CHAPTER – 2

LITERATURE REVIEW

2.1 GENERAL

Masonry is an assemblage of masonry units and mortar. Its properties and behaviour are controlled by the characteristics of masonry units, mortar as well as the bond between them. For the same type of bricks using same proportions of cement and fine aggregate, the strength obtained may be different due to the variation in quality of water, difference in workmanship and on the arrangement of bricks in masonry.

Many earthquake damage reports pointed out the devastating damage to masonry buildings including the recent earthquakes. Due to many natural disasters like earthquakes, most rural houses lacking in the proper building structure were damaged in brittle collapse. Nevertheless, Paulo et al [2006] discussed that the brick masonry is the least understood in the aspect of strength and other performance related parameters because of its complex behaviour and its non homogeneity even in deci-scale. In India about 100 million tonnes of fly ash is generated each year. The Indian government passed a law in October 2005 stating that a minimum of 25 percent of fly ash must be used in the manufacture of clay bricks for use in construction activities within a 50 km radius of coal burning thermal power plants. There are also restrictions on the excavation of top soil for the manufacture of clay bricks. Consequently, the need for the research in material behaviour of brick masonry in India became evident. The study of previous research work is essential in identifying the problem to be investigated and to detect the research gap in the specified field of study.

2.2 REVIEW OF PREVIOUS RESEARCH ON MASONRY

The earlier research works were classified into two different categories: first being the study of physical and mechanical properties of brick masonry and its assemblages; second the effect of in-plane shear behaviour of the masonry wall elements and the wall capacity for un-reinforced and reinforced brick masonry elements with analysis.

2.2.1 Brick

Sarangapani et al [2002] compared the characterization and properties of local low modulus bricks, table moulded bricks and wire cut bricks, mortars and masonry. Leaner mortars such as 1:6:9 cement – soil mortar showed very ductile behaviour which was indicated as the stress-strain curve becoming horizontal after reaching a peak strain value. This indicated that the
presence of a significant amount of soil gave rise to ductility with low strength mortars. Stress-strain characteristics of masonry were examined through prism tests. The modulus of elasticity of brick masonry was found as 265MPa. Simple analysis was carried out to understand the nature of stresses developed in the mortar joint and brick in the masonry. The results revealed that the bricks made around Bangalore had low moduli compared to the cement mortar. This led the masonry where mortar joints developed lateral tension while brick developed lateral compression.

Deodhar and Patel [1997] presented that under compression; mortar deformed more than brick and expanded laterally causing failure of masonry. With the strength of brick and mortar, the compressive strength of brick masonry was evaluated with the constants given. It was found that rich mortar does not improve the strength of masonry but for low strength bricks a mortar ratio 1:4 or 1:5 gave considerably high strength.

Choubey [1993] had done the experiment with brick masonry specimens for flexural tensile strength. The effect of various parameters such as suction rate, type of sand, mortar grade, joint thickness and slenderness ratio on flexural tensile strength of brick masonry were investigated. In the first two minutes, decrease in suction rate was very fast and it became almost constant after an immersion time of five minutes. Maximum strength was obtained by immersion of bricks in water for ten minutes before use which influenced the flexural tensile strength. The behavior was almost similar for all panel specimens irrespective of the type of mortar (1:3, 1:4.5, and 1:6) and size of panel. But the specimens made of richer mortar mixes showed lesser deflections.

Deodhar and Patel [1996] discussed the strength of brick masonry with respect to the strength of the brick and strength of the mortar. Frog in bonding the brick work, shape and size of frog affect the strength of brick masonry. The mortar joint of size 5mm to 10mm gave the maximum strength. The ratio of cement to sand ratio of 1:6 gave reasonably high compressive strength of brick masonry. For mortars richer than 1:6 ratios, though the increase in strength is considerable, the adhesion of cementing materials is very high compared to the benefit of increase in the crushing strength.

Deodhar and Patel [1995] obtained a mathematical model to ascertain compressive strength of brick masonry with that of brick known. The crushing strength of brick prism reduced with the
increase of the cement to sand ratio. A mathematical model in the form of straight line was setup to relate brick strength to brick masonry prism strength as, \( \sigma_{bm} = \sigma_b - 0.5r \) where, \( \sigma_{bm} \) – crushing strength of bricks masonry prism in N/mm\(^2\), \( \sigma_b \) – compressive strength of bricks in N/mm\(^2\) and \( r \) – sum of ratio of mortar mix. Cement to sand ratio of 1:6 was most feasible for mortars in brick masonry. However, the use of 1:8 cement–sand ratios also recommended in framed structure where masonry works as a filler material.

Dayal [1995]\(^21\) described that the fly ash has good shear strength properties and relatively less compressibility. He suggested the usage of fly ash in various modes. With respect to the fly ash bricks, there are two types of bricks which are manufactured from fly ash (i) non-calcinitite bricks (fly ash mixed with bonding agent) and (ii) calcinite bricks (fly ash clay brick). The use of fly ash offered a considerable saving of coal consumption which had been found to vary in the range of 3t – 7t of grade I coal per 10\(^5\) bricks. The percentage of fly ash mixed varied from 10% to 80% and tested for their suitability and 40% by weight of local silty soil found as the optimum percentage of fly ash.

Krishnamoorthy et al [1994]\(^78\) investigated the quantum of fly ash added to soil for making good bricks. Fly ash obtained from Vijayawada thermal power station was mixed with the soil in varying ratios such as 0%, 10%, 20%, 30%, 40% and 50% described that the bricks cannot be manufactured with highly swelling soils without additives. The properties of strength and water absorption of bricks made with replacement of soil by 50% of fly ash were reasonably good and the strengths were ranging from 9.8 to 11.5 N/mm\(^2\) but for the country brick, it was about 3.5N/mm\(^2\) and no marked improvement was there with more addition of fly ash.

Mei-In Chou and Sheng Fu Chou [2006]\(^93\) reported that the paving bricks with 20 % volume of fly ash and building bricks with up to 40% of fly ash were successfully produced in commercial-scale production test. All the final products met the brick plant’s in-house specifications for marketability and far exceeded the ASTM commercial specifications for the severe weather grade. The results showed that the bricks with the fly ash were introduced into the commercial production without acquiring additional machinery, while concurrently reducing plant operation costs. They also suggested that, similar to the regular commercial bricks, the fly ash containing bricks were environmentally safe construction products that can be used for the construction.
Tayfun Cicek and Mehmet Tanriverdi [2007] experimented the fly ash–sand–lime bricks and obtained the compressive strength, unit weight, water absorption and thermal conductivity under optimum test conditions as 10.25 MPa, 1.14 g/cm$^3$, 40.5% and 0.34 W m$^{-1}$ K$^{-1}$ respectively. They suggested that it was possible to produce good quality of light weight bricks from the fly ash of Seyitomer power plant, Turkey. The unit volume weight of the fly ash bricks prepared with quartz sand addition was 1.15 g/cm$^3$, whereas the unit volume weight of the bricks with river sand addition was 1.27 g/cm$^3$. Thus, the unit volume weights of the fly ash bricks were much lower than that of the traditional clay bricks. The water absorption of the fly ash–sand–lime bricks ranged from 30% to 40%. The thermal conductivity of the fly ash–sand–lime bricks was found to be 0.34–0.36 W m$^{-1}$ K$^{-1}$ which was lower than that of the traditional clay bricks. The fly ash–sand–lime bricks produced were suitable for use as construction material. The production of fly ash bricks contributes to the recycling of the fly ash and hence minimizes the negative impact of the fly ash landfills on the environment. On the other hand, the reduction in clay usage for the production of conventional clay bricks helped to protect the environment. Furthermore, the hazardous emissions from the clay brick burning kilns were reduced. The considerably low volume weight and low thermal conductivity of the fly ash bricks will reduce the construction and heating/cooling costs of the buildings.

Mariarosa Raimondo et al [2009] considered the capillarity phenomenon and the suction capacity of brick depends on their micro-structural characteristics, amount, size and shape of pores. Besides some exceptions, the linear relationships between the capillary coefficient $K_s$ and these micro-structural variables substantially confirm the role played by open porosity in increasing the absorption capacities of clay bricks. The capillary coefficient $K_s$, together with the micro-structural variables and phase composition, finally underwent a statistical procedure that confirmed the influence of porosity, as well as coarser pore dimension (in terms of both radius and percentage of pores greater than 3μm) in increasing the liquid adsorbing rate with the highest statistical significance. In addition, the sintering pattern of products, leading to a different amorphous/crystalline phases ratio, proved to be relevant on the definition of the most suitable microstructure: the higher porosity, promoted by the complete CaCO$_3$ decomposition and the smaller pore size, connected with the low sintering degree of clay bricks.
Giulia Baronio and Luigia Bindat [1997] demonstrated that a good degree of hydraulicity of the mortar obtained in ancient mortars were durable for centuries. Modern bricks are seldom pozzolanic, not only because they are fired at high temperature, but also because they can be made of materials which do not contain or have a low content of clays. When the basic material is clay, then a thermal treatment can give pozzolanicity properties in which the temperature and duration of the treatment must be chosen very carefully. The clays were fired for 15 to 30 minutes at two different temperatures at 650°C and 750°C. After treatment, the two clays were subjected to diffractometric analysis and pozzolanicity test. In order to improve the brick characteristics (pozzolanic reaction can take place at brick/joint interface) or simulate the production of pozzolanic materials from common clays useful in the preparation of hydraulic mortars.

Jose Luis Vivancos et al [2009] discussed that the energy consumption of a specific building depends mainly on the building type, climatologic conditions, building construction, occupancy behaviour, installations for heating, cooling, production of domestic hot water and lighting. Heat flux evolution on different types of clay and concrete bricks was studied using a guarded hot-plate. From the data collected a new model to study heat flux was proposed. This model was based on the shape of the typical sinusoidal curves observed for the time dependent heat flux evolution. The heat flux evolution on different types of clay and concrete bricks was studied using a guarded hot-plate based on standards. The model allowed the determination of the thermal resistance ($R_B$), the heat flow for a finite wall thickness in the steady-state ($Φ_∞$) and the time necessary to achieve half of $Φ_∞$ ($t_B$). The proposed model helped to determine the value of $t_B$ in a simple way. The value found showed a linear correlation with the square root of the product between the thermal diffusivity and the geometric characteristics of the brick.

Michele et al [2004] outlined the thermal conductivity of clay and physical or micro-structural parameters which affect their thermal behaviour most significantly. A comparison of the correlation between the thermal conductivity data collected from the literature and those obtained in the present work with the bulk density highlighted that the dependence of thermal conductivity on bulk density, quoted by several authors was not always very obvious and was not able to describe accurately the thermal behaviour of clay bricks. Through a statistical treatment of data, some trends regarding the relationships among the thermal conductivity and the main
mineralogical and micro-structural variables of bricks were revealed. The simple linear binary correlations and the multivariate analyses (factor analysis and multiple linear regression analysis) highlighted the role played by some mineralogical components, in particular Ca-rich silicates (wollastonite and melilite), quartz and amorphous in depressing the insulating properties of clay bricks. On the other hand, among the microstructural parameters, the role of open porosity in improving the thermal performances of bricks was found to be predominant.

Gangadhara Rao et al [1998] made an effort to evaluate the thermal resistivity of class F fly ash using a laboratory thermal needle/probe. The effect of density of compaction and the moisture content on the thermal response of the fly ash were studied. Fly ash was used in conjunction with aggregates to design a proper backfill material. Soil as such was not a good conductor of heat when compared to the metals (normal conductors). Soil thermal resistivity was a measure of the resistance offered by the soil to the passage of heat. Thermal stability was normally related to the ability of moist soil to maintain a relatively constant thermal resistivity when subjected to an imposed temperature difference. Thermal instability occurs when a soil was unable to sustain a rate of heat transfer; to overcome this; the native soil was replaced by materials (backfills) with better thermal properties. When water was added to the ash, it formed a thin film around the fly ash particles that eased the conduction of heat (i.e) increasing its conductivity and reducing its resistivity. This attributed to the fact that the thermal resistivity of air (equal to 4000°C-cm/ W) was higher than that of the water (equal to 165°C-cm/W). The addition of water to the fly ash resulted in decrease in the air voids (and hence the density increases) and as such the thermal resistivity of the fly ash in the near vicinity of its optimum moisture content attained almost a constant value that is the minimum value of thermal resistivity the fly ash can exhibit. At this situation the resultant resistivity of the fly ash, known as “critical moisture content,” was more dependent upon the resistivity of the pore water. There was a rapid increase in the thermal resistivity of the soil, with a small reduction in moisture content as less than the critical moisture content. This critical moisture content depends on the particle size distribution and the density of compaction. From the resistivity-moisture content variations, another important observation was that as compaction density increased, critical moisture content decreased. The critical moisture content values for fly ash were obtained in the range of 28–32%. 

