Earthquake-resistant design of masonry buildings tomazevic pdf

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Series on Innovation in Structures and Construction Vol. 1 Earthquake-Resistant Design of Masonry Buildings Miha Tomaževič Slovenian National Building and Civil Engineering Institute Imperial College Press ...

PREFACE From many points of view, writing this book Earthquake-Resistant Design of Masonry Buildings was not an easy task. In most countries, masonry is still used as a traditional, non-engineered construction material: in this regard, the engineering approach to masonry construction had to be encouraged. ...

Miha Tomaževič CONTENTS PREFACE V 1. INTRODUCTION 1 2. EARTHQUAKES AND SEISMIC PERFORMANCE OF MASONRY BUILDINGS 5 2.1 Introduction 5 2.2 Earthquakes and Seismic Ground Motion 6 2.2.1 Causes of earthquakes 6 2.2.2 Seismic waves and earthquake ground motion 9 2.2.3 Magnitude and intensity 11 2.2.4 Occurrence of earthquakes 17 2.2.5 Earthquake ground motion and effects on buildings 18 2.5 2.3 Seismic Performance of Masonry Buildings 23 ...

CHAPTER 1 INTRODUCTION Besides wood, masonry is the most important construction material used in the history of mankind. Masonry in stone, as well as unburned and burned clay bricks, is used in a wide variety of forms, as a basic construction material for public and residential buildings in the past several thousand years ...

CHAPTER 2 EARTHQUAKES AND SEISMIC PERFORMANCE OF MASONRY BUILDINGS 2.1 Introduction Seismic actions are accidental actions which, depending on the seismicity of the location, rarely occur in the building's life-time. However, because of the destructive power of earthquakes, the stability and safety of buildings located in earthquake-prone areas should be verified for seismic loads. ...

2.2 Earthquakes and Seismic Ground Motion 2.2.1 Causes of earthquakes Earthquakes are caused by various natural phenomena, such as tectonic processes, volcanic eruptions, sudden failure of parts of the ground in karstic terrain, as well as by human activities, such as large excavations in mines, explosions, and large water reservoirs. ...

2.2.3 Magnitude and intensity ... Richter's magnitude is also called "local magnitude" M_L ... $\log E = 4.8 + 1.5 M$ ...

2.2.4 Occurrence of earthquakes ... $P_t = 1 - (1 - 1/T)^t$ ...

Table 2.2. Average values of peak ground acceleration ... Intensity scale Grade VIII ...

2.3 Seismic Performance of Masonry Buildings 2.3.1 Structural typology Masonry has been used as a handicraft material of old. ...

2.3.2 Typical damage patterns ... The analysis of damage patterns can clearly identify the weak and good points of different structural systems. On the basis of damage analysis, the failure mechanisms of individual structural walls and the entire structural system can be defined. On the basis of the observed mechanism, forces that develop in the structural system during earthquakes can be defined. However, any quantitative data regarding seismic action or structural resistance cannot be assessed unless additional experimental investigations that simulate the observed behaviour and damage patterns have been carried out. 32 Eart hq uake-Ies istan t Design of Mas onry Buildings Deformation and the typical type of damage to structural walls of a simple masonry building, subjected to seismic loads, is shown in Fig. 2.19. As can be seen, structural walls which are perpendicular to seismic motion are subjected to out-of-plane bending, causing vertical cracks at the corners and in the middle of the walls. In the plane of the walls, however, bending and shear caused horizontal and diagonal cracks in the walls, respectively. Distribution of forces and stresses, induced in a typical pier of a simple masonry building, subjected to earthquake ground motion, is shown in Fig. 2.20. Figure 2.20. Distribution of forces and stresses in window piers. 2.4 References [1] [2] [3] B.A. Bolt. Earthquakes. A Primer (W.H. Freeman and Co., San Francisco, 1978). J.M. Gere and H.C.Shah. Terra Non Firma (W.H. Freeman and Co., New York, 1984). M. Wakabayashi. Design of Earthquake Resistant Buildings (McGraw Hill, New York, 1986). Earthquakes and Seismic Performance of Masonry Buildings 33 L. Seeber, S.K. Jain, C.V.R. Murty and N. Chandak. 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Guidelines for earthquake resistant non-engineered construction (The International Association for Earthquake Engineering, Tokyo, 1986). Improving Earthquake Resistance of Low Strength Masonry Buildings Guideline, Indian Standard IS 13828 : 1993 (Indian Standard Institute, New Delhi, 1993). - CHAPTER 3 MASONRY MATERIALS AND CONSTRUCTION SYSTEMS 3.1 Introduction Masonry is a typical composite construction material which consists of: Masonry units, Mortar, Concrete infill and/or concrete, and Reinforcing steel. A wide variety of raw materials, both natural and arWicia1, is used for the production of traditionally and industrially made masonry units. Mortar is composed of lime, cement, sand and in different proportions, and mixed with water, with or without additives. Deformed and smooth steel reinforcing bars of different shapes and qualities, embedded in mortar or placed in the holes and grouted, are used to reinforce the masonry. Although each component of a masonry wall has its own specific mechanical characteristics, they are all expected to act together as a homogeneous structural material in case they are subjected to permanent and temporary actions. Of course, not all materials are always encountered together. Depending on which materials and/or how they are composed together in a structure, masonry construction systems are further subdivided in to 0 Unreinforced (plain) masonry, consisting of masonry units, mortar, reinforcing steel and concrete, and 35 Reinforced masonry, consisting of masonry units, mortar, reinforcing steel and concrete infill. Because of specific characteristics of each constituent masonry material, especially masonry units, it is not easy to predict the mechanical characteristics of a specific masonry construction type by knowing only the characteristics of its constituent materials, and masonry units. It is therefore of relevant importance that, for each type of masonry, experiments to correlate the strength characteristics of constituent materials with the characteristics of masonry are carried out. Because of its complexity, masonry and its constituent materials should comply with specific requirements of standards and codes, especially when they are used for the construction of engineered structures, where the resistance of elements and the entire structure to gravity and seismic loads is verified by calculation. In cases where the mechanical characteristics of the constituent materials and masonry structural system as a structural material do not comply with the assumptions of numerical verification, unreliable conclusions may be obtained. The basic requirements to be fulfilled for masonry materials and types of construction are specified in Eurocode 6: Design of masonry structures [1]. Additional requirements for masonry materials and construction systems to be considered in seismic zones are specified in Eurocode 8 : Design provisions for earthquake resistance of structures [2]. A set of European standards, which determine the basic properties of masonry materials and the way of testing, accompanies the Eurocodes. 3.2 Masonry Materials 3.2.1 Masonry Units Apart from the load-bearing capacity, the following aspects should also be considered when selecting the most suitable type of masonry unit: - adequate thermal and sound insulation capacity of masonry, especially in the case of external walls, - reduction of the weight of the building in order to reduce the seismic loads, and - economy of construction. Adobe and natural stone are still used in some countries and regions for the construction of plain, low-cost masonry houses. Adobe units (usual dimensions Masonry Materials and Construction Systems 37 are 300x400(120mm) are made of clayey earth, consisting of 30-40 % clay and 70-60 % earth, which is mixed with water. Straw is added to the mix of clay and earth (7-10 kg per m3 of earth) in order to prevent cracking. Sometimes, cement or gypsum is added as a stabiliser in order to improve the compressive strength of the units. The units are dried under the sun [3]. Besides masonry units manufactured from traditional and modern materials, different kinds of natural stone (lime-stone, slate etc.) are still used for the construction of masonry walls. Usually, in the case of dimensioned stone units, i.e. square dressed units with parallel horizontal faces, is allowed for the construction of masonry buildings in seismic zones. Traditional stone-masonry construction with two outer layers of uncoursed stone with an inner infill of rubble stones is not considered to be earthquake resistant. In any case, natural stone units are considered as units, where Category II of manufacturing control values of partial safety factors for material properties are taken into account. EC 6 gives specifications regarding the use of: Fired clay units, including lightweight clay units. Calcium silicate units. Concrete units, made with dense or lightweight aggregates. Autoclaved aerated concrete units. Manufactured stone units. Dimensioned natural stone units. In all cases, the quality of masonry units should comply with the requirements of relevant European standards (EN 77 1-1-6, respectively). Requirements regarding materials and manufacture, dimensions and tolerances, density, mechanical strength, water absorption, frost resistance, soluble salts content, safety in case of fire, etc., are specified for each type of masonry units. Solid or perforated, hollow, cellular and horizontally perforated masonry units with different dimensions and shapes, made of different materials are produced. Solid or equivalent solid masonry units are either units without recesses or units with recesses that are filled with mortar during construction, or units with up to 25 % by volume of vertical holes. Perforated masonry units are units with more than 25 % but less than 50 % by volume of formed vertical holes, which may pass through the unit. Hollow masonry units are units with more than 50 % by volume of vertical holes. Horizontally perforated units are units, which have no more than 50 % by cross- 38 Earthquake-Resistant Design of Masonry Buildings sectional area of formed horizontal holes, which pass through the units. Typical shapes of masonry units are shown in Fig. 3.1. In the case of hollow block masonry units, the disposition and size of holes should be limited in order to avoid premature cracks in webs and shells, either during manufacture and handling, or use. With regard to the total volume of holes, volume of any hole, area of any hole, as well as combined thickness of webs and shells, EC 6 classifies the units into four groups (Table 3.1). Perforated unit Hollow unit Horizontally perforated unit Cellular unit Figure 3.1. Types of masonry units (after [4]). This classification is used to select the appropriate value of correction factors K in cases where the characteristic compressive fk and shear strength of the masonry fvk are assessed by calculation on the basis of empirically obtained correlation between the strength of the constituent materials and the characteristic compressive strength of masonry units fb and mortarf, (Eqs. 3.2 and 3.4). In addition to that and in order to avoid local brittle failure of hollow units under a combination of vertical and lateral loads, EC 8 requires that masonry units to be used for earthquake-resistant construction of masonry buildings shall also meet the following requirements: - The units have not more than 50 % of holes (in % of the gross volume). - The minimum thickness of shells is 15 mm. - The vertical webs in hollow and cellular units extend over the entire horizontal length of the unit. 39 Masonry Materials and Construction Systems Table 3.1. EC 6 requirements for the grouping of masonry units. Group of masonry units I 2b 2a > 25-45 for > 45-55 for clay units, clay units, > 50-60 for > 25-50 for concrete concrete aggr. units aggr. units S 12.5 for 1 2.5 for clay units, clay units, 1 25 for I25 for concrete concrete aggr. units aggr. units 3 Volume of holes (% of the gross volume) I25 Volume of any hole(% of the gross volume) S 12.5 cL -- Limited by volume (see above) Area of any hole - - - - - - - - - - - - - - Limited by volume (see above) Limited by volume (see above) 1 170 Limited by are (see below) 2 2 800 mm2 except units with a single hole when the hole should be I 1 8 000 mm2 -- Combined No thickness (YO 2 20 2 37.5 2 30 requirement of the overall width3) Notes: 1. Holes may consist of formed vertical holes through the units or frogs or recesses. 2. If there is national experience, based on tests, that confirms that the safety of the masonry is not reduced unacceptably when a higher proportion of holes is incorporated, the limit of 55 % for clay units and 60 % for concrete aggregate units may be increased for masonry units that are used in the country with the national experience. 3. The combined thickness is the thickness of the webs and shells, measured horizontally across the unit at right angles to the face of the wall. Relatively low minimum mean values of compressive strength of masonry units to be used for the construction of structural walls are specified in relevant standards (EN 77 1-1-6): - Clay units: minfb = 2.5 MPa, - Calcium silicate units: minfb = 5.0 MPa (normalised values). 40 - Earthquake-Resistant Design of Masonry Buildings Concrete aggregate units: minfb = 1.8 MPa, Autoclaved aerated concrete units: minfb = 1.5 MPa. According to EC 8, the minimum normalised compressive strength of a = 2.5 MPa. Nevertheless, it is masonry unit, normal to the bed face, is recommended that, in the case of hollow clay and concrete aggregate units, the mean values of the units' compressive strength should not be kept below 7.5 MPa, especially where the units are used for the construction of reinforced masonry walls. The reason for such recommendation is explained in Section 3.3.2. According to EC 6, the so called "normalised compressive strength of masonry units"fb should be used in the design. This is the mean value of a reference strength determined by testing of at least ten (six in the case where the coefficient of variation is not greater than 15 %) equivalent, air dried, 100-mm wide and 100-mmhigh specimens cut from the unit (for example, this is the way to test calcium silicate and autoclaved aerated concrete units). The strength is obtained by testing full-sized units, the mean value of strength is converted to the compressive strength of the unit in its air-dry condition and then to the compressive strength of an equivalent unit of 100 mm width and 100 mm height. If the compressive strength of masonry units is specified as characteristic strength, this should be first converted to the mean equivalent using a conversion factor based on the coefficient of variation of tests. Then the measured value of strength is multiplied by the shape factor 6, which takes into account the actual dimensions of the unit. In the case of horizontally perforated units, the strength is determined in the direction perpendicular to the bed face. The values are given in Table 3.2. Table 3.2. Shape factor 6 for conversion of mean value of unit's strength to normalised value (EC 6). Least horizontal dimension (mm) Height (mm) 50 65 100 150 200 >250 I 50 0.85 0.95 1.15 1.30 1.45 1.55 I I I 00 0.75 0.85 1 .oo I 1.20 1.35 1.45 I I 100 0.70 0.75 0.90 1.10 1.25 1.35 I 200 - I I >250 - ~~~ 0.70 0.80 I .oo 1.15 1.25 0.60 0.75 0.95 1.10 1.15 Interpolation of values of the shape factor 6 is permitted where the dimensions of the unit are different than those specified in Table 3.2. 