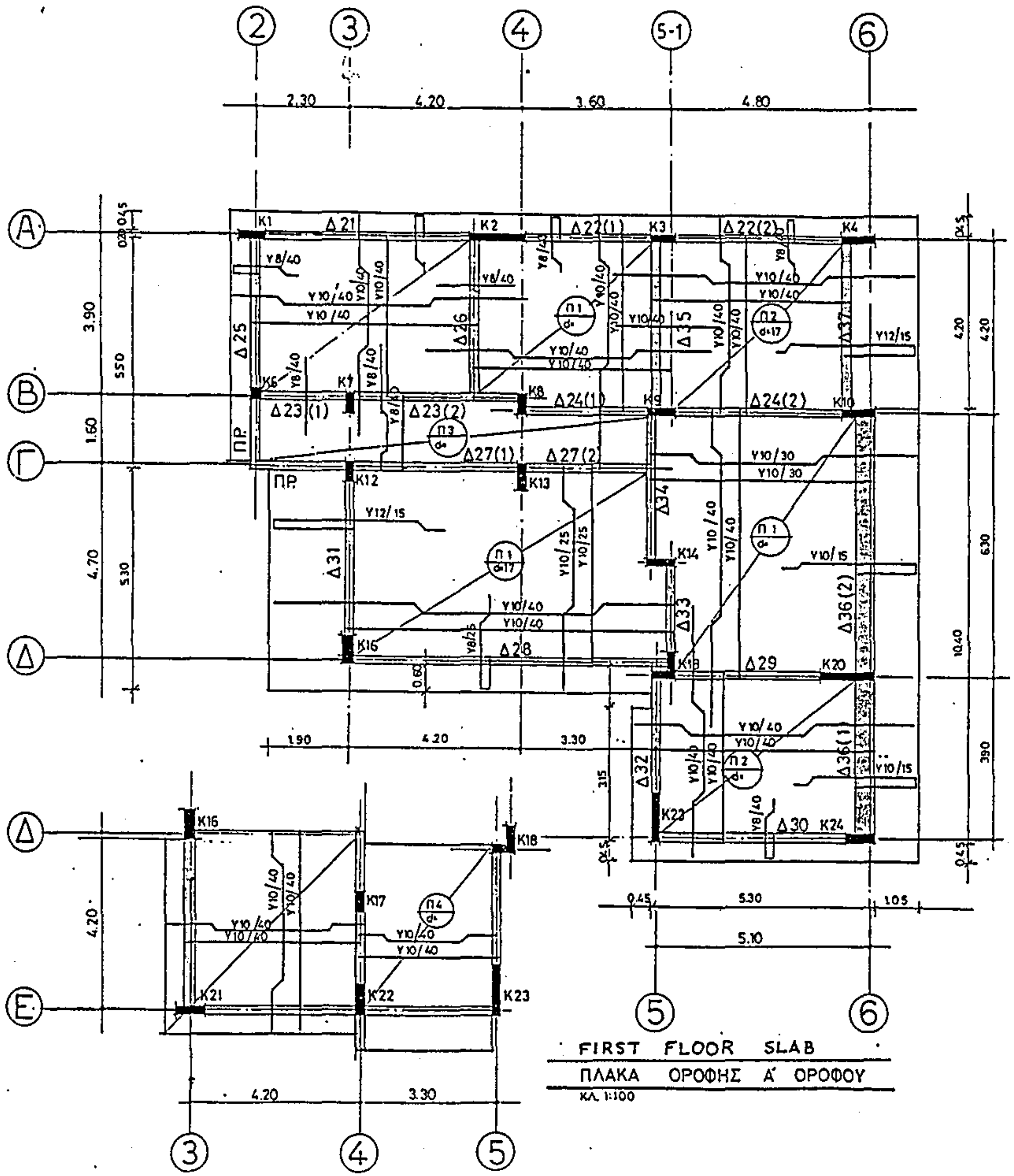
GROUND - BEAMS

DIMENSION / ΔΙΑΤΟΜΗ		REINFORCEMENT / ΟΠΛΙΣΜΟΣ TOP / ΑΝΩ	BOTTOM / ΚΑΤΩ	MIDDLE / ΜΕΣΟ	LINKS / ΣΥΝΔ
Σ1	20 X 100	3 Y 14	3 Y 14	2 Y 10	Φ 8/20
Σ2	20 X 80	3 Y 14	3 Y 14	2 Y 10	-//-
Σ3	20 X 50	3 Y 14	3 Y 14	——	-//-
Σ4	20 X 60	2 Y 12	2 Y 12	2 Y 10	-//-





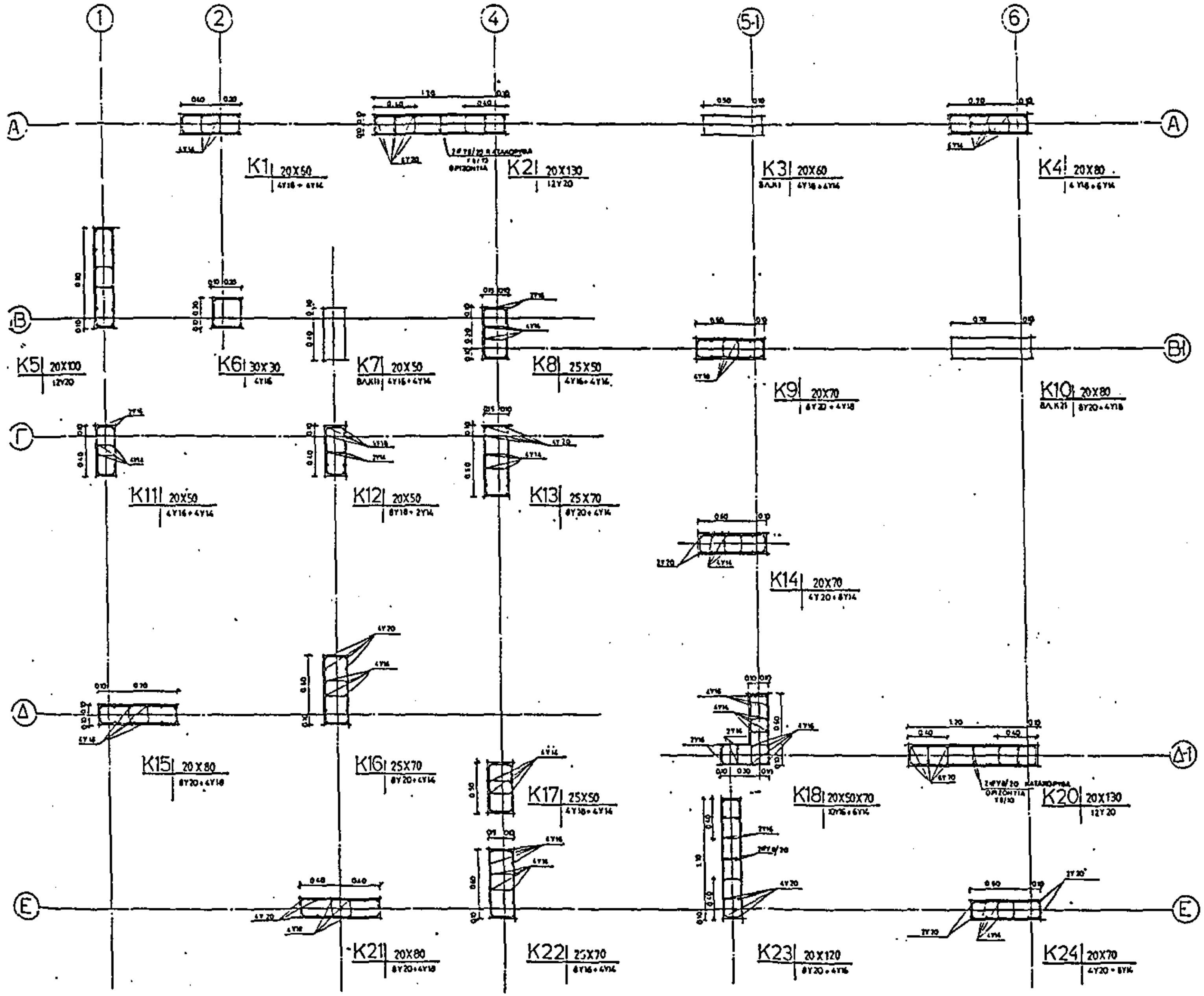
6.4(e) Drawings of the house built in 1991



BEAM	DIMENS.	REINFORCEMENT TOP	BOTT.	BAR	LINKS	SUPPORTS	
ΔΟΚΟΣ	ΔΙΑΤΟΜΗ	ΑΝΩ	ΚΑΤΩ	ΛΟΞΑ	ΣΥΝΔΕΤ.	ΕΔΡΑΣΕΙΣ	ΠΑΡΑΤ.
Δ21	20x60	2Y12	2Y12	2Y12	Φ8/20		
Δ22 (1)	20x60	2Y12	2Y12	2Y12			
Δ22 (2)	20x60	2Y12	2Y14	2Y14	Φ8/20		
Δ23 (1)	20x60	2Y12	2Y12	2Y12	Φ8/15		
Δ23 (2)	20x60	2Y12	2Y14	2Y14	Φ8/20		
Δ24 (1)	20x60	2Y12	2Y12	2Y12	Φ8/20		1Y10
Δ24 (2)	20x105	4Y12	4Y14	2Y14	Φ8/20		
ΠΡ.	20x60	2Y12			Φ8/10		
Δ25	20x60	2Y12	2Y12	2Y12	Φ8/20		
ΠΡ.	20x50	2Y12			Φ8/10		
Δ27 (1)	30x50	2Y12	2Y14	2Y14	Φ8/20		
Δ27 (2)	30x50	2Y12	2Y14	2Y14	Φ8/20		
Δ28	30x80	3Y14	3Y20	3Y20	Φ8/15		1Y10
Δ29	20x60	2Y12	2Y12	2Y12	Φ8/20		
Δ30	20x60	2Y12	2Y14	2Y14	Φ8/20		
Δ31 ΑΝΩ	20x70	2Y12	2Y16	2Y16	Φ8/20		1Y10
Δ32	20x60	2Y12	2Y12	2Y12	Φ8/20		
Δ33	20x70	2Y12	2Y12	2Y12	Φ8/15		
Δ34	20x70	2Y12	2Y16	2Y16	Φ8/15		1Y10
Δ35	20x60	2Y12	2Y14	2Y14	Φ8/20		
Δ36 (1)	40x65	2Y12	2Y16	2Y16	Φ8/20		
Δ36 (2)	40x65	2Y12	2Y20	2Y20	Φ8/20		
Δ37	40x65	2Y12	2Y16	2Y16	Φ8/20		

ΥΛΙΚΑ ΚΑΤΑΣΚΕΥΗΣ
- ΣΚΥΡΟΔΕΜΑ B 225
- ΟΠΛΙΣΜΟΣ Y STIII (ΣΤΡΙΦΤΟ)
  Φ STI (ΜΑΛΑΚΟ)

6.4 (f) Drawings of the house built in 1991



6.4 (g) Drawings of the house built in 1991

TITLE	NAME
A house built in 1991	D. L.
	PAGE 1 OF 3
	DATE April 92

ANALYSIS OF BEAM Δ15

LOADING

Floor Slab = $0.17 \times 2.4 \times g$ = 4.0

Beam = $\frac{1}{3.7}(2.4 \times 0.25 \times 0.6 \times g)$ = 1.0

Floors = = 1.0

Walls = = 0.9

Finishes = = 0.6

7.5 $KN\,m^{-2}$

KN/m^2

$G_K = 7.5$

$Q_K = 2.0$

SEISMIC DESIGN LOAD

$$S_d = S(G_K + E + \psi Q)$$

$$E = G_K + \psi Q$$
$$= 8.0$$

$\psi = 0.25$ (Table 4.1.4)

EQUIVALENT STATIC ANALYSIS

$$F_i = C_d \cdot \gamma_i \cdot W_i$$

$A_{max} = 0.10$
$S = 1.25$
$K = 3.00$
$I = 1.0$
$\alpha = 2.5$

Nicosia
Class II
DL II
Class III
Cl. 6.4.1

$$W_i = E \times \text{Floor Area}$$
$$= 8 \times 201$$
$$= 1608 \text{ (Top floor)}$$

$$= 8 \times 282$$
$$= 2256 \text{ (First floor)}$$

$$C_d = I \cdot A_{max} \cdot S \cdot \alpha \cdot \frac{1}{K}$$
$$= 0.1$$

$$\gamma_i = \frac{\Sigma W_i}{\Sigma W_i \cdot h_i} \cdot h_i$$
$$= 0.228$$

$\therefore$ F_i = 162 First floor
= 231 Top floor

Loads on each frame : F = 40 KN First Floor
= 61 KN Top Floor

TITLE	NAME
	PAGE 2 OF 3
	DATE

ANALYSIS OF THE FRAME

The frame, including Δ15 beam, was analysed using a computer programme and the results are attached.