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Henry Liu et al [2009] developed the brick made of pure fly ash and the manufacture of the brick did not involve high temperature heating in kiln, in contrast to manufacturing clay bricks. Consequently, using of greenest brick not only eliminated waste disposal of fly ash and saved landfill space, it also saved much energy and eliminated all the air pollution and global warming problems caused by burning fossil fuel in kilns to manufacture clay bricks. Fly ash bricks made from fly ash do not emit mercury into air. On the contrary, they absorbed mercury from air, making the ambient air cleaner. Fly ash brick did not emit radon gas, but only at about 50% of that emitted from concrete. Thus, it was considered safe to use concrete or concrete products in buildings and it should be even safer to use fly ash bricks. Leaching of pollutants from fly ash bricks caused by rain was negligible. In addition, long-term observation of the compacted fly ash bricks revealed that the long-term growth of strength of fly ash bricks was due to carbonation caused by absorption of CO₂ from the atmosphere which brings relief to global warming.

Obada Kayali et al [2005] compared the properties of fly ash bricks to the clay bricks. The fly ash bricks produced were about 28% lighter than clay bricks. The bricks manufactured from fly ash possessed compressive strength higher than 40MPa. The technology used less energy than that needed in the manufacture of clay bricks. The mechanical properties of the fly ash bricks exceeded those of the standard load bearing clay bricks. Compressive strength was 24% better than good quality clay bricks. Bond strength of fly ash bricks was 44% higher than the standard clay brick. The density of fly ash brick was 28% less than that of standard clay bricks. This reduction in the weight of bricks resulted in a great deal of savings in the raw materials and reduction in transportation costs. The resistance of the bricks to repeated cycles of salt exposure showed zero loss of mass and indicated excellent resistance to sulphate attack.

Kute and Deodhar [2003] found suitable alternative methods of brick manufacturing process to the existing materials. The properties of different proportions of fly ash at different baking temperatures were tested. Also they investigated their effects on compressive strength and water absorption quality of bricks by casting and testing. The two important properties of bricks namely compressive strength and water absorption improved substantially by adding fly ash in proportion of 40% by weight of brick during moulding and burning at 1000°C. It was found that the bricks that were cast using 40% fly ash resulted in optimum strength. Blending fly ash in different proportions with the soil modified its consistency limits. However, the consistency
limits did not have any noticeable effect on compressive strength of the bricks and it was concluded that fly ash can be mixed with any type of soil for manufacturing good quality bricks.

Gregory Majkrzak et al [2007] studied the two major brick fly ash brick properties such as compressive strength and freeze-thaw resistance with the addition of cenospheres a powdered material derived from the fly ash of coal-fired power plants. and is a small. Burning coal produces fly ash containing small ceramic cenospheres, which are particles made largely of alumina and silica. These cenospheres were produced at high temperatures of 1500 – 1750°C through complicated chemical and physical transformations during the combustion of coal. The addition of cenospheres to fly ash bricks resulted in a significant decrease in brick density. For instance, by adding 10% cenospheres, the brick density reduced by 25%. In addition to the improved freeze/thaw durability and lowered density, the uses of cenospheres were suggested in fly ash bricks.

Henry Liu et al [2005] intended to provide a solution to the fly ash disposal problem by utilization of the natural binder that exists in class C fly ash to make bricks and blocks. Having high concentration of CaO, a key characteristic that distinguishes the class C fly ash from the class F fly ash was studied. Depending on the boiler (burner) used, two types of fly ash were generated, the high grade fly ash (less than 0.1% unburnt carbon) which was generated by pulverized-coal boilers and the low-grade fly ash (close to 10% unburnt carbon) which was generated by cyclone boilers. The most effective and economical method to enhance the freeze/thaw property of the fly ash bricks was the use of air-entrainment chemical. By adding only 0.2% of the air-entrainment chemical by weight, the bricks made of high-grade fly ash passed the 50-cycle ASTM standard, and the bricks made of low-grade fly ash passed 40 cycles of the test.

Aeslina Abdul Kadir et al [2010] studied on recycling of cigarette butts into fired clay bricks. The cigarette butts were disinfected by heat at 105°C for 24 hours and then stored in sealed plastic bags. The soil used was brown silty clayey sand prepared for making fired clay and provided by boral bricks pvt Ltd, Australia. Recycling cigarette butts was difficult because there are no easy mechanisms or procedures to assure efficient and economical separation of the butts and appropriate treatment of the entrapped chemicals. An alternative to incorporate cigarette butts in building material such as fired bricks, four different mixes were used for making fired
brick samples. Cigarette butts (2.5, 5, and 10% by weight, about 10 – 30% by volume) were mixed with the experimental soil and fired to produce bricks. The bricks became more porous as cigarette butts content increased. Low-density or light-weight bricks had great advantages in construction, lower structural dead load, easier handling, lower transport costs, lower thermal conductivity, and a higher number of bricks produced per tonne of raw materials. Light bricks can be substituted for standard bricks in most applications except when bricks of higher strength are needed or when a particular look or finish was desirable for architectural reasons. The light-weight bricks produced by incorporating 2.5% to 10% cigarette butts by mass, equivalent to approximately 10 to 30% by volume could be used in different applications according to the required strength. The percentage of cigarette butts increases the dry density and therefore thermal conductivity of bricks decreases. For example, adding 5% of cigarette butts reduced the thermal conductivity by approximately 51%, which was a very significant amount in terms of energy saving. The density of fired bricks found to decrease by 8.3 – 30% when 2.5 – 10% cigarette butts was incorporated into the raw materials. The compressive strength of bricks was reduced from 25.65 MPa (control) to 12.57, 5.22 and 3.0 MPa for 2.5, 5.0 and 10% cigarette butt content respectively. Based on a model developed in this study, using some experimental data from several previous studies, thermal conductivity of the experimental bricks was estimated to reduce by 21, 51 and 58% for cigarette butt contents of 2.5, 5 and 10% respectively.

2.2.2 Mortar

Pitre et al [1995] suggested the utilization of waste materials - fly ash, kiln ash, surkhi, cinder and crushed stone in building construction along with lime and cement offered a viable alternative. Use of these waste materials with lime was investigated to obtain substitutes for the cement mortar. Fly ash mortars with un-slaked lime developed more strength than those with slaked lime and mortars with surkhi and slaked lime gains more than with the un-slaked lime. Lime mortars with kiln ash attained higher strength than all other mortars therefore, it was recommended as a viable substitute to cement sand mortar. Lime mortar with surkhi and fly ash developed adequate compressive strength and are therefore recommended for use in building construction.

Deodhar [2000] presented that the thickness of mortar material and brick material were very important factors that affect the strength of brick masonry prisms in compression. More the thickness of brick material in brick masonry compared to mortar thickness, more the strength of
masonry. The joint thickness of 5mm to 10mm is optimum for metric bricks and for conventional bricks, and there is considerable reduction in strength of brick masonry beyond 10mm joint thickness. Stress–strain curve of brick masonry are similar to that of concrete. Strain corresponding to maximum stress was always higher and the brick strength does not affect the overall strain of brick work corresponding to maximum stress.

Reda Taha and Shrive [2002] suggested that the poor bond and low bond strength was a major weakness of brickwork. This bond was affected by many interrelated factors associated with both masonry units and mortar. Lime present in masonry mortar as a by-product of cement hydration, particularly at the mortar-unit interface, it produces a weak layer. Hence introduction of varying amounts and types of pozzolans (fly ash types F and C and slag) reacts with the lime, produce strong calcium silicate hydrates. The intent was to enhance the bond strength of the masonry by altering the microstructure of the mortar-unit interface. An experimental programme examining the bond strength of mortar-unit joints was therefore carried out using mortars with and without pozzolonas. Statistically significant increases in bond strength were measured at 28, 90 and 180 days with 20% substitution of fly ash in the cementitious materials. No increase was observed with slag. Introducing pozzolonas as a mineral admixture in masonry mortar, besides being an environmentally positive feature, can therefore be beneficial from the rheological, economic and structural points of view. Also it was suggested that the fly ash in masonry mortar improves long-term bond strength. Partial replacement of the portland cement and lime with class F fly ash significantly improved masonry bond strength. Class C fly ash provided limited enhancement to the long-term bond strength. Both materials can provide more cost-effective, high durable, environmental friendly mortar than mortar without fly ash.

Moinul Islam and Saiful Islam [2010] studied cement as partially replaced with six percentages (10%, 20%, 30%, 40%, 50% and 60%) of class F fly ash by weight. Among the six fly ash mortars, the optimum amount of cement replacement in mortar was about 40%, which provides 14% higher compressive strength and 8% higher tensile strength as compared to ordinary portland cement mortar. The rate of gain in strength of fly ash mortar specimens was observed to be lower than the corresponding ordinary portland cement mortar. Fly ash mortar provides satisfactory or higher strength as compared with ordinary portland cement mortar. Use of high volume fly ash in any construction work as a replacement of cement, provides lower
impact on environment (reduce CO$_2$ emission) and judicious use of resources (energy conservation, use of by-product). Use of fly ash reduced the amount of cement content as well as heat of hydration in a mortar mix. Thus, the construction work with fly ash concrete became environmentally safe and also economical.

Mandal and Majumdar [2009]$^{87}$ studied the effect of various parameters such as fluid to fly ash ratio, concentration of alkali activators, curing temperature and duration of curing on the compressive strength of mortar at different ages of 3, 7 and 28 days. 48 hours curing at about 60-70$^\circ$C was optimum for the present alkali activated fly ash mortar. He concluded that concentration of activator fluid and the fluid to fly ash ratio has a great effect on the compressive strength. At higher concentration and at low fluid to fly ash ratio, the strength of the mortar was maximum and the curing temperature increased in the range of 25$^\circ$C to 90$^\circ$C, the compressive strength of the mortar also increased.

Miranda et al [2005]$^{99}$ studied the corrosion potential ($E_{corr}$) and polarisation resistance ($R_p$) values for steel electrodes embedded in Portland cement mortar and two fly ash mortars respectively activated with NaOH and water glass + NaOH solutions. Chloride-free activated fly ash mortars found to inactivate steel reinforcement as speedily and effectively as portland cement mortars, giving no cause to fear that corrosion limits the durability of reinforced concrete structures built with these new types of activated fly ash cement. Activated fly ash mortars inactivate reinforcing steel as rapidly and effectively as Portland cement mortars. The addition of 2% (by binder weight) of Cl$^-$ multiplies the corrosion rate by a factor of 100 approximately. In this case, the $i_{corr}$ values were slightly higher in fly ash mortars, where the chloride content was likewise higher, due to the higher binder/sand ratio in these mortars.

Aravind Galagali [2004]$^5$ reported that the current IS code provided the use of rich mortar (CM 1:6) in the masonry. But such a rich mortar was not essential in the brick masonry. Hence suitable modifications were made and a provision of use of ‘masonry mortar’ which was produced replacing cement by fly ash up to 30% was studied. This obliviously led to saving in the cost of the construction project.

Sanchez et al [2008]$^{124}$ studied the behaviour of mechanical strength and porosity of mortars made with three types of cement fly ash with different content. The trials were carried out under
real conditions with natural pig slurry. The three mortars were submerged at depths of 1m and 3 m in slurry in an experimental lagoon. Control samples were situated in the open air in the natural environment. The variation in the flexural and compressive strength in the specimens was checked at 3, 12 and 24 months by carrying out standardized tests. An increase in flexural and compressive strength in all cements was noted in the two submerged environments as a result of a decrease of pore size produced in the external part of specimens.

Moriconi et al [2003] studied mortars containing either fly ash or ground brick powder as partial cement replacement. Based on characterization results and performance evaluations, recycled-aggregate mortar was superior in terms of mortar-brick bond strength, mainly because of its rheological properties. In addition, the use of fine recycled aggregate instead of natural sand was in accordance with recycling and reuse of building rubble played a key role in meeting the need to complete the building life cycle. Partial substitution of cement with fly ash was tested, so that a comparison between fly ash and brick powder in terms of pozzolanic activity was made. Because fly ash was an industrial byproduct, its reuse was in-line with sustainable development. When recycled sand was used, a higher water dosage was necessary to achieve the same consistence as that of the other mortars, because of the higher water absorption of the recycled sand with respect to the natural one. The tested model was composed of three bricks; bond strength developed during the shearing of a unit with respect to the other along a mortar layer 10 mm thick was evaluated. Experimental results showed the feasibility of using either recycled instead of natural sand or powder obtained by bricks grinding as partial cement substitution for the production of mortars. Excellent bond strength was found, when recycled-aggregate mortar and red bricks were coupled, due to the low thixotropy showed by that mortar and the appropriate pore size distribution of that bricks.