41 Masonry Materials and Construction Systems 3.2.2 Mortar Mortar is a mixture of inorganic binders (lime and/or cement), aggregates and water, which binds together masonry units. Sometimes, additives are added to mortar to improve workability, or for other reasons. According to the classification used in EC 6, different types of mortar are used in masonry construction: General purpose mortar, which is the traditional type of mortar used in joints with a thickness greater than 3 mm and in which only dense aggregate is used. Thin layer mortar, which is intended for use in masonry with a nominal thickness of joints 1-3 mm. Thin layer mortars are typical mortars, designed to fulfill the specific requirement of masonry construction. Lightweight mortar is made using perlite, pumice, expanded clay, expanded shale, or other lightweight materials. Lightweight mortars are also designed to fulfill specific requirements of masonry construction, and have a dry, hardened density lower than 1500 kg/m3. In order to obtain the specified compressive strength, mortar mixes can be either prescribed or else designed. In Table 3.3, typical compositions of prescribed general purpose mortar mixes and expected mean values of compressive strength are indicated [4]. Table 3.3. Typical prescribed composition and strength of general purpose mortars [4]. Mortar tYPe M2 M5 M10 M20 Mean compressive strength 2.5 MPa 5 MPa 10 MPa 20 MPa Approximate composition in parts of volume Cement Hydrated lime Sand 1 1 1 1 1.25 - 2.50 0.50 - 1.25 0.25 - 0.50 0 - 0.25 2.25-3 times cement and lime According to EC 8, for the construction of unreinforced and confined masonry, only mortars of type M5 (compressive strength($\geq$ 5 MPa) or stronger are allowed. If weaker mortars are used, disintegration of mortar may take place when subjected to vibration during a strong earthquake. In the case of reinforced masonry, however, the minimum mortar type is M10 (compressive 42 Earthquake-Resistant Design of Masonry Buildings strength(= 10 MPa) to ensure the adequate transfer of internal forces from steel reinforcement to masonry. In the case where reinforcement is embedded in mortar, the design of mortar strength should also ensure adequate bond strength of reinforcement in mortar (EC 6). Table 3.4. Characteristic anchorage bond strength of reinforcement in mortar (MPa) for ... (MPa) for highbond bars (MPa) I M5-M9 0.7 1 .o I M10 M4 I M15-M99 I 1 1.2 1.5 1 1 1.4 2.0 I M20 1.5 2.5 Mechanical properties of mortar are determined by testing mortar prisms 40/60/160 mm (EN 1015- 1 1). The prisms are first tested for bending, and then for compressive strength. The compressive strength of the mortar is the mean strength of six specimens. Generally, the thickness of bed and perpend (head) joints in the case of general purpose and lightweight mortars should be between 8 mm and 15 mm. If such is the case, Eqs. (3.2) and (3.4) can be used to assess the characteristic compressive strength and shear strength of the masonry. Except in zones of low seismicity, all perpend joints should be fully filled with mortar! 3.2.3 Concrete Infill A concrete mix of suitable consistency and aggregate size, which is used to fill the holes where reinforcing steel bars are placed in the case of reinforced masonry, is called concrete infill or grout. According to specifications given in EC 6, the maximum aggregate size should not exceed 10 mm where the least dimension of the void is 50 mm and the cover to reinforcement is between 15 mm and 25 mm, and should not exceed 20 mm where the dimensions are not less than 100 mm or 25 mm, respectively. In order to ensure workability and pourability of the infill, sufficient water should be added to the mix. However, in order to reduce the risk of cracking in the infill due to shrinkage resulting from loss of water into the masonry, expanding agents may be used. 43 Masonry Materials and Construction Systems The characteristic compressive cylinder or cube strength of the concrete infill f c k should not be less than 12 MPa or 15 MPa, respectively. The values of characteristic compressive cubeylinder strength of concrete infill fck, as well as the values of characteristic shear strength of concrete infill(fcvk to be used in the design and seismic resistance verification, are specified in Table 3.5 for the relevant concrete design classes. Table 3.5. Characteristic compressivef,k and shear strengthf,k of concrete infill (EC 6). Strength class of concrete fck (m a fcvk (ma) 12/1 5 12 0.27 C 16/20 16 0.33 C20/25 20 0.39 C25/30or stronger 25 0.45 I 3.2.4 Reinforcing steel Smooth or high bond (deformed) reinforcing steel with distinct yield plateau, which is used for reinforced concrete structures, is also used for reinforced and confined masonry construction. Especially shaped prefabricated ladder-type or truss-type reinforcement is often placed in mortar bed joints (Fig. 3.2). Figure 3.2. Typical ladder- and truss-type prefabricated bed joint reinforcement. According to EC 6, reinforcing steel may be assumed to possess adequate elongation ductility, if the following requirements are satisfied: - For high ductility class: E U > ~ 5 % and (ft lfy)k > 1.08, - For normal ductility class: Euk > 2.5 % and (ft lfy)k > 1.05, 44 Earthquake-Resistant Design of Masonry Buildings where: EUK = the characteristic value of the strain at maximum load, ft = the tensile strength of the reinforcing steel, fy = the yield strength of the reinforcing steel, and (ft/fy)k = the characteristic value of the ratio ft/fy. a(MPa) 800 600 400 200 0 I 6y - 0.0 0.05 0.10 0.15 0.20 0.25 E Figure 3.3. Typical stress-strain relationship of a high ductility steel for tension. " t I E, = 200 kN/mm2 Figure 3.4. Idealised stress-strain relationship of a reinforcing steel for tension and compression (EC 6). Masonry Materials and Construction Systems 45 In the case where high bond reinforcing steel with diameter less than 6 mm, including prefabricated ladder-type and truss-type bed joint reinforcement, is used, it should not be considered as having high ductility. A typical stress-strain diagram, obtained by tension test of a plain reinforcing steel (reinforcing bar), is shown in Fig. 3.3. An idealised diagram, used in the design of reinforced masonry structural elements, is shown in Fig. 3.4. 3.2.5 Masonry When verifying the load-bearing capacity of masonry walls and structures to vertical and lateral loads, the values of mechanical properties of masonry, considered as a composite material, and not mechanical properties of constituent materials, such as masonry units and mortar, are used. In EC 6, the following intrinsic mechanical properties of the masonry, which should be obtained by standard test methods, are specified: The compressive strength, f, The shear strength,f,, The flexural strength,fx, The stress-strain relationship, 0~E. EC 6 makes a distinction between the masonry itself - considered as an assemblage of masonry units and mortar, which has intrinsic mechanical properties - and the structural masonry element, a wall, the mechanical properties of which depend on the intrinsic mechanical properties of the masonry, the geometry of the element, and the interaction of adjacent parts. In addition to mechanical characteristics specified by EC 6, the following mechanical properties of masonry and masonry elements should be used in seismic resistance numerical verification: The tensile strength ft, as an equivalent to shear strength, f v , The modulus of elasticity, E, The shear modulus, G, and The ductility factor, p . The ductility factor is a typical mechanical property of a masonry wall. It cannot be attributed to the masonry alone. As a rule, mechanical characteristics of masonry are determined by testing standard specimens of masonry wallets and walls according to a set of standards EN 1052. Compressive strength is determined by testing either small wallets of at 46 Earthquake-Resistant Design of Masonry Buildings least 1.5 units length and 3 units height, or walls of 1.0-1.8 m long and 2.4-2.7 m high (Fig. 3.5). Specimens are placed in a testing machine, and the vertical load is increased at a uniform rate so that failure occurs after 15-30 minutes after the beginning of testing. In cases where the wall is slender (height/thickness ratio greater than 20), lateral displacements at the mid-height of the wall are measured in order to take into account the slenderness. If 6 is the displacement just before the attainment of maximum vertical load, and t is the thickness of the wall, the test value can be increased by a factor of t /(t' provided that the increase is not greater than 15 %. DT VDT 40 4 Figure 3.5. Masonry specimens for compression tests (EN 1052-1). According to EN 1052- 1, three identical specimens are tested and the results evaluated. The mean compressive strength f of masonry is adjusted if the compressive strength of masonry units and mortar deviate from the design mean values within f 25 % of the specified strength. The characteristic compressive is determined as the smaller value of either fk = f /1.2 strength of masonry or fk = fmin. 47 Masonry Materials and Construction Systems In case that no test data are available, the characteristic compressive strength of unreinforced masonry made with general purpose mortar may be calculated on the basis of normalised compressive strength of mortar fm using the equation: fk = KfbO.65fm 0.25 (in MPa), (34 provided that fm is not greater than 20 MPa nor greater than 2 fb, whichever is the 0.10 smaller. The value of constant K (in MPa) depends on the classification of masonry units into groups according to Table 3.1. EC 6 recommends that the constant K may be taken as: 0.60 for Group 1 masonry units in a wall without longitudinal mortar joint, - 0.55 for Group 2a masonry units in a wall without longitudinal mortar joint, - 0.50 for Group 2b masonry units in a wall without longitudinal mortar joint, - 0.40 for Group 3 masonry units in a wall with longitudinal mortar joint and for Group 3 masonry units. - The correlation between typical, experimentally-obtained values of characteristic compressive strength of masonry fk, obtained by testing wall specimens [9, lo], and expected values, calculated by Eq. (3.1), is given in Table 3.6. Table 3.6. Correlation between experimentally-obtained and predicted values of characteristic compressive strength of masonry. 11 Unit (kPa) Group Mortar I5 I 1 - clay I 2 a - clay I 2.5 7.5 2 I 1 2.5 5.0 I I 4.4 2.4 I I ~~ 15 I I I 7.5 7.5 7.5 2a - clay 2.5 b - clay 5 12a- other I 5 I 2a - other I 5 I 2b - clay 2.5 2.5 I I I 48 4.8 3.0 3.5 I 4.0 4.5 I 1 3.0 3.0 2.3 I I I 48 Earthquake-Resistant Design of Masonry Buildings Good correlation between experimental and calculated values of compressive strength of masonry can be obtained in some cases. In others, however, the correlation is far from good. It is therefore recommended that tests to determine the mechanical characteristics of masonry should be carried out wherever a new shape of masonry unit, or a new construction technology, is introduced. Shear strength, which is the strength of masonry subjected to shear forces, is defined as a combination of initial shear strength under zero compressive strength and increment in strength due to compressive stress perpendicular to shear in the member at the level under consideration. Initial shear strength at zero compressive stress is determined by testing so called triplet specimens according to EN 1052-3 (Fig. 3.6). The way of supporting the specimens and applying the load should ensure that only shear stresses develop in the mortar to masonry unit contact planes. Figure 3.6. Determination of initial shear strength (EN 1052-3). A minimum of five triplets are tested. The minimum acceptable value of shear strength at zero compressive stress is 0.03 MPa. The characteristic shear strength of unreinforced masonry is then calculated by using the equation: fvk =fvko + 0.4 Od, where Od is the design compressive stress perpendicular to shear. However, this value should be not less than 0.065 (which is not less than fvko), or less than a limiting value specified in EC 6 and depending on masonry unit's group and mortar quality. In Table 3.7, typical values for masonry, constructed with units and mortar quality recommended for the construction in seismic zones, are indicated. 49 Masonry Materials and Construction Systems A different approach is also used, which defines the shear strength of unreinforced masonry as the maximum value of principal tensile stress, developed in a masonry wall of a specified geometry, idealised as an elastic, homogeneous and isotropic panel at the attained shear resistance of the wall 6). The value of principal tensile stress, developed in the wall under those assumptions at shear failure, is called tensile strength, ft. Eq. (7.16), which is used to evaluate the value of tensile strength from experimental results, and the assumptions of this theory are explained in details in Chapter 7.4.1. There is yet no standard testing procedure proposed to determine the shear strength ft or tensile strength of masonry by direct testing. Either cyclic and monotonic racking or simple diagonal compression tests can be used to determine the values of bothf, andft. As has been found by correlating the results obtained from different types of testing, the differences are not significant [73. Whether the results are evaluated forf" erft, the effect of compressive stresses in the masonry wall panel is taken into account. Typical experimentally-obtained values of characteristic tensile strength of masonry fk are correlated with initial shear strength values/fvko at zero compressive stresses, recommended by EC 6, in Table 3.8. Table 3.7. Typical values of initial shear strength at zero compressivefko, and limiting values of characteristic shear strength/fvk for masonry with general purpose mortar (EC6). Units (I f" ,ax (MPa) Mortar M10 - M20 M2.5-M9 M10- M20 M2.5-M9 I M10(a other 26 clay M20 M10- M20 M2.5-M9 I 0.2 0.2 0.15 I I 0.3 0.2 0.15 I I I 1.7 1.5 I I 1.4 I I 1.2 I I Eqs. (3.2) and (7.16), which define the shear and tensile strength of masonry, respectively, are considered as statistical regression equations used to present the same experimental results in two different ways, it can be seen that that no 50 Earthquake-Resistant Design of Masonry Buildings statistically important difference in results exists (Fig. 3.7). Table 3.8. Correlation between experimental values of tensile strengthfk and initial shear strengthfvko for different masonry types. I Unit (kPa) Group Mortar I Strength (MPa) I 1 I 1 I I 2 2 5 I I I I I I I I I I R 25 2.0 2.5 I 5 I 0 I 0 0.20 I 0 I 2 I 5 (MPa) I I 0 I 0.0 I 0.0 0 0.20 I 0.0 0 0 20 I 0.0 0 I I 0 ... 2 0 I 3 I 2 4 5 6 7 8 9 I 0 I 0 I 0 I 0 I 0 Figure 3.7. Correlation between experimental values and values calculated by Eq. (3.2). Masonry Materials and Construction Systems 51 As can be seen, the values obtained from shear tests on triplet specimens by means of Eq. (3.2), as well as values obtained from tests of walls according to Eq. (7.16), correlate well with each other. The differences in ... Modulus of elasticity E, measured for a typical masonry, is determined on the basis of the secant modulus in compression, and a shear modulus G, usually taken as 40 % of E. Although EC 6 allows the value of modulus of elasticity E to be taken as zero, i.e. no contribution of unreinforced masonry should be taken into account in the case of design for seismic resistance. Depending on the direction of failure plane, flexural strength parallel to bed joints,fxl, or flexural strength perpendicular to bed joints, fx2, are distinguished (Fig. 3.8). The values of flexural strength fxl and fxl should be determined by testing specially-made wallets according to EN 1052-2. Figure 3.8. Definition of failure planes to determine flexural strength(fx1 andfx2 (EC6). On the basis of experimental results, the modulus of elasticity of masonry can also be evaluated. Modulus of elasticity E is defined as a secant modulus at service load conditions, i.e. at 1/3 of the maximum vertical load. It is calculated on the basis of the mean strain p measured at that level of vertical load (Fig. 3.9). On the basis of experimental values, EC 6 proposes idealised stress-strain relationship for the design of masonry in bending and compression (Fig. 3.10). In 52 Earth uake-Resistant Design of Masonry Buildings absence of a value of E determined tests, the value to be taken into consideration in the structural analysis can be assessed to be equal to E = lOOOfk. (3 -5) fk(MPa) 7 4 3 2 1 0 2 6 8 Figure 3.9. Typical experimental stress-strain relationship of masonry in compression [5]. -0 A Idealised ------/ I I 1 /r= 0" ow.