DESIGN OF BEAM Δ15

$$\frac{1.4}{410} < p < \frac{7}{410} \qquad p = \frac{A_s}{A_c}$$

$$\therefore \quad 480 < A_s < 2400$$

At K17: $K = \frac{M}{bd^2 \cdot f_{cu}}$ $\qquad M = 74\ KNm$

$= 0.075$ $\qquad z = 0.93d$

$$A_s = \frac{M}{0.87 \cdot f_y \cdot z}$$

$$= \underline{\underline{546}}\ mm^2$$

At K13: $K = 0.122$ $\qquad M = 152\ KNm$

$A_s = \frac{M}{0.87 \cdot f_y \cdot z}$ $\qquad z = 0.90\ d$

$$= \underline{\underline{975}}\ mm^2$$

Midspan Δ15(2): $K = 0.05$ $\qquad M = 62\ KNm$, $z = 0.95d$

$A_s = 375\ mm^2$

$\Rightarrow \underline{\underline{480}}\ mm^2$

DESIGN OF COLUMNS K13, K17

$$l_e = \beta l_o$$
$$= 1.2 \times 3.15$$

$$l_e/b = \frac{3.15}{0.2}$$

$$= 18.9 > 10 \quad \therefore \underline{\underline{\text{Slender}}}$$

$\beta_\alpha = 0{\cdot}18$, $K = 1$, $h = \ldots$

$\therefore\ \alpha_u = \beta_\alpha \cdot K \cdot h$
$= 0{\cdot}18\, h \quad \Rightarrow$

K22 : 126 mm
K17 : 90 mm
K13 : 126 mm
K 8 : 90 mm
K 2 : 36 mm

$M_{add} = \alpha_u \cdot N$

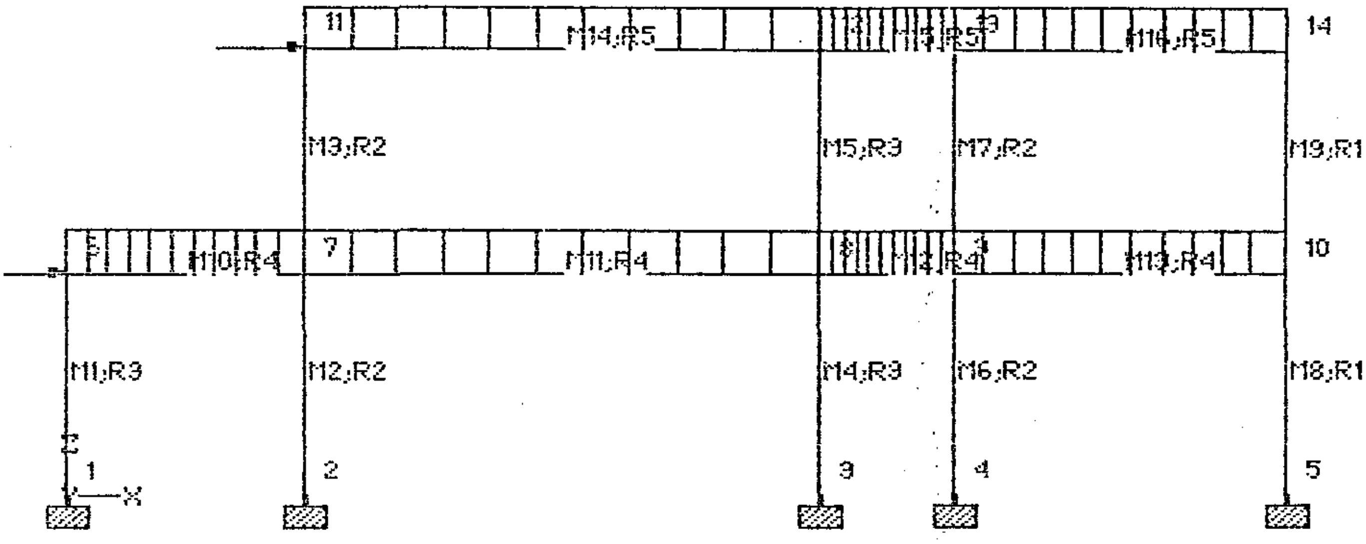
$1{\cdot}0\% < \rho < 4{\cdot}0\%$

Using chart 29 (BS 8110) and the limitations above the following results were obtained:

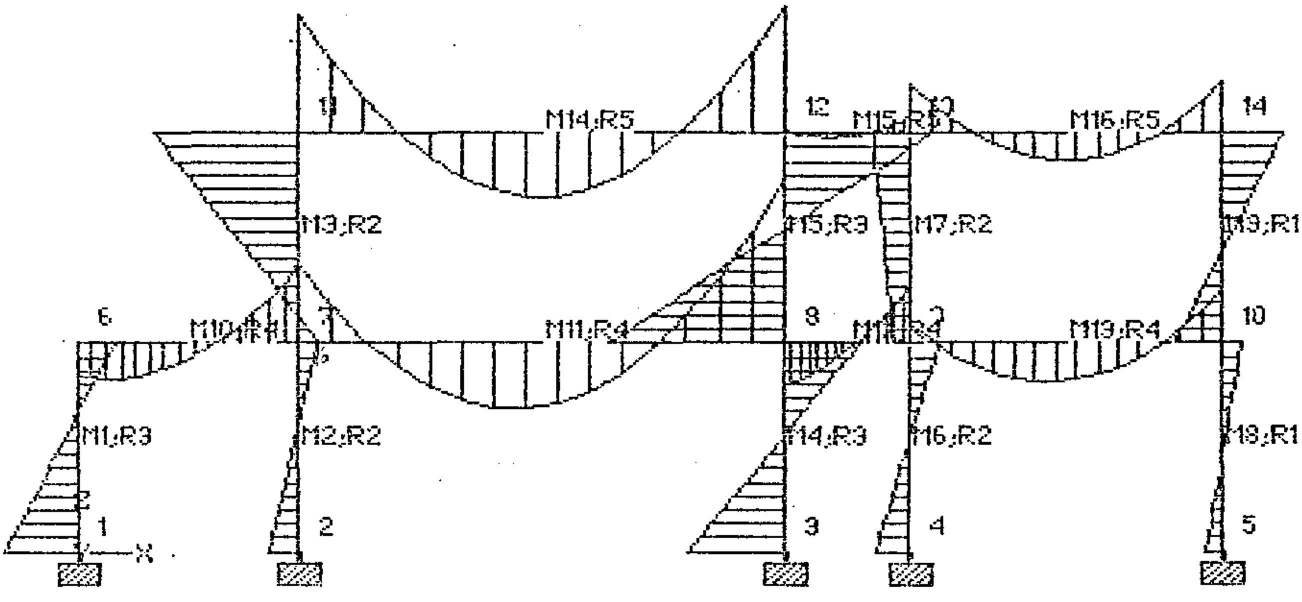
Column		N(KN)	N/bh	M(KNm)	M/bh^2	ρ(%)	Asc (mm^2)
K22	Gr. Floor	75	0·5	67	1·1	1	1750
K17	Gr. Floor	175	1·4	38	0·3	1	1750
	Fst. Floor	350	2·8	143	1·2	1	1750
K13	Gr. Floor	125	0·9	89	1·5	1	1750
	Fst. Floor	250	1·8	168	2·7	1·5	2650
K 8	Gr. Floor	105	1·1	37	0·3	1	1450
	Fst. Floor	210	2·2	48	0·4	1	1450
K 2	Gr. Floor	80	0·4	17	0·3	1	2600
	Fst. Floor	160	0·8	53	1·1	1	2600

NOTE: Problems appear in columns,

- K17 (existing reinforcement is 1620 mm^2)
- K13 (existing reinforcement is 2500 mm^2)



Bending Moment Diagram.



M14;R5
M15;R5
M16;R5
M3;R2
M15;R3
M7;R2
M9;R1
M10;R1
M11;R4
M12;R4
M13;R4
M1;R3
M2;R2
M4;R3
M6;R2
M8;R1
1
2
3
4
5
6
7
8
9
10
11
12
13
14
X

Nodes

No.	X(m)	Z(m)	Supports
1	0.000	0.000	FB
2	2.800	0.000	FB
3	8.900	0.000	FB
4	10.500	0.000	FB
5	14.400	0.000	FB
6	0.000	3.150	NULL
7	2.800	3.150	NULL
8	8.900	3.150	NULL
9	10.500	3.150	NULL
10	14.400	3.150	NULL
11	2.800	6.300	NULL
12	8.900	6.300	NULL
13	10.500	6.300	NULL
14	14.400	6.300	NULL

Members

No.	Node A	Node B	Ref	Pins- A/ y	; B/ y	Length
1	1	6	3	f	f	3.150m
2	2	7	2	f	f	3.150m
3	7	11	2	f	f	3.150m
4	3	8	3	f	f	3.150m
5	8	12	3	f	f	3.150m
6	4	9	2	f	f	3.150m
7	9	13	2	f	f	3.150m
8	5	10	1	f	f	3.150m
9	10	14	1	f	f	3.150m
10	6	7	4	f	f	2.800m
11	7	8	4	f	f	6.100m
12	8	9	4	f	f	1.600m
13	9	10	4	f	f	3.900m
14	11	12	5	f	f	6.100m
15	12	13	5	f	f	1.600m
16	13	14	5	f	f	3.900m

Properties

No.	A $\times 10^3 mm^2$	I $\times 10^6 mm^4$	E kN/mm^2
1	260.00	900.00	28.00
2	125.00	2600.00	28.00
3	175.00	7100.00	28.00
4	188.00	8800.00	28.00
5	34.00	100.00	28.00

Loadings

Node	Load No.	Type	Dir.	Mag(kN).
6	1	POINT	X	44.000
11	1	POINT	X	61.000

Member	Load No.	Type	Glob/Loc		Mag. A (kN/m)	Mag. B (kN/m)
10	1	DISTRIBUTED	GLOBAL	Z	-37.20	-37.20
11	1	DISTRIBUTED	GLOBAL	Z	-37.20	-37.20
12	1	DISTRIBUTED	GLOBAL	Z	-37.20	-37.20
13	1	DISTRIBUTED	GLOBAL	Z	-37.20	-37.20
14	1	DISTRIBUTED	GLOBAL	Z	-37.20	-37.20
15	1	DISTRIBUTED	GLOBAL	Z	-37.20	-37.20
16	1	DISTRIBUTED	GLOBAL	Z	-37.20	-37.20

Analysed Member Forces (Local Axes)

No.	Node	x-Force Axial (kN)	z-Force Shear (kN)	y-Force Shear (kN)	x-Moment Torsion (kNm)	z-Moment B.M. (kNm)	y-Moment B.M. (kNm)
1	1	15.558	27.527	-------	-------	-------	-56.552
	6	-15.558	-27.527	-------	-------	-------	-30.158
2	2	301.523	11.501	-------	-------	-------	-22.072
	7	-301.523	-11.501	-------	-------	-------	-14.156
3	7	112.310	-40.245	-------	-------	-------	15.828
	11	-112.310	40.245	-------	-------	-------	110.945
4	3	232.755	41.625	-------	-------	-------	-72.642
	8	-232.755	-41.625	-------	-------	-------	-58.478
5	8	136.565	79.491	-------	-------	-------	-135.460
	12	-136.565	-79.491	-------	-------	-------	-114.937
6	4	263.106	15.740	-------	-------	-------	-27.111
	9	-263.106	-15.740	-------	-------	-------	-22.469
7	9	109.604	-4.928	-------	-------	-------	-13.886
	13	-109.604	4.928	-------	-------	-------	29.407
8	5	154.259	8.607	-------	-------	-------	-12.769
	10	-154.259	-8.607	-------	-------	-------	-14.343
9	10	73.041	26.682	-------	-------	-------	-37.171
	14	-73.041	-26.682	-------	-------	-------	-46.877
10	6	16.473	15.558	-------	-------	-------	30.158
	7	-16.473	88.602	-------	-------	-------	72.105
11	7	-35.273	100.611	-------	-------	-------	-73.777
	8	35.273	126.309	-------	-------	-------	152.155

Analysed Member Forces (Local Axes)

No.	Node	x-Force Axial (kN)	z-Force Shear (kN)	y-Force Shear (kN)	x-Moment Torsion (kNm)	z-Moment B.M. (kNm)	y-Moment B.M. (kNm)
12	8	2.593	-30.119	....	....	....	41.783
	9	-2.593	89.639	....	....	....	54.025
13	9	-18.075	63.862	....	....	....	-17.670
	10	18.075	81.218	....	....	....	51.514
14	11	101.245	112.310	....	....	....	-110.945
	12	-101.245	114.610	....	....	....	117.963
15	12	21.754	21.955	....	....	....	-3.026
	13	-21.754	37.565	....	....	....	15.514
16	13	26.682	72.039	....	....	....	-44.923
	14	-26.682	73.041	....	....	....	46.877

Maximum Member Forces (Local Axes)

No.	Node	x-Force Axial (kN)	z-Force Shear (kN)	y-Force Shear (kN)	x-Moment Torsion (kNm)	z-Moment B.M. (kNm)	y-Moment B.M. (kNm)
2	2	301.523					
11	8		126.309				
11	8						152.155

Minimum Member Forces (Local Axes)

No.	Node	x-Force Axial (kN)	z-Force Shear (kN)	y-Force Shear (kN)	x-Moment Torsion (kNm)	z-Moment B.M. (kNm)	y-Moment B.M. (kNm)
12	8	2.593					
7	9		-4.928 ;				
15	12						-3.026

Reactions (Global Axes)

Node	X (kN)	Z (kN)	Y (kN)	X Moment (kNm)	Z Moment (kNm)	Y Moment (kNm)
1	-27.327	15.558	....	....	....	-56.552
2	-11.501	301.523	....	....	....	-22.072
3	-41.625	232.755	....	....	....	-72.642
4	-15.740	263.106	....	....	....	-27.111
5	-8.607	154.259	....	....	....	-12.769

b. The transverse reinforcement, increasing ductility, is not adequate in some beams. Critical zones are treated correctly in most of the beams but they are not recognised in the columns.

c. The concrete compressive strength, 20 N/mm^2 for the beams and 25 N/mm^2 for the columns is adequate for ductility level II.

d. Some beams and columns having widths 250 mm instead of 200 mm will achieve higher ductility.