Yilmaz kocak [2010] indicated that fly ash and silica fume have shown different surface features compared to Portland cement. These variations on compressive strength of mortar samples were studied. The ternary use of fly ash and silica fume provided the best performance, when the compressive strength properties of the cement mortars were taken into account. During hydration of cement mortars, calcium hydrate formation is reduced due to fly ash and silica fume substitution, therefore a lower compressive strength was obtained at the early ages when compared to Portland cement. In the following hydration days, fly ash and silica fume having pozzolanic structure bind calcium hydrate in time and turn it into new (pozzolanic) C-S-H gel.
and cause the strength values to reach that of plain cement (except for 30FA coded cement). It is thought that, durable cement and concrete mortars can be produced with these cements without any compromise on strength. Therefore, it will be beneficial to carry out research on other pozzolanas and their properties.

Hayen et al [2008] addressed that during the failure of the masonry the cement mortar remains as quasi-brittle material with its linear elastic deformation, the lime mortar instead transforms into a viscous material with total deformation obtaining values which are up to 50 times higher in comparison to its uni-axial value. The study on the pore structure of the mortar samples showed some evidence for the alteration of the internal structure of the mortar upon tri-axial loading with the ratio of confining pressure to vertical pressure. For the hydraulic lime mortar the decrease in total porosity, a normal variation in material properties of the material, is substantial. The study of the pore structure by mercury intrusion on the other hand showed that the internal structure of the hydrated lime-mortar undergoes an important transformation. The tri-axial loading condition, again in the case of the presence of an important horizontal component, is responsible for the formation of both a network of fine to large cracks and the closing of the medium sized pores.

Wong et al [1999] investigated the effect of fly ash on strength and fracture properties of the interfaces between the cement mortar and aggregates. The mortars were prepared in the proportion of 0.3:1:1.5 (water: cementitious materials: sand) by mass. Fly ash was used as replacement of cement, at the levels of 0, 15, 25, 45, and 55% by mass. Mortar-aggregate interface cubes were tested to determine the splitting strength of the interface. It was found that a 15% fly ash replacement with cement increased the interfacial bond strength and fracture toughness. Fly ash replacements at the levels of 45 and 55% reduced the interfacial bond strength and fracture toughness at 28 days, but recovered almost all the reduction at 90 days. Fly ash replacement at all levels studied, increased the interfacial fracture energy. Fly ash contributed to the interfacial properties mainly through the pozzolanic effect. For higher percentages of replacement, the development of interfacial bond strength initially fell behind the development of compressive strength. Many researchers indicated that low-calcium fly ash (class F) also improves the interfacial zone microstructures, although it is generally coarser and less reactive than silica fume. Fly ash contributed to the interfacial properties mainly by the pozzolanic effect.
Strengthening the interfaces of the brick and the mortar by the high volume fly ash in the mortar account for the long term strength and the excellent durability properties.

Rafat Siddique [2003] investigated the mechanical properties of concrete mixtures in which fine aggregate (sand) was partially replaced with class F fly ash. Fine aggregate (sand) was replaced with five percentages (10%, 20%, 30%, 40%, and 50%) of class F fly ash by weight. Concrete containing fly ash as partial replacement of fine aggregate had not delayed early strength development, but rather enhanced its strength on long-term basis. This study explored the possibility of replacing part of fine aggregate with fly ash as a means of incorporating significant amounts of fly ash. Compressive strength, splitting tensile strength, flexural strength, and modulus of elasticity of fine aggregate (sand) replaced fly ash concrete, continued to increase with age for all fly ash percentages. The maximum compressive strength occurred with 50% fly ash content at all ages. It was 40.0 MPa at 28 days, 51.4 MPa at 91 days, and 54.8 MPa at 365 days. It was suggested that class F fly ash could be very conveniently used in structural mortar.

Katsioti et al [2009] investigated the substitution of the limestone filler with pozzolanic additives in mortars, in specific perlite and fly ash and their compatibility for construction use. The partial substitution of the aggregate with 5% perlite or 5% fly ash in mortars after 28 days seems to be the same for all mortar samples with a tendency for increase in the fly ash containing mortar samples. The elastic modulus was measured at about 4000 MPa with no significant differences for all mortar samples. The elastic modulus being of almost equal magnitude for perlite and fly ash containing mortars and the reference mortar, it was estimated that it results from the compatibility in their mechanical behaviour. The X-ray diffraction, Differential thermal analysis (DTA), Thermogrametric analysis (TGA) and scanning electron microscope (SEM) analysis evaluation of the perlite and fly ash containing mortars, proved the presence of hydraulic compounds: portlandite, calcium silicate hydrate and ettringite.

Abdou et al [2006] carried out an experimental study on half brick couplet specimen as load/unload shear tests to assess the type of the shear behaviour of the joint mortar. In the brick-mortar interface, two failure modes are possible: tensile failure and shear failure. The first led to the joint opening and the latter to the joint sliding with friction. Two types of clay bricks (solid and hollow), made of the same basic material were used in combination with the same type of
mortar in order to study the influence of the holes. The presence of holes does not affect the internal friction angle (ultimate and residual). Indeed, the friction phenomenon occurs in both specimens between the mortar and bricks. The internal walls of the hollow bricks fail step by step so that the hollow specimens do not fail suddenly. For solid bricks it was observed that: as the applied compressive stress was extremely low or near to zero, a brittle behaviour was recorded (sudden failure). When the compressive stress became more important, a quasi-brittle failure of the mortar joint was obtained.

Mike Lawrence et al [2008] examined the bonding properties of a range of mortars with a number of commercially available unfired clay bricks. The materials used for the unfired masonry are largely the same as those used for fired clay bricks as commercial brick manufacturers would prefer to use existing materials and manufacturing plants for the mass production of unfired clay masonry. The clay: sand mortars used without a bonding agent and with thick 300mm walls in traditional earth masonry construction had low bond strengths. These materials cannot be used in thin 100mm wall without a substantial reduction in load carrying capacity. In order to produce a similar structural capacity under lateral loading for a 100mm thick wall, the bond strength increased to 0.20 N/mm². All of the joints, with exception of the sodium silicate mortars exhibited an interface failure when no bonding agent was used. This changed to a majority of joints exhibiting failure within the mortar joint after the interface strength had been increased with the bonding agent. Failure of the sodium silicate mortared joints was through the face of the bricks rather than along the interface or mortar; bond strength was therefore limited by brick strength, indicating that sodium silicate mortar mix used was too strong for these bricks. Mixes with sand to clay as 3:1 had 5% sodium lignosulphonate at 55% concentration added to the clay-sand and water. Three different forms of failure were seen: (i) Interface failure, where the bond between the mortar and brick was weaker than tensile strength of either the mortar or the brick. (ii) Bond failure within the mortar, where the bond between mortar and brick was stronger than the tensile strength of the brick. (iii) Mortar failure within the brick, where both the bond between mortar and brick and the tensile strength of the mortar are stronger than the tensile strength of the brick. Sodium silicate based mortars showed to perform more consistently and to a higher level of performance than other mortar types. In addition, these mortars had low carbon content and are environmentally attractive for use in low carbon construction. The use of sodium silicate mortars with 100mm thin walls appeared promising.
Rajamane et al [2007]\textsuperscript{121} suggested that fly ash acts as a partial replacement material for both Portland cement and fine aggregate. The published information on fly ash as sand (fine aggregate) replacement material is limited and rational guidelines to estimate the compressive strength of concrete are not available. They derived an equation in which the cementing efficiency of fly ash is used based on modified Bolomey equation. The derived equation considered the different levels of replacement of sand and also it was possible to account for cases when the quantity of fly ash added was more than that of sand replaced on weight basis.

Vimal Kumar et al [2005]\textsuperscript{139} suggested the use of fly ash in the manufacture of cement, part substitution of cement in concrete/ mortar, manufacture of bricks etc., at current annual levels. This may save the generation of carbon dioxide by 25 million tonne, good quality lime by 35 million tonne and coal by 15 million tonne a year. It has been found that Indian fly ashes are very low on radioactivity and heavy metal counts. Use of fly ash in cement, mortar and concrete as a pozzolanic material, saves equal amount of cement, which otherwise had been used. Use of fly ash in brick manufacturing would also reduce / eliminate the release of carbon dioxide and other harmful gases. For the manufacture of bricks using fly ash / lime / gypsum / cement etc., no firing is required, which completely eliminates the harmful emissions. Even in the manufacturing of clay fly ash bricks, it was found that there was a fuel saving of 15 to 20%, which would again result in similar savings in emission of carbon dioxide and other gases. With the fly ash through all its utilizations and the huge potential for the environment to be preserved, it can rightly be termed as an environment savior for the society and country.

Tarun Naik et al [1992]\textsuperscript{130} studied the performance of ASTM class C and F fly ash in mortars under varying water to cementitious materials ratio. Four different basic mixtures were proportioned. These mixes were proportioned to have cement replacements in the range of 20 - 40 percent by the weight of fly ash. For each basic mix, water to cementitious materials ratio varied between 0.25 - 5.0. The optimum water to cementitious materials ratio (weight of water divided by total weight of cement plus class C or class F fly ash) was found to range between 0.35 and 0.6 for mixes tested in their investigation. The compressive strength increased with increasing water to cementitious ratio up to 0.57 and then diminished for both 7-day and 28-day test ages. The data presented above revealed that increase in class F fly caused reduction in mortar strength, because of the slower pozzolanic reaction that occurred at an early age due to
poor reactivity of the class F fly ash used. Improvement in compressive strength was not achieved even at 20% class F fly ash with cement replacement.

Yucel [2006] described the behaviour of concrete under flow. The mortar can provide cohesion of the concrete, but should be fluid enough for seeping through the concrete and forming a gliding layer on pipe wall. The mortar possessing these properties had a low yield value and moderately high plastic viscosity. The cohesion of the mortar increased, as fine mineral additive such as fly ash was added, and the fluidity improved, by adding plasticizer or super plasticizer. Fly ash addition also improved the impermeability of the concrete to water and chloride ions. The chloride diffusion was the main cause of the embedded steel corrosion in reinforced concretes; the chloride ions suppress the beneficial effect of alkaline passivation due to the presence of Ca(OH)$_2$ of the hydrated cement. Minimum cement content was needed for obtaining the pozzolanic efficiency of the fly ash. Replacement of the cement by fly ash decreased the compressive strength of the concrete. Investigations were made on methods of measuring viscosities of fresh mortar and pipe flow of mortar as a fundamental study for rationalization of fresh concrete behaviour. The rheological constants of mortar and concrete of relatively wet consistency measured fairly satisfactorily with the double-cylinder rotation. The flow curves of the mortar confirmed the Bingham model.

### 2.2.3 Brick masonry

Milad Alshebani and Sinha [2000] conducted tests on half-scale and plast brick masonry specimens, subjected to cyclic biaxial compression in which the angles between the principal stresses and the bed joint were limited to 90° and 0°. Masonry under biaxial stress was representative of a number of walls subjected to in-plane loads which mostly occur cyclically (e.g. seismic loads) thus inducing a cyclic biaxial stress state in various regions of the masonry structural element. Failure of the specimens was characterized by splitting at mid thickness of the bearing area.

Jagadish et al [2002] examined an additional feature known as containment reinforcement which controls the post-cracking deflections and impart flexural ductility of masonry walls. Model-1: had no earthquake resistant features: one of the cross walls collapsed after the fifth base shock and the other after 8 base shocks. Model-2: provided with the horizontal band: this model never developed vertical flexural crack propagating from the top edge and above the lintel
level. However, a lot of horizontal cracks were formed, particularly between the lintel and sill band and below the sill band. Model-3: with containment reinforcement: It withstood 60 base shocks without collapsing, although it developed a large number of cracks. This model also developed horizontal cracks below the lintel and sill level; however they were prevented from growing significantly by the containment reinforcement. Masonry buildings in mud mortar or lime mortar are prone to severe damage due to lack of bond strength. Use of rounded stones in withes without through-stones can further aggravate the problem. The failures of such structures were essentially due to out-of-plane flexure. Masonry with cement mortar (which has higher bond strength) generally behaved better. The provision of corner reinforcement in corners and junctions as suggested by BIS, has to be properly bonded with the surrounding masonry possibly with dowels or keys to prevent separation. Since the brittle nature of masonry buildings is the major cause for collapse of buildings and loss of lives, there is a need to introduce remedial measures in the construction of such buildings. The horizontal bands are helpful in tying the walls together at the junctions and also in preventing the growth of vertical cracks and in-plane shear cracks.

Bryan and Mervyn [2004]\(^{16}\) captured the stress-strain characteristics of unconfined and confined clay brick masonry. Confinement plates dramatically improved the compressive strength of clay brick masonry. The plates increased the ultimate strength by as much as 40%. It was noted that confinement plates placed within the mortar bed joints restricted the lateral expansion of the joint and the differential expansion between the clay brick unit and the joint.