-- I I I I fk f d I/ #3m 7 I I I I Design I I I 0.0020 0.0035 E Figure 3.10. Stress-strain relationship for the design of masonry in bending and compression (EC 6). 53 Masonry Materials and Construction Systems As was the case of compressive strength, the values of modulus E, assessed by Eq. (3.4), are sometimes far from reality. As can be seen by comparing the values in Tables 3.6 and 3.9, the actual values vary between (3.4x) 2 0 0 0 5 E I 2000fk. As specified in EC 6 , it may be assumed that the shear modulus, G, is 40 % of the elastic modulus, E. However, as experimental results indicate, the actual values of shear modulus are much lower than assumed by EC 6 . Typical correlation between experimental and predicted values is shown in Table 3.9. Table 3.9. Correlation between experimental and predicted values of moduli E and G. 15 7.5 7.5 7.5 2a - clay 2b - clay 2a - other I 2a - other 2 b - clay 5 5 5 3 5000 5000 6000 1800 3000 3500 6000 4500 3200 500 500 270 1200 1400 2400 1800 It can be seen that the values of shear modulus G vary from 6 % to 25 % of the elastic modulus E. In no case have values as high as 40 % of E, as specified in EC 6 , been obtained. If shear modulus G is expressed in terms of tensile Strengthft, typical values are obtained: which most results falling close to G = 2000ftk. Taking into consideration the wide range of variation of the possible values of strength and deformability characteristics of masonry, it is recommended that the values of mechanical characteristics of masonry, which do not depend on the relevant characteristics of constituent materials in a uniform way, the testing of masonry becomes one of the basic aspects of seismic resistance verification of 54 Earthquake-Resistant Design of Masonry Buildings constructions systems. Using the data obtained by testing each specific type of masonry, and not relying upon the code recommended relationships, will make the results of the seismic resistance verification accurate. 3.3 Construction Systems Masonry buildings are box-type structural systems composed of vertical and horizontal structural elements, walls and floors, connected in both directions. Reinforced-concrete bond-beams (tie-beams) are provided at floor levels to connect the walls. During earthquakes, floors should act as rigid horizontal diaphragm, which distribute the seismic inertia forces, developed in them, among the structural walls in proportion to their stiffnesses. Any type of floors may be used, provided that general requirements of continuity and effective diaphragm action are satisfied. According to EC 6, the main types of structural walls are distinguished as follows: Single-leaf wall, which is a wall without a cavity or continuous vertical joint in its plane. Double-leaf wall, which is a wall consisting of two parallel leaves with a longitudinal joint between them, not exceeding 25 mm, and solidly filled with mortar. The leaves are tied together with wall ties to achieve common action under vertical and lateral loading. Cavity wall, which is a wall consisting of two parallel single-leaf walls, tied together with wall ties or bed joint reinforcement, with either one or both leaves supporting vertical loads. The space between the leaves is left as a continuous cavity or filled, or partially-filled, with non-load bearing thermal insulating material. Grouted cavity wall, which is a cavity wall with two parallel leaves, spaced at least 50 mm apart and tied securely together with wall ties and bed joint reinforcement, and with a cavity filled with concrete to achieve common action under vertical and lateral loading. Typical examples of wall types are shown in Fig. 3.11. Although no restriction regarding the use of any of the structural walls' type in seismic zones is specified in EC 8, it is obvious that single-leaf walls should be preferred to double-leaf walls, and reinforced grouted cavity walls to cavity walls, since they ensure monolithic behaviour of the wall under seismic conditions. Masonry Materials and Construction Systems 55 Figure 3.11. Cross-section of a (a) singleleaf, (b) double-leaf and (c) cavity wall (EC6). Various types of masonry construction systems, both traditional and engineered, are used in various countries. Because of the characteristics of masonry, the behaviour of different construction systems subjected to seismic loads is different: whereas unreinforced, plain masonry represents basically a non-ductile structural material, confined and especially reinforced masonry represent structural systems of improved strength and ductility. 3.3.I Unreinforced Masonry Although it is not considered to be earthquake-resistant by EC 6 and EC 8, traditional stone-masonry construction, constructed as two-layered walls with an inner infill, can be made earthquake-resistant by providing connecting stones at least one per m2 of the vertical area of the wall. Mortar type M2 should be used for the construction, and the voids between the stones are filled with mortar, especially the inner infill part of the wall. Dimensioned stones must be used at the corners and wall intersections to provide adequate connection in these, critical, zones. Construction course must be levelled at each 1.0 m height in the zones of high, and at each 2.0 m height in the zones of moderate and low seismicity. R.c. bond-beams should be provided, connecting the structural walls all around the building (Fig. 3.12). Brick and block masonry walls should be constructed according to the rules of good mason workmanship, such as: - When necessary, masonry units should be soaked in water before the construction in order to prevent burning of the mortar, especially cement mortar. 56 Earthquake-Resistant Design of Masonry Buildings stone 0.5 m Figure 3.12. Construction of earthquake-resistanttraditional stonemasonry wall. Figure 3.13. Bonding arrangements using Group 1 masonry units (EC 6) . - Masonry units should be overlapped on alternate courses so that the wall acts as a single structural element. To ensure adequate bonding, masonry units should overlap by a length equal to at least 0.4 times the height of the unit or 40 mm, whichever is the greater (Figs. 3.13 and 3.14). At the corners and wall - - intersections, the overlap should not be less than the thickness of the units. Units should be cut in order to achieve the specified overlap. Vertical stacked bond in not allowed in seismic regions. If general purpose mortar is used, the thickness of mortar joints should be not less than 8 mm and not more than 15 mm. Perpend joints should be fully filled with mortar. Figure 3.14. Bonding arrangements using Group 2a and 2b masonry units (EC 6). Although the recommended minimum thickness of load-bearing walls by EC 6 is only 100 mm, EC 8 specifies that, in seismic zones, the thickness of unreinforced masonry walls should be not less than 400 mm in the case of natural stone, 300 mm in the case of manufactured stone, and 240 mm in the case of confined and reinforced masonry. This value is also recommended for masonry units made of materials other than stone. In many countries, however, 190-mm thickness is considered as the minimum for masonry construction in seismic zones. To take into account the stability of structural walls, the ratio of the effective height to thickness of the wall should not be more than 9 in the case of natural stone, 12 in the case of manufactured stone, and 15 in all other cases. Also, the 58 Earthquake-Resistant Design of Masonry Buildings length of a structural wall should be at least 1/2 of the greater clear height of the openings adjacent to the wall in the case of natural and manufactured stone, and 1/3 in the case of confined masonry. As a rule, the same type of masonry units and mortar should be used for structural walls in the same storey. Bracing walls should be constructed simultaneously with load-bearing walls. It is also recommended that the thickness of individual walls be kept constant over the entire height of the building. 3.3.2 Confined Masonry 3.3.2 Confined Masonry is a construction system, where masonry structural walls are surrounded on all four sides with reinforced concrete or reinforced masonry vertical and horizontal confining elements, which are not intended to carry either vertical or horizontal loads, and are consequently not designed to perform as a moment-resisting frames (Fig. 3.15). Figure 3.15. Masonry confined masonry and (b) reinforced concrete bond-beams and columns in the case of masonry-infilled frames, the r.c. frame structure, which is designed to resist vertical and seismic loads, is constructed first, and infill masonry is constructed in a subsequent phase, whereas in confined masonry construction it is the other way round: masonry walls are constructed first and confining r.c. structure. In the case of confined masonry, however, masonry walls are intended to carry all vertical and seismic loading. The structural walls, which support the floors, are constructed first. Then, Masonry Materials and Construction Systems 59 floors with horizontal bond-beams are put in place, and finally, r.c. vertical confining elements are constructed, well connected with horizontal confining elements. As the experimental investigations and the experiences obtained after earthquakes have shown, confining the masonry walls with bond-beams and tiecolumns results in: - Improvement in the connection between structural walls. - Improvement in the stability of slender structural walls. - Improvement in strength and ductility of masonry walls. - Reduction of the risk of disintegration of masonry panels damaged by the earthquake. In order to ensure structural integrity, vertical confining elements should be located at all corners and recesses of the building, and at all joints and wall intersections. In addition, they should be placed at both sides of any wall opening 2 which, according to EC 8, has an area of more than 1.5 m . This is a severe 2 requirement which, as experiences indicate, can be relaxed to 2.5 m . Vertical confining elements should be placed at all free ends of structural walls. As is the case of horizontal bond-beams, the distance between the vertical confining elements should not exceed 4.0 m (Fig. 3.16). H Figure 3.16. Typical distribution of vertical confining elements in the plan of a building. One of the dimensions of the cross-section of r.c. vertical tie-columns is usually equal to the thickness of the wall, but not less than 150 mm. The dimension may be less than the thickness of the wall in the case of double-leaf walls. The minimum cross-sectional area of the r.c. columns should be approximately 5 % in the case of load-bearing walls. It is recommended to provide at least 4 longitudinal bars of 10 mm diameter. Figure 3.17. Typical confining of masonry walls with r.c. frames. Masonry Materials and Construction Systems 61 The design of reinforcement of confining elements should be aimed at ensuring their rapid activation by forming the hinge in the column at the point where the wall is separated. Confining elements, Indian experience has shown that in the cross-section of the confining elements is the 60 Earthquake-Resistant Design of Masonry Buildings minimum (Fig. 3.17). The minimum dimension in the direction of the 60 Earthquake-Resistant Design of Masonry Buildings wall should be 150 mm. Concrete of at least grade C15 should be used. According to EC 6, the minimum cross-section area of steel reinforcement of tie2 columns is 200 mm , corresponding to 4 plain steel (yield stress 240 m a) ; 8-mm diameter reinforcement bars. Although the minimum cross-section of the tie2 columns' reinforcement is increased to 240 mm in seismic zones (EC 8), it is recommended that, at corners and wall intersections, tie-columns are reinforced with at least 4 plain steel, 10-mm diameter reinforcement bars, with a total cross2 sectional area of 314 mm . Stirrups of 6-mm diameter plain bars at 200 mm intervals are used to confine the longitudinal reinforcement. According to EC 6, the resistance of the r.c. or reinforced masonry confining elements should not be taken into account in the design of confined masonry structures in seismic situations. This is a mere consequence of fact that very little experimental information regarding the mechanism of action and distribution of lateral seismic loads among confining elements during earthquakes is available. Therefore, the amount of reinforcement in confining elements is determined on empirical basis [so]. In Table 3.10, a proposal is given to choose the adequate dimensions and number of plain steel (yield stress 240 MPa) reinforcing bars of vertical confining elements on the basis of the number of storeys of the building under consideration and the seismicity of the location. Table 3.10. Typical reinforcement of vertical confining elements (after [lo]). No. of storeys 2 1 2 4 I 2-4 I 2 -6 I 2 4 5 4 6 Low : a g < 0.1 g 448 448 448 4410 448 448 Moderate: High 0.1 g = ~g <0.2 g 0.2 g < ag 4410 4412 4410 4412 4410 4412 4410 4412 In order to provide the integrity of the walls' connecting system, care should be taken such that the reinforcement of the bond-beams and tie-columns is adequately placed and anchored at the corners and wall intersections. Sixty diameters overlaps are required by EC 8. In addition, tie-columns should be connected with masonry walls with reinforcing bars of not less than 6 ~Ifn Masonry Materials and Construction Systems 61 diameter placed in bed joints, spaced at not more than 600 cm apart, adequately anchored into the mortar joints. Fig. 3.17 shows the damage to a confined masonry wall pier at the end of a "shaking-table" test of a confined masonry building model [11]. Although the confinement significantly improved the seismic performance of unreinforced masonry buildings, it did not prevent the disintegration of the middle part of the masonry panel at ultimate state. It can be seen that, in order to fully utilise the resistance and energy dissipation capacity of masonry, a masonry wall panel should be reinforced with horizontal bed joint reinforcement. Figure 3.17. Disintegration of a confined masonry wall pier without bed joint reinforcement at the end of "shaking-table" test of a confined masonry building model. To reinforce the masonry, the use of specially shaped ladder-type or trusstype reinforcement (Fig. 3.2), embedded in mortar with a vertical spacing of no more than 600 mm, and adequately anchored into the tie-columns at the ends, is recommended. In case where the masonry bed joint reinforcement is anchored into the tie-columns, the connectors, required by EC 8 to connect the masonry and confining elements, are not necessary. 