Structural Design of Shallow Masonry Domes

By

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Abstract

This thesis investigates the viability of shallow, unreinforced masonry domes for the roof and floor systems of residential construction. In recent years, reinforced concrete (RCC) framed construction has been established as the dominant structural form for residential and commercial usage in the developing world, with the flat two-way slab as the most common (almost universal in northern India) spanning solution for these designs. Over the same time period, local artisans in the Muzzafarnagar region of Uttar Pradesh in northern India have combined shallow brick vaulting techniques with an RCC tension ring as a small to medium scale spanning solution. While these vaults are a cost-effective alternative to a concrete slab roof/floor system, improperly designed and detailed masonry construction can prove dangerous, especially in seismic zones.

This thesis is an exploration of the structural behavior and design of these domes, intended to produce broadly applicable design guidelines to ensure the strength and stability of this structural typology in order to valorize their broad usage, where appropriate, in India's housing sector. Simplified design calculations for unreinforced masonry which match experimental data are generated using equilibrium methods and plastic design theory. Influence of geometric and material parameters on strength and stability are investigated, and discussion of proper detailing and the limitations of this spanning technology is included.

Thesis Supervisor: John Ochsendorf
Title: Class of 1942 Professor of Civil and Environmental Engineering and Architecture

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Thank you to the Hunnarshala team in Bhuj – Sandeep Virmani, Tejas Kotak, Mahavir Acharya, and Hardika Dayalani. Their work with these flat domes is the foundation of my research and they were of invaluable assistance to my research. Thanks also to Lara Davis for sharing her expertise in masonry design and construction. Special thanks to Piyush, Hugo, Divya, Brad, Emma, Mark, Maddie, Hugh and Johanna for being great travel partners. Thanks also to the Course 1 staff – Jeanette Marchocki, Kathleen Ross, and Kiley Clapper were always there to help out. Thanks to Steven Rudolph for his help in the lab downstairs.

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Chapter 1: Introduction

1.1: Motivation

In 2013, the Hunnarshala Foundation was invited to Muzzafarnagar, Uttar Pradesh, India to assist in the resettlement of residents displaced by violence between the Muslim and Hindu communities of the region. While there, the flat dome typology which is the subject of this thesis was discovered and documented. Described briefly in §1.2.4, this documentation is presented in full as Appendix B. This, as well as a brief description on the website of the Indian Rural Housing Knowledge Network [RHKN, 2016] are the only pieces of available information on these particular flat domes; there are no published pieces of scholarly work or engineering calculations available. The literature review in this chapter consists of a review of published material on masonry design as well as a summary of personal research conducted by the author in Muzzafarnagar and Bhuj, Gujarat, India in January of 2016.

The Hunnarshala Foundation conducted two full-scale load tests at their headquarters in Bhuj in 2015 – it is the goal of this thesis to develop design methods and calculations (which can be validated by the data from these tests) to promote the broad usage of these domes where applicable. Figure 1.1 below shows the configuration of the domes tested.

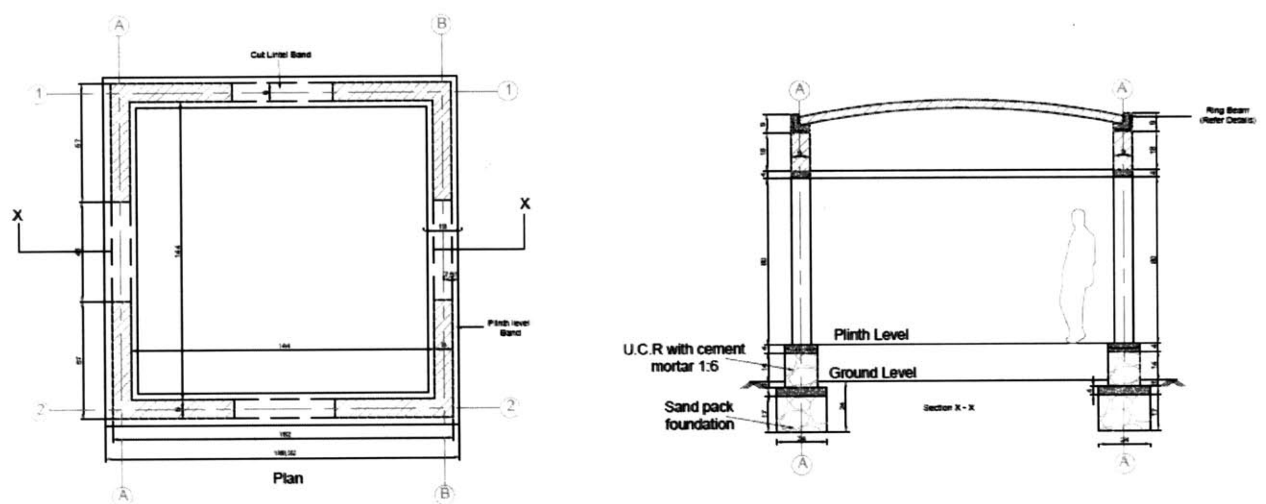


Figure 1.1: Drawings of domes for full-scale test at Bhuj, 2015. Drawings by Hunnarshala Foundation

1.2: Literature Review

1.2.1: Masonry theory

Masonry arches, domes, and vaults have been used as spanning structures for thousands of years by disparate cultures across the globe. Many of these structures remain structurally sound and stable to this day, a tribute to the durability of masonry and the expert structural knowledge of the master builders of antiquity. Generally, while a beam or slab system spans and bears loads in bending (both tension and compression are found within the section of the spanning element), masonry arch or vault systems resist loads through their shape – the arch or vault is in pure compression, generating thrust at the base to be resisted by another structural element (tension ring, buttress, tie, etc.). There is extensive scholarly work on the behavior of unreinforced masonry structures. Heyman (1995) provides an excellent introduction of the lower-bound plastic theory of masonry design and analysis of arches, vaults and domes, particularly in stone. Heyman introduces three assumptions for masonry design: first, that masonry has infinite compressive strength, second, there is no tensile strength in the joints between masonry units, and third, that there is no sliding between masonry units. These three assumptions form the foundation of the limit state analysis of masonry structures.

Plastic theory is used extensively in masonry analysis – since masonry structures are highly indeterminate and subject to boundary conditions which are often impossible to quantify exactly, elastic analysis will provide misleading results and in some cases severely underestimate the capacity of masonry structures. The essence of plastic theory is a focus on collapse – while elastic theory aims to determine the exact state of stress inside a structure under service loads, plastic theory is concerned with determining a collapse (limit) state of a structure and the associated loading required to cause that collapse. Plastic theory states that at collapse, three conditions are satisfied: there is equilibrium within the structure, the stresses within the structure are at or below the yield stress, and there is a mechanism – an arrangement of hinges which allow for kinematic deformation of the structure. The lower-bound theorem states that a load case for which the first two conditions are satisfied is a lower bound for the collapse load. For masonry structures (in 2 dimensions), following Heyman's three assumptions, this means that if we can find a compression-only thrust line which falls entirely within the effective depth of the masonry arch (first condition), our structure will be stable (the second condition is automatically satisfied since we assume that masonry has infinite strength in compression). A masonry structure thus fails when the thrust line – a line representing the path of compressive stress through the arch – can no longer be contained within the thickness of the masonry elements, causing a kinematic mode of collapse. The primary conclusion from limit state analysis of masonry applicable to this thesis is that a well-shaped dome will experience relatively low compressive stresses within the surface of the dome, and thus the failure of such a structure will occur either by the failure of the structural element to resist thrust (tension ring or buttressing system) or by a mechanism brought about by improper shaping of the dome (thrust line can no longer be contained within the thickness of the masonry).