Masonry – Openings

e. The infill walls, in most of the cases 200 mm thick have no reinforcement and are not separated from the frame.

f. A characteristic of the house is the number and the size of openings being large. Assuming that the walls may act as shear walls during an earthquake, then small lengths of the wall will have to resist the amount of strain that the whole wall should resist.

Building Configuration

g. The plan may be considered as irregular: In both floors re-entrant corners are exceeding the limit of 25%. As shown in the drawings at the ground floor the width of the corner is 43% of the width of the house and the length is 27% of the length of the house. At the first floor the two sides of the corner have dimensions of 49% and 29% of the external building dimensions.

h. The centre of mass and the centre of rigidity were found to be 2.6 metres apart in the ground floor and 2.9 apart in the top floor.

e. The elevation of the building also presents some irregularities. Setbacks at the top floor are greater than 20% of the plan dimensions of the ground floor.

Shear Walls

j. Only three columns have a length of more than 1.20 metres and can be considered as shear walls. They are not enough, however, and are badly positioned.

Using,

$$\frac{bd^2}{6} = \frac{H^2Fe}{5500}$$

where

$$b = 0.20 \text{ m}$$
$$H = 6 \text{ m}$$
$$F = 282 \text{ m}$$
$$e = 0.15$$

Then, $d^2 = 8.31$

Assuming four walls in each direction,

$$d^2 = \frac{8.31}{4}$$

$$= 1.50 \text{ m}$$

Non-Structural Elements

k. The large openings already mentioned give large areas of glass.

l. Heavy finishes (marble) were used on some external walls and on the chimney.

Detailing (clause 5.1 – Cyprus Code)

m. **Beams:**

Geometrical constraints: Minimum acceptable width was used. The ration b/h and l/h are generally satisfied with the exception of beams Δ6(2), Δ13(2), Δ17, Δ5(1), Δ5(2).

Longitudinal reinforcement: A comparison between the results obtained by seismic analysis and the existing amount of

reinforcement used in shown in figure 6.5 and 6.6.
Generally the right amount of reinforcement was used. Some problems appear in beam Δ6(2), which should be treated as critical along its entire length. Serious problems appear in beam Δ15(2) at the supports (columns K17 and K13).
Transverse reinforcement: Generally adequate.
Splicing is done within the critical zone

n. **Columns:**
Geometrical Constraints: The width of most of the columns being 200 mm instead of 250 mm is not in accordance with the seismic code and deprives the structure of a better ductility level. Ratio l/b is satisfied.
Longitudinal reinforcement: Generally in accordance with the code. Transverse reinforcement: No critical zones are treated according to the code.
Splicing is done within the critical zones.

6.3.4 STRENGTHENING

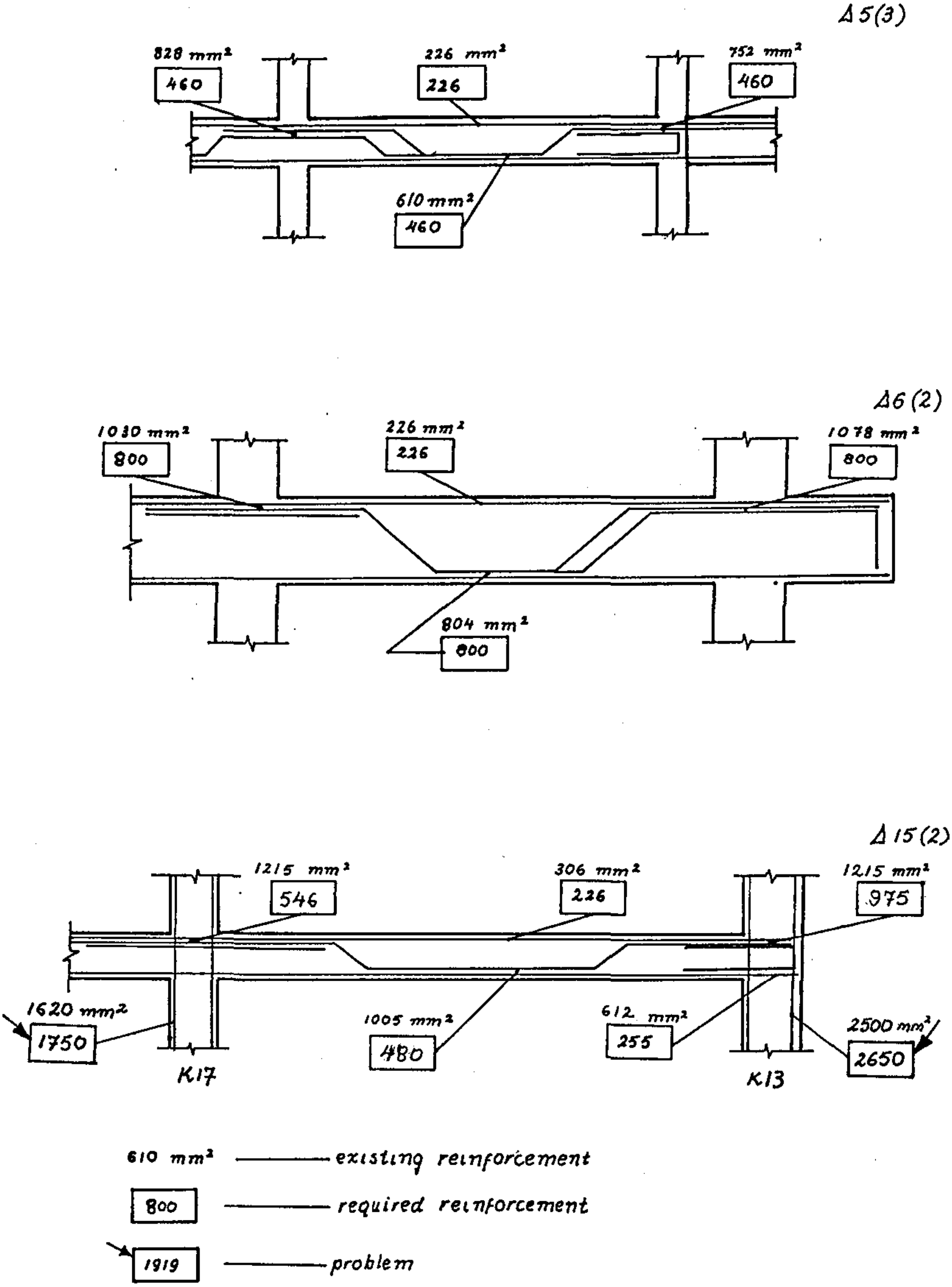
This case presents us with great difficulties due to its irregular configuration. Methods for improving the earthquake resistance are obstructed by the large rooms, the lack of walls and the non-continuity of the structural elements from the ground floor to the first floor. Nevertheless the following measures may increase the earthquake resistance.

a. Construction of shear walls, positioned in such a way to mollify the possible torsional effects during an earthquake.

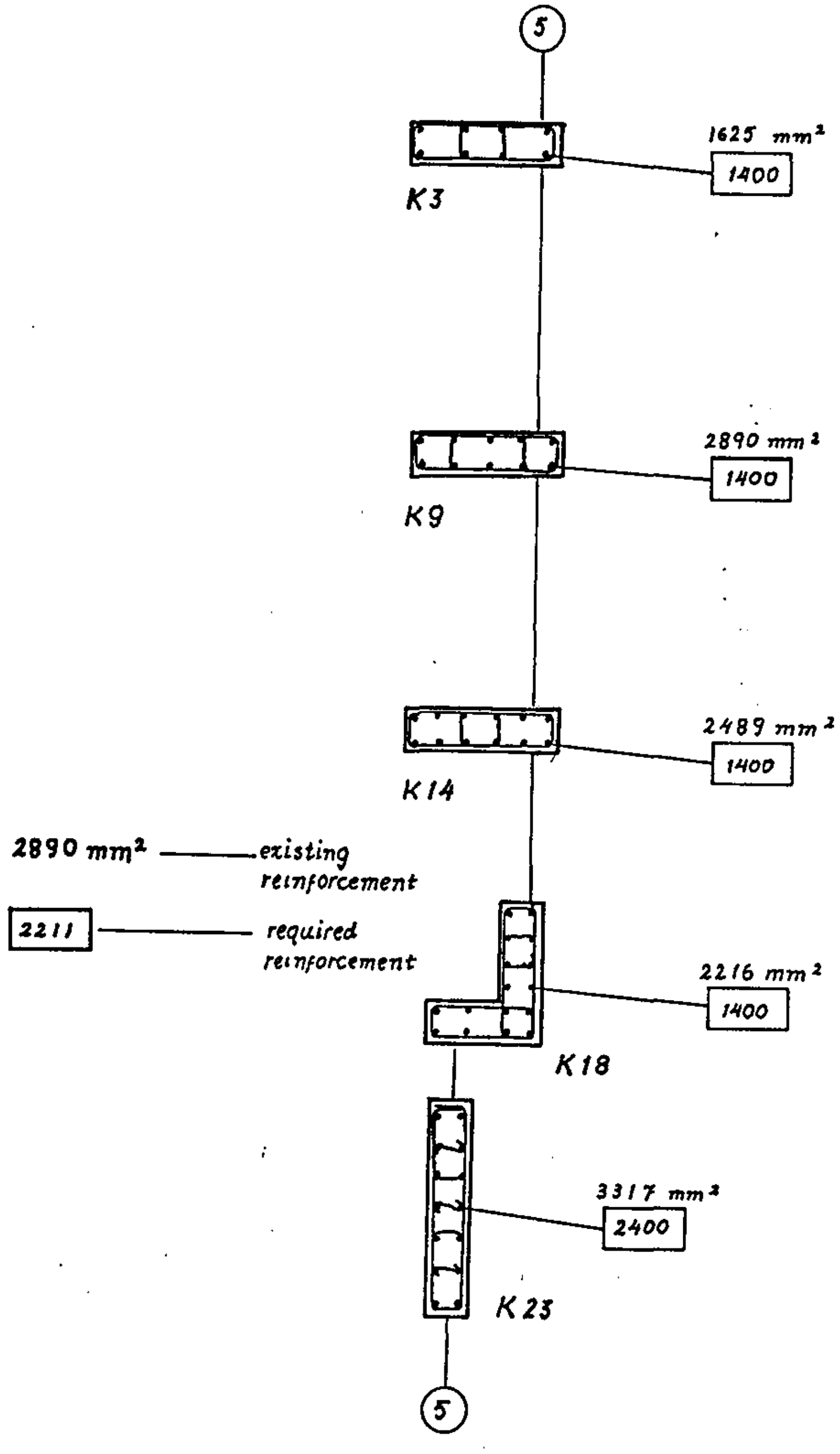
b. Strengthening of beam Δ15(2) and adjacent columns and beams.

c. Removal of heavy finishes at points where human safety is affected.

The measures above are discussed in chapter 5. Further to the methods suggested the following specifications are given:



6.5 Comparison of existing steel reinforcement to required reinforcement after analysis with the seismic loads



6.6 Comparison of existing steel reinforcement to required reinforcement after analysis with the seismic loads

6.3.4.1 Introducting of shear walls

- Introduction of four reinforced concrete walls in each direction. 200 mm thick and 1.50 metres long. Positioning as shown in figure 6.7 (a) and (b). Some extra walls were added to bring the centres of mass and rotation closer.

- Steel reinforcement minimum possible (within limits according to the code) since existing reinforcement is already adequate to hold the additional weight due to the strengthening procedures. Use Y12/200 vertically and horizontally. The reinforcement should be continuous from the foundation to the roof-slab. The horizontal bars (every second) should be anchored on the columns using an epoxy grout.

- Concrete mix to be used (achieving 30 - 35 N/mm^2):

Cement	325 kg	
Sand	850 kg	
10 mm Aggregates	720 kg	
Water	130 ltrs	
Polymer Latex	65 ltrs	(Yields 1 cubic metre)

- All faces should be rough to accept rendering.