62 Earthquake-Resistant Design of Masonry Buildings 3.3.3 Reinforced Masonry Reinforced masonry is a construction system, where steel reinforcement in the form of reinforcing bars or meshes is embedded in the mortar or placed in the holes and filled with concrete or grout. By reinforcing the masonry with steel reinforcement, the resistance to seismic loads and energy dissipation capacity may be improved significantly. To achieve this, the reinforcement should be integrated with masonry so that all materials of the composite masonry system act monolithically when resisting gravity and seismic loading. There are various ways in which steel reinforcement can be used in a masonry structural system. Basically, however, reinforced masonry systems can be classified into: Reinforced hollow unit masonry, Reinforced grouted cavity masonry, and Reinforced pocket type walls. Concrete infill Reinforcement laid in mortar I Mortar or / concrete infill Figure 3.18. Reinforced hollow unit masonry (EC 6). Reinforced hollow unit masonry represents the basic form of reinforced masonry construction (Fig. 3.18). Specially shaped units with vertical holes where vertical reinforcement is placed and infilled with grout or concrete, and at the cross-webs for horizontal reinforcement, are used for the construction of masonry walls. Before laying the masonry units, vertical reinforcement is placed in position. Then, the first course of units is laid in the mortar and horizontal bars or bed joint reinforcement are placed in the grooves or in the mortar joints. The holes containing vertical bars are filled with either concrete or grout, and the grooves containing the horizontal steel are filled with either grout or mortar, as the construction of the wall progresses. In order to 63 Masonry Materials and Construction Systems improve the resistance and ductility depending on the shape of the units, all holes in the hollow blocks are often grouted or filled with concrete infill. Reinforced cavity masonry, however, is different by technology of construction, and consequently, by structural characteristics and behaviour. As can be seen in Fig. 3.19, it consists of two leaves (wythes) of masonry units, separated by a cavity into which the vertical and horizontal reinforcement is placed and grouted with either concrete infill or grout. The two leaves of a cavity wall are tied together with wall ties or connectors, which should be designed to carry lateral loads, induced by earthquakes. It is recommended that, in seismic regions, at least one 6-mm diameter stirrup, or an equivalent tie, should be 2 provided at every 0.25 m of the wall area. Concrete infill Reinforcement laid in mortar Figure 3.19. Reinforced grouted cavity masonry construction systems (EC 6). The wythes are usually 100 mm (the thickness of masonry units) thick and the cavity 60-1 00 mm wide. The masonry units should be laid in running or stretcher bond: vertical stacked bond is not allowed in seismic zones. The grout can be poured either as the work progresses or after the masonry units in the whole storey have been laid. In the first case, vertical reinforcing bars are first placed into position. Then, the horizontal bars (or connectors) are placed and grouted as the laying of courses of masonry progresses. In the second case, however, the mesh of vertical and horizontal reinforcement is first placed into position. Then, masonry units are laid on each side of the mesh, connected together with wall ties. The ties should be laid in the bed joints along the same vertical line in order to facilitate the vibrating of the grout pours. After the masonry is built to a full storey height, the cavity is filled with grout. Before 64 Earthq uake-Resistant Design of Masonry Buildings grouting, all mortar droppings should be removed from the foundation or other bearing surfaces and reinforcement. Cleanout openings should be provided to allow flushing away of mortar droppings and debris at the bottom of each pour. Sometimes, vertical reinforcement is placed in the pockets formed in the wall by special bonding arrangement (Fig. 3.20). As in the case of reinforced hollow unit masonry, vertical reinforcing bars are placed into position before the laying of masonry units. Depending on the units used, horizontal bed joint reinforcement is placed in the mortar joints at vertical spacing not exceeding 600 mm. The pockets containing vertical bars are filled with either concrete or grout, as the construction of the wall progresses. Mortar or concrete infill Figure 3.20. Reinforced pocket type walls (EC 6). Reinforcing steel in masonry should be corrosion-protected adequately against corrosion due to environmental conditions. If ordinary carbon steel is placed in the holes and embedded in concrete infill, it should be protected by a concrete cover at least 20 mm thick in the case of a dry environment, 25 mm thick in a humid environment, and 40 mm thick in the case of an aggressive chemical environment. In the case of the mortar bed joint reinforcement, however, the minimum depth of mortar cover from the reinforcing steel to the face of the masonry should be 15 mm. As specified in EC 8, horizontal reinforcement should be placed in the bed joints or suitable grooves in the units with a vertical spacing not exceeding 600 mm, and the minimum percentage, referred to as the gross area of the section, should be not less than 0.05 %. However, high percentages of horizontal reinforcement leading to compressive failure of the units prior to yielding of the steel should be avoided. Bars of not less than 4-mm diameter should be bent around the vertical steel at the edges of the wall. 65 Masonry Materials and Construction Systems Vertical reinforcement of a cross-sectional area not less than 400 mm should be placed at both free edges of every wall element, at every wall intersection, or at least 4 m intervals within the wall. The reinforcing bars should be provided with an internal forces can be transmitted from reinforcement bars to the mortar and concrete infill, and, subsequently, to masonry units. Anchorage may be achieved by straight anchorage, hooks, bends or loops (Fig. 3.21). Straight anchorage or bents are not allowed for plain reinforcing steel of more than 8 mm diameter. Hooks, bends or loops are not allowed to anchor reinforcing steel in compression. 4 1 1 1 1 lb &-L 0.7 lb Straight anchorage Hook Figure 3.21. Typical details of anchorage of reinforcing steel (EC 6). EC 6 specifies the straight anchorage length lb depends on the diameter of a reinforcement bar. It should be determined by calculation assuming that the bond stress along the bar is constant: Q = the effective diameter of reinforcing steel, fyk = the characteristic strength of reinforcing steel, fbok = the characteristic anchorage bond strength ys, YM = the partial safety factors. For hooks, bends and loops, the anchorage length of the bars in tension can be reduced by 30 %. In the case where a larger area of reinforcement is provided 66 Earthquake-Resistant Design of Masonry Buildings than required by the design, the anchorage length can be reduced proportionally, provided that for a reinforcing bar in tension the anchorage length is not less than the greater value of 0.3lb, 10 bar diameters, or 100 mm, and for a bar in compression not less than the greater value of 0.6lb, 10 bar diameters, or 100 mm. The lap length is also determined on the basis of Eq. (3.6). In the calculation, however, the smaller of the two bars lapped is considered. The lap length provided between two reinforcing bars should be equal to - 2lb for bars in compression and for bars in tension where less than 30 % of the bars in the section are lapped, and where the clear distance between the lapped bars in a transverse direction is not less than 10 bar diameters and the concrete or mortar cover is not less than 5 bar diameters. - 1.4Zb for reinforcing bars in tension where either 30 % or more of the bars at the section are lapped, or if the clear distance between the lapped bars in a transverse direction is less than 10 bar diameters or the concrete or mortar cover is less than 5 bar diameters. - 2Zb for reinforcing bars in tension where both 30 % or more of the bars at the section are lapped and the clear distance between the lapped bars in a transverse direction is less than 10 bar diameters or the concrete or mortar cover is less than 5 bar diameters. However, code requirements regarding the amount and distribution of reinforcement in reinforced hollow unit masonry need careful consideration. As has been shown by experiments, the effectiveness of horizontal and vertical reinforcement strongly depends on the type and quality of masonry units and mortar, as well as on the way the reinforcing bars are anchored at the ends. Obviously, the minimum and maximum amount of reinforcement which can be effectively used in masonry should be modified by a truss-beam mechanism (Fig. 3.22). Under seismic situation, the bending moment and shear action on the wall are resisted by tension and compression forces in the tension and compression chord, as well as by tension and compression that develop in verticals and diagonals of an equivalent truss, respectively. The tension chord is represented by vertical reinforcing bars at the tensioned side, the compression chord, however, by compressed masonry and vertical steel at the compressed side of the wall. As can be seen in Fig. 3.22, "verticals" are Masonry Materials and Construction Systems 67 represented by horizontal reinforcing bars in mortar bed joints, and compressed diagonals are represented by masonry. H Figure 3.22. Simplified mechanism of reinforced action. l Figure 3.2-3 Failure of reinforced hollow block masonry wall due to crushing of shells of masonry units. Earthquake-Res istant Design of Masonry Buildings 68 The mechanism works if there is enough vertical and horizontal steel adequately embedded and anchored to resist the induced tension forces, and 2 masonry units are strong enough to resist the induced compression forces, as well as the anchorage of reinforcing steel without splitting or disintegration (Fig. 3.23). Therefore, the recommendation of EC 8 that "high percentages of horizontal reinforcement leading to compressive failure of the units prior to yielding of the steel, should be avoided" should be modified to take into account the balance between the load-bearing capacity of vertical and horizontal steel, and masonry. In this regard, experimental investigations are inevitable, particularly in the case where new reinforced masonry construction systems are introduced. When subjected to cyclic loads, such as seismic loads, the bond between mortar and steel deteriorates. Consequently, the tension capacity of steel cannot be fully utilised, although the bars are properly embedded and bent around the vertical steel at the edges of the wall. Additional limitations should be given to improve the behaviour. Flexural type of failure is preferred to shear type because it represents ductile type of seismic behaviour. However, crushing of face shells of hollow units under cyclic lateral loads may lead to unexpected failure mechanism, although the compressive strength of the units might be adequate. As a result, the resistance capacity of such walls, calculated on the basis of the materials' strength characteristics, may be overestimated. Fig. 3.24 shows a typical example. 100 80 60 % 40 20 + I f orB Unreinforced I I I 1 I 1 5 0 15 20 25 30 s (mm) Figure 3.24. Vertical reinforcement at the edges did not improve the resistance of the walls because of weak hollow masonry units [9]. Masonry Materials and Construction Systems 69 Although the walls have been designed and a sufficient amount of horizontal reinforcement to resist the shear has been provided, the resistance envelopes, obtained by testing the walls under simulated seismic loads, clearly show that the increased amount of vertical steel at the borders of the walls had no effect on resistance because of the crushing of face shells of hollow masonry units. Whereas reinforced grouted masonry behaves more or less as reinforced concrete, the seismic behaviour of reinforced masonry with hollow or perforated units and reinforcing bars in the vertical holes and bed joints depends on the type and quality of masonry units. For optimum behaviour, balance should be always found between the compression and shear capacity of masonry units, tension capacity of vertical and horizontal steel, and available bond between mortar and steel. In this regard, the minimum percentage of steel, either vertical or horizontal, depends on the quality of the basic masonry wall. The maximum amount of reinforcement should also be limited in correlation with the quality of masonry units, mortar and grout, and not only with regard to the spacing between the bars and depth of cover for protection against corrosion. 3.4 References Eurocode 6: Design of masonry structures. Part 1-1: General rules for buildings. Rules for reinforced and unreinforced masonry. ENV 1996-1-1: 1995 (CEN, Brussels, 1995). Eurocode 8: Design provisions for earthquake resistance of structures. Part 1-3: General rules - Specific rules for various materials and elements. ENV 1998-1-3 : 1995 (CEN, Brussels, 1995). K. Bayülke. The state of the masonry buildings in Turkey. EAEE WG8 Input Report (Ankara, 1989). International Recommendations for Design and Erection of Unreinforced and Reinforced Masonry Structures. 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Seismic behavior of masonry buildings. Part two: Shaking-table tests of model buildings with M2 and M4 analysis of test results. Report ZAG/PI-95/06 (Slovenian National Building and Civil Engineering Institute, Ljubljana, 1996). - CHAPTER 4 ARCHITECTURAL AND STRUCTURAL CONCEPTS OF EARTHQUAKE-RESISTANT BUILDING CONFIGURATION 4.1 Introduction Many lessons have been learned from earthquakes regarding the influence of structural layout on the seismic behaviour of masonry buildings. The observations of earthquake damage and subsequent analysis of the causes of damage indicated clearly that the shape of structural systems, building configuration is of relevant importance. The buildings with regular structural layout, with the walls properly connected together at floor levels, have often performed well, even when they are not designed to resist earthquakes. Adequate seismic behaviour of masonry buildings can be obtained, if it is possible to improve the seismic resistance by considering simple principles of architectural and structural planning, and meeting the requirements for the quality of materials and construction at the same time. The following basic principles should always be considered when conceiving a seismically-resistantmasonry structure: Structural simplicity and regularity, i.e. uniformity and symmetry, 0 Redundancy (robustness), Rigid floor diaphragm action, and 0 Adequate foundation. In the case where the structure is regular and simple, gravity and seismic loads are transmitted in a clear and undisturbed way from element to element. 71 72 Earthquake-Resistant Design of Masonry Buildings Under seismic conditions, the inclined seismic energy will dissipate uniformly over the entire structure. If structural elements are not distributed uniformly in the plan and elevation of the structural system, however, concentration of stresses might occur in the zones of nonuniformity, resulting in heavy damage and collapse of the structure. Redundancy, however, is not known which will be the main direction of ground motion during an expected seismic event. Therefore, the resisting elements of a structure in a seismic zone should be designed to resist the seismic excitation in both principal directions of the building. The consequences of neglecting the necessity of having an adequate number of sufficiently resistant structural walls in both principal directions of the building are severe (Fig. 2.18). Symmetric distribution of resisting elements in the plan of the building will prevent possible torsional vibration, which often causes unexpected behaviour of the structure when subjected to strong seismic ground motion. For the same reason, the dimensions of setbacks and recesses should be limited. In the case of masonry structures, the importance of good connection of walls and rigid horizontal diaphragm action of floors has already been emphasised. Last but not least, an adequate foundation system should be provided to transmit the ultimate seismic loads, developed in the upper structure, into the foundation soil. If the foundation soil fails before the ultimate resistance of the structure is attained, the building is exposed to a severe risk of overturning. 