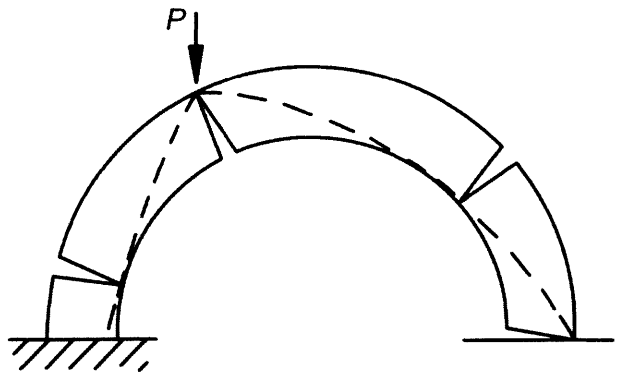


Figure 1.2: Collapse of arch under a point load, after Heyman (1995)

The geometry of this thrust line is a compression only network representing a compression-only structure inside the masonry structure, and can be found graphically. Chapter 8 of Allen and Zalewski (2010) provides a clear introduction to equilibrium analysis and design of unreinforced masonry structures. The ideal shape for a 2-dimensional compression-only structure is, after Robert Hooke's 1675 anagram*, the inverse of the catenary shape formed by a chain hanging under its own weight. Additional loading can be represented by adding representative weights to the hanging chain – this technique has been used for many years in the analysis of historic masonry structures (notably by Poleni in 1748 for the dome of St. Peter's in Rome) and the design of new masonry structures (most notably by Antoni Gaudí, see also Block et al (2010)).

* *ut pendet continuum flexile, sic stabit contiguum rigidum inversum* – translates to "as hangs the flexible line, so but inverted will stand the rigid arch."

While graphical analysis provides a simple solution for two-dimensional masonry problems, three-dimensional analysis becomes more complex. As domes are commonly defined as the rotation of an arch around a central axis, the simplest solution for the analysis of a hemispherical dome is to analyze radial slices of the dome as arches – if a representative arch is stable, so is the dome. This is a conservative method – the continuous surface of a dome allows for forces in the transverse or "hoop" direction to develop along its surface, increasing capacity for stability. Membrane theory, developed in 1866 by Johann Schwedler, allows for the development of these hoop forces, but only up to a point to equilibrate meridional forces. Lau (2006) focuses on the magnitude of these hoop forces, developing a new method (modified thrust line method) which allows for variation in their value in order to find satisfactory thrust line solutions. This progression illustrates the lower bound plastic theory – that any equilibrium solution that can be found within the surface of the masonry structure, either by making proper assumptions about the 3-dimensional behavior of a dome or by considering a 3-dimensional structure as a collection of 2-dimensional structures, represents a lower (safe)* bound on the unique collapse load. While contemporary researchers such as Block and Ochsendorf (2007) have developed computer applications which use the principles of graphic statics in three dimensions to generate vault shapes, conservative solutions can be found with significantly less computation by simplifying 3-dimensional problems as collections of 2-dimensional ones.

In general, a dome is a surface generated by a revolved curve and a vault is a surface generated by extruding or sweeping a curve, or intersecting these vaults. Common dome types are hemispherical domes and pointed domes, while common vault types are the barrel vault (extruded arch) or the groin vault (generated by intersection of two barrel vaults). The domes studied in this thesis do not have a prescribed shape and thus several possible geometries are considered. Parameters that describe dome typology are the span-to-rise ratio (D/z) and the span-to-thickness ratio (D/t). The domes considered in this thesis have D/z ratios in the range of 15-25 – high values indicating flat vaults. Examples of spanning systems in this range are many vaults by the Guastavino Company: detailed extensively in Ochsendorf (2010). The Guastavino system

* The "safe" notation is used since a capacity which is a lower bound on the actual collapse load will produce a conservative (safe) design.

of tile vaulting is a classic example of an unreinforced masonry spanning system used both to enclose space below and provide support for the floor above.

1.2.2: Application of masonry vaulting to the developing world

In the recent past, masonry has emerged as a cost-effective solution for construction in the developing world. Much of the traditional architecture of the world is in unreinforced masonry, and its revitalization as a cost-effective solution is a way to engage the vernacular architecture of communities. Many common methods of low-cost construction, for example prefabricated concrete panels or solutions using metal decking, actually introduce high fiscal (transportation) and social (local architectural methods are often subjugated to the whim of the international aid community) cost to the local communities who are ostensibly being helped. Masonry was encouraged as an affordable alternative to reinforced concrete construction in India by Laurie Baker (1917-2007), a British architect who dedicated his life to developing affordable solutions for residential construction in easily digestible form (see Baker, 1987). Hassan Fathy (1900-1989) also pioneered a similar revival of vernacular architecture, promoting the usage of sun-dried adobe bricks, in Egypt and Mexico – see Hebel (2016) for a contemporary application of his leaning-brick vaulting technique.

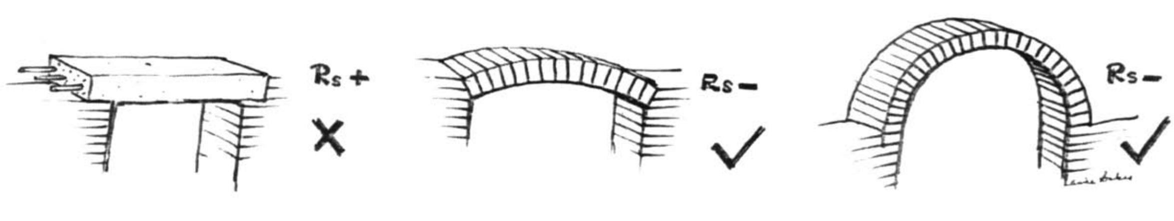


Figure 1.3: Design guide for lintel over door or window from Baker (1987)

The rise in interest in low-energy material solutions has brought compressed, cement-stabilized earthen masonry blocks (CSEB), and thus unreinforced masonry construction, into prominence as a sustainable material solution for building in the developing world. In the 21st century, international design team have delivered two innovative projects in Africa – the Mapungubwe National Park Interpretation Center in South Africa and the SUDU (Sustainable Urban Dwelling Unit) in Ethiopia – both utilizing CSEB and masonry vaulting techniques. While the Mapungubwe center is a daring, structurally expressive form in thin tile vaulting, the SUDU is a relatively simple design for a two-story house utilizing similar vaulting techniques. In India, organizations such as the Hunnarshala Foundation and Auroville Earth Institute (AVEI) aim to encourage sustainable architecture and development by providing technical and logistical support to local artisans and architects.* In conclusion, masonry vaulting can reduce total material usage and dependence on imported construction materials (steel, cement) while promoting the usage of local labor and embracing traditional building culture.