Fatma E-Refai et al [1984]\(^{41}\) attempted to explore the degree of compliance between theoretical values and measured values yielded from computer programmes and experimental tests respectively. The theoretical analyses were based on the lateral interaction between the bricks and mortar and concluded that with mortar softer than bricks, the masonry compressive strength increased as the brick height increased. As the mortar rigidity increased higher than that of bricks, it introduced lateral restraints at the ends of bricks and consequently the masonry strength increased. It was concluded that when using soft mortar, the thickness of horizontal joint kept to a minimum as masonry strength was a primary concern.

Mojsilović [2005]\(^{104}\) derived masonry characteristics from compression tests and examined the stress-strain relationship and the applicability of orthotropic elasticity to masonry. It was
concluded that masonry behaved more or less as a linear-elastic material, in particular for working loads (loads up to 30% of the failure load); for higher loads concrete and calcium-silicate block masonry exhibited nonlinear behaviour, while clay brick masonry remained linear-elastic up to failure. At the same time, concrete block masonry assumed to be isotropic, and calcium-silicate block and clay brick masonry to be orthotropic materials.

Durgesh Rai and Subhash Goel [2007] used a simplified mechanics model to obtain the system capacity curve for an unreinforced masonry wall in which rocking piers were stabilized. The undesirable compressive modes of failure of stabilized rocking piers at larger drifts were eliminated by the use of yielding energy dissipation device to limit the forces in verticals and thereby the compression force in rocking piers. The rocking resistance increased with lateral displacement as the pier compression reaches a peak value. The model was first used to obtain the load-deflection curves of each rocking pier in the story, which are then simply added to obtain the story capacity curve. Further, the system capacity curve for the entire wall was simply derived by assuming the pattern of story displacements after the first mode shape as it is the dominant mode for earthquake response in most structures. The capacity spectrum method was then used to estimate the seismic demand on the rocking pier system described by the system capacity curve. A strengthening scheme using steel vertical elements and energy dissipation devices have been proposed to enhance seismic performance of rocking piers which may be inadequate. A rocking pier stabilized by vertical elements maintains its deformation controlled behavior, ductility and stable hysteretic performance, despite significant enhancement in its lateral strength. This strengthening system was shown to possess considerable load sharing between the masonry and the added elements at almost all load stages.

Mohamad et al [2005] carried out experimental tests on masonry prisms to determine the response of masonry subjected to compression. The stress-strain diagrams were obtained with prisms made of concrete blocks and a wide range of mortar strengths. Here the cement : hydraulic lime : sand proportions in volume of the mortar type 1:0.5:4.5 agrees well with experimental results, while mortars 1:0.25:3 and 1:2:9 exhibited reasonable agreement for the initial stress but only moderate agreement close to the ultimate stress. The failure mechanism of masonry depends on the difference of elasticity modulus between unit and mortar. The mortar governs the non-linear behavior of masonry. A polynomial expression was the best fit curve
between the elasticity modulus and compressive strength of masonry. This demonstrates that there was a non-linear relation between strength and the elasticity modulus. The initial tangent modulus of masonry obtained from transformed hyperbolic stress-strain diagram shows values rather similar for prisms built with mortar types 1:1:6 and 1:2:9.

Hemant et al [2007]55 developed a simple analytical equation by regression analysis of the experimental data to estimate the modulus of elasticity and to plot the stress–strain curves for masonry. Hand-moulded burnt clay solid bricks were used in constructing masonry prisms. Different grades of mortar (cement: lime: sand by volume) used in the study were: 1:0:6 (weak), 1:0:3 (strong), and 1:½:4½ (intermediate). The compression testing was performed according to ASTM specifications, which are quite similar to those given in the Indian masonry code (IS: 1905–1987). Load and displacement measurements were recorded in real time using a computer-based data acquisition system. Stress–strain curves obtained from the experimental testing were summarized including prism strength ($f_m$), failure strain and modulus of elasticity of masonry ($E_m$). Performance of masonry with intermediate mortar, with lime content was much better than that of masonry with the other two mortar grades. Prism strength of masonry with intermediate mortar was only about 13% lesser than that with strong mortar, while strain failure was about 50% more. A significant improvement in ductility of masonry was observed because of the presence of lime in the mortar without any considerable reduction in its compressive strength. This showed that lime in the mortar offered distinct structural advantages. The compressive strength of masonry was found to increase with the compressive strength of bricks and mortar. The trend was more prominent in case of masonry constructed with weaker mortar. Therefore, using a mortar grade of higher strength than required may not always produce high-strength masonry. Masonry with lime mortar was found to undergo about 50% more compressive deformation than that constructed using mortar without lime, while the reduction in compressive strength was only about 13% when lime mortar was used. Therefore, adding lime to mortar was a recommended practice in masonry construction. Experimental verification was required for extension of these results for bricks found in other regions, and mortar of different grades.

Oliveira et al [2000]111 carried out the tests on prisms under cyclic loading in order to evaluate the importance of stiffness degradation. The results obtained from each masonry component (bricks and mortar) were compared with the results from the masonry prisms and presented the
specimen’s behaviour based on the failure modes. The stress-strain diagrams of the brick prisms showed a bilinear pre-peak behaviour. Peak load was preceded by visible crack initiation, and post-peak was characterized by a stable behaviour. While the extreme bricks presented slight damage, the central bricks were very damaged with visible cracks along the entire surface and aligned with the load direction. Stiffness degradation of the reloading branches occurred especially during post-peak, where stiffness suffered important decreases. It was observed that energy dissipation increased with the strain level. The average strength value of the prisms was much higher when compared to the mortar specimens, but less than the average strength of the single bricks. Mortar had a very large influence on prism deformation. A reduction on the peak strength was compensated by stable post peak behaviour. The compressive strength of the masonry was highly influenced by the characteristics of the single components - brick and mortar.

Lourenco [2004] conducted the triplet test to assess the shear behavior of stack bonded masonry with micro-concrete joints. Standard masonry bond was the running bond, which results in discontinuous vertical joints. Typical failure modes were obtained and the shear strength adequately followed Coulomb friction law. Therefore, both the use of a stacked configuration and the use of micro-concrete for the joints were acceptable. The mechanical strength parameters that characterize the interface of the joints were cohesion ‘c’ of 1.39N/mm² and a tangent of the friction angle tanΦ of 1.03. According to European code (prEN1052-3 1996), the characteristic value of the cohesion ‘c’ is 1.11 N/mm². It was also found that the dilatancy of the masonry micro-concrete joints in the stack bond configuration was similar to standard masonry.

Ritchie and Davidson [1963] presented the influence of properties of bricks and mortars on leakage and bond strength. The resistance of brick masonry to moisture penetration and the strength of bond between brick and mortar primarily depend on the properties of the materials used and the manner in which brick and mortar were brought together in masonry. Leakage took place through the brick, depending on its permeability, but more usually it occurs through channels at the brick mortar interface. Factors affecting the condition of the mortar include the rate of water absorption of the bricks on which the mortar was spread, the inherent resistance of the fresh mortar to loss of moisture (water retention value), the amount of water in the fresh mortar, the thickness of the mortar bed, the length of time that elapses before a brick was placed in the mortar and the energy used to bed a brick. The strength of bond between brick and mortar
also depends on the nature of the bond between the two. A particular brick and mortar combination may however, have a complete extent of bond at interface and have relatively low strength of bond, whereas another combination may have a patchy or incomplete extent of bond with greater strength. The extent of contact between brick and mortar and the tensile strength of the mortar influenced the bond strength. The extents of these effects were difficult to assess because the factors were interdependent.

Gumaste et al [2007] attempted to study the properties of brick masonry using table moulded bricks and wire-cut bricks of India with various types of mortars. The strength and elastic modulus of brick masonry under compression were evaluated for strong-brick soft-mortar and soft-brick strong-mortar combinations. Various sizes of prisms and wallettes were tested to study the size effect and different bonding arrangements. It was concluded that for table moulded brick masonry, the failure of masonry specimens using lean mortar was primarily due to loss of bond between brick and mortar. In the case of 1:6 cement-sand mortars, the specimens showed failure due to splitting of bricks. The masonry efficiency was in the range of 17.7% to 31% for prisms and in the range of 20–27% for wallettes. Due to a large coefficient of variation for the brick strength (40%), the crushing of the weakest brick in the specimen determined the masonry strength rather than the interaction between brick and mortar, and mortar strength influenced the masonry strength. The secant modulus (at 25% of ultimate stress) of prisms was observed in the range of 345–467MPa. A larger scatter in the range of 260–735MPa was observed in case of wallette specimens. The poor correlation in the strength pattern and the scatter in the modulus values of table moulded brick specimens attributed to the large coefficient of variation in the brick strength. It was concluded that wire-cut brick masonry exhibited a better correlation between mortar strength and masonry strength. The stack bonded prisms and English bonded prisms showed masonry efficiencies in the range of 21–43% whereas the stretcher bonded wallettes had efficiencies between 35-53%. The wire-cut brick masonry showed a lesser scatter in the strength values as compared to the table moulded brick masonry specimens. Relatively higher secant moduli values in the range of 2393–5232 MPa were observed in these specimens. Specimens with 1:6 cement mortar 1:1:6 cement–soil mortars failed due to loss of bond between brick and mortar, whereas the specimens with 1:½:4 cement–lime mortar failed by diagonal shear failure and splitting of bricks. The results provided a guide to the design of brick masonry in a developing country like India, where low/moderate strength bricks were common.
Mosalam et al [2009] investigated the mechanical properties of masonry which was a heterogeneous composite in which brick units made from clay, compressed earth, stone or concrete were held together by mortar. Mortar of lime or a mixture of cement, lime, sand and water in various proportions were used. Consequently, masonry properties vary from one structure to the next depending on the type of brick units and mortar used. For each type of brick units and mortar, their properties depend upon the properties and composition of the constituents. The interface between mortar and unit was known as the weak link in the system with minimal tensile bond strength, thus masonry had limited tensile strength and is usually negligible. Under uniaxial compression, state of stresses in the brick in a masonry prism was compression-tension-tension; whereas softer mortar joint was under tri-axial compression. Under tension, masonry was linear elastic material; tensile failure was characterized by splitting along the interface. Masonry exhibited nonlinearity under biaxial loading. Mohr-Coulomb model was appropriate for modeling shear behaviour in the joint. Cyclic loading reduced compressive strength of masonry prism by 30%. Shear behaviour of masonry depends upon normal stress; under high normal stress, dilatancy was insignificant.

Maurenbrecher [1980] described the effects of various factors on prism strength. The Canadian masonry design standard for buildings allows two methods of determining compressive strength of masonry, (i) tabular values based on unit strength and mortar type, (ii) axially loaded prisms such as two-course blockwork stacks. The latter method was more accurate and usually gave higher allowable design stresses, but in the result a number of factors were considered. Although the pressed brick prisms did not show a large difference between mason built and jig-built prisms, a large change was observed for the extruded brick prisms. It was even more marked in terms of characteristic strength, since the variation in strength is greater for the mason -built prisms. The characteristic strength of these prisms was 23.2 MPa in comparison with 40.3 MPa for the jig-built prisms. The masonry standard tabular value, based on unit strength and mortar type was also higher than for the mason-built prisms as 25 MPa for Type S mortar and characteristic brick strength of 99 MPa.

Elizabeth Vintzileou and Eleni-Eva Toumbakari [2001] investigated the effect of deep rejointing on the behaviour of brick masonry subjected to axial compression. In order to determine the initial compressive strength of masonry prisms, three of the specimens were tested
in axial compression as constructed. In the remaining prisms deep rejointing was applied as follows: The mortar was chiseled out of all horizontal joints along the four faces of the specimens, at a depth of approximately 40mm. Thus, almost 35% of the total volume of mortar was removed. Loose mortar pieces and dust were removed using air under pressure. Subsequently, the prisms were saturated and the rejointing mortar was mixed and introduced to the joints by hand. All masonry prisms were subjected to axial compression. In all specimens, typical vertical cracks due to compression appeared both along the length and the width of prisms. In addition to those cracks, spalling of bricks was observed in prisms to which deep rejointing was applied. The spalling of bricks was attributed to the concentration of stresses in the region of the new mortar having substantially higher strength and modulus of elasticity than the old one. Such a concentration of stresses was more pronounced in case of defective application of deep rejointing, in which case horizontal joints remain partly void. Deep rejointing leads to substantial increase of the compressive strength of masonry, provided that horizontal joints were completely filled.