4.2 Building Configuration Masonry is a specific construction material, which, because of its mechanical properties, requires specific structural configuration even when it is intended to carry only vertical loads. The basic rules for the construction of masonry structures are based on tradition and experience. As a result, traditional structural systems are generally simple and regular, consisting of load-bearing walls and cross walls which do not change their position and shape along the height of the building, and are evenly distributed in both directions of the building. The simplicity and regularity of a structure not only improve the expected structural behaviour, but also make possible simplification of methods and ways of seismic resistance verification. Experience and subsequent parametric analysis of response of buildings to earthquakes have indicated that the following general criteria for structural regularity in plan and elevation should be considered [1-5 3: Architectural and Structural Concepts of Earthquake-Resistant Building Configuration - 73 The building structure is approximately symmetrical along each principal axis in plan, regarding both lateral stiffness and mass distribution. A sufficient number of structural walls, with approximately the same crosssectional area and stiffness, should be provided in each direction of the building (Fig. 4.1). U + n Adequate Not adequate Figure 4.1, Distribution of structural walls in plan. - The plan configuration should be simple. Simple square or rectangular buildings perform better when subjected to earthquake actions than those with many projections. In this regard, the total dimension of projections, reentrant comers or recesses in one direction should not exceed 25 % of the overall dimension of the building in the corresponding direction (Fig. 4.2). - In order to avoid torsional effects resulting from differences in ground motion in the case of long rectangular buildings, it is desirable to limit the length of a single part of the building to four times its width. If longer 74 - Earthquake-Resistant Design of Masonry Buildings buildings are required, the building should be divided into separate parts with adequate separation. Separation of large buildings with composite shape in plan (L, T, U or + shaped plan) into several parts may be required in order to obtain symmetry and rectangularity of each individual part (Fig. 4.3). To prevent hammering effects between the adjacent parts, a sufficient separation between individual parts should be provided. It is recommended that the width of separation should not be less than 30 mm, and 10 mm should be added for each storey (or 3.0 m) when the building height exceeds 9.0 m. 1-L 4B -t Separation Separation Figure 4.3. Irregular masonry buildings should be separated in regular sections. - - The building should also be regular in elevation. The distribution of resisting elements, stiffnesses and masses along the height of the building should be as uniform as possible. Concentration of masses at upper storeys should be avoided. Sudden changes in stifness due to changes in dimensions in plan, distribution and type of structural elements in the adjacent stories along the height of the building result in severe concentration of stresses, energy dissipation demand and possible damage to those zones (Fig. 4.4). Mixed structural systems, such as a combination of masonry structural walls in one storey and r.c. frame structural system in the adjacent storey, are not allowed (Fig. 4.5). Sometimes, a combination of r.c. columns and masonry shear walls within the storey is required because of architectural reasons. If this is the case, structural masonry walls should be reinforced with vertical Architectural and Structural Concepts of Earthquake-Resistant Building Configuration 75 and horizontal reinforcement to improve their ductility and energy dissipation capacity. R.c. columns should be part of a frame system, connected with masonry shear walls with rigid horizontal diaphragms, to ensure a uniform distribution of seismic loads among the reinforced masonry shear walls and r.c. columns. A detailed structural analysis should be carried out to verify the transfer of seismic forces from one type of structural element to another. In the design, special attention should be paid to structural details to ensure that the structure is able to resist the assumed effects of seismic actions. Adequate Not adequate Bad Figure 4.4. Principles for building configuration in elevation. Figure 4.5. Mixed structural system should be avoided. Although the criteria for structural regularity and simplicity are fulfilled, the potential seismic resistance of the structure is not fully utilised, unless the requirements for rigid horizontal floor diaphragm action and good connection of the walls are fulfilled at the same time. Namely, if the walls are not connected together at floor levels, out-of-plane vibration will cause their separation along vertical joints at the corners and wall intersections. Uncoupled vibration of separated walls during earthquakes will significantly reduce the resistance of the 16 Earthquake-Resistant Design of Masonry Buildings building to lateral loads and will, consequently, lead to the partial or total collapse of a building. In order to ensure rigid horizontal diaphragm action of floors, the following requirements should be taken into consideration: - Each floor should be situated in a single plane. Sharp dislevelements should be avoided. - The rigid behaviour of horizontal diaphragm should not be altered by the presence of discontinuity, such as stairways. Large opening zones should be strengthened with special reinforcement or bond-beams. - Two-way slabs are preferred to one-way slabs, as they distribute vertical gravity loads more uniformly onto the structural walls. 4.3 Dimensions, Building Height and Number of Stories On the basis of experience and tradition, on the extent of earthquake damage due to the type of materials used and structural systems employed, and considering the current level of technical knowledge and construction technology, limitations regarding the dimensions and height of masonry buildings have been set in most existing seismic codes. However, based on the results of recent experimental and theoretical investigations, and on improvements in technology and methods of design, limitations regarding the dimensions, building height and number of stories have been relaxed. Except for unreinforced masonry located in seismic zones with ag 2 0.3 g, which is not allowed for earthquake-resistant shear walls in buildings higher than two storeys, no limitations regarding the size and height of masonry buildings are specified in EC 6 and 8. In the case of confined and reinforced masonry buildings, which fulfill the basic requirements for the quality of materials and structural configuration, the size and height of the building depend on the load-bearing capacity of masonry materials. However, the structure should undergo subsequent verification of serviceability and ultimate limit states. In order to reduce typical effects of temperature differences, shrinkage of r.c. floors, and differential soil settlements common to long buildings, which may all result into cracking of masonry structural walls even before the earthquake, and to avoid the unfavourable effects, such as torsion due to differences in ground motion along the length of the building in the case of earthquakes, it is recommended that the length of masonry buildings of all masonry structural Architectural and Structural Concepts of Earthquake-Resistant Building Configuration 77 systems or their separated parts should be limited to 40 m in the zones of high (ug 2 0.3 g), and to 50 m in the zones of moderate and low seismic intensity (ag< 0.3 g). In the case of unfavourable soil conditions, further limitations regarding the maximum dimensions of masonry buildings should be observed, depending on the specific soil characteristics. Although the seismic resistance of all masonry buildings should be verified by calculation, it is recommended that, having in mind the available quality of masonry materials and technology, the height and number of stories of masonry buildings constructed in one of the construction systems may not exceed the recommended values specified in Table 4.1. The limitations are specified for the traditional brick or hollow unit masonry construction. Reinforced grouted cavity masonry is exempted from these limitations. Table 4.1. Recommended maximum building height Hand number of stories n. 4.4 Distribution of Structural Walls In order to obtain a good performance when subjected to an earthquake, structural walls in a masonry buildings should be uniformly distributed in two orthogonal directions. Their number and strength should be sufficient to resist the induced seismic loads. The walls should be firmly connected to the floors, which, acting as rigid horizontal diaphragms, distribute horizontal seismic forces to the walls in proportion to their stiffness. From the viewpoint of the structural system, which resists the seismic loads, the walls of a masonry building can be classified as: - Structural walls, which resist the horizontal loads acting on the building, and 78 - Earthquake-Resistant Design of Masonry Buildings Non-structural walls, having exclusively the function of partitioning the building space. With regard to the carrying of vertical loads, the structural walls can be further subdivided into two categories, namely: - Load-bearing walls, which carry their own weight and vertical loads from the floors, and Bracing walls, which carry their own weight, but do not support the floors. - Considering the significance of structural walls, these walls should have a minimum thickness of 400 mm in the case of stone and 300 mm in the case of unreinforced masonry with artificial units, whereas 240 mm is the minimum thickness in the case of confined and reinforced masonry (see Section 3.3.1). However, there is no specification in EC 6 and 8 regarding the minimum distance between them. In order to obtain adequate performance for different masonry structural systems, it is recommended that the distance between the structural walls should be limited depending on the structural system and seismicity of the zone as indicated in Table 4.2. Table 4.2. Recommended maximum distance between structural walls [3]. I Design ground acceleration ag

Unreinforcedmasonry Confined masonry Reinforced masonry < 0.2 g 1 10 m 15 m 15 m 0.2-0.3 g 1 ~~~ 8m 12 m 12 m 2 0.3 g r ~~ 6m 8m 8m 1 Although the above recommended values are used when conceiving the structural layout of the building, its structural safety should be verified by calculation. Limiting factors may be the vertical load-bearing capacity and the out-of-plane bending capacity of these walls. 4.5. Wall Openings The size and position of wall openings, such as windows and doors, have a strong effect on the in-plane resistance of a masonry shear wall. When subjected to seismic loads, stress concentration takes place in the opening zones, which Architectural and Structural Concepts of Earthquake-Resistant Building Configuration 79 may result in unexpected cracking of masonry and the subsequent deterioration of resistance of masonry elements to in-plane lateral loads. In order to improve the behaviour of masonry buildings when subjected to earthquakes, the following recommendations should be observed regarding the location and size of the wall openings: Openings should be located in those walls which are subjected to smaller intensity of vertical gravity loads. Openings should be located outside the zones of direct influence of concentrated loads at beam supports. On each storey, openings should be located in the same position along the vertical line. In order to provide an uniform distribution of resistance and stiffness in two orthogonal directions, openings should be located symmetrically in the plan of the building. The top of the openings in the storey should be at the same horizontal level. Openings should not interrupt r.c. bond-beams at the top of structural walls. In addition to the above, the total length of openings in a shear wall should not exceed half of the wall's length. It is also recommended that, in the case of brick and hollow unit masonry construction in the zones of high expected seismic intensity, the total cross-sectional area of structural walls in each of two orthogonal directions should not be less than 3 % of the gross floor area. 4.6 Simple Buildings For masonry buildings which comply with the provisions regarding the quality of masonry materials and construction rules (see Chapter 3), and with additional structural limitations specified in EC 8, an explicit safety verification is not mandatory. Such buildings are called "simple buildings". Simple buildings are regular buildings with an approximately rectangular plan, where the ratio between the length of the long and short side is not more than 4, and the projections or recesses from the rectangular shape are not greater than 15 % of the length of the side parallel to the direction of the projection. The number of stories above ground of different construction systems is limited depending on seismic zones to the values given in Table 4.3. The resisting walls (shear walls, structural walls) should be arranged almost symmetrically in plan in two orthogonal directions. A minimum of two parallel walls should be placed in each orthogonal direction, the length of each wall being Earthquake-Resistant Design of Masonry Buildings 80 greater than 30 % of the length of the building in the same direction, and the distance between these walls not being greater than 75 % of the length of the building in the other direction. In the case of unreinforced masonry buildings, walls in one direction should be connected with walls in the orthogonal direction at a maximum spacing of 7.0 m. Table 4.3. Number of stories above ground, allowed for simple buildings @C 8). Design ground acceleration ag < 0.2 g 0.2-0.3 g 2 0.3 g I Unreinforced masonry Confined masonry Reinforced masonry 3 4 5 2 3 4 1 2 3 At every floor, the cross-sectional area of structural walls in two orthogonal directions, given as a percentage of the total floor area above the level considered, should be not less than the values given in Table 4.4. Table 4.4. Minimum horizontal shear wall cross-section, given as % of the total floor area above the level considered. Design ground acceleration ag < 0.2 g 0.2-0.3 g 2 0.3 g Unreinforced masonry Confined masonry Reinforced masonry 3 2 2 5 6 4 5 5 4 In addition to that, at least 75 % of the vertical load should be supported by the structural walls, and the difference in the mass and in horizontal cross-section of structural walls between two orthogonal directions should not be greater than 20 %. 