1.2.3: Indian construction industry

The population of India is over 1.2 billion people – it contains more than 15% of the world's population and it is projected to be the most populous country in the world (surpassing China) by 2022, per Maithel and Uma (2012). It is projected that the demand for building stock in India will increase by 400% between 2005 and 2030. A significant portion of this increase is expected to occur in urban regions – thus it is expected there will be a corresponding rise in demand for modern construction materials such as steel reinforcing bars and Portland cement. Even with the rise in popularity of reinforced concrete (RCC) construction in urban areas, the traditional clay fired brick remains the most popular building material in India. India accounts for more than 10% of all global brick production, producing about 150-200 billion bricks annually. The price of a single clay-fired brick ranges from 4-5 Indian rupees (Rs) in the North to up to 8 Rs/brick in the southern and western regions of India where poor soil precludes the large-scale production of structurally sound bricks. Per Indian Standard (IS) 1077, traditional clay-fired bricks have a minimum allowable compressive strength of 3.5 MPa (~500 psi). According to sources in India, high-quality bricks will have strength of about 7.5 MPa (~1000 psi). Clay-fired bricks are a widely-available construction material which can be utilized as the main components of unreinforced masonry roof construction.

The author, as part of an investigation into low-energy masonry [Laracy, 2015] in Muzzafarnagar, Uttar Pradesh, conducted research on current building costs and typologies as well as potential applications of the shallow dome spanning system for residential construction.† The dominant building typology in northern India is a reinforced concrete frame with masonry infill walls and a two-way RCC slab or one-way slab on beams as the spanning system. These

* For more information, see AVEI (2016) and Hunnarshala (2016).

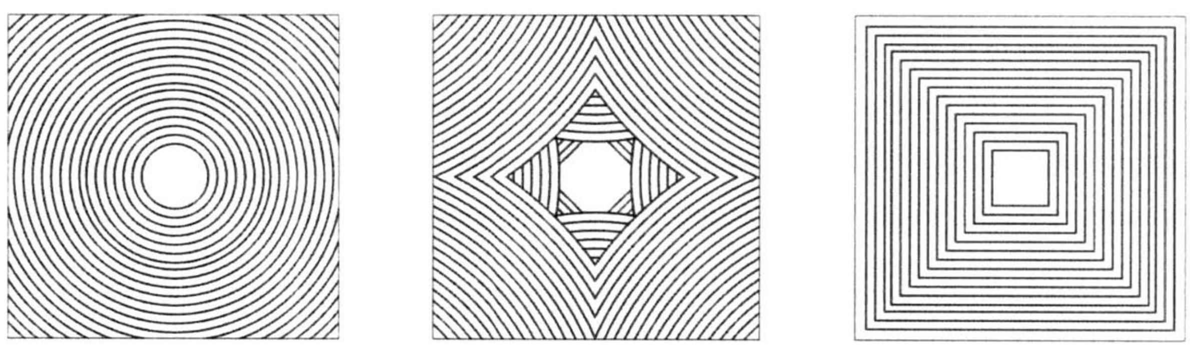
† The information in this paragraph comes from informal interviews conducted by the author in January of 2016 with Pankaj Aggarwal, the owner of Bindlas paper mill, Shri Madhukar Shyani, an architect, and an unidentified contractor, all in Muzzafarnagar.

systems are adopted primarily on the basis of material availability (all necessary materials except cement may be acquired locally) and social factors – local residents see RCC construction in wealthy Indian cities and desire the same for their own homes. Similarly, Indian architects and contractors are beholden to tradition – once a form (such as the RCC frame) is established, it is held as the standard method of construction and formal or structural experimentation is not common. In Muzzafarnagar (a city of over 300,000 residents), greater than 90% of all new construction is done with an RCC slab roof, with either a concrete frame or load-bearing masonry walls as the gravity system. In this region, the typical cost of this type of concrete construction is 150-200 Rs/sf – of which a significant portion (up to 1/3 of total cost) is the cost of steel reinforcing bars. The average size for a house in the city is between 1000 and 1500 square feet, which is relatively large. In the context of affordable housing, Gopalan (2015) gives 1200 square feet as the criteria for affordable housing in the "middle income group", whereas the minimum for the "low income group" where 99% of India's housing shortage is concentrated [Ministry of Housing and Urban Poverty Alleviation, 2007] is only 300 square feet. Another design driver in the residential sector is the relatively high price of land: Indians prefer to build a house with a flat roof, so that expansion in the vertical direction is possible without purchasing additional land. Essentially all small-to-medium scale construction of this type is non-engineered – architects refer to IS-456 (Indian concrete design guide) to design beams and slabs. For buildings in more rural areas not necessarily designed by a licensed architect, excessive reinforcement is often used in slabs and beams – while conservative on the basis of strength, there is great potential to increase material efficiency in this sector.

1.2.4: Existing design guidelines for unreinforced masonry construction

Although masonry vaults and domes have been designed and constructed in different forms in India for centuries [Tappin, 2003], there exist few official guidelines for design of new masonry spanning systems. The Auroville Earth Institute (AVEI) produces the manual "Building with Arches, Vaults, and Domes: Training manual for architects and engineers" [Maïni and Davis, 2015], a comprehensive summary of the stability and design of arches and vaults, using the graphical principles described in §1.2.1. However, the document offers limited guidance for the construction of domes. It correctly notes that the stability of domes generated by the intersection of vaults (groin and cloister domes) may be studied like the arch of their generating geometry, and that particular attention must be paid to the detailing of joints in a concrete ring beam used to resist the thrust of a dome.

The Rural Housing Knowledge Network (RHKN), an initiative of the Indian government's Ministry of Rural Development, lists on its Web site [RHKN, 2016] descriptions of housing technologies applicable to rural locations. As mentioned in §1.1, one of these technologies is the "funicular shell roof" – similar to the brick vaults which are the subject of this thesis. Both this guide and a report on the technology prepared by the Hunnarshala Foundation describe well the principal components and construction process of this roof system, reproduced here and in Appendix B. First, flat scaffolding or shuttering is constructed at the desired ceiling level. A concrete ring beam (design to be specified by an engineer) is cast, and an earthen mound is formed on the scaffolding as the form of the roof. After the ring beam has set, bricks are arranged in one of several possible patterns (see Fig. 1.4 below) and cement is poured over the top of the vault to tie the bricks together. After an appropriate amount of time, the scaffolding is removed. After the dome is set, an earthen or other lightweight fill can be used to level off the dome to allow its use as a flooring system.



(a) (b) (c)

Figure 1.4: Possible arrangements of brick courses for a flat dome

Neither the Hunnarshala documentation nor the RHKN guide provide instructions for ring beam design, detailing, or calculations for the required amount of tension steel. As per the Hunnarshala documentation, little is done to ensure proper shaping of the dome – the phrase "domical shape" is used, but no particular requirement is given besides ensuring the center of the dome is at the required rise. The RHKN guide does prescribe a shape for the dome – it gives values for the rise on a rectangular grid adopted from IS 6332, a design guideline for roofing/flooring systems composed of precast concrete shell units. The shape, given by Equation (1.1) below, consists of parabolas along sections cut on each major axis of a dome over a rectangular (a x b) plan. IS 6332 also limits the D/z ratio for domes of this type to between 10 and 20. This shape was developed by Suresh (1985) such that a thin concrete shell will develop only biaxial compression under uniform vertical loading, given:

$$z(x, y) = Z\left(\frac{4x^2}{a^2}\right)\left(\frac{4y^2}{b^2}\right) \tag{1.1}$$

This funicular shell shape was used in 1971 to construct a large vaulted space for a materials testing laboratory at SERC, the design and construction of which is described in George, et. al (1971). The authors correctly noted that the vault, if well-shaped, would experience low compressive stresses and used bricks with reinforced concrete ribs instead of a monolithic reinforced concrete (RC) dome. The authors also observed that using the equation above to determine the shape of the dome will produce areas of anticlastic curvature (saddle shape) near the corners of the square dome. Instead of changing the shape, the corners were simply more heavily reinforced and left supported by formwork for a longer period of time. The three domes each span 13.5 m x 12 m, with a rise of 1.58 m and a thickness of 10 cm. This gives values for D/z as 8.5 and D/t as 135. The cost of this roof is given as 470 Rs/square meter – considering historical rates of exchange*, this is approximately equivalent to 275 Rs (about $4)/sf today.