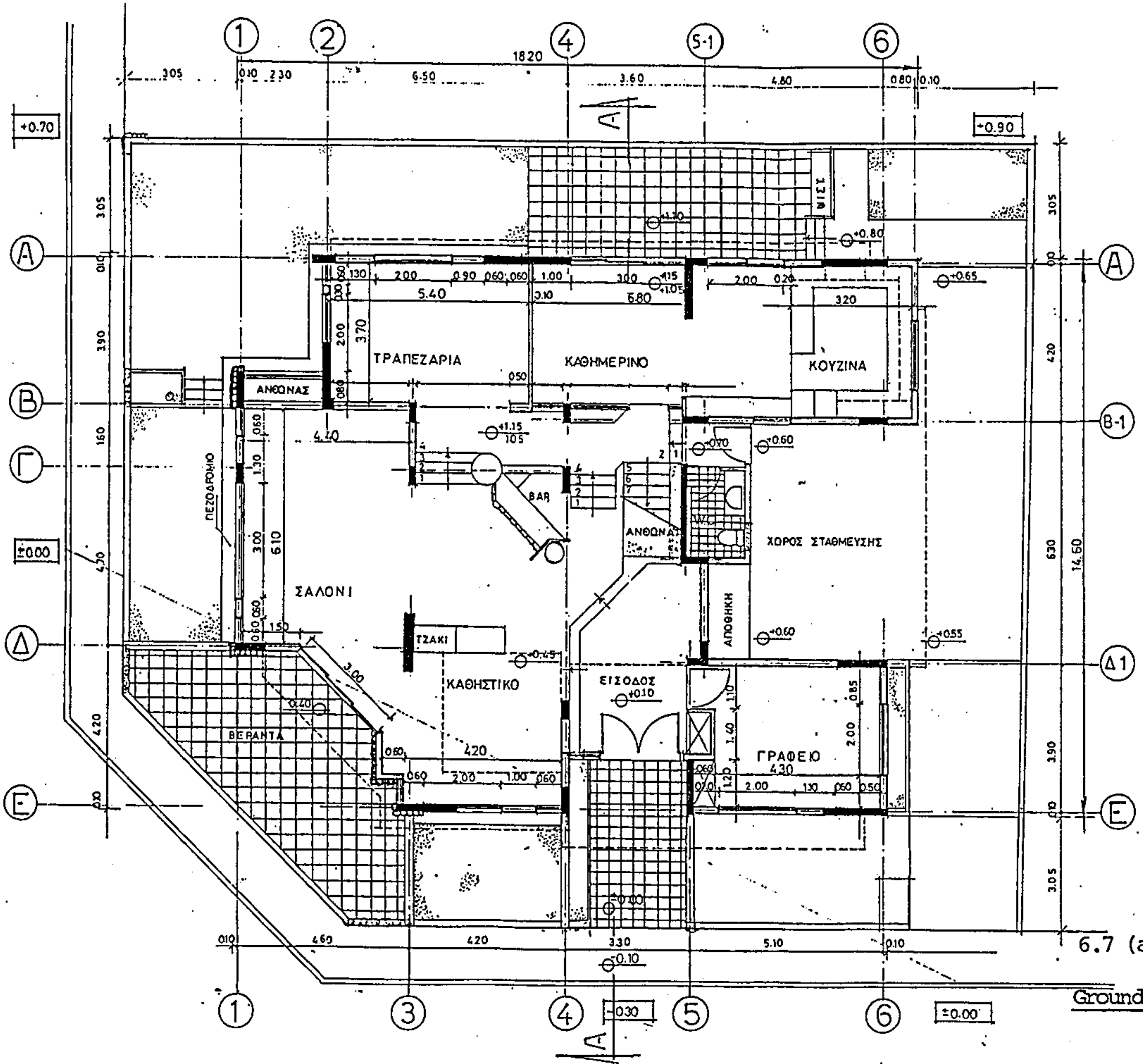
6.3.4.2 Strengthening of beam Δ15(2) (see figure 6.8)

- Necessary to strengthen Δ15(1) and columns K17, K13 and especially the joints.

- New dimensions:

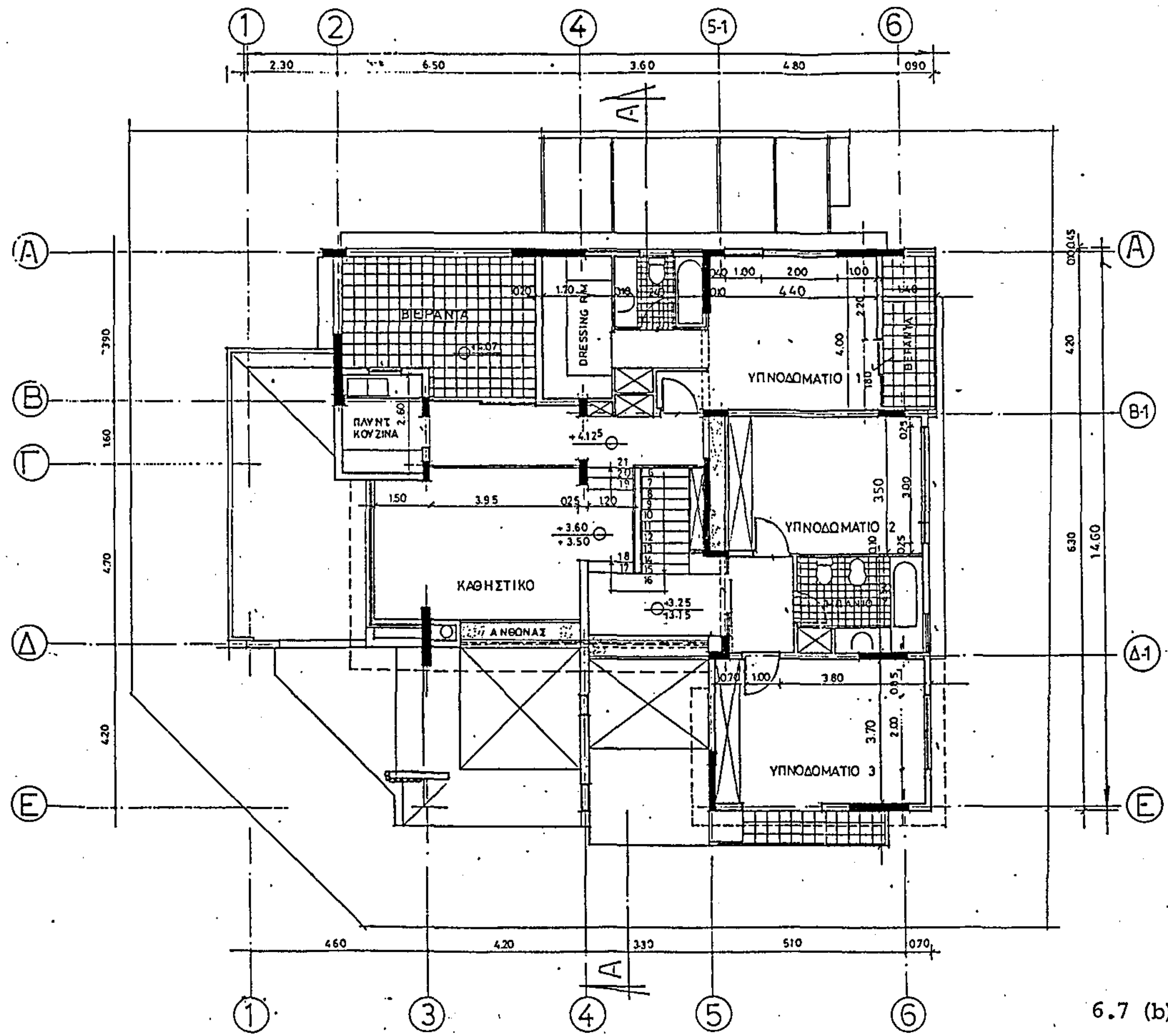
Beams:	32 x 60
Column K13:	32 x 80
Column K17:	32 x 60

- By increasing the dimension already some of the problems are solved.



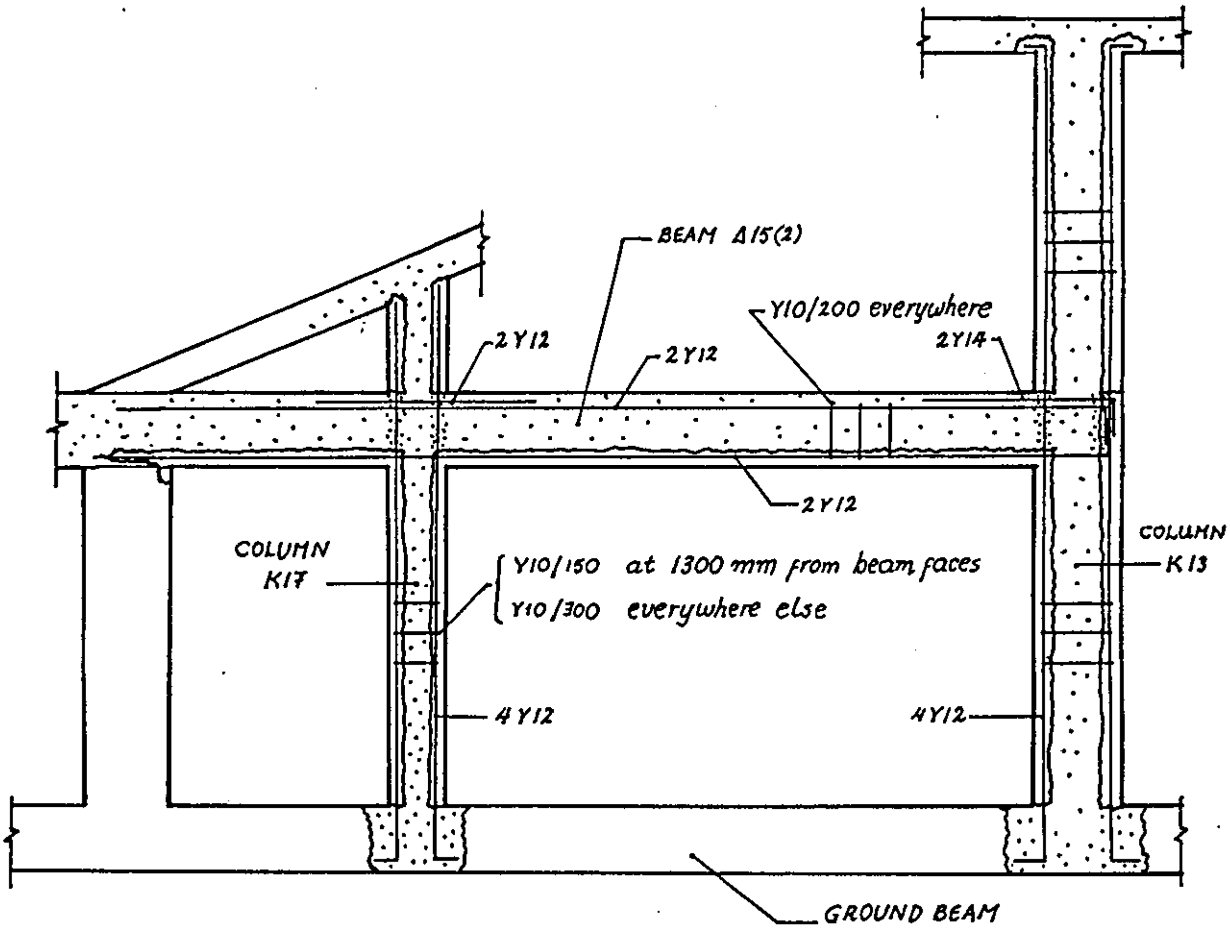
6.7 (a) Strengthening alterations

Ground Floor



6.7 (b) Strengthening alterations

First Floor



6.8 Strengthening of beam Δ 15(2) and adjacent beam and columns

- Additional reinforcement required:

Beams:	Top:	2 Y12
	Bottom:	2 Y12
	Links:	Y10/150 – 300
	At support K17:	2 Y12
	At support K13:	2 Y14 (on top)

Columns: 4 Y12
Y10/150 – 300 for links

- Concrete mix should be pourable achieving 30 N/mm^2:

Polymer grout	25 kg	
5 mm Aggregates	20 kg	
Water	5 ltrs	(Yield 25 ltrs)

- All faces should be rough and able to accept rendering.

6.3.4.3 Removal of heavy finishes

- Heavy finishes consist of marble and natural stone. They should be removed from the entrance and the staircases. Note that due to the previous measures – introduction of shear walls – most of the heavy finishes at the staircases will be removed.

- Render should be strengthened by an admixture – polymer latex – for better adhesion.

A complete cost analysis of the suggested measures follows:

The total cost of £7,374 Cyprus pounds is within acceptable limits. It is stressed, however, that despite the above measures, the aseismic behaviour of the building is still unpredictable. Such 'irregular' houses should be avoided.

COST ANALYSIS

No.	Item description	Unit	Quantity	Rate	Amount £	c.
A	SHEAR WALLS					
A1	Demolish part of brickwork to be replaced by concrete	m^2	27	8.-	202	
A2	Cut back to reinforcemnt, roughen edges of columns and beams sides in contact with new walls	m^2	10	15.-	150	
A3	Drill through beams and ground beams for steel bars to pass	m	16	15.-	210	
A4	Steel reinforcement - High yield steel bars to BS 4449: Y12/200 #	kg	540	0.40	216	
A 5	Drill columns 100 mm deep at 400 mm intervals and anchor horizontal bars using an epoxy anchor grout	cm^3	4800	0.40	1920	
A6	Erect watertight formwork - 'letter box' type - rogh face	m^2	58	3.50	203	
A7	Place concrete (as specified	m^3	6	150.-	900	
A8	Repair damages				300	
A9	Render all affected areas adding propylene fibres in the mortar	m^2	58	6.-	348	
A10	Paint	m^2	60	5.-	300	
B	STRENGTHENING OF BEAM Δ15(2)					
B1	Remove plaster, cut back to reinforcement of beams Δ15(2), Δ15(1) and columns K13,K17	m^2	36	12.-	432	
B2	Drill through beams for new links to pass	m	9.5	13.-	114	
B3	Drill through slabs and ground floor for column reinforcement to pass	m^2	1.0	30.-	30	
B4	Steel reinforcement (columns and beams) - High yield steel bars to BS 4449:					
	Y12	kg	90	0.4	36	
	Y14	kg	5	0.4	2.	
	Y10	kg	100	0.4	40	

COST ANALYSIS

No.	Item description	Unit	Quantity	Rate	Amount £	c.
B5	Erect watertight fromwork, rough-face	m^2	38	3.5	133	
B6	Place concrete (as specified)	m^3	2	400.-	800	
B7	Repair damages				200	
B8	Render with propylene fibres added to the mortar	m^2	38	6.-	228	
B9	Paint affected areas	m^2	38	5.-	190	
C	REMOVE HEAVY FINISHES					
C1	Remove finishes (marble - natural stone)	m^2	18	15.-	270	
C2	Render with mortar reinforced by latex	m^2	12	5.-	60	
C3	Paint	m^2	12	5.-	60	
					7,374	

6.4 Conclusions

In this chapter two cases were deliberately selected and assessed as far as their earthquake resistance is concerned. The first case, a house built in 1984, was adequately designed to resist normal loads but no aseismic design was employed, since the concept of such design was not known at the time.