2.2.4 Masonry wall
Mustafa Taghdi et al [2000]\textsuperscript{107} described the retrofitting measures taken to strengthen the unreinforced walls and partially reinforced walls. The retrofitted walls with steel strip system of diagonal and vertical strips were attached with through-thick bolts. Stiff steel angles and anchor bolts were used to connect the steel strips to the foundation and the top loading beam. All walls were tested under combined constant gravity load and incrementally increasing in-plane lateral deformation reversals. The lightly reinforced concrete walls were also repaired using only vertical strips and retested. These tests showed that the complete steel strip system was effective and significantly increasing the in-plane strength and ductility of low-rise unreinforced and partially reinforced masonry walls and lightly reinforced concrete walls. The capability of unreinforced masonry walls to resist lateral loads is limited by the strength of both masonry units and bed joint mortar. Shear failures can be eliminated by using heavy horizontal reinforcement and relatively lighter vertical reinforcement, thus promoting flexural behavior.

Sivarama Sarma et al [2003]\textsuperscript{129} said that confined masonry panels in a building were considered to provide better, cost-effective, seismic resistant structural elements as confined column in hollow block masonry shear wall improved the ductility and shear load characteristics. In case of panels with opening, however the ductility of hollow block panel was superior when compared
with the brick panels. The vertical reinforcement had significant influence in improving the ductility behaviour. The ultimate lateral load was governed by shear failure when compared with flexural capacity, even in the case of fully reinforced brick wall panel systems.

Shambu Sinha [2006] investigated the most common retrofit technique applying a 76mm thick layer of shotcrete to either the outside or inside surface of the walls to one surface strengthened to provide earthquake resistance. The shotcrete greatly increased the strength of the unreinforced brick panels. Panels reinforced with the welded wire fabric showed a significant increase in strength after first cracking and large inelastic deflection capacity. The shotcrete plus reinforcement permitted the panels to deflect in-elastically and to remain intact even after the full reversed cycle loading. Bond strength between the shotcrete and bricks directly influenced the strengthening of the structural panels including its stiffness properties.

Khan Mahmud [2007] conducted the experiment and investigated the use of ferrocement laminates for repairing and retrofitting masonry infill. It was concluded that ferrocement overlay was a highly effective method of strengthening/repairing distressed reinforced concrete frame with masonry infill. Since the tested capacity of the repaired frame was more than the capacity of the original frame, it was quite logical if ferrocement overlay was applied to any existing distressed infill, the lateral load capacity of the frame significantly increased.

Gabor et al [2006] described that the shear behaviour of unreinforced masonry panels strengthened by fiber reinforced polymer composite strips in diagonal compression. Three types of fiber reinforced polymer composites were employed: a unidirectional glass fiber (noted RFV), a unidirectional carbon fiber (noted RFC) and a bidirectional glass fiber (noted RFW). The global behaviour was described by the applied load vs. strain along the compressed diagonal curve. It is quasi elastic with a very weak yield plateau. Indeed, the failure strength was conditioned by the shear strength induced by the interaction of the mortar notches with the internal wallettes at the brick/joint interface. The load corresponding to the elastic limit and the ultimate load of the reinforced panels are much higher than one of the unreinforced panels. The gain in strength was quite remarkable: 42% for the RFV reinforcement and over 65% for the RFW. The deformations corresponding to the maximum loads of the reinforced walls are three times higher than those of the unreinforced walls. Therefore, the seismic behaviour is enhanced. The panel reinforced with the RFV composite failed suddenly due to a cracking along the
compressed diagonal at the ends of the composite strips. The two other walls, strengthened with RFC and RFW strips were failed locally at the compressed corners in the loading shoes. Finite element modeling was developed with commercial software for the analysis of the behaviour of unreinforced and fiber reinforced polymer composites strengthened masonry walls subjected to a predominant shear load. The obtained numerical results validated experimentally in the case of diagonal compression test of masonry panels.

Valluzzi [2002]\(^{138}\) conducted experiments to study the shear behaviour of masonry brick panels using fiber reinforced polymer laminates and tested for the shear strength by the technique of diagonal compression. The finite element modeling was studied on masonry panels and offered a limited effectiveness. The diagonal configuration was more efficient in terms of shear capacity than the grid set up; however the latter offered a better stress redistribution that caused crack spreading and a less brittle failure.

Corradi [1999]\(^{19}\) experimented on the strength of masonry brick panel of various buildings struck by 1997-1998 earthquake typically at that part of Italy. Tests were performed in two parts: in the laboratory and in-situ, in order to determine the correct parameters describing masonry behaviour. The walls were tested under diagonal compression and shear–compression. These tests involved the use of panels of two different dimensions: 120x120 cm\(^2\) for the diagonal compression tests and 90x180 cm\(^2\) for the shear–compression tests. All panels were cut using the diamond-wire technique and isolated from the remaining masonry walls in order to leave the panels undisturbed. Regarding the solid brick panel, it was significant to note that the particular brick texture caused a nominal shear strength \(\tau_{k}^{\text{diag nom}}\) of 0.069 MPa. Working with three couples of results related to the three above mentioned buildings, the average ratio of diagonal compression test and shear compression test was equal to 2.06.

Maria Rosa et al [2005]\(^{88}\) proposed a strengthening technique based on the insertion of steel bars in the bed joints. It is particularly suitable for regular brick masonry showing a critical crack pattern due to high compressive loads. Experimental tests and numerical analyses showed that the presence of the bars allowed control of the cracking phenomena, keeping the structure in the desired safety conditions. Both experimental and numerical analyses showed that the most significant result concerns the reduction of the tensile stresses in the bricks and of the dilatancy of the wall.
Gabor [1998] studied the shear behaviour of hollow brick masonry panels. The panels were subjected to horizontal loading and the out of plane failure and the diagonal tensile failure was studied. Finite element modeling was done with the elasto plastic properties of the mortar joints cohesion, and residual friction was studied. It was concluded that finite element modeling approaches with a good accuracy with respect to the behaviour of masonry panels, ultimate loads, ultimate strains, plastic strain evolution and failure modes.

Essy Arijoeni Basoenondo, [2008] investigated the behaviour of brick masonry wall without surface mortar, with surface mortar and added with different surface mortar plaster for monotonic, repeated and cyclic loading. It was concluded that the capacity of the wall under cyclic loads was 50% less than that under monotonic and repeated lateral in-plane loads. All walls collapsed due to brittle failure mechanism, without the presence of ductility. It was also recorded that the presence of surface mortar plaster as wall confinement system increased the stiffness of the wall, but did not affect the improvement of wall ductility. Various kinds of masonry walls were suggested for different seismic zones in Indonesia.

Navaratnarajah Sathiparan et al [2005] conducted a series of diagonal compression tests and out-of-plane tests using non-retrofitted and retrofitted wallettes by polypropylene (PP) band meshes. The retrofitted wallettes achieved 2.5 times larger strengths and 45 times larger deformations than the non-retrofitted wallettes did. In out-of-plane tests, the effect of mesh was not observed before the wall cracked. After cracking, the presence of mesh positively influenced the behaviour wallettes. In the retrofitted case, although the initial cracking was followed by a sharp drop at least 45% of the peak strength remained. After this, the strength was regained by readjusting and packing by PP band mesh. The final strength of the specimen was equal to 1.2kN much higher than initial strength of 0.6kN. The retrofitted wallettes achieved 2 times larger strengths and 60 times larger deformations than the non-retrofitted wallettes.

Totoev and Nichols [1997] constructed three high - stack bonded masonry prisms from seven different brick types. Prisms were made using a mortar with the following properties by volume, 1:1:6 (cement: lime: sand). The water cement ratio was maintained in the range of 1.9 to 1.96. The masonry panels used in the experiments are square panels. These panels were 1200 millimeters on each side. The final test program was undertaken using a standard, commercially available, solid-pressed brick, which was 230 x 110 x 75 mm. Panel construction, used a high
quality research grade mortar. The measurement of the damage variable appeared feasible with the shear rig developed at the University of Newcastle. The initial results from panel 246-c suggested a slow degrading of failure, as tensile split along the plane of the first principal stress, under the test conditions. The use of a 30-second sinusoidal test pattern of varying amplitude and frequency provided acceptable results.

Nichols and Beavers [2003] developed a procedure to estimate the probable death rates in some earthquake events for a given set of circumstances. A number of factors affecting the level of the death toll were identified for these tragic events. These factors were, the timing of the events, and the time between events impacts on the death toll. The felt intensity had a casual relationship to the death toll through the rate of building collapse. The magnitude of the earthquake, the distance between each population area and the epicenter, and the distribution of energy release relative to the population’s location affected the death toll. The construction quality of the buildings and the ground conditions affected the fatality rates in the different buildings or areas. The number of people exposed to the earthquake inside buildings also affected the death toll. The similarities between geological conditions and the soil conditions are of interest in assessing the impact of historical earthquakes. A simple equation was established and calibrated the fatalities in earthquakes having tolls lower than the bounding function. This equation and the calibration data, essentially for unreinforced masonry and timber-framed buildings, provides a procedure for estimating fatality counts in future theoretical events with a specific combination of circumstances. A function has been established that relates the earthquake magnitude to the high fatality count events of the twentieth century.

$$\log(\Xi (M)) = 9.335M - 0.577M^2 - 32.405$$

The function had a regression coefficient of 0.95 for a fatality count of \( (M) \) \( B \) \( \Xi \) and an earthquake magnitude \( M \). This interpolation function, \( (M) \) \( B \) \( \Xi \), provided fatality estimation for an earthquake of magnitude \( M \) to predict the future losses in human terms for specific earthquakes and conditions near or in an urban area. The function was informally estimating the likely fatalities in earthquakes. The function provided a basis to allow the real time estimating of potential earthquake losses for planning purposes.

Badoux et al [2002] investigated the dynamic in-plane behaviour of unreinforced masonry walls. Half scale hollow clay masonry walls were subjected to a series of simulated seismic
motions on an earthquake simulator before and after upgrading with glass and carbon fiber reinforced plastics. The wall started to rock under a lateral load of about 27 kN. This lateral force doubled at failure as the normal force doubled. Generally, the presence of the glass fibre reinforced polymer (GFRP) system prevented development of cracks through the wall panel itself, i.e. the wall didn’t experience observable damage until masonry crushing at the bottom corners. Rupture of the glass fiber reinforced polymer at the wall base was simultaneous with crushing of the masonry wall. In this respect, the upgraded wall reinforcement was “balanced”. The lateral drift at failure was about 1%. The use of a classic flexural beam model with an elastic-plastic material stress deformation law (stress block approach) gave a good estimate of the lateral resistance of the masonry wall in rocking failure. As expected, no significant asymmetry or out-of-plane behaviour was observed even though the wall was strengthened on one side only. The hysteretic force displacement relationship was linear and the loops indicated the small energy dissipation. Three half-scale unreinforced masonry walls were subjected to a series of simulated earthquake motions on an earthquake simulator. The first wall was a reference specimen without upgrading, the next two were upgraded with glass fiber wrap and carbon fiber laminates. Test confirmed that wall rocking was a stable nonlinear response in slender unreinforced masonry walls, providing significant lateral deformation capacity. In spite of relatively poor mortar, the wall friction coefficient exceeded 0.55. The presence of one-sided glass fiber wrap in the wall improved the lateral resistance by a factor of about two.

Pankaj Agarwal and Thakkar [2001] demonstrated the differences in the behaviour of brick masonry model subjected to either shock table motion or quasi-static loading. The shock model responds with a significantly higher initial strength and stiffness as compared to the quasi-static model subjected to equivalent lateral displacements. Severity of damage was greater in quasi-static test due to increased crack propagation. The shock test suggested that at low levels of excitation at the base, acceleration gets amplified at the roof, with an almost elastic behaviour of the model. Marked reduction in both strength and stiffness has been observed when the model was loaded statically rather than dynamically. The crack patterns obtained under both the test methods were nearly similar.

Jagadish et al [2002] attempted to evaluate the behaviour of masonry structures based on the type of masonry used in places like Bhuj, Anjar, Bhachau, Morbi, Samakhyali and several other
places. A variety of masonry structures suffered damage during the recent Bhuj earthquake. Some of the traditional masonry structures had no earthquake resistant features and suffered considerable damage. The behaviour of masonry buildings after an earthquake was investigated so as to identify the inadequacies in earthquake resistant design. In the new Bhuj town, most of the one or two-storied buildings using brick/stone in cement mortar behaved reasonably well with minor cracks. The cracking and failure patterns of various buildings were examined. A three-storied stone masonry building with cement mortar, which had performed rather well, while a near-by framed RC structure, had collapsed. Since the brittle nature of masonry buildings was the major cause for collapse of buildings and loss of lives, there is a need to introduce remedial measures in the construction of such buildings. The horizontal bands are helpful in tying the walls together at the junctions and also in preventing the growth of vertical cracks and in-plane shear cracks. The concept of ‘containment reinforcement’ was developed to contain the flexural tensile cracks from growing. This helped in imparting ductility and in absorbing a lot of energy during earthquakes.