More recently, a team based in Switzerland and Addis Ababa has produced a complete design manual for the SUDU (see §1.2.2, Hebel et. al 2016). The SUDU contains two unreinforced masonry vaults, one a barrel vault using thin tiles in the Guastavino technique, and the second a dome using a leaning-brick method originated in Ancient Egypt and common in Mexico. The SUDU manual provides detailed instructions for construction of the example house, however for many critical components of the design (ring beam, rise of dome), the only instruction given is to consult an engineer. While this is understandable from the perspective of liability, one can imagine a design guide or series of design tables giving appropriate designs for these structural elements for common geometries and material parameters.

* https://en.wikipedia.org/wiki/Indian_rupee_exchange_rate_history#1974_to_1980

1.2.5: Caveats on construction in seismic zones

Masonry structures are susceptible to damage during seismic activity – in recent years, earthquakes in Iran (2003), Peru (2007), and Nepal (2015) have caused severe damage to infrastructure and loss of life. An adage in the engineering community, "earthquakes don't kill people – unsafe buildings do" rings especially true for unsafe masonry buildings. Due to the discrete nature of masonry construction and the high strength of its components, failure often occurs in a sudden mode. Unlike how a well-designed steel or reinforced concrete structure can fail in a ductile manner and allow evacuation before collapse, a masonry structure may undergo failure by forming a kinematic mechanism and collapsing suddenly. Due to their relatively high self-weight (especially true for stone vaults, less so for bricks), large horizontal forces are generated by seismic ground acceleration. DeJong (2009) gives an overview of existing (limited) seismic assessment strategies for unreinforced masonry structures and introduces new tools to predict the behavior of these structures, including tilting thrust-line analysis (used in this thesis).

Much of India, especially the northern regions, experience high seismic activity. The Indian seismic design code (IS-1893) contains a map designating locations as part of one of four earthquake zones, with each zone having a prescribed horizontal acceleration factor (comparable to C_s in ASCE 7-10) for design. These factors range from 0.10 (structures must be designed to resist 10% of their weight as a statically-applied horizontal force) in Zone II to 0.36 in Zone V. The Muzzafarnagar region is in Zone IV and the Kutch region of Gujarat, the location of the Hunnarshala Foundation's headquarters, is in Zone V. It is thus imperative to ensure the safety of this structural typology under seismic loading before it can be adopted in these areas. Dynamic analysis of the domes is beyond the scope of this thesis – while a simple, first-order approximation of the behavior under seismic loading is performed, this thesis intends only to validate the usage of the domes as a spanning solution in areas of low seismic activity.

1.3: Problem Statement

This thesis aims to validate flat masonry domes as a low-cost, low-energy roofing/flooring system for residential construction in non-seismic regions of India. The key components of structural performance are strength and stability – while the limits of masonry domes in these two respects are well understood in general, this particular structural typology has not been investigated in particular. In order to provide safe and sustainable housing using this technology, it is imperative that engineering guidance must be given to the local artisans and builders actually constructing these domes to ensure proper design and detailing.

This guidance is given primarily through two routes – first, by parametrizing the design of these domes, calculations using equilibrium methods to predict their load capacity are developed. Strengths obtained by these calculations are compared to the limited experimental results available. Second, issues related to detailing and construction are studied and recommendations are made to ensure the theoretical strength and stability are achieved in the field. Through a synthesis of these two approaches, a recommended shape for the dome – in order to resist loads economically and facilitate ease of construction – is presented and design tables which can be used to determine required material quantities are developed.

Chapter 2: Structural Design

2.1: Parameters

Based on the available documentation of this shallow dome typology, the design of a particular dome can be represented by several numerical geometrical and material parameters, as follows:*

Geometrical parameters:

- Dimension (X): the primary dimension of the enclosed area, given in feet.
- Aspect ratio (c): determines the shape of the dome, for $c = 1$ the dome is square, otherwise dome is rectangular with dimensions X by $c*X$.
- Rise (z): height of center of dome, given in inches.

Material parameters:

- Thickness (t): Usually the thickness of one brick (3.5"). While extremely thin masonry vaults are achievable, additional thickness acts as a safety factor against improper dome shaping (see §2.2.2).
- Brick strength (f_b): compression strength of bricks in uniaxial compression, given in psi. Per IS 1077, the minimum value of this parameter is 500 psi, although it is important to consider lower or higher strength masonry units in order to broadly validate the use of this technology.
- Tension steel (A_s): area of reinforcing steel in reinforced concrete ring beam, given in square inches. Typically, reinforcing steel is provided as deformed bars – usually with diameters of 8, 10, or 12 mm (giving areas of 0.078, 0.12, and 0.18 in^2, respectively).
- Steel strength (f_y): yield strength of steel rebars. Commercially-available high-strength rebars have strength of 500 MPa (~70 ksi), while mild steel has strength of 240 MPa (~36 ksi)

* Usage of units in India is strange. Metric and Imperial units are often used side-by-side – dimensions are usually given in feet and inches, while material strengths are usually denoted in MPa. Standard rebar sizes are also given in metric units (mm). This thesis attempts to replicate this convention with conversion factors included where appropriate.

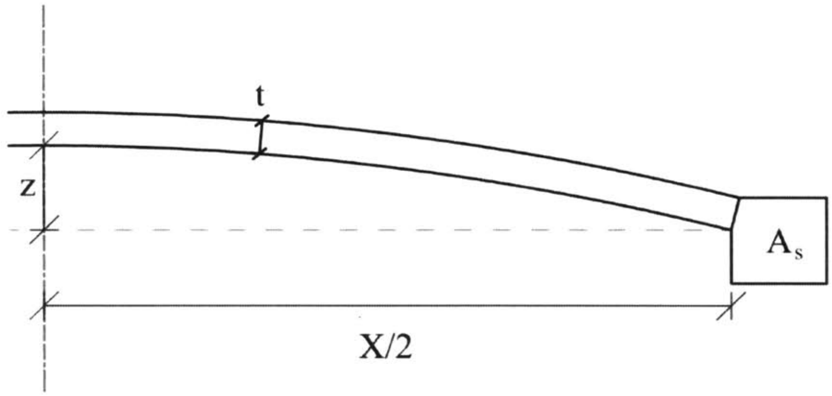


Figure 2.1: Section of example dome showing geometrical parameters

Two of the primary objectives of this thesis are to determine a reasonable maximum value for X – how large of a space can be spanned safely and economically by these flat domes – and a reasonable range of values for c – do the derived analysis methods apply to rectangular as well as square domes. These parameters can also be divided roughly into independent and dependent design variables. For example, one can imagine being given prescribed values of X and z and then determining required values of A_s, f_y, and f_b.