The second case, a house built in 1991, was again adequately designed for normal loads and some aseismic ideas were employed, since the owner asked for an aseismic design. In 1991, however, a seismic code did not exist.

The procedure involved was demonstrated:

a. Analysis under seismic loads
b. Design of the reinforcement and comparison with the existing reinforcement
c. Inspection of the architectural layout
d. Identification of deficient areas
e. Design of specific solutions
f. Analysis of the cost of the strengthening measures

Having analyzed the structures and assessed the architecture of each building, it was found that both buildings have a number of deficiencies, which makes them inadequate for seismic load.

The 1984 house has the greater number of problems, whereas the 1991 house still has serious layout problems. In both cases the column design appears to indicate that only the minimum area of steel (1%) is required and that the existing design is adequate. The main problem areas appear to be in the beam hogging steel at the column connections.

Though the 1984 house has more problems, it has more chances to be strengthened adequately and to be brought into an almost aseismic structure, thanks to its symmetrical plan. In comparison with the 1991 house, it seems impossible to reach such a stage and the suggested strengthening steps will only improve partially the resistance of the structure, due to the irregular layout.

*

The estimated strengthening cost were,

Case 1 (1984) CP 11,681 on house valued at 60,000
Case 2 (1991) CP 7,374 on house valued at 80,000

These are relatively high figures (10-15% of the capital cost), and it will be rather difficult to convince the owners to proceed. Nevertheless they are within reasonable limits and if a strong campaign is organised, more and more people will realise the necessity of the strengthening works. Hopefully, they will not have to be persuaded by a destructive earthquake.

* The two cases studied demonstrate that the main problem with the existing structures is not so much the steel reinforcement content although the lower limit seems to govern. More important is that they do not comply with some of the rules of the aseismic design included in the seismic code, mainly:

  a. No special detailing of the joints.

  b. Incorrect positioning of shear walls creating torsional problems.

  c. Non-desirable stiffness variations due to soft-storeys and stub columns.

  d. Non-symmetrical plans and elevations.

7. CONCLUSIONS

The objectivesof this thesis, presented in a form of a manual of seismic design practice, were to show the need for strengthening existing buildings in Cyprus and to supply engineers with the necessary information so that they can proceed with the necessary measures required for strengthening.

Going through the existing historic documents and the geological facts, it was demonstrated (in chapter 2) that the seismic risk is high. Although there has been no recent major seismic event, Cyprus has been shaken many times in the past by destructive earthquakes. The creation of the island itself was actually due to seismic activity. Most of the seismic activity is concentrated on the southern coasts where, unfortunately, problematic soils exist such as soft clay and saturated sand.

Before going into the detailed methodology for strengthening existing structures a good knowledge of the main requirements for the seismic design of reinforced concrete building is necessary. In chapter 3 the priciples of good aseismic design were presented and a number of desirable design features were identified:

a. The need for ductility to allow large deformations occur and absorb energy.
b. Buildings should be regular with their centres of mass and rigidity being close enough.
c. Stiffness should be distributed uniformly whereas "soft stories" should be avoided.
d. Beams should be designed to fail before columns to prevent total collapse of the building.
e. Beam-column joints should be adequately reinforced and strengthened zones should be provided in adjacent beams and columns to shift plastic hinge formation far enough away fromthe joint.
f. Shear walls greatly improve the earthquake resistance of buildings but must be positioned correctly.

g. Non-structural elements should be designed to resist seismic loads.

Having established the required design features, chapter 4 reviews building practice and identifies deficiencies with regard to seismic resistance. A substantial stock of reinforced concrete building is of irregular shape, standing on soft stories, invariably rely on un reinforced masonry and often include the unacceptable stub-columns. In many cases the absence of professional supervision allowed bad workmanship to assist.

Relating the requirement of good aseismic design, (chapter 3) with building practice (chapter 4) enables areas to be identified where strengthening of existing buildings is required. The development of polymers and their use for modifying cementitious or epoxy mortars have enhanced considerably the possibilities of strengthening reinforced concrete structures. These special materials have properties that render them ideal for concrete repairs and strengthening.

Strengthening techniques include:

a. Strengthening of reinforced concrete elements by improving ductility.
b. Strengthening of critical zones around a beam-column joint.
c. Improving existing shear walls or introducing new ones carefully positioned.
d. Eliminating the stub-columns by enlarging existing foundations.
e. Strengthening of unreinforced brickwork.
f. Improvement of large openings.
g. Prevention of non-structural damage.

A number of the same techniques may be used to repair damage after an earthquake. It is vital that any damaged structures are properly repaired otherwise they will be especially vulnerable to subsequent seismic events.

Chapter 5 identified strengthening techniques that could be applied to the existing stock. As one of the objectives of this thesis was to provide Cypriot engineers with a manual on strengthening of existing buildings to resist seismic loading then it is neccesary to demonstrate how to assess existing structures. Chapter 6 presents two case studies. The first was selected being constructed prior to 1984 when the concept of earthquake resistance was not known. The second case is of a house built recently when basic provisions of the aseismic design were applied. These case studies demonstrate the procedure involved:

a. Analysis under seismic loads.
b. Design of the reinforcement and comparison with the existing reinforcement.
c. Inspection of the architectural layout.
d. Identification of deficient areas.
e. Design of specific solutions.
f. Analysis of the cost of the strengthening measures.

Following the above procedure, it was found that both buildings have number of deficiencies, no matter if built prior to 1984 or after. Therefore all structures designed without following the new seismic code should be assessed as far as their earthquake resistance is concerned. The case studies also demonstrated the importance of symmetrical layout. Strengthening is greatly aided by the symmetry of the structure and therefore the result is of a higher standard.

The improvement of a structure's earthquake resistance and its strengthening is feasible. The suggested methods and the materials are already available on the Cyprus market. Costs in the order of 10-15% of the capital cost of the building involved are within reasonable limits. However it is still not easy to persuade the owners that such work is essential. Ironically, it appears that only a destructive earthquake will achieve this.

Nevertheless the Cypriot Engineers should continue their efforts persuading others that the existing structures are inadequate to sustain the probable seismic load likely to occur in the next few years. Strengthening techniques can be identified that if carried out would reduce the likelihood of catastrophic collapse.

APPENDIX I General information about earthquakes[21,35,54,65]

The entire surface of the Earth is composed of a series of rigid and undeformable plates. Seven major plates cover most of the Earth's surface (as shown in figure 2.3 in chapter 2), namely the Pacific Plate, the North and South American Plate, the Nazca, the African Plate, the Indian Plate and the Eurasian Plate. Many other smaller plates exist in addition to the major ones. The plates are continuously in motion, and sometimes collide with each other along a destructive plate margin. Due to these collisions island arcs, such as Cyprus may be form on the surface. More than that, the collision creates excessive stresses built up. Rocks suddenly fail and move and thus energy is released. This energy shakes the ground making it to 'quake' and this phenomenon is called an earthquake.

The centre of the cause of the earthquake is called the focus and it lies usually somewhere along the boundary of the two plates colliding. The position vertically above on the Earth's surface is called the epicentre.

To describe an earthquake two different terms are used: magnitude and intensity.

INTENSITY:

It is a measure of the amplitude of ground vibration at one locality. Although there are several scales measuring the intensity, the most common one is the Modified Mercalli Scale. This scale is actually used to assess the results or the damages in a certain area. It measures from 1 to 12 (in Latin numbers) and each number corresponds to a certain degree of damages observed during the earthquake as shown in Table 7.1 The advantage of such a descriptional scale is that even past earthquakes can be assessed.

SCALE	DESCRIPTION	MAXIMUM ACCELERATION OF SOIL (mm^{*}/s^{2})	RICHTER SCALE OF MAGNITUDE (M) (APPROXIMATELY)
I	Felt only by instruments	10	2
II	Felt by a few people at rest	10	3
III	Slight. Rattling of windows	25	
IV	Generally perceptible, Rocking of things in tall buildings	50	4
V	Rather strong. Shaking of hanging items. Trembling of furniture	100	5
VI	Strong. Cracking of plaster Small damages	250	
VII	Very strong. Considerable damages especially to poor construction	500	6
VIII	Destructive. Much damage to normal buildings - partial collapse. Over-turning of tanks, monuments and chimneys.	1000	
IX	Ruinous. Ground cracked	2500	7
X	Disastrous. Most of the buildings collapse.	5000	
XI	Few structures left standing. Flooding	7500	8
XII	Catastrophic - Total destruction	9800	9

TABLE 7.1 The Modified Mercalli Scale of earthquake intensity

MAGNITUDE:

It is a quantative measure of the size of an earthquake, that is, the energy generated at its focus. It is measured directly by instruments. The Richter Magnitude Scale is commonly used nowadays measuring from 0 to 9. An earthquake is considered to be a strong one if its magnitude is greater than 5 – 5.5 on the Richter Scale. Such earthquakes can be recorded on seismographs over the entire Earth. The results of an earthquake of a certain degree on the Richter Scale will vary from place to place according to the distance from the focus and the geology.

The two terms cannot be easily compared. Nevertheless, a rough comparison between the Modified Mercalli Scale and the Richter Scale can be made as also shown in Table 7.1.

The difference from one degree on the Richter Scale to the next one is enormous. This fact will become more clear looking at Table 7.2[54] where magnitude and energy released by some earthquakes are shown.

The seismic activity around the Earth is frequent. On average twenty earthquakes of magnitudes over than 4 degrees on the Richter scale occur every day! More statistical data are shown in Table 7.3[54].

PLACE OF EARTHQUAKE	MAGNITUDE M	ENERGY RELEASED ERG	RELATIVE ENERGY
Agadir 1960	5.75	10^{20}	1
Orleanwil 1954	6.75	$3x10^{21}$	30
Messina 1908	7.50	$4.5x10^{22}$	450
San Francisco 1906	8.20	$5x10^{23}$	5000
Tokyo 1923	8.30	$6.9x10^{23}$	6900
Assam 1950	8.60	$2x10^{24}$	20000
Lisbon 1755	$\sim$9.00	$7.9x10^{24}$	79000

TABLE 7.2 Magnitude and energy released

TYPE OF EARTHQUAKE	MAGNITUDE (M)	AVERAGE ANNUAL NUMBER
Catastrophic of global scale	> 8	1-2
Major regional	7-8	15-20
Major local	6-7	100-150
Medium	5-6	750-1000
Minor local	4-5	5000-7000

TABLE 7.3 Average annual number of earthquakes according to their magnitude

APPENDIX II: EXTRACTS FROM THE CYPRUS SEISMIC CODE FOR REINFORCED CONCRETE STRUCTURES

3. CRITERIA OF CALCULATIONS

3.1 DEFINITION

The criteria of the calculations consist of a set of procedures to be followed in order to satisfy the general requirements in chapter 2. These procedures include:

a. Study of the limit state of the structural behaviour and the control of this state.
b. Design of the structural elements according to the provisions of this Code.
c. The adaption of quality control procedures during construction.

3.1.1 Collapse mechanism

The provisions of this code have been developed on the choice that stuctures should resist earthquake actions by means of a stable, non linear, energy dissipating mechanism of response. This aim will be achieved by following the dimensioning rules of the various elements given in chapter 4.

3.1.3 Strength and Ductility

The critical regions, i.e. regions where most of the energy dissipation is expected, must be provided by a suitable balance of strength and ductility, to ensure safety and serviceability of the structure. Specific analytical provisions which take into account the influence of accumulative damage and degrading of mechanical properties are given in chapters 4 and 5.

3.1.4 Limit deformations

The amplitude of the structure's deformation under the seismic forces must be limited in accordance with clause 4.4.6.4 .

3.1.5 Global Ductility

The use of appropriate materials (chapter 4.1) as well as of detailing

3.2 RELIABILITY DIFFERENTIATION

Target reliabilities shall be established on the basis of the consequences of failure, cosidering both aspects of safety and serviceability.
Consequences of failure in which momentary and non momentary losses are included, depend principally on the use given to buildings, on their contents, and on the importance of their functions.
Five different to reliability levels are recognised for the structures.
According to their importance, the structures shall be classified as follows:

Class I: Buildings where collapse may have catastrophic consequences - like nuclear stations, stores of inflammable of toxic material, dams - or buildings with more than 15 floors or very important buildings.

Class II: Buildings with likely large number of occupants - cinemas, theatres, halls etc - or important communal indurstrial buildings with expensive equipment.

Class III: Houses, multistory buildings, restaurants, hotels and other buildings not included in classes I and II.

Class IV: Auxiliary buildings and farms.

Class V: Temporary structures where collapse will not create any danger to people.

The different reliability levels proper to each class shall be obtained by amplifying the design action with a factor I, called 'importance factor' given in table 3.2 .

Table 3.2:

CLASS	FACTOR I
I	not covered by this code
II	1.5
III	1.0
IV	0.5
V	seismic analysis not necessary

In addition or in alternative of the use of the factor I, checking of specific limit-states relevant to damage or loss of function can be required for certain types of buildings.