Emília Juhasovaa et al [2002] investigated the seismic effects on masonry structures to real structure response during earthquake or intensive artificial seismic excitation. The model was designed as an asymmetrical one with two rooms in the first storey and one room in the second one. The retrofitting procedure applied special lime cement fiber plaster reinforced by plastic grids. The applied mineralized polypropylene fiber ‘DIMAPOS’ was especially produced for fiber concrete or fiber mortar. This synthetic fiber of staple type was produced from isotactic polypropylene. The fiber had circular cross-section and its surface was hydrophilysed because of its basic properties in concrete and plaster products, substantial decrease of cracks created during prematuring and in ready product; higher impact toughness (200% increase); lower abrasion of the product surface; higher resistance against fracture of edges; higher surface hardness (about 15% increase) and higher resistance against penetration; lower thermal conductivity (about 13% decrease); higher tension strength in bending (about 10–15% increase); lower water seepage, higher frost resistance and higher resistance of fiber against alkali, acids and solvents. Properties of tested fiber mortar samples confirmed that they have much higher resistance against origin of cracks and their development. Brick or other masonry buildings had load carrying system based on compression shear transfer of loads from superstructure to the base foundation and into the surrounding ground. Usually, the strength of mortar and plaster was lower than that of bricks,
therefore the cracks initiation was observed in mortar or in the connection between bricks and mortar. The contribution of subsoil effects in-view of soil-structure interaction was partially included into shaking table tests of heavy models. Both schemes of stiff and flexible supports should be analyzed to obtain the appropriate data.

Emeritus and Hendry [2001] reviewed different type of masonry wall construction with their applications. Masonry materials include clay, concrete and calcium silicate in which a wide variety of unit sizes, forms and colours were produced. Clay bricks were obtainable in strengths of up to 100 N/mm² but much lower strengths, say 20-40 N/mm² for domestic buildings and for cladding walls for taller buildings. Concrete blocks had lower apparent compressive strengths in the range of 2.8 - 35N/mm². The tensile strength of masonry units both direct and flexural strength had an influence on the resistance of masonry under various stress conditions but was not normally specified except in relation to concrete blocks used in partition walls where typically a breaking strength of 0.05N/mm² was required. Although mortar accounts for as little as 7% of the total volume of masonry, it influences performance far more than this proportion indicated. Mortars were usually cement-sand with either lime or a plasticizer added to improve workability. In recent years, new types of mortars were developed including thin bed mortars for use with accurately dimensioned units and mortars with improved thermal properties. Stronger the mortar, lesser was the accommodation of the movement. It was inadvisable to use a stronger mix than necessary to meet the structural requirements. Hardened mortar was sufficiently strong and developed adequate adhesion to the units and also set without excessive shrinkage which would reduce the resistance of the masonry to rain water penetration or even cause cracking of the units. Masonry wall construction had undergone considerable change in the course of the last few decades with the introduction or extended use of lightweight materials and new types of units.

Brignola et al [2008] presented the role of the in-plane stiffness of timber floors in the seismic response of un-reinforced masonry buildings. The past experiences of earthquakes and the seismic response of existing masonry buildings were strongly dependent on the characteristics of wooden floors and in particular on their in-plane stiffness and on the connections quality between the floors and masonry elements. The diaphragm action clearly depends on the type of floor. Focusing herein the attention on timber floors was to evaluate the in-plane stiffness of
existing (as-built) and retrofitted configurations. According to international guidelines on seismic rehabilitation of buildings, both the global and local behaviour of unreinforced masonry buildings were assessed, accounted for partial/local collapse mechanisms, either in-plane or out-of-plane. The presence of a rigid diaphragm limits the out-of-plane rotation of the masonry units while causing a concentration of outward forces in the corners. The diaphragm flexibility was evaluated by analyzing the contribution to the in-plane deformation of the timber floor under simple loading conditions. Monotonic tests on small size floor specimen (1mx2m) and cyclic tests on real size floor specimens (one-way, 4m x 5m, aspect ratio equal to 1.25) were performed. The floor specimen was linked to the laboratory reaction floor by means of two external hinges due to the non-linear response of the diaphragm shear force vs. displacement (or diagonal deformation). The value of in-plane stiffness derived by each test was strongly affected by the floor stiffness adopted.

Turco et al [2006]\textsuperscript{135} reported the results of an experimental program under three phases; in the first phase mechanical properties of the materials used were determined. Then, the fiber reinforced polymer bars technique was used to strengthen unreinforced masonry walls to resist out-of-plane forces (second phase) and in-plane forces (third phase). Basically, glass and carbon FRP bars, having a rectangular and circular cross-section, and with a smooth or twisted sand-coated finish, were used as reinforcement. They were mounted vertically or horizontally into two different embedding materials: latex modified cementitious paste and an epoxy-based paste. Two kinds of masonry type, built with clay and concrete masonry units, were also considered. The walls exhibited the following modes of failure: (1) debonding of the fiber reinforced polymer reinforcement and (2) shear failure in the masonry near the support. The specimens were diagonally loaded and tested in a closed loop fashion. The force was applied to the wall by steel shoes placed at the top corner, and transmitted to similar shoes at the bottom corner through high strength steel bars. Linear variable displacement transducers were placed diagonally along the wall to monitor deformations. The failure of the control wall was brittle, controlled by bonding between the masonry units and mortar. Some materials came loose after reaching the ultimate load. Strength and pseudo-ductility substantially increased; the capacity by a factor of up to 2.5 in the case of shear strengthening and by 4.5–26 times in the case of flexural strengthening. The glass fiber reinforced polymer in spite of its low elastic modulus, had proved to be a good
material for masonry strengthening: often the performances were better than those obtained using the carbon fiber reinforced polymer.

Badarloo et al [2009] conducted a series of uni-axial and biaxial tests on full-scale grouted unreinforced brick masonry square panels. A failure criterion, with the principal compressive stresses oriented at $0^\circ$ and $90^\circ$ angles to the bed joints were obtained. It was concluded that the ratio of the horizontal to the vertical load had significant influence on the failure mode of the clay brick and hollow clay brick layers of panels, while it had little influence on the failure mode of the grout layer. The fundamental failure mode of all specimens corresponding to the loading ratio was splitting the grout layer from the solid clay brick (C-brick) or F-brick layer. The results showed that the behaviour of grouted unreinforced brick masonry panels was isotropic and the bed joint orientation did not play a significant role in the failure criterion. Test results indicated that the masonry strength under equal biaxial compression is higher by about 36% on average than that under uniaxial compression; the influence of joint orientation is very insignificant and negligible for these models. The comparison between experimental failure and Hill’s criteria exhibited reasonable agreement.

Luciano and Sacco [1998] presented a damage model for masonry material with a periodic microstructure characterized by a finite number of possible fractures. A damage model was derived by micromechanical approach and considering a typical unit cell of a periodic masonry material. Hence, a damage law for masonry was carried out and derived a model to analyze the possible damaged states of the masonry. Then, a finite element code was developed by using the model proposed in order to analyze the damage evolution of some simple masonry structures. Finally, numerical results were carried out in order to test the effectiveness of the model proposed which allowed individuality at each instant of the loading path, the distribution of the possible fractures in the structure and failure load of the structure support.

Bal et al [2008] described a simplified nonlinear method (DBELA) that defines the vulnerability of a masonry building by relating its deformation potential at different limit states and comparing this with the displacement demand from an over-damped displacement response spectrum at the period of vibration of the structure. A procedure for the displacement-based earthquake loss assessment for the masonry buildings within the northern marmara region along with the geometrical properties (i.e. storey height and pier height values) of these building types
was explained. Nonlinear time history analyses conducted on 28 different case studies of the buildings from the region with 20 randomly chosen acceleration records with peak ground acceleration (PGA) values varying from 0.02g to 0.51g. Period-height relationships and drift limit states for timber and reinforced concrete slab structures extracted from the results of these analyses.

Daniel Abrams [1997] gave an overview of newly proposed engineering guidelines with an emphasis on evaluation and retrofit of existing masonry buildings to enhance both strength and deformation capacity of masonry wall and pier components through the retrofit process. Although much of the guidelines were focused towards analytical procedures used with systematic rehabilitation, there were some rehabilitation methods for masonry buildings such as, grout and epoxy injections, surface coatings, adhered fabrics, shotcrete overlays, reinforced cores, prestressed masonry, infilled openings, enlarged openings and steel bracing. Most of the wall enhancement methods were researched at increasing the lateral in-plane strength of a component. Grout and epoxy injections had proved to be useful for increasing the shear resistance of unreinforced brick. Surface coatings and shotcrete overlays or adhered fabric also found to increase shear resistance of unreinforced masonry components as well as in-plane and out-of plane flexural strength. Reinforced cores and prestressing were known to increase flexural strength of unreinforced brick walls and piers. Adding vertical reinforcement increased flexural strength such that shear became critical. Grout cores for added reinforcement preclude bed-joint sliding shear mechanisms because of the dowel-type action that were provided. In such case, an existing deformation controlled component became force controlled and the acceptability criteria became more stringent. Bed-joint sliding mechanisms will be enhanced through increased frictional forces. If the cores are grouted, the bed-joint sliding will be restrained through the keying action provided by the grouted cores and diagonal tension governed for shear. The addition of steel bracing parallel to the plane of an unreinforced masonry shear wall added strength and ductility or otherwise brittle component provided the wall shear strength and governed by diagonal tension.

Cultrone et al [2007] evaluated the compatibility of a selected representative set of building materials (brick and calcarenite) and lime-based conservation mortars (hydraulic and/or non-hydraulic) by focusing on the study of their physical–mechanical properties. The study of
combinations of either bricks or calcarenites with different restoration mortars was an important contribution to understand the processes and factors that damage historic buildings. As the water cannot flow at the same speed through different materials as it could cause water to build up in certain areas of the buildings and this would lead them to deteriorate more quickly. With bricks, the loss of water absorbed by the test samples was slightly quicker if the contact area between the two materials was in a vertical position, for any type of mortar. This led to the conclusion that either the pores in the area of the test samples were better connected (the capillaries joining the pores are of an appreciable size) or the contact areas between the two different types of materials (brick or stone with mortar) were not continuous and therefore the water prefers to circulate through these routes. Pore size fell significantly in the brick + mortar (or calcarenite + mortar) contact area, because there was a larger concentration of binder in this area due to capillarity. This surface acts as a barrier slowing down the movement of water inside the composite material system. It was observed that the use of different additives in the mortars had not modified the texture in this contact area. Both systems suffer damage, but calcarenite creates a stronger and more continuous degree of adherence to lime mortars.

Mohamed Elgawady et al [2004]\textsuperscript{102} presented preliminary comparisons between the test results of the dynamic and static cyclic tests. The test specimens are half-scale specimens built using half-scale hollow clay masonry units and weak mortar. The specimens, before and after retrofitting, are subjected to a series of either synthetic earthquakes or static cyclic test runs. The tests showed that the composites improve the cracking and ultimate load of the retrofitted specimen by a factor of 3 and 2.6, respectively. The lateral resistance of the reference specimen measured in the static cyclic tests is 1.2 times the lateral resistance of the similar reference specimen measured in the dynamic test. In spite of relatively poor mortar, the specimen friction coefficient exceeded 1.0. However, after heavy damage and a drift of about 2% the specimen coefficient of friction reduced to 0.7. The initial stiffness for the reference and retrofitted specimens was approximately the same in the static cyclic and dynamic tests. The lateral resistance of the reference specimen in the static cyclic test is approximately 20% higher than the lateral resistance in the dynamic test.

Olivito and Stump [2001]\textsuperscript{112} carried out experimental tests on specially built masonry specimens subjected to compressive and tensile stress conditions. In particular, four-point bending tests
allowed the characterization of both the tensile behaviour and elastic parameters of masonry subjected to tensile stress. The tests also gave effective results with regard to material toughness. The results obtained on masonry supported low tensile stress conditions; masonry composites exhibited bimodular behaviour which is related to masonry composition and texture; masonry exhibited a strain-softening behaviour, caused by the start and growth of cracking. These results used to define an analytical model which took the elastic behaviour of the masonry up to the critical condition into account and defined by the fracture toughness, which then considered the softening behaviour.