Depending on the configuration of the dome and the boundary conditions (how the dome interacts with the ring beam), the ring beam may also act in bending. If this is the case, the design of the ring beam becomes a separate parameter – it is necessary to calculate the flexural capacity (M_n) of the reinforced concrete section. This thesis focuses primarily on simply-supported domes which engage the ring beam exclusively in tension, however some configurations in which the ring beam is subjected to combined tension and flexural loading are also examined.

2.2: Performance under uniform loading

2.2.1: Design for strength

As discussed previously, the required capacity of both the steel rebars in the ring beam and the bricks forming the surface of the dome must be determined. As a continuous surface, there are infinite possible load paths through the masonry components of the dome – one cannot expect, and in fact has no real need, to determine the exact state of stress in a given brick or a given location. Instead, a stress path in equilibrium with the external loads leading to a mode of failure is postulated based on our knowledge of global behavior of the system and the external loads required to cause this failure are determined. While this method produces a collapse load which is a nonconservative (upper bound) solution for the collapse load, these values can be compared with test results and the choice of failure mode can be validated or refuted.

In 2015, the Hunnarshala Foundation performed two full-scale load tests on flat domes at their headquarters in Bhuj. These tests were performed on domes covering a 10' square plan with rises z at the center of 7" (intended to be 9", but settled due to improper compaction of earthen formwork) and 9". The domes were laid with compressed-stabilized earth blocks with a compressive strength f_b of 700 psi (~5 MPa) and a thickness of 3.5". The concrete ring beam contained 3 high-strength (f_y = 72 ksi), 10 mm Ø rebars (total A_s = 0.37 in^2). The first dome was loaded to failure by piling sandbags evenly onto its upper surface. The dome failed at an ultimate load of 155 psf. Significant cracking at the corners of the ring beam was observed and the surface of the dome collapsed inwards. The second dome was not loaded to failure, instead loading was stop when a yielding behavior (significant increase in deflection without corresponding increase in load) was observed at a load of 200 psf. Photos from these tests show no evidence of bending in the ring beam – there are no cracks visible on the exterior (see Fig. 2.2), which would be expected if any thrust was acting outwards on the beam. There is significant cracking in the corner regions: based on these load tests to failure, it is assumed that the load path within the dome carries the thrust to the corners of the dome.

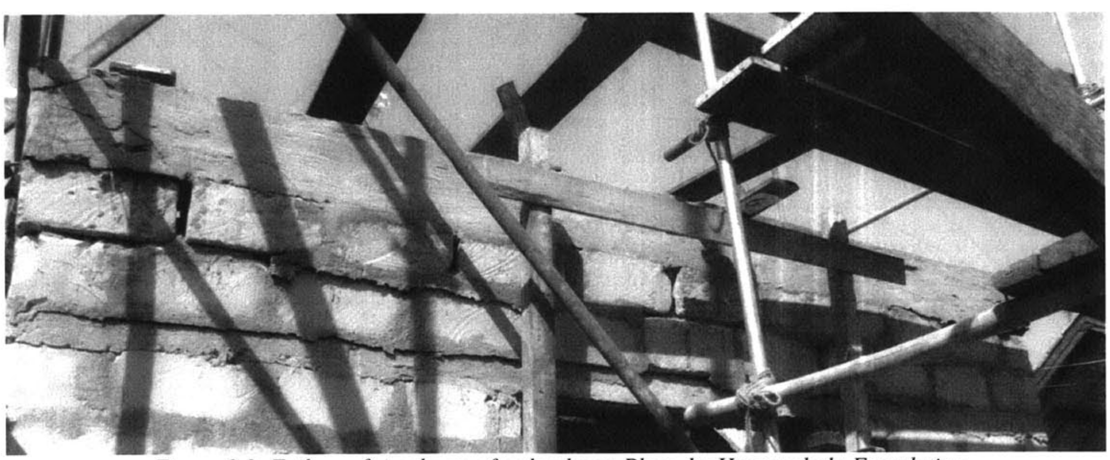


Figure 2.2: Failure of ring beam after load test. Photo by Hunnarshala Foundation

The simplest representation of this failure mode is considered first – the dome is idealized as a pair of arches spanning from corner to corner, as in Figure 2.3(a). Each arch has span D =

$\sqrt{2}X$, and the tributary area for half of each of these arches is a quarter of the total surface of the dome, resulting in a triangular distributed load along each arch with its resultant at the quarter-point. The free-body diagram of half of each arch is shown in Figure 2.4. Using moment equilibrium about the base of the arch, an expression is developed for the ultimate load σ_0 which will yield the tension steel.