3.3 DUCTILITY LEVELS

The structural systems covered by this Code can be designed to possess different 'ductility' levels according to the following classification:

Ductility level I (DL I): This level is defined as that proper to structures proportioned in accordance to the usual code of reinforced concrete, with the few additional requierements included in chapter 5.

Ductility level II (DL II): For this level specific aseismic requirements should be adapted, to enable the structure to reach non-elastic limits of behaviour, under repeated reversed loading, avoiding brittle failure.

Ductility level III (DL III): For this level specific procedures should be adapted for estimating design loads and dimensions and the detailing of the elements to ensure the development of selected stable mechanisms able to dissipate significant amount of energy.

The greater the ductility level adapted in a structure the lower is the seismic action to be considered in the design. This is given and numerically by the 'behaviour factor' K. (chapter 4.1.3)

4.1.3 STRUCUTRE BEHAVIOUR FACOTRS

The values of the behaviour factor K, defining the intensity of the design action (ch. 6.4.4) as a fuction of the structural type and of the selected ductility level, are given in Table 4.1.3

Table 4.1.3

STRUCTURAL - SYSTEM	DUCTILITY LEVEL		
	DL I	DL II	DL III
frame	2	3.5	5
wall and combined	2	3	4

The values of K in table 4.1.3 for wall and combined structures apply if at least 50% of the lateral force in both directions is resisted by coupled walls. If this condition is not satisfied the K values shall be reduced by a factor of 0.7 .

Ductility level I is permitted only for Class III, IV, V structures in areas of moderate seismicity. Class II structures to be built in high seismicity areas shall be preferably designed for DL III. If appropriate, and for more safety, K values relative to DL II could also be used in this case.

4.1.4 Design Load Combination

The fundamental combination of load effects to be used for all the limit-states verifications is:

$$S_d = S(G + E + \psi Q)$$

where:

G. includes all the permanent load at their nominal value.

E: the design seismic action as defined in ch. 6.4.4

Q: includes all the imposed load at their nominal value whose duration of application is long enough for the probability of their joint occurred with earthquake action being not negligible.

ψ: factor defining the fraction of the imposed loads to be included in the seismic analysis calculations. Values for factor ψ are given in table 4.1.4

Table 4.1.4

TYPE OF STRUCTURE	FACTOR ϕ
Roofs	0.00
Houses, Multistory builings	0.25
Public halls, Hospitals, Schools	0.50
Stores, Factories	0.75
Water tanks	1.00

The evaluation of the seismic action shall be based on all the gravity loads appearing in formula 4.1.4

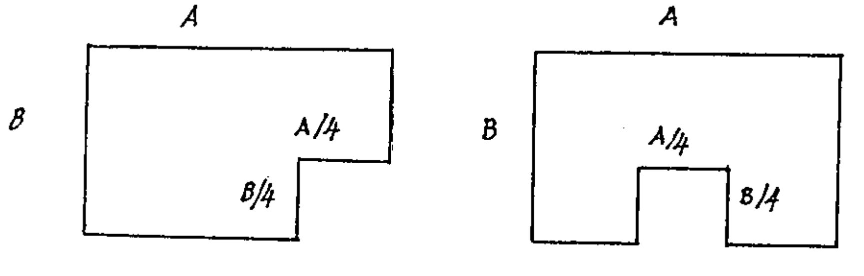
4.2 STRUCTURAL ANALYSIS

4.2.1 Suggested methods of structural analysis shall be different for buildings which, according to the definition given in this chapter, are classified as 'regular' or irregular'

Regular buildings can be designed according to the simplified method of analysis (indicated as equivalent static analysis) described in ch. 4.2.4, provided their height does not exceed 50 m and the fundamental period is shorter than 2 sec.
If these conditions are not satisfied or if the building is of irregular type, the dynamic method in ch. 4.2.5 shall be applied.
A building shall be classified as regular when the following conditions, regarding both plan and vertical configuration are satisfied.

4.2.1.1 Plan configuration

The building has an approximately symmetrical plan configuration with respect to at least two orthogonal directions along which the earthquake resisting elements are oriented. When re-entrant corners are present, they do not exceed 25 percent of the building external dimensions.



4.2.4 EQUIVALENT STATIC ANALYSIS

The 'equivalent static analysis' can be adopted for buildings classified as 'regular' according to 4.2.1, provided their height does not exceed 50 m, and the fundamental period is not greater than 2 secs. The limits given are in recognition of the importance of higher modes of vibration for long-peroid (generally taller) structures.

4.2.4.1 Horizontal Design Forces

The design lateral force to be applied at each floor level, in the direction being analyzed, shall be given by,

$$F = C_d \ \gamma_i \ W_i \qquad (4.2.4.1.1)$$

where:

C_d: design seismic coefficient, equal in value to the design response spectrum, as given in Ch. 6.4.4.

γ_i; distribution factor, depending on the height of the floor measured from the building.

W_i: total gravity load at floor i.

The fundamental period of the building, which is required for the evaluation of C_d, shall be calculated using the elastic properties of the structure by means of ordinary methods of mechanics, taking into account all the elements which can contribute to the building stiffness.
For frame structures an approximate expression of the fundamental period, based on analytical and experimental results, is:

$$T = N/12$$

where:

N is the number of storeys.

In many cases, a sufficiently accurate estimate of the period can be obtained with reference to an 'equivalent' uniform cantilever, whose period is given by the expression:

$$T = 1.8 \left(\frac{m}{E} \frac{h}{I} \right)^{1/2}$$

where:

m is the building mass per unit length

h is the heigt of the building from the foundation level.

E I is the flexural stiffness of the 'equivalent' cantilever.

In case the period is not calculate, C_d shall be taken as:

$$C_d = I \quad A \quad S \quad \hat{a} \quad \frac{1}{K}$$

The distribution factor γ_i is given by the expression:

$$\gamma_i = h_i \frac{\Sigma W_i}{\Sigma W_i h_i}$$

where:

h_i is the height of floor i from the foundation level.

5. DETAILING, EXECUTION, USE

When no explicit distinction is made, the provisions in this chapter apply to both DL II and DL III structures. Provisions applicable to DL I structures are always explicit .

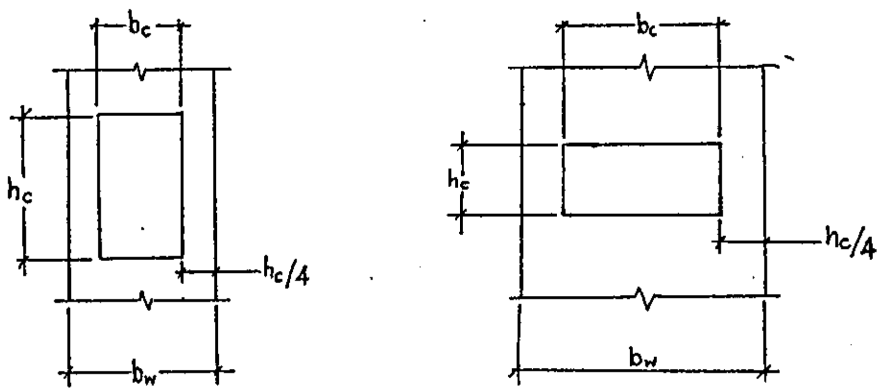
5.1 Elements

5.1.1 Geometrical Constraints

DL II and DL III structures

Unless special proofs for exemption are given, the following dimensional limitations shall be satisfied:

a. To ensure efficient transfer of moment from beam to column, the width of beam shall not be less than 200 mm or more than the width of the supporting column, plus length on each side of the beam not exceeding ¼ of the depth of the column.

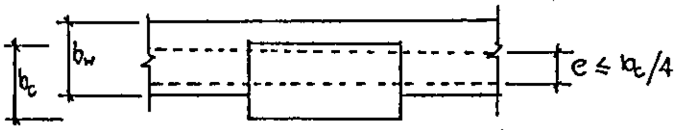


$$b_w \leq b_c + h_c/2 \leq 2b_c$$

b. To avoid any possible danger of transverse instability in the non linear range of response, the ratio b/h shall not be less than 4.

c. Behaviour of frame components having l/h ratio less than 4 is is substantially different from the overall behaviour of slender components. So the ratio l/h shall not be less than 4. (This does not apply to coupling beams in wall structures Cl 4.4.3.3)

d. The eccentricity of any beam relative to the columns into which it frames as measured by the distance between the geometrical centrelines of the two member, shall not exceed ¼ b_c.



5.1.2 Longitudinal Reinforcement

DL II and DL iii structures

a. At any section of the member the tensile reinforcement ratio for the top or the bottom reinforcement shall not be less than:

$$\rho_{min} = \frac{1.4}{f_{yk}} \qquad (f_{yk} \text{ in MPa}) \qquad (5.1.2.1)$$

and to ensure a sufficient ductility shall not be greater than:

$$\rho_{max} = \frac{7}{f_{yk}} \qquad (f_{yk} \text{ in MPa}) \qquad (5.1.2.2)$$

with ρ_{min} and ρ_{max} referred to the gross concrete area A_g.

b. At least two 12 mm bars shall be provided both top and bottom throughout the length of the members.

c. To ensure adequate ductility at potential plastic hinges and to provide a reasonable strength for the reverse action, the compression reinforcement ratio ρ shall not be less than one half of the tension reinforcement ratio at the same section

$$\rho \nless 0.5$$

d. At least one quarter of the larger of the reinforcement required at either end of the member shall be continued throughout its length.

e. In T and L beams built integrally with slabs, the effective reinforcement to be considered near column faces in addition to all longitudinal bars placed within the web width of the beam shall be as follows:

I. At interior columns when a transverse beam of similiar dimensions frames into the column, all reinforcement within that part of the slab which extends a distance 4 times the slab thickness from each side of the columns.

II. At interior columns where no transverse beam exists, all reinforcement within that part of the slab which extends a distance of 2.5 times the thickness of the slab from each side of the column.

III. At exterior columns where transverse beam of similar dimensions frames into the columns and where the beam reinforcement is to be anchored all reinforcement within that part of the slab which extends a distance of twice the slab thickness form each side of the columns.

IV. At exterior columns where no transverse beam exists, all reinforcement within the width of the column.

In all cases at least 75% of the reinforcement in each face providing the required flexural capacity, must pass through or be anchored in the column core.

DL I structures

Only clause 5.1.2 (a) needs to be satisfied.

5.1.3 Minimum Transverse Reinforcement

Transverse reinforcement as specified in this section shall be provided unless larger amount is required to resist shear (Sec. 4.4.1.4). The purposes of transverse reinforcement are:

a. to confine the concrete in order to increase its ultimate deformation and to increase bond strength
b. to restrain laterally the longitudinal bars so to prevent them from buckling
c. to provide shear resistance.

Portions of the beams to be considered as 'critical' regions are:

a. Twice the member depth, measured from the face of the supporting column, or beam, towards midspan at both ends of the beam.

b. Twice the member depth on both sides of a section where yielding may occur

c. Wherever compression reinforcement is required.

DL II structures

In the critical regions as defined above, stirrup-ties of not less than 8 mm diameter shall be provided, with maximum spacings not exceeding the smaller of

a. $h/4$

b. $8\ \Phi_l$ (Φ_l: diameter of the longitudinal bars)

c. $24\ \Phi_h$ (Φ_h: diameter of the stirrup-ties bars)

d. 200 mm

The first stirrup-tie shall be located not more distant than 50 mm form the face of the column.

At least one out of every two separate longitudinal bars included in the web width of the beam should be restrained by a 90^o bend of a stirrup-tie.

DL III structures

In the critical regions as defined above stirrup-ties of not less than 8 mm diameter shall be provided, with maximum spacings not exceeding the smaller of:

a. $h/4$

b. $6\ \Phi_l$

c. 150 mm

The minimum area of one leg of the transverse reinforcement shall be:

$$A_{s\ min} = \frac{\Sigma A_b\ f_{yk}}{16\ f_{ykt}}\ \frac{S}{100}$$

to prevent buckling of longutudinal bars subjected to severe reverse plastic deformations

ΣA_b = sum of the areas of longitudinal bars at the section considered to be restrained by the transverse leg

f_{yk} = yield strength of longitudinal bar

f_{ykt} = yield strength of stirrups

S = spacing of the stirrups in mm

The first stirrup-tie shall be located not more distant than 50 mm from the face of the supporting member.
At least one out of every two separate longitudinal bars included in the web width of the beam should be restrained by a 90^{o} bend of a stirrup-tie.

5.2 Elements Subject To Bending And Axial Force ($N_d > 0.1\ A_g\ f_{cd}$)

The purpose of the provisions in this clause, is to provide columns with a sufficient reserve of ductility which might prove essential should some deviation occur from the expected structure's response.
Observations of damages produced by earthquakes frequently show that corner columns are more vulnerable than interior ones, due to unanticipated torsional effects.
It is therefore recommended that corner columns be subjected to particular care on detailing, or even be made somewhat stronger than required by analysis.

5.2.1 Gemetrical Constaints

DL II structures

a. The minimum cross-section dimension shall not be less than 250 mm
b. The ratio L/b shall not exceed 25

DL III structures

a. The minimum cross-section dimension shall not be less than 300 mm
b. The ratio L/b shall not exceed the values of
   - 16 for columns having moments of opposite sign at the two extremities
   - 10 for centilever columns.