4.7 Non-structural Elements Failures or fall-downs of non-structural elements, such as partition walls, chimneys, masonry veneer, ornamentations, etc., might cause casualties and structural damage during strong earthquakes. The falling-down of non-structural Architectural and Structural Concepts of Earthquake-Resistant Building Configuration 81 elements might also obstruct passages and emergency exits, hence preventing emergency interventions after the earthquakes. In this regard, when designing masonry buildings to resist seismic loads, attention should also be paid to adequate structural detailing of non-structural elements. Partition walls are made of Group 3 masonry units (see Table 3.1) and are usually about 100 mm thick, or less. Depending on their dimensions and the seismic zone, partition walls may be either unreinforced or reinforced with bed joint reinforcement to prevent their out-of-plane instability. If reinforced, 4-6 mm diameter bars are usually placed in bed joints with a vertical spacing of 400-600 mm. Partition walls are fixed between the floor slabs by means of cement mortar joints, whereas their connection with structural walls or tiecolumns along the vertical borders is achieved either by bond or by steel anchors. The out-of-plane stability of partition walls should be verified by calculation (see Sections 7.8 and 7.9). It is recommended that masonry gable end walls and attics higher than 0.5 m are anchored to the uppermost floor bond-beams. In order to connect those walls, r.c. bond-beams should also be provided on top of those walls. In the case where the height of those walls exceeds 4 m, intermediate bond-beams should be added at intervals not exceeding 2 m. In addition to that, r.c. tie-columns, as specified in Section 3.3.2, should be provided at distances not exceeding 4 m, and should be well connected together with r.c. bond-beams (Fig. 4.6). Figure 4.6. Tying of gable end walls and attics with r.c.tie-beams and columns. 82 Earthquake-Resistant Design of Masonry Buildings Masonry veneer is supported by the main structural system and can either be adhered or anchored to the backing structure. If veneer is not a part of a structural wall, and is made as a free standing wall of special veneer units, it should be adequately bonded to the backing structure, even though it does not contribute to its strength. In the case where masonry veneer wall is attached to a masonry wall, many problems related to differential movements between veneer and support due to shrinkage, short and long term deflections, temperature differences, and the like, occurring in the case of masonry veneer attached to r.c. or steel structures, disappear. However, in order to prevent its falling-out during earthquakes, even adhered masonry veneer should be adequately anchored to structural walls with steel anchors or connectors. Although no specific requirements regarding masonry veneer are provided in EC 6 and EC 8, it is obvious that similar rules as in the case of non-structural walls should be considered to verify the stability of masonry veneer (see Sections 7.8 and 7.9). Free standing chimneys and ventilation stacks should be constructed using cement mortars. Adequate anchoring into the top floor and reinforcement above the top floor level should be provided. Ornamentations, such as cornices, vertical or horizontal cantilever projections, etc., should be reinforced with steel reinforcement and adequately anchored into the main structural system of the building. The adequacy of the anchoring should be verified by calculation as specified in Sections 7.8 and 7.9. 4.8 References Eurocode 6: Design of masonry structures, Part 1-1: General rules for buildings. Rules for reinforced and unreinforced masonry. ENV 1996-1-1 : 1995 (CEN, Brussels, 1995). Eurocode 8: Design provisions for earthquake resistance of structures, Part 1-2: General rules and rules for buildings - General rules for buildings. ENV 1998-1-2: 1995 (CEN, Brussels, 1995). Eurocode 8: Design provisions for earthquake resistance of structures, Part 1-3: General rules - Specific rules for various materials and elements. ENV 1998-1-3 : 1995 (CEN, Brussels, 1995). Architectural and Structural Concepts of Earthquake-Resistant Building Configuration [4] [5] 83 Construction Under Seismic Condition in the Balkan Region. Vol. 3 : Design and Construction of Stone and Brick-masonry Buildings (UNIDO/UNDP, Vienna, 1984). International Recommendations for Design and Erection of Unreinforced and Reinforced Masonry Structures. CIB Recommendations, Publication 94 (CIB, Rotterdam, 1987). CHAPTER 5 FLOORS AND ROOFS 5.1 Introduction Masonry buildings represent box-type structural system composed of vertical structural elements - walls, and horizontal structural elements - floors and roofs. Vertical gravity loads are transferred from the floors and roof, which act as horizontal flexural elements, to the bearing walls, which support the floors and act as vertical compression members. Finally, the loads are transferred fkom the bearing walls to the foundation system and into the ground. In the case of earthquakes, however, floors and roof act as horizontal diaphragms which transfer the seismic forces, developed at floor levels, into the walls, which support them in both orthogonal directions. In addition to this, floors and roofs connect the structural walls together and distribute the horizontal seismic forces, developed in a masonry building, among the structural walls in proportion to their lateral stiffness. Bond-beams are provided along each structural wall at floor levels to assist the floors in connecting the structural walls. 5.2 Floors According to EC 6 and EC 8 [I, 2], a floor or roof structure can be made of reinforced or prestressed concrete or timber joists incorporating boarding, provided the floor or roof structure is capable of developing horizontal diaphragm action. The connection between the floors and walls should be provided by steel ties or r.c. bond-beams. The design lateral loads should be transmitted between the walls and interconnecting elements either by means of anchors (straps) or by the frictional resistance between the walls and the floors or roofs. 85 86 Earthquake-Resistant Design of Masonry Buildings Different types of floors can be used in the earthquake resistant construction of masonry buildings. Monolithic r.c. slabs, which are cast simultaneously with r.c. bond-beams (Fig. 5. l), represent the most simple solution. Sufficient bearing length, being not less than 65 mm in normal cases, should provide the required bearing capacity and transfer of shear forces. - -- Rc. slab - Figure 5.1. Typical example of monolithic cast-in-place r.c. slabs with bond-beams (after [3]). prefabricated element sit-in-place concrete earn U Figure 5.2. Typical example of prefabricated slabs with r.c topping and bean-beams (after [3]). 87 Floors and Roofs In cases where the floors are made of prefabricated, reinforced-concrete or reinforced-masonry elements, precast elements should be well anchored into the bond-beams along the wall (Fig. 5.2). Typically, in that case, the topping should be cast simultaneously with r.c. bond-beams (Fig. 5.2). In cases where the floors are made of large prefabricated elements without r.c. topping, steel connectors should be provided along the connections between two elements to transfer the shear and tension forces developed in the horizontal diaphragm during an earthquake from one element to another. Steel connectors should be strong enough to ensure monolithic rigid diaphragm action of the floor during the strongest expected earthquake. Adequate anchors should also be provided at supports to ensure good connection of structural walls and rigid horizontal diaphragm action of such type of floors (Fig. 5.3). ictctd elcmcnl elem m- I Figure 5.3. Typical example of a slab made of large prefabricated elements (after [3]). Wooden floors represent flexible horizontal diaphragms. As some recent experimental and analytical studies indicate, the flexibility of horizontal wooden floors, adequately anchored to structural walls, may improve the seismic behaviour of masonry buildings in the specific case of long span structural walls 141 88 Earthquake-Resistant Design of Masonry Buildings In the case of wooden floors, rigid horizontal diaphragm action is provided by nailing plywood to timber joists at the bottom and top, or by nailing boards or planks to timber joists in both diagonal directions: in one direction at the bottom and in the other one at the top (Fig. 5.4). Sometimes, wooden floors are stiffened by nailing boards or planks in both orthogonal directions only at the top of the joists. In any case, however, r.c. bond-beams or steel ties should be provided along the walls, and timber joists should be anchored into the bond-beams or walls with steel anchors. Figure 5.4. Stiffening of wooden floors. 5.3 Bond-beams Horizontal r.c. bond-beams (tie-beams) should be constructed at the top of all structural walls at every floor level, or with a distance not exceeding 4 m between them. Bond-beams represent a horizontal framing system which - Transfers the horizontal shear induced by the earthquakes from the floors to the structural walls; - Connects the structural walls; - Improves the in-plane rigidity of horizontal floor diaphragms. - In combination with vertical tie-columns, improves the strength and energy dissipation capacity of masonry walls. In order to achieve the assumed behaviour, a number of structural details and recommendations have been developed on the basis of practice and experimental research. Concrete at of least class C 15 should be used. According to specifications given in EC 8, the cross-section of horizontal bond-beams should not be less than 150 by 150 mm. Usually, the vertical dimension of a r.c. section of a bond-beam is equal to the thickness of the floor structure, whereas its Floors and Roofs 89 horizontal dimension may be less than the thickness of the wall in order to accommodate an outer thermal insulation layer, but no less than 150 mm. Following the experiments and observations, at least four plain steel (yield stress 2 240 m a) , 10-mm diameter reinforcing bars, with a total of 314 mm crosssectional area, should be used to reinforce the bond-beam, although a cross2 section of 240 mm is specified as the minimum by EC 8. These requirements need to be considered in correlation with the limitation of the height of unreinforced masonry buildings in seismic zones (see Section 4.3). In order to provide the integrity of the walls' connecting system, care should be taken that the reinforcement of the bond-beams is adequately spliced and anchored at the corners and at wall intersections. Stirrups of 6-mm diameter plain bars at 200 mm intervals are used to confine the longitudinal reinforcement. Sixty bar diameters overlaps are required by EC 8 for the longitudinal reinforcement of the bond-beams. According to EC 6 and EC 8, the resistance of the r.c. bond-beams should not be taken into account in the design, and the bond-beams do not need to be designed for seismic loads. As was the case of vertical confining elements, this is a mere consequence of fact that limited experimental information regarding the mechanism of action and distribution of lateral seismic loads onto bond-beams and structural walls during earthquakes is available. The amount of reinforcement in the bond-beams is determined on empirical basis [3]. In Table 5.1, a proposal is given to choose the adequate dimensions and number of plain steel (yield stress 240 MPa) reinforcing bars of horizontal confining elements on the basis of the number of storeys of the building under consideration and the seismicity of the location. Table 5.1. Typical reinforcement of horizontal r.c. bond beams (after [3]). No. of Position Low- Moderate : 0.2 g 5 ag < 0.3 g ag < 0.2 g storeys (storey) 4 bars, 4 8 mm 4 bars, 4 10 mm 2 1-2 4 1-2 4 bars, 4 1 0 mm 2 bars, 4 8 mm 4 bars, 4 8 mm 4 bars, 4 10 mm 2-4 4 6 1-2 4 bars, 4 12 mm 4 bars, 4 14 mm 6 3-4 4 bars, 4 10 mm 4 bars, 4 12 mm 4 bars, 4 8 m m 4 bars, 4 10mm 5-6 6 - ~ ~ ~ I High: 0.3 g 5 ag 4 bars, 4 12 mm 4 bars, 4 16 mm 4 bars, 4 14 mm I 4 bars, 4 12mm 90 Earthquake-Resistant Design of Masonry Buildings However, it should be pointed out that, in spite of the fact that the resistance of bond-beams is neglected in design, the bond-beams do play an important role in the seismic behaviour of buildings, as indicated by experiments (see Section 10.5.1.), two mechanisms govern the behaviour of bond-beams in the case of flexible floors: bending due to out-of-plane vibration of the walls and tension due to seismic shear which developed in the walls in the direction of seismic action. However, in the case of monolithic floors and rigid horizontal floor diaphragm action, the bond-beams represent a constituent part of the diaphragm. In this particular case, their reinforcement is carrying the tension developed in the tensioned chord of the diaphragm, subjected to seismic loads. 5.4 Lintels, Balconies and Overhangs For vertical loads lintels function as beams, which support the weight of the wall and floor above the opening. Lintels can be made of either cast-in-place concrete, or prefabricated reinforced concrete, or reinforced masonry elements. Depending on the distance from the top of the opening to the top of the adjoining floor, cast-in-place lintels can be cast either independently or monolithically with the bond-beam and floor slab. The latter solution provides improved behaviour in the case of an earthquake (Fig. 5.5). Monolithic bond-beam and lintel Bond-beam Lintel Figure 5.5. Typical configuration of lintels in seismic zones (after [3]). In seismic zones, the lintels and parapets should be properly bonded to the masonry of the adjoining walls and connected to the bond-beams and tie-columns, where provided. In the case where r.c. tie-columns are used to confine the walls along the openings, the reinforcement of the lintels should be anchored into r.c. tie-columns. In order to prevent local collapse due to crushing of supports in earthquakes, sufficient bearing length should be provided at the end of the lintels. Floors and Roofi 91 It is recommended that a minimum of 250 mm bearing length should be provided at both ends. As a rule, the lintel width should be equal to the thickness of the wall. In the case of external walls, however, the lintel width may be reduced to accommodate the outer thermal insulation layer, but in no case should be less than 150 mm. Figure 5.6. Typical disposition of cantilever slabs and overhangs (after [3]). Balconies and overhangs are typical cantilever structural elements, where non-desirable vertical vibration can be induced during earthquakes. In order to reduce the vertical oscillation, it is recommended that the span of balconies, overhangs and other cantilever elements should be limited to (Fig. 5.6): - 1.20 m for cantilever slabs cast continuously with the floor slabs, and - 0.50 m for cantilever slabs anchored into the bond-beams without the continuity with the floor slab. The stability of cantilever balconies and overhangs, exceeding the recommended span, should be verified by taking into account the vertical component of seismic loads. According to EC 8 [6], the analysis for determining the effects of the vertical component of the seismic action can be carried out on the basis of a partial model of the structure, which includes the elements under consideration and takes into account the stiffness of the adjacent elements. According to EC 8 [7], the effects of vertical components of seismic motion are modelled by the response spectrum as defined in Section 2.2.5, but with the ordinates reduced by a factor of 0.7 for vibration periods T shorter than 0.15 s, by a factor of 0.50 for vibration periods longer than 0.50 s, and linearly interpolated value of factor for a vibration period between 0.15 s and 0.50 s. 92 Earthquake-Resistant Design of Masonry Buildings 5.5 Roofs In order to transfer inertia forces developed at the roof level into the supporting walls, the roof system should be adequately braced in both orthogonal directions and should be well connected to the r.c. bond-beam, constructed at the top of the structural walls (Fig. 5.7). Structural systems which exert lateral forces on the attic masonry walls and gable walls should be avoided. In order to avoid damage, the attic walls should be anchored to the uppermost floor by means of adequately spaced r.c. tie-columns. Figure 5.7. Typical timber roof structure. In order to reduce seismic loads, light roof structural systems and roof cover (tiles) are preferred to heavy roof structural systems. Where prefabricated elements are used, r.c. cast-in-place topping with a minimum thickness of 40 mm should be provided. In such a case the ends of the prefabricated elements should be embedded into the r.c. bond-beam along the complete perimeter of the roof. 5.6 References [I] Eurocode 6: Design of masonry structures, Part 1-1: General rules for buildings. Rules for reinforced and unreinforced masonry. ENV 1996-1 - 1 : 1995 (CEN, Brussels, 1995). [2] Eurocode 8: Design provisions for earthquake resistance of structures, Part 1-3: General rules - Specific rules for various materials and elements. ENV 1998-1-3:1995 (CEN,Brussels, 1995). Floors and Roofs [3] [4] [5] [6] [7] 93 Construction Under Seismic Condition in the Balkan Region. 