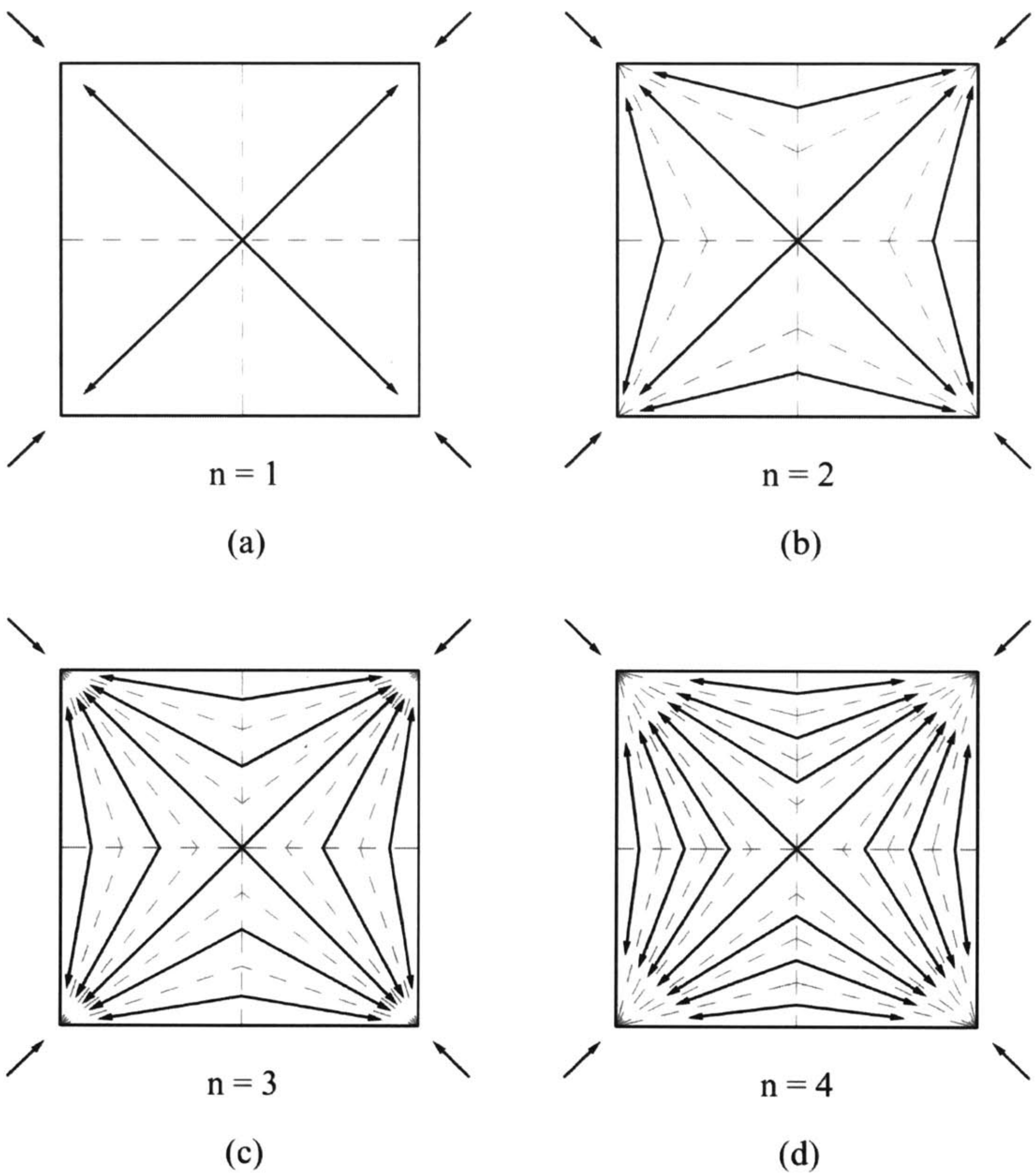


Figure 2.3: Possible stress paths within the surface of a flat dome

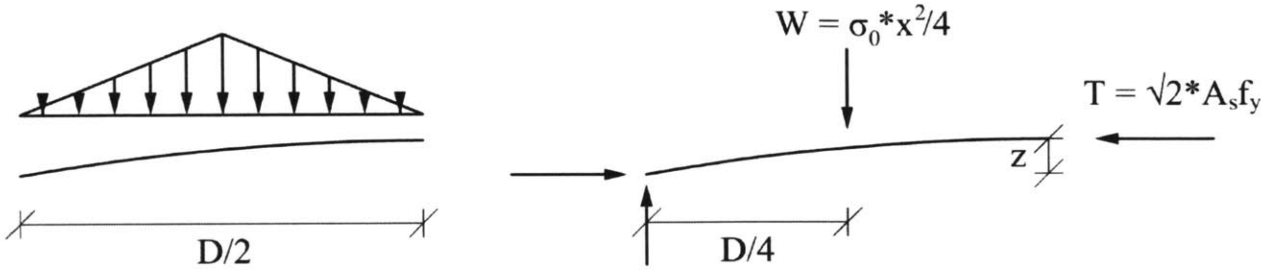


Figure 2.4: Free-body diagram of arch spanning from corner to corner of square plan

For a square dome with dimension *X*, rise *z*, and total reinforcing steel area A_s, the required uniform load σ_0 to cause failure is:

$$\sigma_0 = 16\frac{A_s f_y * z}{X^3} \tag{2.1}$$

We then consider the load required to load the bricks in the surface of the dome to their ultimate strength in compression. This is done by considering a 1-foot wide arch strip, also spanning in the critical direction (diagonally across the plan). The maximum compression in an arch occurs at the base and is the vector sum of the vertical and horizontal reactions at the support. Again using basic equations of equilibrium, an expression is developed for the ultimate load which will crush the bricks.

$$\sigma_0 = \frac{t * (f_b * 144)}{\sqrt{\frac{X^2}{2} + \frac{X^4}{16z^2}}} \tag{2.2}$$

Thus, the following equation is obtained to determine the capacity σ_0 of a square dome with dimension *X* (span $D = \sqrt{2}X$), and rise *z*:

$$\sigma_0 = \min\left(16\frac{A_s f_y * z}{X^3}, \frac{t * (f_b * 144)}{\sqrt{\frac{X^2}{2} + \frac{X^4}{16z^2}}}\right) \tag{2.3}$$

Equation (2.3) generates strength values which match closely (<10% error) the results of the two load tests performed by the Hunnarshala Foundation. A comparison of these results is presented graphically below. Both results are within the regime where the failure is governed by yielding of the steel reinforcement.

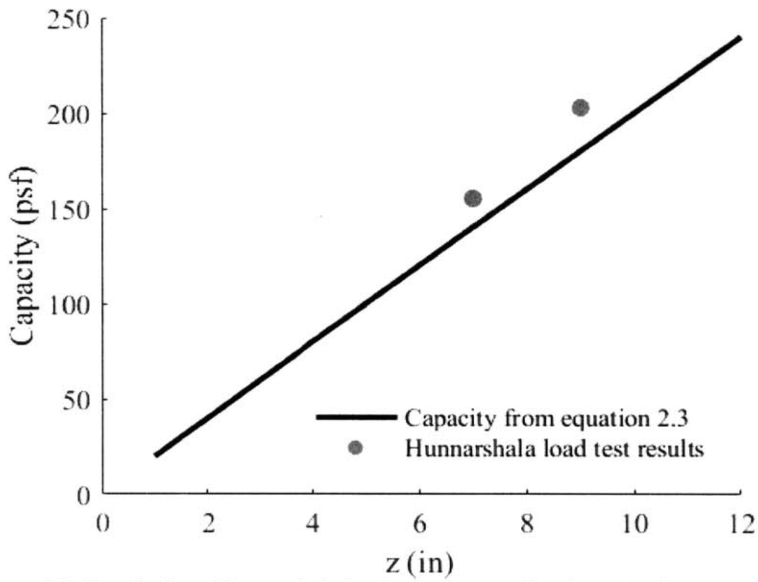


Figure 2.5: Results from Hunnarshala load test compared with results from Equation (2.3)

This failure is controlled by the yielding of the steel rebars in tension. For typical conditions, the failure of the dome is almost always controlled by this limit state (as opposed to the bricks crushing). Figure 2.6 below illustrates that even for designs with low brick strength and large amounts of reinforcing steel, Equation (2.2) will usually give a higher strength than Equation (2.1).

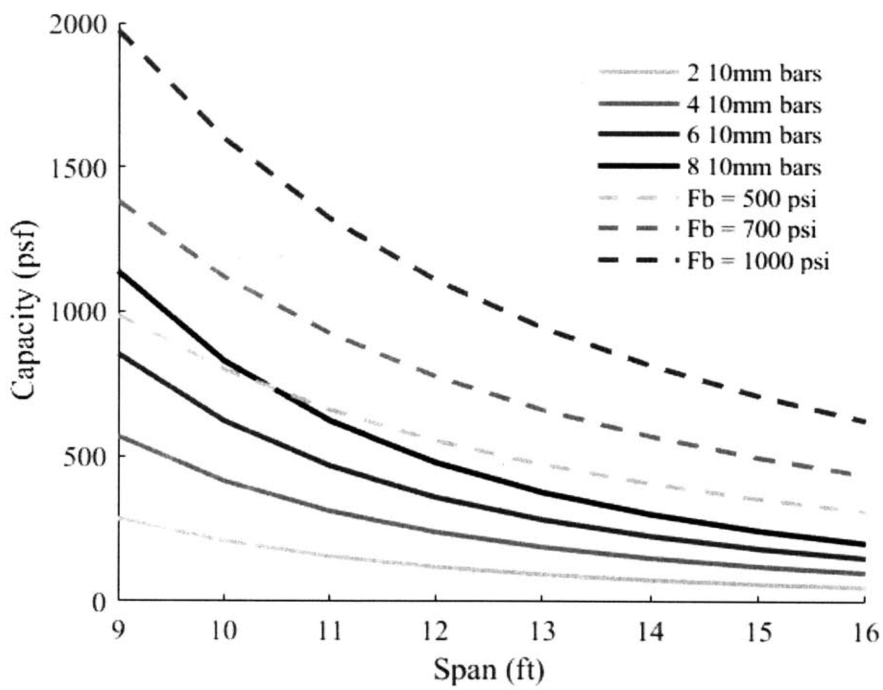


Figure 2.6: Comparison of dome capacity considering steel yielding (solid lines) or bricks crushing (dashed lines). All results shown for $z = 9''$, $X = 10'$, $f_y = 72$ ksi, $t = 3.5''$.