5.2.2 Longitudinal Reinforcement

The reinforcement ratio shall not be less than 1.0% nor greater than 6% including the region of lap splices.
For S 400 steel, the reinforcement ratio outside the splices shall not be greater than 4%.
When the section is confined for architectural purposes, the minimum reinforcement ratio may be reduced. Reinforcement calculated as less than 0.5% may be doubled. However, in no case should the reinforcement be less than 0.5%.

The bars shall not be spaced further apart between centres than 250 mm for DL II structures or 200 mm for DL III structures.

DL I structures

The provisions above must be satisfied also by DL I structures.

5.2.3 Transverse Reinforcement

A basic amount of reinforcement shall be provided all over the height of the columns, while special reinforcement shall be placed in the column critical regions, defined in the following Cl. 5.2.3.1.

The amount of reinforcement required by the present clause shall be provided unless a larger amount is required to resist shear according to Cl 4.4.1.4

5.2.3.1 Column Critical Regions

a. For usual cases, critical regions are considered to be the regions at each end of a column above and below connections over a length from the faces of the connection of not less than the larger of:
   - the longer column cross-section dimension
   - one-sixth of the clear height of the column
   - 450 mm
b. When a masonry infill wall is in contact with one or both of the two opposite sides of a column, over the whole height or part of it, the entire column height shall be considered as a critical region.
c. In case of columns with part of their height restrained due to a connection with a wall, the free part of the column shall be considered as a critical region.

Critical column regions require greater amount of closely spaced, well anchored transverse reinforcement than the remainder of the column, in order to provide confinement to concrete (hence adequate curvature ductility), lateral support for the longitudinal bars, and shear resistance.

5.2.3.2 DL II structures

Critical region

Special transverse reinforcement having a minimum diameter of 8 mm in the form of spiral or hoop reinforcement shall be provided.

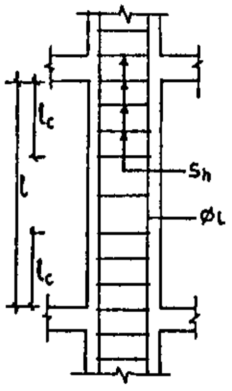
Cross-ties to restrain longitudinal bars not directly held by hoops shall not exceed the lesser of:

a. eight times the minimum diameter of the longitudinal bars

b. half the least cross-sectional dimension of the section

c. 200 mm

The tranverse reinforcement in the amount specified above shall be continued throughout the length of the beam-column joint.



DUCTILITY LEVEL II

critical region,	l_c = max (h, L/6, 450 mm)	
Spacing	critical region,	S_h = min(8 Φ_l, b/2, 200 mm)
	Elsewhere,	S_h = min(12Φ_l, b, 300 mm)

Non-critical regions

The minimum transverse reinforcement in non-critical regions shall be in accordance with the Code for Concrete.

5.2.3.3 DL III Structures

Critical regions

The volumetric ratio of transverse reinforcement (spiral of hoops) shall not be less than the greater of

$$\rho_s = \lambda_1 \frac{f_{ck}}{f_{yk}} \qquad (5.2.3.3.1)$$

or

$$\rho_s = \lambda_2 \left(\frac{A_g}{A_c} - 1 \right) \frac{f_{ck}}{f_{yk}} \qquad (5.2.3.3.2)$$

where A_g = gross sectional area

A_c = confined area of concrete

and the values of λ_1 and λ_2 are given in the following table.

N_d / A_c f_{ck}	0.10	0.20	0.30	0.40	0.50
λ_1	0.05	0.06	0.07	0.08	0.09
λ_2	0.18	0.22	0.26	0.30	0.34

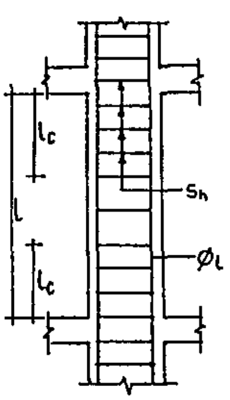
The volumetric ratio is the ratio of volume of spiral or hoop reinforcement to total volume of concrete core (out-to-out of bars) within spacing s_h.

The volumetric ratio: ρ_s for rectangular sections is defined as:

$$\rho_s = A_{sh} / S_h \; h' \qquad (5.2.3.3.3)$$

where A_{sh} is the total area of hoop bars and supplementary cross ties in each of the principal directions of the cross section, S_h is the spacing and the h' is the distance between centres of outer bars.

1. The minimum diameter of spiral or hoops shall be 8 mm
2. The maximum spacing between spirals of hoops shall not exceed the smaller of:
   a. six times the minimum diameter of the longitudinal bars
   b. one fourth of the smallest lateral dimension of the section
   c. 150 mm



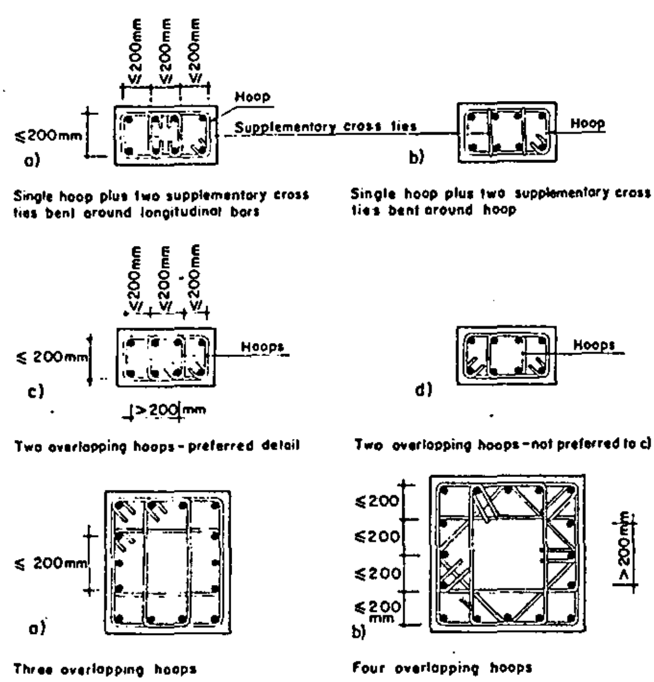
DUCTILITY LEVEL III

critical region,	l_c = max (h, 1/6, 450 mm)	
Spacing	critical region	S_h = min($6\Phi_1$, b/4, 150 mm)
	elsewhere,	S_h = min($8\Phi_1$, b/2, 200 mm)

3. Each longitudinal reinforcing bar of bundle of bars shall be laterally supported by the corner of a hoop, at least 135^{0} or by additional cross ties, except for:

a. bars of bundles of bars which lie between two bars supported by the same hoop where the distance between them does not exceed exceed 200 mm beween centres.

b. inner layers of bars with concrete core centred more than 75 mm from the inner face of hoops



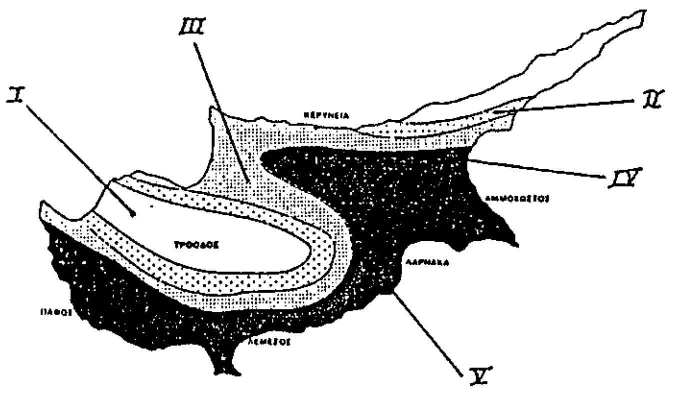
4. The yield force of the hoop bar or the additional tie shall be at least one-sixteenth of the yield force of the bars, it is to restrain including the contribution from the bars exempted under 3 (a) above.

5. Each end of an additional tie shall engage either a longitudinal bar or the peripheral hoop besides a longitudinal bar with a bend of at least 135^{0} and an extension beyond the bend of at least 10 tie bar diameters. Additional ties and legs of hoops shall not spaced transversely more than either 200 mm or one-quarter of the column section dimension perpendicular

6. SEISMIC ACTION

6.1 Regional Seismicity

The seismic activity in Cyprus is described by the seismic map of Cyprus (figure 6.1);



For design purposes the most suitable parameter is the maximum value for the soil accleleration, A_{max}.

6.2 Seismic Zones

For the purposes of this Code, Cyprus is divided into five zones according to the seismic intensities expected. For each zone, the design values for the maximum soil acceleration.A_{max}, are given in Table 6,2.

Table 6.2

Zone	A_{max}
I, II: III	0.075
IV	0.10
V	0.15

6.3 Characteristics of Seismic Actions

The seismic actions are a result of soil vibrations transformed to the structures during the earthquakes.

For the purposes of this Code the ground motions are described by:

a. the maximum soil acceleration A_{max}

b. the response spectrum for horizontal motion for stiff soils.

c. the response spectrum for vertical motion reduced to 2/3 of the respective response spectrum for the horizontal motion.

In regions where the geological evidence shows the possibility of 'short type' vibrations (where the response spectrum is not satisfactory) or where there is a large scàle and deep soil layering, the expected characteristics of the ground motion must be analysed more rigorously.

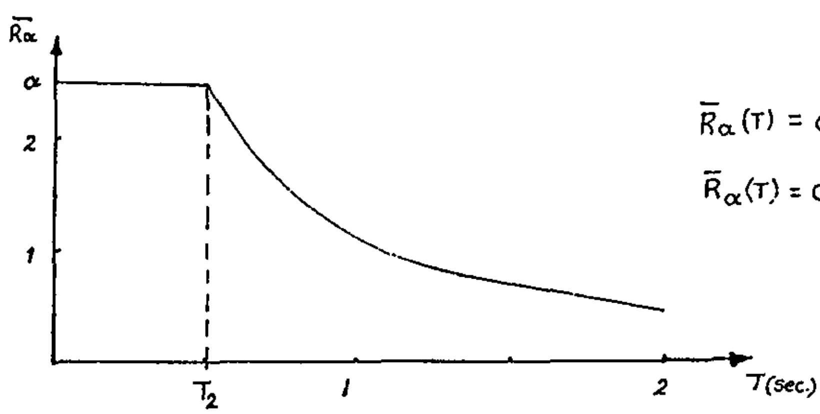
6.4 Design Seismic Action

The design seismic action is defined as the force, which when used in combination with other dead or imposed load to design structures according to these provisions and the provisions of the Code for Concrete satisfies the general requirements shown in Ch. 2, in relation to the determined safety level.

6.4.1 Elastic Response Spectrum

For the purposes of this Code, the shape of the 'standard' (rocky or stiff-stable soils) response spectrum is given in figure 6.4.1 . The spectrum is for 5% structural damping.

Different shapes for spectral strengthening may be adopted according to specific site records and/or geophysical evidence.



$\bar{R}_\alpha(T) = \alpha \quad$ for $T < T_2$

$\bar{R}_\alpha(T) = \alpha\,(T_2/T)^{\beta} \quad$ for $T > T_2$

$\alpha = 2.5$ $\beta = 1.0$

When there is no information

$T_2 = 0.40$

6.4.2 Site characteristics

When there is no detailed knowledge on the local soil conditions. procedure followed shall be as recommended in Clauses 6.4.2.1/2/3.

6.4.2.1 Soil classes

The soil classes are defined as below:
SOIL S 1: Rock of any characteristic, either schistolithic or crystalline, or stiff soil where the depth of its layer is less than 60 m and the soil overlying the rock is stable deposits of sand, gravel or stiff clay.
SOIL S 2: deep, cohesionless or stiff clayly soils where the depth of the soil is more than 60 m and the soil overlying the rock is stable deposits of sand, gravel or stiff clay.
SOIL S 3: soft to medium stiff clayly and sandy soils characterised by 10 m or more of soft to medium stiff clay with or without intervening layers of sand or other cohesionless soils.
In places where the soil characteristics are not known to classify the soil or where the soil is not matching with any of the three classes, class S 2 shall be used.
Additional in-situ or laboratory tests should be performed to check conditions where there is a possibility of:

a. dynamic instability and liquefaction of sand
b. significant settlement
c. rock falls
d. faults

It is recommended to avoid such sites, if possible.

6.4.2.2 Site factor S

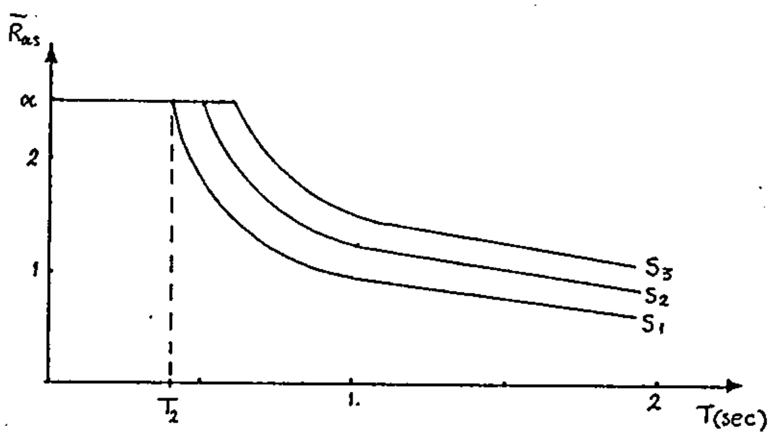
The site factor, S is employed to take in account the soil conditions of the site. Its values are given in the Table 6.4.2.2

Soil class	S 1	S 2	S 3
Site factor, S	1.0	1.2	1.5

Table 6.4.2.2

6.4.3 Elastic Site Response Spectra

The elastic response spectra is shown in figure 6.4.3.