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CHAPTER 6 BASIC CONCEPTS OF LIMIT STATES VERIFICATION OF SEISMIC RESISTANCE OF MASONRY BUILDINGS 6.1 Fundamentals After many centuries of traditional use and decades of allowable stresses methods, limit states verification of seismic resistance of masonry structures has been recently introduced to seismic codes. The philosophy of Eurocode 6: Design of masonry structures [1] and Eurocode 8: Design provisions for earthquake resistance of structures [2], which regulate the design and construction of masonry structures, is based on the fundamental requirement that a structure should be designed so that, with acceptable probability, it will remain in use within the expected life period and under expected maintenance conditions. This means that the structure should withstand all actions and influences likely to occur in its life time without substantial damage, but will also not be damaged disproportionally in cases where accidental events such as explosions, impacts, earthquakes or human errors might occur. In seismic regions, two basic requirements are considered in the design: 0 No collapse requirement, and Damage limitation requirement. The structure should be designed and constructed to withstand the design seismic action without local or general collapse. It should also retain structural integrity and load-bearing capacity after being subjected to an earthquake with expected intensity (design earthquake). If the structure is subjected to seismic 95 96 Earthquake-Resistant Design of Masonry Buildings action having higher probability of occurrence than the design earthquake, but of a lesser intensity, no damage to structural or non-structural elements should occur that might limit the use of the building, or the costs of which would be disproportionally high. Therefore, two basic limit states, corresponding to the above criteria, need to be verified in the design of a structure to resist seismic loads: Ultimate limit state which is associated with collapse or other forms of structural failure which may endanger the safety of people, and Serviceability limit state which is associated with the occurrence of damage, deformations or deflections, beyond which the specified service requirements of the building are no longer met. Because of specific characteristics of masonry structures and their materials, there is usually no need to check the serviceability limit states. Generally, masonry buildings are rigid structures in the case of which even ultimate deformations and displacements are relatively small. In most cases, if a masonry structure is verified for ultimate state, the requirements for serviceability limit will be automatically fulfilled. 6.2 Safety Verification and Partial Safety Factors for Materials The safety of a structure against earthquakes is a probabilistic function which depends on the expected seismic action and ability of the structural system to resist the earthquake. According to EC 6 and EC 8, the following general relationship shall be satisfied for all structural elements: where Ed is the design value of the actions' effects, and Rd is the design resistance capacity of a structural member under consideration. When considering a limit state of transformation of the structure into a mechanism, it should be verified that a mechanism does not occur unless the actions exceed their design values. According to EC 8, the design value Ed of the actions' effects, i.e. the design value of bending moments, axial and shear forces in the seismic design situation is determined by combining the characteristic values of the relevant actions defined in EC 8 and Eurocode 1: Basis of design and action on structures [3]: 97 Basic Concepts of Limit States Verification of Seismic Resistance where Gkj = the characteristic value of permanent action j, i.e. self-weight of the structure (dead load), fixed equipment, etc., AEd = the design value of seismic action for the reference return period, Pk = the characteristic value of prestressing action, if any, 4ii = the characteristic value of variable action i. Snow, wind and fire are not taken into account in the case of an earthquake. y = the importance factor, ~ 2 ,= i the combination coefficient for quasi permanent value of variable action i (live load). In the case of residential and office buildings, ~2 = 0.3, in the case of congregation areas and shopping, however, ~2 = 06. Importance categories and importance factors for buildings y1 are given in Table 6.1. The importance factor = 1.O is associated with the design earthquake having a reference return period of 475 years. Table 6.1. Importance categories and importance factors for buildings (EC 8). Importance I Buildings ~ II 111 ~~ Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc. Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, etc. Buildings of minor importance for public safety, e.g. agricultural buildings, etc. ~ Importance factors, y~ ~ I1 ~ 1.4 1.2 I 1.0 0.8 According to design philosophy of the Eurocodes, characteristic values of material mechanical properties should be taken into account in the calculations of design resistance capacity Rd of masonry structural members. However, to be in compliance with the design actions' effects in the seismic situation, the characteristic values of material mechanical properties are further reduced by partial safety factors for materials YM. The values of partial safety factors for masonry in a normal situation are given in Table 6.2. Table 6.2. Partial safety factors for material properties "(M (EC 6). YM Category of execution C A B Category of I 1.7 2.2 2.7 manufacturing control of I1 2.0 2.5 3.0 masonry units Anchorage and tensile and compressive 2.5 2.5 2.5 resistance of wall ties and straps Anchorage bond of reinforcing steel 1.7 2.2 Steel (Ys) 1.15 1.15 Note: The value of y~ for concrete infill should be taken as that appropriate to the category of manufacturing control of the masonry units in the location where the infill is being used Masonry (see note) i In a seismic situation, however, the values of partial safety factors for masonry strength ym are reduced. The recommended values are specified in Table 6.3. The partial safety factor for reinforcing steel should be taken as ys = 1.O. Table 6.3. Partial safety factors for masonry strength ym in seismic situations (EC8). ICategory of execution I I A I B I C I Basic Concepts of Limit States Verification of Seismic Resistance 99 In the seismic design situation it is clear that, once the safety level of the structure against earthquakes have been decided upon, the design seismic action and design resistance capacity become an interdependent procedure. The philosophy of determination of partial and global safety factors used in the calculation of R d and structural behaviour factors used in the calculation of Ed should be compatible, otherwise the verification according to Eq.(6.1) would lead to 99 Basic Concepts of Limit States Verification of Seismic Resistance of Masonry Buildings wrong conclusions. In order to estimate accurate values of parameters appearing on both sides of Eq. (6.1), realistic data about the seismic behaviour of the structure under consideration and the expected seismic ground motion should be known. Since limit states of structural behaviour are considered, mathematical models should be developed on the basis of the experimentally verified theoretical hypotheses that take into account the characteristics of the actual seismic behaviour of structures. 6.3 Design Seismic Action To determine the design value of seismic action, the presence of all gravity loads (in other words: masses, inducing seismic forces) should be taken into account in the following combination: where: VEi = ψ V2i, and the values of (p are given in Table 6.4. Table 6.4. Values of 3.5 m or n = 4, H < 3.5 m: Ph = 2, n = 4, H > 3.5 m or n = 5, H < 3.5 m: Ph = 4, n = 5 , H > 3.5 m or n = 6, H < 3.5 m: Ph = 6, H < 3.5 m: Ph = 6. The seismicity of micro-location of the building, expressed in terms of design ground acceleration ag is defined by the parameter Pr: ag =0.l5: Pr=0, ag = 0.20: Pr = 1, 3 0.30: ~ g = 0.25: Pr = 9, ag = 0.30: Pr = 15. The seismic vulnerability index of individual buildings is then calculated by (see Fig. 10.17), where Fs is the factor which takes into account the structural system. According to the proposed methodology, Fs = 1.0 for plain and Fs= 0.75 for confined masonry buildings. The method has not yet been calibrated to earthquake-damaged buildings. However, as can be seen in Fig. 10.17, where the assessed values are correlated with the values obtained by detailed seismic resistance verification of a group of existing masonry buildings in the City of Ljubljana [27], reliable estimates can be obtained by this, although simple, method. Recently, expert systems, based on large databases of the values of many parameters of buildings, have been developed [283. In most cases, non-parametric models and neural networks are used to model the seismic vulnerability. 10.4 Methods of Strengthening of Masonry Walls Different technical measures have been developed for repairing and strengthening masonry buildings. Some of them are based only on the analysis of earthquake damage and engineering judgement, and have never been actually verified. Others, however, are based on both earthquake damage observation and experimental investigations, and have been verified either in the laboratory or by a real earthquake. Different methodologies are available for different types of masonry walls. The type and quality of masonry materials and a building's structural layout are the main criteria to be considered when choosing the Repair and Strengthening of Masonry Buildings 217 adequate method of strengthening. However, the choice of the method also depends on the required degree of improving the resistance of the wall. This is indicated by seismic resistance verification and redesign of the existing structure. While efficient interventions in stone-masonry are more or less limited to injecting the grout into the void parts of the walls, various possibilities are available for the repair and strengthening of brick- and block-masonry walls. The procedures can be classified into the following main groups: Repair of cracks. Repointing the joints with cement mortar. Application of reinforced cement coating on one or both sides of the walls. Grouting with cement, modified cement, or epoxy grout. Prestressing the walls in vertical or horizontal direction. Reconstruction of the most damaged parts of the walls. 10.4.1 Repair of cracks According to recommendations given in Annex K to Part 1-4 of EC 8, cracks may be sealed with mortar if the crack width is large (about 10 mm), and the thickness of the wall is relatively small. Otherwise, the cracks should be injected with cement grout which contains admixtures against shrinkage. If the cracks are fine, the use of epoxy is recommended, but if the cracks are larger than 10 m, the damaged area should be reconstructed using stitching bricks. In the case of brick and/or block masonry, the cracks can be injected either with epoxy, if their width does not exceed 1.0 mm, or with cement grout, if their width varies between 0.3-3.0 mm. Taking into account the effectiveness of intervention and the cost of materials, the use of cement grouts has been preferred to epoxy injections for the repair of earthquake-damaged masonry walls. However, experience has shown that the price of epoxy grout can be reduced and application range increased by adding inert filler materials, such as fine-grained quartz sand, to epoxy resins. In order to repair the crack by grouting, the surface of the wall is cleaned and the damaged part of plaster is removed from the wall, and the injection nozzles are built into the holes, drilled into the wall along the crack at 0.3-0.6 m intervals. The surface of the wall is then cleaned, and the cracks are sealed and the nozzles fixed to the wall with fast binding mortar. Before injecting, cracks are washed with water. Usually, the mix consisting of 90% of Portland cement and 10% of Pozzolana, is used. In the case of 218 Earthquake-Resistant Design of Masonry Buildings of grouting of 5-10 mm wide cracks, fine sand can be added to the mix for better results. When injecting the grout, the pressure in the container is slowly increased up to 3 bar and is kept constant until the wall absorbs the grout. After that, the pressure is increased to 4 bar and kept constant for 5-10 minutes in order to densify the mix and to drain excess water. The effect of grouting the cracks on the lateral resistance of brick- and blockmasonry walls has been tested experimentally [29]. It has been found that by injecting the cracks with cement or epoxy grout, the original load-bearing capacity of the walls is recovered (sometimes even improved, as in the case of poor quality masonry), but the rigidity in most cases is not. Therefore, the grouting of cracks in brick- or block-masonry walls represents a typical method of structural repair. Typical results are given in Table 10.3. Table 10.3. Effect of injecting the cracks on tensile strength and shear modulus (after[29]). I Grouted cracks Original Masonry unit ~ ~ Brick (B 20) Brick (B 20) 0.5 3 .0 0.07 0.20 - 0.1 1 0.25 - Ceramic block (B 20) Ceramic block (B20) 4.8 6.1 0.15 0.19 360 240 0.26 0.18 250 380 Light concrete block (B 7.5) 2.9 0.19 380 280 Fly-ash block (B 15) 1.3 1.3 0.14 0.16 370 480 0.14 0.22 230 490 In the case of excessively wide cracks in brick and/or block masonry walls, the damaged area is reconstructed as recommended in Part 1-4 of EC 8, or cracks are sealed with cement mortar or grouted, but in addition, the area around the crack is coated with reinforced-cement coating (Fig. 10.8), as described in subsequent sections. In such a way, good connection between the separated parts of the wall is provided and the integrity of resisting elements ensured. 219 Repair and Strengthening of Masonry Buildings 2 3 m 0 10-0.15 m 0 3 m 3 m 0.10-0.15 m 0 3 m = ~ . . . . , . . . ~ - Figure 10.8.Repairing of heavily cracked brick-masonry wall with r.c. coating (after [5]). Heavily damaged part Rebuilt with original material Figure 10.9. Reconstruction of the central part of a heavily cracked stone-masonry wall (after [S I). In the case of stone-masonry walls, however, reconstruction of parts or complete walls is usually necessary where severe cracks occur in the middle part of the wall, or if that part of the wall has been disintegrated during the earthquake. In the case of reconstruction, original materials, stone and mortar, which are compatible with the materials of the remaining part of the wall, should be used (Fig. 10.9). If, for example, the part of the masonry that is removed is 220 Earthquake-Resistant Design of Masonry Buildings replaced by concrete, the rigid concrete element represents a potential cause of serious damage to neighbouring masonry during the future seismic events. However, care should be taken to provide good connection between the existing and new masonry. 