While Equation (2.3) replicates the test results with reasonable precision, we have drastically simplified the load path within the dome and have only generated an upper (unsafe) bound on the load capacity. While the compressive forces we have postulated within the dome – see Fig. 2.3 (a) – are in equilibrium with the external reactions (tension force in steel reinforcement), there are infinite other possible stress paths within the surface which we must consider in order to determine a safe value of the load capacity. Figure 2.3 (b) – (d) shows an extension of our original method of generating tributary areas while resolving all compressive forces in the dome as tension in the steel rebars at the corners. We postulate that each "slice" of the dome, under uniform loading, has an area (A), a centroid (r), an orientation relative to the diagonal (R), and an effective height (z). Each slice acts as an arch which leans against an equivalent slice across the dome. This equilibrium is shown in Figure 2.8 below, and we find σ_0 such that the sum of all T_i*R_i is equal to the capacity of the tension steel. Equation (2.1) can be modified to (2.1a) below, and we can use Equation (2.4) to find the scalar factor α as a function of n, the number of slices.

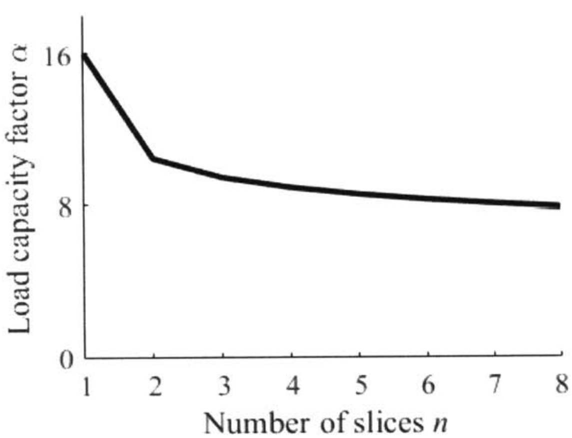


Figure 2.7: Variation of load capacity factor with n

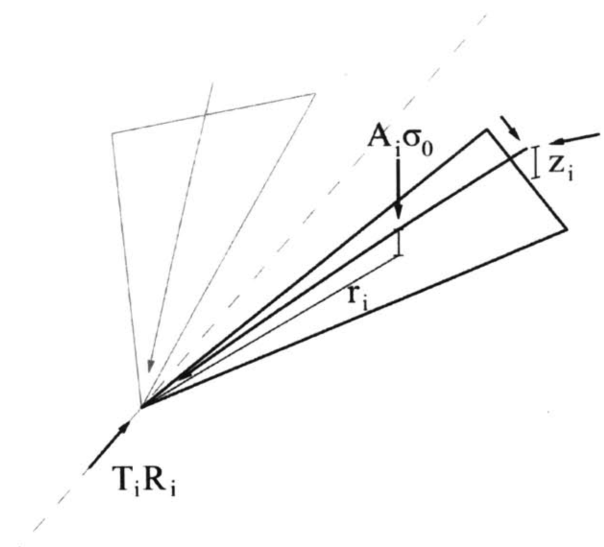


Figure 2.8: Equilibrium of arbitrary slice of dome

$$\sigma_0 = \alpha \frac{A_s F_y * z}{x^3} \tag{2.1a}$$

$$\alpha(n) = \frac{\sqrt{2}X^3}{Z * \sum_{i=1}^{n} \frac{R_i A_i r_i}{z_i}} \tag{2.4}$$

This method follows the fundamental tenets of plastic theory – the chosen stress path is in equilibrium with the external loads and reactions, and the postulated failure mechanism (yielding of the rebar at the corners and the spreading of the dome supports) will produce a kinematic failure mechanism. However, as n increases to infinity, $\alpha(n)$ decreases without bound. This is then, obviously, not an entirely accurate description of the flow of forces within the surface of the dome – experimental data and common sense confirm that the capacity of the dome is not 0, and indeed is close to the capacity predicted by taking $n = 1$.

As n increases, both A_i and z_i tend towards zero – however A_i approaches 0 as $1/n$ and z_i goes as $1/n^2$. This discrepancy leads to the apparent reduction in strength to 0 of the domes, and setting each z_i equal to the maximum z at the center of the dome results in α remaining constant at a value of 16 for all n. While extending the slicing method shown in Figure 2.3(a) – (d) is thus an inaccurate description of the path of forces in the dome as n gets very large, it is reasonable to assume that low values of n still provide a safe estimate for the load capacity of these domes. While further testing is recommended, a value of $\alpha = 8$ is recommended to be used for design – this produces very conservative values when compared with experimental results without significantly compromising economy.

Possible explanations for the discrepancy between the theoretical strengths predicted by Equation (2.1a) and the experimental results are as follows:

- The test results are inaccurate. When not placed carefully, a pile of sandbags (used for the load testing of both domes in Bhuj) can develop arching action and redistribute load to the edges of the pile (the load-bearing masonry walls). This would result in the measured capacity (the total weight of the sandbags) being higher than the actual capacity (the weight of the sandbags transferred to the actual surface of the dome).
- The postulated failure method is inaccurate. While there is photographic evidence suggesting that the ring beam is placed only in tension, it is certainly possible that some of the load on the surface of the dome is transmitted as a transverse load on the ring beam, introducing bending stress into the steel rebars. For future load tests it is

recommended to instrument the ring beam with strain gages on the inside and outside faces to determine the magnitude and distribution of bending stress in the ring beam (see §4.2).

- The assumed load distribution is inaccurate. We assume that the load on the surface is distributed evenly to each of our "slices" proportional to its area – however it is possible that the distribution is skewed towards the "slices" with a higher effective depth (closer to the center).* This can be forced in a way by designing the dome as a cloister dome (see §3.1) with creases at the corners – these act like the ribs in classical Gothic vaulting, attracting more forces to the creases or ribs and then to the supports.

2.2.2: Design for stability

These calculations for the dome capacity are derived with the assumption that the dome is "properly shaped" – that it is formed in such a way in which the compressive load path can travel entirely through the bricks which are the surface of the dome. Since the masonry units are assumed to be discrete units which carry only compressive forces, any applied bending moment will produce a hinging mechanism causing the dome to fail.