If there is no specific infomation for the site, T_2, α and β may be taken as given in Cl 6.4.1.

Spectra for vertical vibrations may be defined quite accurately multiplying the coordinates of the spectra for horizontal vibrations by 2/3.

6.4.4 Design Response Spectra

The coordinates of the design response spectrum are given multiplying the coordinates of the site response spectrum by factor:

$$R_a = \overline{R}_{as} \cdot I \cdot \frac{1}{K} \cdot A_{max}$$

where:

- I: Importance factor given in Ch. 3.2
- K: Behaviour factor given in Table 4.1.3
- A_{max}: Maximum soil acceleration for the considered seismic zone given in Table 6.2

APPENDIX III
LIST OF REFERENCES

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5.7.1 Hair cracks (< 0.3 mm)[25]

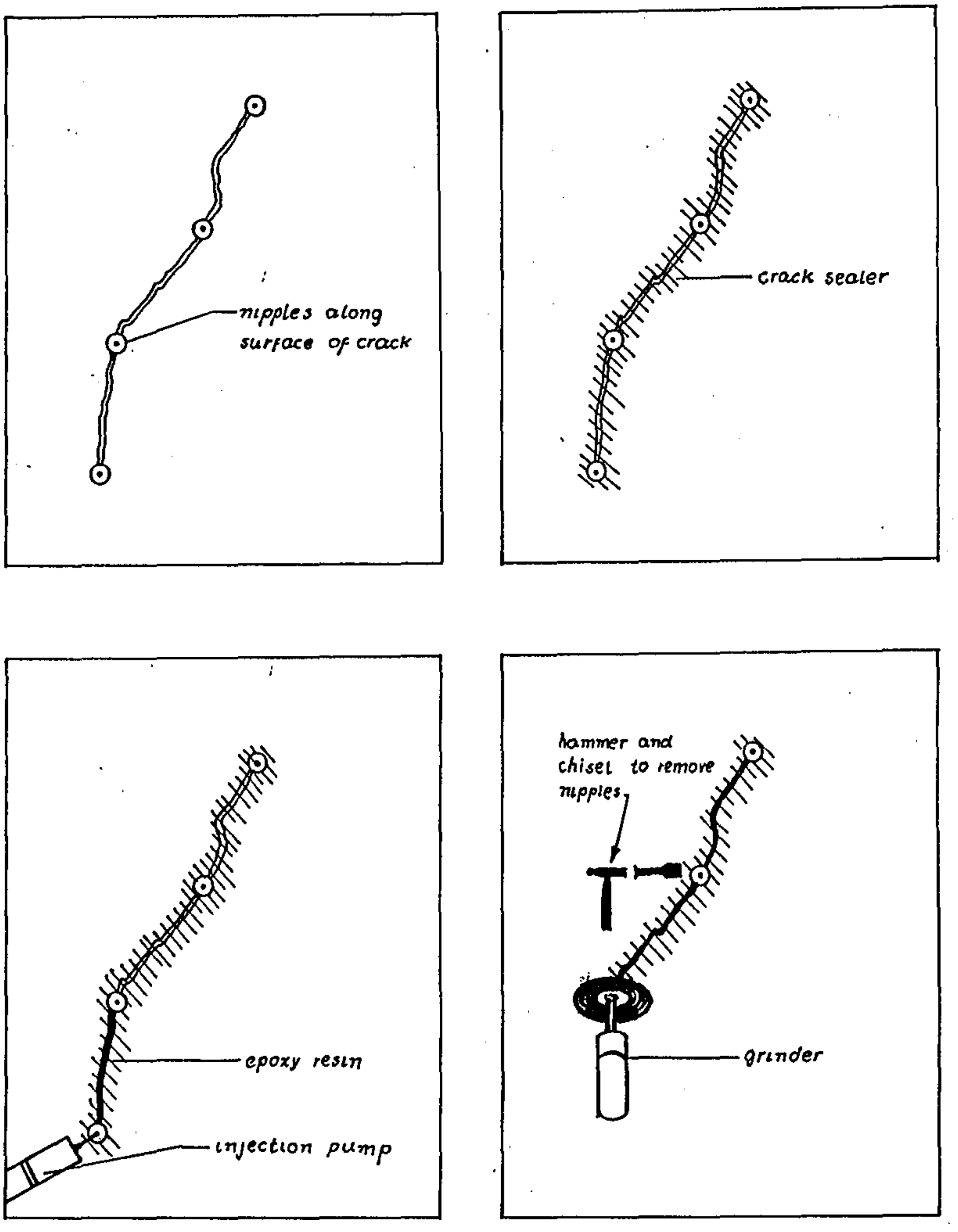
A sealer of low viscosity and penetrating properties is required to seal hair cracks. Special polymer sealers can be used which offer flexibility and bond strength. Fine hair cracks are best treated by isolating the areas with dams formed from putty or mastic. If there are no obvious cracks best results will be achieved by drilling holes through the surface taking care to remove the dust created and forming little wells around the holes to ease application.

5.7.2 Cracks (0.3 mm - 2 mm)[26]

Narrow cracks are repaired by crack injection. Where all exits from the cracks can be found and sealed, the standard low viscosity epoxy resin may be used. Where this is not feasible a thixotropic grade should be used, which will not pour away uselessly, out of the back of crack. The procedure for the injection system is (see figure 5.12):

a. The crack must be thoroughly clean and dry.

b. Using a sealer the copper injection nipples are bonded on to the crack taking care that entry into the crack through the nipples is not blocked. Then all external entries into the crack must be sealed, so that the resin will not drain away. The spacing of the nipples should be 200 – 250 mm.



5.12 Crack repairing by injection resins

c. The resin is then injected using an applicator gun. The injection starts form the lowest nipple (when vertical crack) or from the one extremity to the other (for horizontal cracks). Injecting continues until resin begins to exude from the next nipple.

d. The first nipple is then sealed and the injection proceeds in the same way from the next nipple until the whole crack is filled.

e. The nipples are knocked off and a sealer is used to make good their points. This must be done after 24 hours.

Cyclic loading tests of injected epoxy repaired concrete beams were operated in the United States.[6] The repaired beams showed excellent energy absorption and resisted numerous application of cyclic loads. They prove to be stronger than uncracked normal beams!

<u>5.7.3 Cracks (2 mm - 6 mm)</u>

Where cracks are wider, epoxy systems become rather unsuitable due to the high cost and their chemical properties (exothermic heat build up). A non-shrink cement based grout is more appropriate for such cracks and more economical. The procedure is very similar to the previous method. The grout is injected in the same way using an applicator gun. When complete filling has taken place it should be left to harden at least 48 hours. Then the nipples are removed and the holes are sealed.

Where extensive cracking has taken place, then the recommended repair method is the removal of the loosened sections and repairing as in the cases of spalled and damaged concrete which follow.

<u>5.7.4 Small spalled areas - Cover replacement less than 12 mm</u>

Repair to small spalled areas of concrete where replaceable cover is less than 12 mm must be done with epoxy mortars. According to

the producers no cementitious material can be used with a layer less than 12 mm (8 mm was given by a German company).[25] The minimum depth of repair would be 3 mm since anything less than that could be made up with a high-build surface coating.
The procedure for repairing[26,28] such concrete surfaces is as follows:

a. Cut out and scabble off unsound concrete leaving a minimum depth of 3 mm. If reinforcement is exposed, concrete should be cut back behind by at least 15 mm.

b. The surface must be then thoroughly cleaned and wetted.

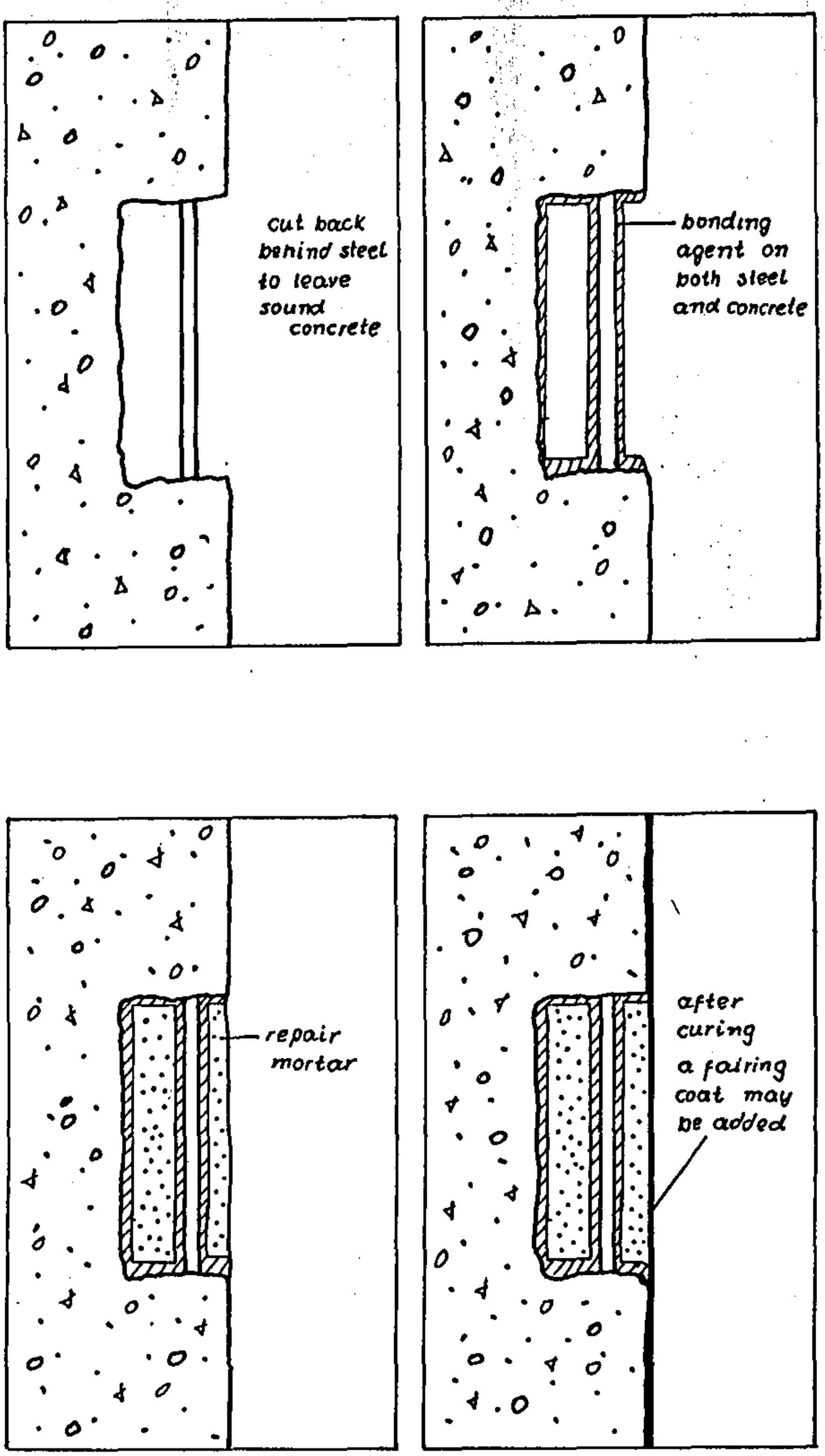
c. One coat of an epoxy bonding agent is applied to the concrete (and the steel) so a tacky layer is formed prior to the application of the mortar.

d. While the bonding agent is still tacky an epoxy mortar is placed using a trowel. The material must be pressed firmly into position.

<u>5.7.5 Spalled areas – Cover replacement more than 12 mm (see figure 5.13)</u>

The use of a polymer modified cementitious based system is more suitable for larger repairs mainly for economy. The result of the method will be comparable to the epoxy based system. The compressive strength achieved by a polymer cementitious mortar will be lower than that obtained by epoxy mortar. Nevertheless it will be more than enough for our purposes (according to producers a standard polymer cementitious mortar can achieve a compressive strength of 45 N/mm^2).

The procedure is the same as for small spalled areas with the epoxy materials replaced by cementitious mortar. The maximum thickness of repair specified by the producers should not be exceeded. If the repair depth is greater than the one allowed then more than one layers of mortar should be applied. Each time



5.13 Repairing of a spalled area with cementitious materials a bonding agent should be used. Special lightweight mortars exist which make work easier for vertical surfaces and soffits. Such mortars can achieve compressive strengths for 30 N/mm^2, which is sufficient for most applications.

5.7.6 Large area repairs

Large scale repairs to large volumes of damaged concrete can be done using any cementitious mortars. It can be done more effectively and economically however, with the use of a pourable grout. A high-strength gout, even in its flowing form, will obtain sufficient compressive strengths (up to 50 N/mm^2).

The method to be followed is specified below:

a. Remove all loose concrete and cut back around edge of repair to give minimum depth of 12 mm. If steel reinforcement is exposed, concrete should be cut back behind by approximately 20 mm.

b. Steel reinforcement should be clean from any loose material. Bars that have suffered severe damage must be replaced.

c. Formwork of the 'letter box' type should be prepared. All forms require to be watertight. One section of the formwork remain open and should be able to be fixed quickly.

d. A bonding aid is then applied to the existing concrete and steel reinforcement.

e. While surface is still tacky the remaining section of the formwork should be fixed and then a pourable grout may be poured into void through the 'letter box' opening.

f. Curing methods are equally important like for all cementitious mortars.

These specifications are supplied as general suggestions for different areas. Further specific directions should be sought.