Alfaiate et al. [1997] used the finite element method to study mixed mode crack propagation in concrete and masonry. A discrete approach was adopted: cracks were allowed to evolve along discontinuity surfaces called fictitious cracks. These discontinuities were modeled using: i) interface elements, in which case a numerical algorithm was adopted which avoided the need to remesh and ii) embedding discontinuities, according to a discrete strong embedded discontinuity approach. The effect of shear stresses which develop at the discontinuity surfaces was analyzed. It was found that the amount of shear stresses present in the discontinuity is the factor which influenced the most significant structural behaviour of both concrete and masonry. From all tests analyzed, it was verified that the amount of shear stresses present at the discontinuities was the most important factor in mixed-mode fracture of both concrete and masonry. In concrete, it was found that larger shear stresses led to both a stiffer post peak response and to a better approximation of the softening regime. It was also found that mixed-mode fracture does not depend significantly either on the mode-II fracture energy or on the cohesion. In masonry, if slippage at the mortar interfaces was allowed to fully develop, the limitation of shear stresses under high compressive stresses led to a smaller peak load as well as different failure mechanisms, which was also confirmed experimentally.

Haroun et al. [2005] evaluated the in-plane shear behavior of masonry walls externally reinforced with fiber reinforced polymer composite laminates. The wall specimens were built with a height-to-length aspect ratio of 1:1 to promote a shear dominated behavior under in-plane loading. The control as-built wall was cyclically tested to failure and demonstrated a pure shear mode. The failure of the specimen was initiated by diagonal shear cracks and developed a diagonal strut action resulting in the crushing of the wall edge boundaries. The other specimen
was cyclically pre-cracked and then repaired and retested. In the pre-cracked wall specimen, two major localized damages appeared: diagonal shear cracks across the wall and local compression failure of the wall toe on one side. The performance of the repaired wall was compared with that of the as-built wall. From this it became clear that the repair technique improved the strength and energy dissipation of the wall. It not only succeeded in restoring the capacity of the original wall, but also increased it to a level 120% of that of the original wall capacity. The energy dissipation observed for repaired specimen was also increased to 167% of that of the control wall. The ductility of the carbon/epoxy repaired specimen was 1.7 times that of the as-built specimen. For the retrofitted specimens, the enhancement in the ductility ranged from 3.4 times that of the as-built in case of double-side carbon/epoxy retrofit to 6 folds in the case of pre-cured carbon/epoxy strips. Despite the premature failure caused by localized compression failure of the masonry at the wall toes, notable gains in strength, stiffness and ductility were achieved by applying the fiber reinforced polymer laminates to either one or two sides of the walls.

Balasubramanian et al [2006] reviewed the features of some of the popular models to predict the shear capacity for monumental structures (using the in-situ properties obtained from field investigations). The strut model was used for determining the in-plane shear capacity of unreinforced masonry walls. The unreinforced masonry wall was considered to be consisting of several piers, one per storey, separated by relatively stiff joint regions. The probabilistic capacity assessment of one wall of a monumental building made of unreinforced brick masonry against in-plane loading using strut model was presented. From the results obtained, it was noted that the shear capacity of the wall was relatively independent of the variations in random variables considered. Attempts were made to evaluate the lateral resistance adopting suitable theoretical models. It was noted that for in-plane lateral resistance, the formulae proposed by Magenes and Calvi was improved over those given in Eurocode and are rational. The Indian code provisions were comprehensive and had modified in accordance with various failure modes. An approach integrating the limit analysis method with the dynamic analysis was proposed for seismic safety evaluation.

Theofanis and Thanasis [2005] investigated the application of fiber-reinforced polymer (FRP) as a means of increasing the axial capacity of masonry through confinement. Four series of uniaxial compression tests with a total were conducted on model masonry columns with these
variables: number of layers, radius at the corners, cross-section aspect ratio and type of fibers. Masonry column specimens in four series were prepared using clay bricks with dimensions of 55mm-width, 40 mm-height and 115mm-length, bonded together with a mortar containing cement and lime as binder, at a water: cement: lime: sand ratio equal to 0.9:1:3:7.5 by weight. The corners of all specimens were rounded using a grinding machine at a radius of 10 mm in the first, third and fourth series and at a radius of 20 mm in the second series. Within each series, specimens were wrapped with one; two or three layers of unidirectional carbon fiber reinforced polymer sheets or with five layers of unidirectional glass fiber reinforced polymer sheets, applied through the use of a two-part epoxy adhesive. When the corner radius was increased from 10 to 20 mm, the strength increased by about 25 – 40% with carbon fiber reinforced polymer jackets and by about 12% with the very thick glass fiber reinforced polymer jackets. Hence, the beneficial effect of increasing the corner radius was verified. It was concluded that fiber reinforced polymer-confined masonry behaved very much like fiber reinforced polymer-confined concrete. Confinement increased both the load-carrying capacity and the deformability of masonry almost linearly with the average confining stress. The uniaxial compression test results enabled the development of a simple confinement model for strength and ultimate strain of fiber reinforced polymer-confined masonry.

Giancarlo Marcari et al [2007] dealt with a specific type of masonry (Tuff masonry structures) largely used in South Italy and particularly in campania region, whose territory exhibits a relevant seismic risk. Tuff is a rock composed of volcanic particles, ranging from ash to small pebble size, compacted or cemented in a consolidated state. Uni-axial compression test on masonry units were performed according to Italian standards. A mean compressive strength $f_c=2.1$ MPa and coefficient of variation of 10% were obtained. An experimental program focused on the monotonic response of tuff masonry panels subjected to shear loading was investigated. The panels consisted of two-layered walls with the inner filled with mortar and chips from yellow tuff blocks. The overall dimensions of the tested panels were 1570 mm in height, 1480 mm in width and 530 mm in thickness, with an aspect height-to-width ratio equal to 1.06. At failure, a typical splitting mode of failure through bricks and perpendicular joints occurred, which was induced by the different deformability of the mortar and tuff units. In fact, an average compressive strength $f_{wc,m}$ equal to 1.4 MPa was measured over the net cross sectional area. Experimental elastic modulus of the panels was about 630 MPa. Shear-compression tests
were performed in two different steps. Firstly, a vertical load of 400kN was applied at the top of the wall and was kept constant during the test. Such a value represents the service gravity loads that typically act at the lowest storey’s of traditional tuff masonry buildings. Then a horizontal load was applied up to failure under monotonic displacement control, at a loading rate of 2 mm/s. Shear tests were deemed to stop as the panels lateral load degradation reached a limit of about 25% of the peak one. The test set-up was designed in order to reproduce static and kinematic boundary conditions of panels located in shear type structures and provided the basic behavioral parameters such as strength, cracking pattern and deformation characteristics at various states of performance for both as-built and strengthened panels. Parametric analysis explained the type of finite element and relevant mechanical parameters pointed out the sensitivity of the proposed anisotropic model and its reliability for seismic assessment procedures. Review of numerical data led to recognize that, if macro-modeling strategy applied, the overall response of masonry panels well predicted in terms of collapse load values, as well as sufficiently accurate failure mechanisms. It was observed that, for practical purposes, the results can be considered mesh insensitive in terms of the peak load and failure mechanisms. A good agreement was also found with respect to the collapse load, which resulted just 16% lower than the mean experimental value. Even if the exact sharp reproduction of the post peak behaviour was not the main issue here, an overall good agreement was found because the same trend can be observed in both numerical and experimental plots. In particular, from a displacement of about 8 mm the numerical and experimental load-displacement curves displayed a similar post-peak response.

Guido Magenes [2006] discussed some of the selected issues pertaining to seismic design and the assessment of the masonry buildings. Buildings from one to three storey for unreinforced and from two to four storey for reinforced masonry were considered, characterized by a rather simple plan configuration and regularity in elevation with a total area of shear walls ranging from 3.5% to 7.5% of the total floor area in each principal direction. The tests were performed on up to three-story, 1:2 to 1:1 scale models of unreinforced stone and brick masonry buildings or walls, representative of common Italian existing typologies. The seismic force was in turn reduced by a behaviour factor of the wall element, which for primary elements was assumed as $q_a = 3$, whereas for non-structural walls $q_a = 2$ would be used. Since force-based approach may not produce consistent results, it is suggested that specific experimental procedures and criteria should be developed and codified to assess the conformity of new systems to specific seismic
performance requirement. Elements that favour masonry construction with respect to other techniques were ease of construction, durability, good insulating properties, fire resistance, sustainability and sometimes aesthetics. On the other hand, the use of structural masonry determines a strong interconnection between structural and architectural design, whereas frame structures allow more freedom in distribution of internal space. A global model of the structure was usually needed when the resistance of the building to horizontal actions was provided by the combined effect of floor diaphragms and in-plane response of structural walls.

Korkmaz [2009] investigated the seismic safety of unreinforced masonry buildings that dominated the building inventory in the Pakistan region by multiple approaches. Four different representative buildings were modeled to demonstrate the building stack in the region. Nonlinear time history analysis and probabilistic based seismic assessment analysis were performed on the representative buildings. The analysis showed that unreinforced masonry low-rise buildings present higher displacements; shear forces and probability of damage were directly related to damage or collapse. The non-linear time history analysis indicated that masonry buildings were susceptible to seismic damage with higher displacements. A detailed seismic safety assessment for all unreinforced masonry low-rise building types in the region was highlighted.

Wang et al [2006] carried out experiments on eight brick masonry walls with pilasters reinforced by glass fiber reinforced polymer to study the seismic shear capacity of brick masonry wall structures. First, the interaction coefficient of the pilaster $\psi$, modified coefficient $\delta_{gf}$, statistic coefficient $\beta$ and effective participation coefficient $\zeta$ were determined. Then, based on failure model of brick masonry walls and truss model of fiber reinforced polymer, the formulae of seismic shear capacity of brick masonry wall with pilaster reinforced by fiber reinforced polymer were established, which had an excellent coincidence with the experimental results. Finally, the simplified design formula was proposed to study the seismic shear capacity of brick masonry wall reinforced by fiber reinforced polymer. Based on the results of the experimental program, strip fiber reinforced polymer was applied as masonry strengthening material and it increased the load carrying capacity of masonry subjected to in-plane shear loading. A statistical coefficient $\beta$ was regressed from the data and then a shear compression for masonry structures was obtained based on the principle tensile stress theory. The concept of the interaction coefficient of pilaster $\psi$ was proposed and the concrete calculating expression was obtained.
based on the equivalent rigidity theory, which solved the difficult problem of calculating the shear area of masonry wall with pilaster. The effective working coefficient $\xi$ was not only corrected to the area reinforcement rate but also was corrected to reinforcement mode. The value of $\xi$ for horizontal reinforcement mode ($\xi_h$) is generally smaller than that of for diagonal reinforcement mode ($\xi_x$) in the same loading state. The shear capacity of fiber reinforced polymer-reinforced masonry wall with pilaster was expressed by a simple superposition method and shear capacity calculation method. The results obtained from those methods agreed well with the test results n the safety point of views.

### 2.2.5 Finite element modeling

Paulo Lourenço et al [2006] studied the numerical representation of masonry to focus on the micro-modeling of the individual components, viz. unit (brick, block etc.) and mortar, or the macro-modeling of masonry as a composite. Depending on the level of accuracy and the simplicity desired, it was possible to use the following modeling strategies. Detailed micro-modeling - units and mortar in the joints were represented by continuum elements whereas the unit-mortar interface was represented by discontinuum elements; Simplified micro-modeling - expanded units were represented by continuum elements whereas the behaviour of the mortar joints and unit-mortar interface was lumped in discontinuum elements; Macro-modeling - units, mortar and unit-mortar interface were smeared out in a homogeneous continuum. Simplified divisions of the elementary cell such as layered approaches were inadequate for the non-linear range. In finite element analysis, the failure mechanisms and collapse loads were similar to more complex approaches based on nonlinear incremental and iterative finite element simulations and compared to the non-linear simulations. Nevertheless, significant caution was always recommended when trying to reproduce existing damage patterns, advanced non-linear simulations.

Jahangir et al [2004] presented the numerical verifications of the experimental investigation on the effect of mortar joint thickness on compressive strength characteristics of axially loaded brick-mortar prisms. The three dimensional micro modeling of the prisms was based on two approaches: firstly, models were assumed to be made of homogeneous material; the second approach envisaged the models as composite materials made of brick and mortar. The later modeling approach assumed the prism to be made of composite material, gave more accurate prediction of the stress distribution in the prisms. The failure load predictions were in good
agreement with the experimental results, suggesting that this modeling approach with composite material assumption was more appropriate than the homogenous material assumption. The study proved that for a normal case, where elastic modulus of mortar \( (E_j) \), is less than elastic modulus of brick \( (E_b) \), the increase in mortar thickness results in reduction of elastic modulus of the masonry, an increase of elastic modulus of mortar, led to an increase in the elastic modulus of masonry. The stress - strain curve obtained from both analyses showed that the maximum compressive strength of brickwork was slightly higher for the case of homogenized material than that of the composite material. Homogenized model behaves as one material that is the dispersion of load in the model at about 45°. The mortar joint having lower strength than the brick strength caused a reduction in the compressive strength of masonry. The actual compressive strength of the masonry determined by experimental method was much higher than the strength obtained by numerical method. However, in reality, the brickwork was constructed from two layered materials namely brick and mortar, therefore, the idealization of composite material for the analysis should be adopted. Homogenized model exhibited about 4% higher strength than the model assumed as a composite material.