10.4.2 Repointing Where the bed-joints are relatively level, the mortar is poor and units are good, the resistance of a wall to vertical and lateral loading can be considerably improved by replacing a part of the existing mortar with mortar of significantly better quality, e.g. cement mortar. Initially, 10 t3i3 t t3 t3 -il L t3 1 1 t3 t3 t3 tt-t Figure 10.10. Repointing of a brick-masonry wall. For that purpose, existing mortar up to 1/3 of the wall's thickness is removed from the joints on one or both sides of the wall using clamps or electric tools. In order to prevent vertical instability, the wall is made to be supported by temporary wooden supports, and the repointing is first completed on one and then the other side. Care should be taken to avoid horizontal and vertical cut of masonry units and maintain the required verification of stability (buckling) is needed to determine the allowable depth of removal of mortar. Once the existing mortar has been removed, the surface of the wall is cleaned and moistened with water jet. Steel reinforcement is sometimes placed in the bedjoints to improve the ductility and energy dissipation capacity of the structure. In Repair and Strengthening of Masonry Buildings 22 1 the case of normal bed-joint thickness (1.0-1.5 cm), 6-mm bars are placed at 0.3-0.5 m intervals along the height of the wall, adequately anchored at the ends of the wall, e.g. into the vertical tie-columns. The joints are then repointed with cement mortar. After sufficient strength of the mortar is attained, the procedure of repointing is repeated on the other side of the wall (Fig. 10.10). 10.4.3Reinforced-cement coating In the case of serious damaged brick- and block-masonry walls, or where there is a need to strengthen the existing structure, the application of reinforced-cement coating (jacket) on one or both sides of the wall represents a logical way of improving the lateral resistance and energy dissipation capacity of the system. Since the method is easy to apply and very efficient, it has been widely used for strengthening existing masonry walls all over the world. The possibilities of using ferro-cement, and most recently, carbon fibre coating instead of ordinary reinforcing steel, have been experimentally studied. If reinforcing steel is used, plaster is first removed from the wall. Mortar is removed from the joints between the bricks or blocks, 10-15 mm deep, and the cracks in the wall are grouted. The wall surface is cleaned, moistened with water and spattered with cement milk. The first layer of cement coating, i.e. 10-15 mm thick cement mortar layer (compressive strength 20 MPa) is then applied. The reinforcing mesh is then placed on both sides of the wall (4-6-mm diameter bars at 100-150 mm intervals in the vertical and horizontal directions), connected together with steel anchors (6-mm diameter bars, placed in the pre-drilled holes and cemented or epoxied, 4-6 pieces per m2 of the wall's surface). After the reinforcing mesh is connected to the anchors, the second layer of cement coating is applied, so that the total thickness of coating does not exceed 30 mm (Fig. 10.1 1). In a similar way, fibre-reinforced polymer (FRP) coating can be applied. Similar results as in the case of conventional r.c. coating have been reported with regard to the improvement in lateral resistance of brick masonry walls, in the case where adhesion between coating and masonry has been achieved [29]. For better results, reinforcement can be placed diagonally in order to follow the possible direction of cracks. Although both technologies have several advantages in comparison with conventional methods, they have not yet been widely applied because of their relatively high costs. 222 Earthquake-Resistant Design of Masonry Buildings The effect of strengthening the brick- and block-masonry walls by coating has been experimentally investigated in the laboratory and in situ [18, 29, 31, 32]. The experiments, which have all shown that r.c. coating improves the lateral resistance, have also indicated the importance of adequate anchoring of coating reinforcement to the existing masonry. If the connection was not adequate to prevent splitting, the coating separated from the wall at the occurrence of cracks in masonry walls and buckled. ; 0.5 m 10-15 mm Figure 10.11. Application of r.c. coating to stone-masonry wall (after [5]). As shown by experiments, by applying r.c. coating, the shear resistance of the tested walls was significantly improved (Table 10.4). Consequently, in the case of walls with geometry aspect (height/width) ratio greater than 1.0, the failure mechanism changed from shear to bending. As indicated by experiments, the improvement in the lateral resistance is inversely proportional to the quality of the original walls. It is significant in the case of poor-quality masonry, but not so in the case of good-quality walls. Other systems of application of cement coating can also be used (e.g. shotcreting), otherwise the coating is simply applied by concreting. In the latter case, however, the thickness of coating is increased to 80-100 mm. If concrete is poured into forms, reinforcing mesh consisting of 8-10 mm diameter bars at 250 mm intervals, is used. Concrete is used for stone-masonry walls, in the case of 223 Repair and Strengthening of Masonry Buildings which 30 mm thick cement-plaster layers would not be enough to strengthen a two-leaf wall which is more than 50 cm thick. Typical examples of strengthening stone-masonry walls with reinforced-concrete coating are shown in Figs. 10.12 and 10.13. Table 10.4. Effect of coating on the lateral resistance of a wall. Type of Grade Grade Brick I 3 20 I M 0.4 Brick B 10 I M 0 3 C. BlockB 7.5 I M 5 Brick B 20 I M 7.2 Brick B 15 I Resistance cement I Steel I Steel I Steel I Ferro-cem. I Carbon 34 47 128 276 199 I 11 I 118 167 167 693 426 I II I 11 I Multiplier 3.5 3.6 1.3 2.5* 1.4 Note: * Diagonal compression tests [31]. Existing wall Cage reinforcement detail 10-20 cm ~ 5 cm Cage 7 Rc. Figure 10.12.Connecting the r.c. coating to stone-masonry wall with steel connectors (after [5]). In the case of stone-masonry, plaster and loose stones are first removed from the wall and any cavities are ealed or injected with cement mortar or grout, Next Page 224 Earthquake-Resistant Design of Masonry Buildings respectively. Then, holes for steel anchors-6-8-mm diameter reinforcing steel bars placed on either side of the wall-that will connect the coating reinforcement-are drilled through the wall at 50-60 cm intervals. Sometimes, stones are removed from the wall at regular intervals and a reinforcement cage is placed in the chase or the void created by removing the stone. By filling the chase with concrete, a shear connector is formed which efficiently transfers the forces from the new coating to the wall. This system is used where only one-side coating is provided and where heavy and thick stone-masonry makes the drilling of holes for the anchors difficult. Existing Figure 10.13. Application of r.c. coating on one side of the wall with new r.c. slab (after [5]). Before concreting, dust is removed from the wall and the wall is washed with water jet. Depending on the thickness of the coating, the concrete is either shotcreted (80 mm) or poured in the form (at least 100 mm). In the case of shotcrete, the reinforcing mesh (8-10 mm diameter bars at 250 mm intervals) is placed on the finished first layer of shotcrete, the anchors are tied to the bars or welded mesh fabric, and the second layer of the shotcrete is applied. If the concrete is poured in, the reinforcing mesh is fixed to the position before concreting. As the test results and subsequent analyses indicate, it is not possible to estimate the lateral resistance of a coated wall panel by simple calculation. As a rule, the calculated values, obtained on the basis of simple combination of the theoretical lateral resistance of the existing wall and reinforced-cement coating, do not correlate with experimentally obtained test results. Therefore, practical SYMBOLS = area of horizontal cross-section of r.c. column = equivalent area of composite section of infilled frame = design value of action = area of horizontal cross-section of infill = area of horizontal reinforcement = area of vertical reinforcement = area of horizontal cross-section of wall = stress block = peak ground acceleration = width of a building = design base shear coefficient = stiffness degradation parameter = depth of stress block = seismic force factor; Ccff Cl Ccr CR crh crv Cd Dmin d dcr do dmax shear stress distribution factor = coefficient of effectiveness of horizontal reinforcement = interaction coefficient = ReFHmax ratio = masonry infill capacity reduction factor = strength degradation factor = minimum bar diameter = displacement; effective depth of wall = displacement at crack limit = displacement at elastic limit = maximum displacement 257 258 dHmax drv E Ec Ecl e eU e, eY F Fi Fa Fbd Fid Fr Fw f fb fbok fck fcvk fij fm fv fvk fv0 fvko h hk fs fc k 1 fc2 Earthquake-Resistant Design of Masonry Buildings = displacement at maximum resistance = diameter of vertical reinforcing bar = energy = modulus of elasticity; modulus of elasticity of masonry = modulus of elasticity of concrete = eccentricity = design value of action effect = eccentricity = accidental eccentricity = eccentricity in x-direction = eccentricity in y-direction = seismic force, acting at i-th storey = seismic force, acting on the i-th wall in a storey = seismic force, acting on element = design base shear force = design seismic force, acting at the i-th storey = compression force in compressed reinforcement = frequency; compressive strength of masonry = normalised compressive strength of masonry unit = characteristic anchorage bond strength = characteristic compressive strength of concrete = characteristic compressive strength of concrete = characteristic compressive strength of masonry = compressive strength of mortar = shear strength of masonry at zero compressive stress = initial characteristic shear strength of masonry or grout = characteristic shear strength of masonry; tensile strength of masonry; tensile strength of masonry; characteristic tensile strength of masonry; characteristic tensile strength of steel = flexural strength of masonry perpendicular to bed joints 259 Symbols = yield strength of steel = height = horizontal forces = height of storey = horizontal compressive strength of masonry = number of storeys: shear modulus: shear modulus of masonry = shear modulus of concrete = characteristic value (value of permanent action) = acceleration of gravity (9.8 l ms-2) = height of building; lateral load = resistance at crack limit = resistance at maximum displacement = resistance of vertical steel due to dowel action = elastic load = flexural resistance of r.c. column = flexural resistance of reinforced masonry wall = flexural resistance of plain masonry wall = design flexural capacity = maximum resistance of horizontally reinforced wall = tension in horizontal steel = tension capacity of horizontal steel = shear resistance of confined masonry wall; shear resistance of r.c. column = design lateral resistance of r.c. coated masonry wall = design shear resistance of prestressed masonry wall = design shear resistance of a reinforced masonry wall = contribution of horizontal steel = resistance of wall to sliding = shear resistance of plain masonry wall = design shear resistance of infill = resistance of confined masonry panel before the separation of infill = ultimate resistance of confined masonry panel = ultimate design load of plain masonry wall = ultimate resistance of frame 260 Earthquake- Res istant Design of Masonry Buildings = shear resistance of infill = resisting storey shear = ultimate lateral load = frame = height of compressed strut = intensity of earthquake = moment of inertia of r.c. column's section = damage index = equivalent moment of inertia of composite section = moment of inertia of masonry infill's section = moment of inertia of masonry wall's section = characteristic compression strength = stiffness of wall = effective stiffness of wall = effective stiffness of equivalent r.c. coated wall = effective stiffness of masonry wall = effective stiffness of the i-th wall = stiffness of wall at maximum resistance = stiffness of infilled frame = stiffness of wall at w and x- and y-direction = exponents influencing the shape of elastic response spectrum = exponents influencing the shape of design response spectrum = length of building = wavelength = length of span of frame = length of compressed strut = magnitude of earthquake = bending moment of fixed-ended i-th wall = bending moment at the base of the i-th wall = bending moment at the top of the i-th wall = design flexural capacity of wall's section = ultimate flexural capacity of wall's section = flexural capacity of r.c. column's section = prestressing effect multiplier = multiplier for horizontal prestressing = mass at the i-th storey level Symbols 261 = vertical load = number of storeys; number of events = probability of occurrence = characteristic value of variable action i = behaviour factor = behaviour factor associated with structural element = return period = design resistance capacity = soil parameter = seismic coefficient associated with structural element = ordinate of design response spectrum = ordinate of elastic response spectrum = seismic resistance coefficient = spacing of horizontal reinforcement = displacements of masses m and mj in the first mode shape = period of vibration; time = period of vibration of structural element = first vibration mode period of bare r.c. frame without infills = first vibration mode period of structure with infills = tension force in reinforcing steel = i h e s s = weight of building (gravity loads) = weight of structural element = weights of masses mi and mj = co-ordinates of area centre of the i-th wall = co-ordinates of storey mass centre = co-ordinates of storey stiffness centre = section modulus: height of centre of structural element above the base = heights of masses mi and mj above the level of seismic loads 262 Ear thquake-Resistant Design of Masonry Buildings = design ground accelerationfactor of gravity ratio; stiffness degradation parameter = coefficient defining position of moment's inflection point = stiffness degradation parameter = maximum normalised spectral value = importance factor, associated with structural element = importance factor = partial safety factor for masonry = global safety factor = partial safety factor for steel = shape factor; displacement = characteristic value of unit elongation at maximum stress = damping correction coefficient = shear coefficient = ultimate ductility factor = horizontal reinforcement ratio = principal compression stress = design compression stress = average compression stress = compression stress due to horizontal prestressing = compression stress due to vertical prestressing = integral seismic stress = average shear stress at maximum resistance = diameter of reinforcing steel = storey drift rotation angle = storey drift rotation angle = ultimate limit = principal tension stress angle = ultimate storey drift rotation angle = partial safety factor for participation factor = seismic action combination coefficient = action combination coefficient SUBJECT INDEX Index Terms Links A Acceleration response spectrum: design 101-102 elastic 22-24 Acceptable seismic risk 207 Allowable stress method 179 Arch-beam mechanism 130 B Balconies 91 Base shear coefficient 151 Behaviour factor 103 179 105-106 212 Behaviour factor: masonry structures non-structural elements 158 Bond-beams: function 88 dimension 88-89 installation of new 243-244 reinforcement 89 Brick (see masonry units) Bulged stone-masonry wall 233 C Cavity wall 54 Compressive strength 44-47 Compression test of masonry 27 Concrete infill 46 42-43 Strengthening of walls by confinement: installation of tie-columns 247-248 installation of vertical ties 248-249 This page has been reformatted by Knovel to provide easier navigation. 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