Krit Chaimoon and Mario Attard [2007] used a simplified micro-modeling approach wherein the two masonry components: bricks and mortar joints were modeled separately. The mortar joint, which was the plane of weakness, was represented through interface nodes of zero thickness. A simplified micro-modeling approach was proposed to model masonry. The mortar thickness and the brick–mortar interfaces were lumped into a zero-thickness interface while the dimensions of the brick units were expanded to keep the geometry of a masonry structure unchanged. A masonry structure was thus modeled as a set of masonry units. Each masonry unit was further subdivided into interior brick elements which had boundaries either representing the mortar interfaces or internal brick interfaces. Shear wall tests carried out by the performance of the proposed model for the case of shear compression. Vermeltfort’s specimens had a width/height ratio of 0.99 (990 × 1000 mm). The walls were built with 18 courses (16 courses were active and 2 courses were clamped in steel beams). The bricks were ‘Joosten’ solid clay bricks (dimensions 204 × 98 × 50 mm) and a 10 mm thick mortar joint was used. Masonry was modeled using a 2D representation by means of triangular units connected at boundary nodes along the interfaces between the units. The triangular units were grouped into rectangular zones mimicking brick units and mortar joints. The mortar–brick interface was modeled as a zero
thickness interface. The inelastic failure surface for the mortar interfaces were modeled using a Mohr–coulomb failure surface with a tension cut-off and a linear compression cap. Fracture was modeled through a constitutive softening-fracture law at the interface boundary nodes. The constitutive law was a single branch softening law. Comparison of the formulation with the experimental results on the masonry shear walls under pre-compression and shear showed reasonable agreement. The formulation was able to capture the pre-peak and post-peak response and the cracking characteristics reasonably well.

Siro Casolo [2004] proposed rigid elements for a specific simplified model of the in-plane behaviour of masonry walls made of regular textures. The elements were plane, quadrilateral and connected by two normal springs and one shear spring on each side. The mechanical characteristics of these connections are defined in consideration of the texture effects arising due to the mechanical degradation of mortar. The present approach proved effective when transferring the essential texture information from micro-scale to macro-scale. In particular, the local rotation of the blocks was obtained by assigning different stiffness to the shear springs according to their orientation, while the in-plane bending stiffness reproduced by properly disposing the two normal springs. Depending on the geometry of the textures, these aspects were significant in case of large differences in the elastic modulus of the constituents, as the masonry walls subjected to heavy seismic loading. Numerical simulations proved to deal with these aspects with a reduced computational effort which was promising for non-linear dynamical analysis applications. An extensive numerical investigation proved that micro-structure effects become particularly relevant for high ratios between the elastic moduli of the constituents, when texture played an important role giving additional resources of stiffness and strength.

Fazia Fouchal et al [2009] discussed the experimental characterization of the materials (bricks and mortar) and the brick/mortar interface. Since masonry was a composite structure, failure of these structures depend on the properties of the materials (mortar, bricks, etc.), as well as on the characteristics of the bonding between the various components. It was thus possible to model the fracture process occurring along the interface as well as the mortar. Structures composed of hollow bricks modeled using two adhesive characteristics. In particular, the model was used to study the fracture process occurring along the interface of a small structure consisting of three hollow bricks. Further studies were required on the fracture processes crossing the mortar in
structures. The results showed that the model accurately described the behaviour of the structures. In particular, the cracking of the mortar in a small wall structure was accurately predicted by the model.

Dilrukshi and Dias [2008] proposed the analysis of a typical wall connected to the effective slab area, rather than analyzing a whole building. The behaviour of the cracks was predicted based on surveys of buildings the methodology of cracks formed. Also, typical structural arrangements were mathematically modeled using 3D brick finite element models, with link elements between the masonry and concrete elements in order to model interfaces. Locations and directions where cracking would occur were identified using the principal stresses developed in the finite element model and a failure criterion developed based on the modified Von-Mises theory. Also, using these numerical models, the effect of wall length and structural form of the wall (i.e. load bearing walls and reinforced concrete framed walls) on the formation of these cracks was studied. These results were compared with the information obtained from the survey.

In the context of the finite element method, there were two major groups of interface elements/models known as the “zero thickness” interface element and “thin layer” interface elements. Numerical modeling was able to reproduce the phenomena observed in buildings where masonry walls were subjected to thermal movements of an overlying slab. The type and location of cracking depend significantly on whether the wall is load bearing or framed by reinforced concrete elements. Both types of arrangements gave diagonal cracking near the ends of walls, although the crack orientation was steeper in load bearing walls. The survey also indicated that more cracking was formed in walls oriented in the long way direction of the building.

Luisa Berto et al [2008] proposed two identification procedures based on standard homogenization method and on direct identification method. In particular, the homogenization method adopted was based on an analytical approach under the hypothesis of mortar joint modeled as interfaces of zero thickness. On the other hand, the direct identification method was based on a finite element micro-modeling of a periodic masonry pattern subjected to homogeneous deformation state. From a structural point of view, two basic classes for masonry buildings: regular and irregular masonries with regular texture in which brick or stone blocks were regularly shaped and characterized by a disposition of the units along horizontal lines. Two
Further sub-classes were identified: periodic and quasi-periodic masonry, being the difference related to the possibility of identifying a representative volume cell, which generates panel as a whole by repetition. Masonry constituted by irregular stones was characterized by the presence of irregularly shaped and irregularly disposed stones with variable dimensions.

Guineaa et al [2000] presented a micromechanical model for the analysis of mode-I fracture of brick masonry. The analysis was based on a detailed modeling of brick and mortar fracture by means of the fictitious (or cohesive) crack model. Fracture properties for brick and mortar were determined by specific tests and then a numerical model in which bricks and mortar joints were treated separately. The composite fracture model predicted the fracture of masonry panels accurately. Homogeneous linear elastic behaviour was assumed for all the elements outside the fracture line, with an average elastic modulus of 22.9MPa. This value was obtained from a fit of the initial linear part of the test records before the cracking started. Finite elements in the panel were condensed in a super element that was connected to the interface elements. This method was highly recommended in a general non-linear analysis where a portion of the model remains linear elastic. Distributed cracking or damage in the structure was not considered when a macro-crack developed. In the failed specimen single dominant crack was observed which runs vertically through the mortar beds and the brick units. For medium or small masonry panels, a more detailed analysis of brick and mortar interaction required.

Abdou et al [2006] carried out an experimental study to understand better on failure mode of the masonry walls in the shearing process in joint mortar. The overall behaviour of this set was influenced by several factors, such as: brick and mortar properties, brick size and its aspect ratio, joint thickness, joint orientation, relative position of head and bed joints, properties of the unit/mortar bond and workmanship (joint quality) load/unload shear tests were performed to assess the type of the shear behaviour of the joint mortar. The cohesion and the internal friction angle were then derived from linear regression while assuming Mohr–Coulomb criterion. In particular, the influence of holes on the joint behaviour was studied by comparing results obtained with both solid and hollow bricks. In both cases, the experimental results showed that there was not any stiffness degradation even in the softening regime. Actually, the shear modulus remained constant. Hence, the joint behaviour was considered to be elasto-plastic, independently on the brick type. The presence of holes increased the stiffness but did not affect the internal
friction angle of the joint mortar. Filling the holes with mortar constitutes an abutment during the shear process, and consequently increased the shear modulus of the hollow specimens. For hollow brick, the post peak behaviour was characterized by the residual strength. Its value was related to the compressive stress level, and was estimated approximately to 50% of the ultimate shear strength when $\tau_n = 0.3$ N/mm$^2$. One should notice that the ultimate and the residual friction angle were independent on the brick type (solid or hollow) and the ultimate cohesion decreased as the hollow bricks were used because of the cracking of the brick webs. For the residual regime, the failure of the couplet was described by the Mohr–coulomb criterion. It was found that the residual cohesion for hollow bricks was approximately equal to 35% of the ultimate cohesion.

Mauro et al [2010]$^{92}$ presented a homogenization procedure and the effect of the bond and the poisons-type interaction between mortar and brick. Assuming a simplified kinematics for the phases belonging to the representative volume element (RVE), the so-called localization problem was solved by imposing the minimization of the average internal strain energy. Closed form formulations were then derived for the equivalent in-plane elastic constants of masonry. The expressions found were limited to masonry as a stratified medium or where the joints were treated as interfaces. The accuracy of the results was investigated by means of a comparison with finite element analysis. A parametric study, conducted varying the geometries and the mechanical properties of the phases showed that the error introduced over a very wide range of values for the elastic properties was lower than 8%, meaning that the procedure was ready to be used for non-linear analysis. A homogenization procedure for running bond masonry was presented. The procedure relied upon a simplified kinematics defined within the representative volume element and provided analytically the elastic properties of masonry as a function of the geometry and of the individual properties of mortar and brick. By comparing the results with FE analysis, the errors introduced by the model were low from an engineering view point, even when large differences between mortar and brick stiffness were considered or when thick joints were taken into account. On the basis of the results obtained, the proposed formulation reproduces the essential feature of masonry behaviour and thus it constituted a promising tool which was adopted in the framework of multi-scale analysis of masonry structures.
2.3 RESEARCH GAP
The brittle nature of masonry buildings is the major cause for collapse of buildings and loss of lives and thereby there is a need to introduce remedial measures in the construction of such buildings. According to the review of existing literature on research carried out in many institutions and in many different countries, there is a need to investigate the in-plane structural behaviour of masonry walls especially to increase the load carrying capacity and to delay the crack propagation with caution. Also, the proposed new methodology to save the buildings to some extent from the earthquakes should be easily affordable by the society. The research work is carried out to bridge the gap in the in-plane behaviour of masonry structure and contribution towards determining the enhanced performance of brick masonry wall structures in various seismic zones in India. Consequently, the comparative responses of un-reinforced masonry brick walls and reinforced masonry brick walls with introduction of hexagonal woven wire mesh along bed joint in alternate layer of the masonry brick walls are extensively investigated and discussed elaborately in this research thesis.

2.4 RESEARCH METHODOLOGY
Due to the heterogeneous nature present in the masonry wall, tests were conducted to study the behaviour of masonry wall for in-plane loading. The observations obtained were used to develop mathematical equations.

2.4.1 Focus of the research work
The research investigations are supported by experimental work and the analytical work using ANSYS. The detailed experimental investigations carried out in this research are depicted in a schematic flow chart in Fig. 2.1.
Fig 2.1 Schematic diagram depicting the experimental work carried out in this research
The aim of this research work is to study the in-plane shear behaviour of brick masonry. First being the study of structural properties of masonry, the tests were conducted with clay bricks and fly ash bricks. The mortars have the composition of 1:6 cement mortar with partial replacement of fine aggregate with fly ash as 0%, 10% and 20% respectively were used in this study. Initially, the masonry strength was obtained for the unreinforced and reinforced brick masonry prism in 1:6 cement mortar with partial replacement of fine aggregate with fly ash; secondly the in-plane shear behaviour of the unreinforced and reinforced masonry wall panel (with woven wire mesh along the horizontal bed joint) was studied in 1:6 cement mortar with partial replacement of fine aggregate with fly ash using shear compression loading test and diagonal compression test respectively; thirdly the numerical prediction of masonry strength and in-plane shear strength was evaluated; and finally, the wall capacity for un-reinforced and reinforced masonry wall in seismic zones was recommended.

2.5 CONCLUSIONS OBTAINED FROM THE LITERATURE REVIEW
A survey of existing literature on brick masonry reveals that majority of the research on masonry was focused on high quality bricks with compressive strength ranges from 17MPa to 42MPa. The compressive strength of clay bricks commonly used in rural areas in India varies from 2.5MPa to 5.0MPa. These clay bricks are generally adopted in the construction because of their low cost. Although the bricks and mortar are strong enough and safe to retain gravity loads, their behaviour especially under lateral in-plane loads is unpredictable. Fly ash bricks are found to be much stronger with less water absorption and cheaper than clay bricks. If fly ash bricks are considered for construction, saving of cement/ fine aggregate could be achieved easily in addition to the utilization of the waste product like fly ash. These fly ash bricks could be manufactured very easily at village level and can be more suitable than good burnt clay bricks for any kind of building construction. Since the fly ash is a pozzolanic material, as it is used in the cement mortar, strengthens the interfaces of the brick and the mortar in the brick masonry. However, the experimental data justifying the improvement effect of fly ash replacements with fine aggregate/ cement on the interfacial bond are still unavailable and not reported clearly till date. Hence, in this research work the effects of fly ash on the mechanical properties of the brick masonry are investigated and reported.