Both domes and arches carry applied loads to their supports through compressive stresses in their plane of primary curvature – however domes can further rely on stresses in the perpendicular or hoop direction to ensure stability. In this thesis, both in order to simplify calculation and to perform a conservative analysis, three-dimensional behavior is neglected – all stability analysis is considered for two-dimensional "slices" of the dome. As discussed in §1.2.1, the funicular shape to resist uniform self-weight in compression is a catenary – the shape of a hanging chain. For shallow arches such as those considered in this thesis, the loading under self-weight can be approximated as a uniform horizontal load, for which the funicular shape is a parabola. Both of these curves, for the ranges of values of interest in this thesis, can also be approximated by a circular arc.

* This is almost like applying elastic theory to the problem – assuming that "stiffer" (higher z_i) members will attract more load.

Figure 2.9: Parabola overlaid on catenary overlaid on circular arc for X = 12', z = 9". The three curves are indistinguishable at this scale. After Allen and Zalewski (2010).

These results can be obtained analytically by solving for a function, $y(x)$ for which there is no bending moment in the arch, or graphically, by using graphic statics. While the analytical model is convenient for loading conditions which can also be expressed as a function, graphic statics is utilized here in Rhino/Grasshopper in order to study the position of the thrust line parametrically, with the ability to easily observe the influence of different loading conditions.

For a uniform load case, the self-weight of the bricks, the weight of the fill used to level off the top of the dome, and an applied uniform load must be considered. The thrust line is generated by:

- Partitioning the arch into segments
- Assigning each segment a representative weight according to its tributary area
- Generating a load line from these weights
- Selecting a pole on a line perpendicular from the midpoint of the load line (for asymmetrical arches or arches under nonuniform loading the vertical position of the pole is generated by aligning the vertical reactions tip to tail along the load line)
- Drawing rays from the pole to each point on the load line
- Drawing a line segment parallel to each ray between points of application of the loads (midpoints of arch segments)

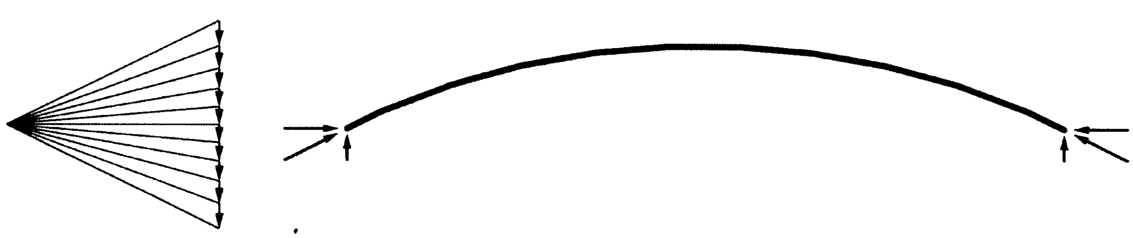


Figure 2.10:Thrust line generated using graphic statics, T = W

Figure 2.10 shows the thrust line and force diagram generated for an arch with a span of 12', a brick self-weight of 40 psf (5lbs/brick), and a fill weight of 150 pcf (packed earth), an applied uniform load of 100 psf, and a thrust equivalent to the total load acting on the system. During this process, the thrust (horizontal position of the pole) is an independent variable – by increasing or decreasing its value, the thrust line becomes shallower or deeper. Considering a parabolic arch, for any value of uniform loading a thrust value can be found for which the thrust line replicates the shape of the dome. This is obvious by looking at the force polygon of the entire arch as a single unit (Figure 2.11). As W (the total load acting on the arch) increases, in order for the segments of the thrust line to remain tangent to the geometry of the arch, the thrust must increase at a rate proportional to the increase in loading. Geometrically, as long as the triangle formed by the reactions and the applied load remains proportional as the applied load increases, the arch will be stable. In order for the thrust line generated and shown in Figure 2.10 to lie inside the thickness of the arch, the magnitude of the thrust is increased to twice the total load acting on the system. This value can be determined graphically, by simply varying the horizontal position of the pole until the rise of the thrust line is equal to the rise of the arch, or by considering the thrust as half of a couple resisting the moment due to the distributed load and obtaining $T = \frac{qD^2}{8z}$*. Because of the thickness of the arch, there are a range of thrust values which will produce parabolas that are statically admissible.

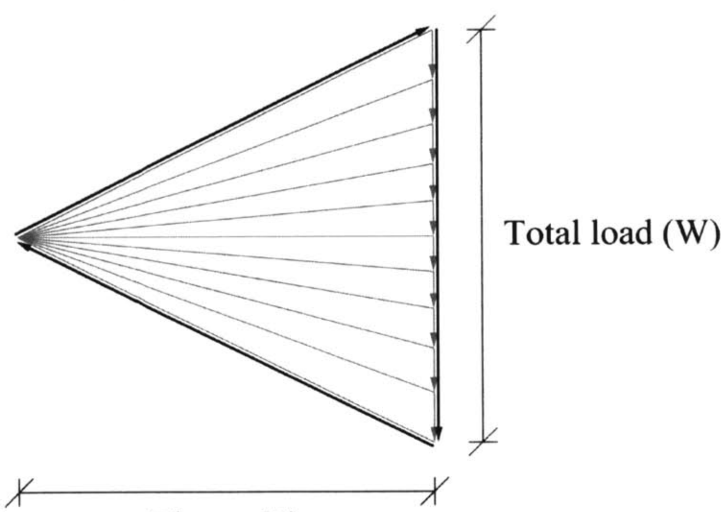


Figure 2.11: Force polygon showing relationship between applied loads and reactions

* While this equation only holds for a uniformly distributed load q, for the flat arches considered here it approximates the required thrust reasonably well.

These results show that for uniform loading, as long as the steel rebars can develop tension within the range determined graphically, the dome will not form a kinematic failure (hinging) mechanism. Thus, the upper bound postulated in §2.2.1 is also the lower bound and thus the unique collapse load – the yielding of the steel reinforcement will cause the supports of the dome to spread while the thrust is unable to increase, causing a collapse mechanism.

Figure 2.12: Minimum and maximum thrust lines for parabolic arch

Again, this statement is only true for domes which are "properly shaped" – have parabolas as their sections. While this is true of the dome shape prescribed by IS 6332 (described in §1.2.4), these parabolas span the dome in orthogonal directions, which per the (albeit limited) experimental results do not represent the flow of compressive stress within the dome. If at the profile of these domes along the diagonal is drawn, a suboptimal shape is revealed. Figure 2.13 below shows a dome with the same parameters that were investigated in the preceding paragraphs (X = 12', z = 9", brick weight = 40 psf, fill weight = 150 pcf, uniform load = 100 psf). While the thrust line is contained within the thickness of the bricks, this is only due to the stabilizing effect of the applied load. Without the additional 100 psf (shown as a dashed line in Fig. 2.13), the distribution of the load from the fill causes the curvature of the thrust line to become greater near the supports – in contrast to the reversal of curvature occurring in the shape of the dome.

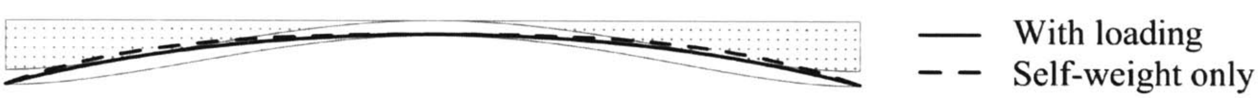


Figure 2.13: Thrust lines in IS 6332 dome shape

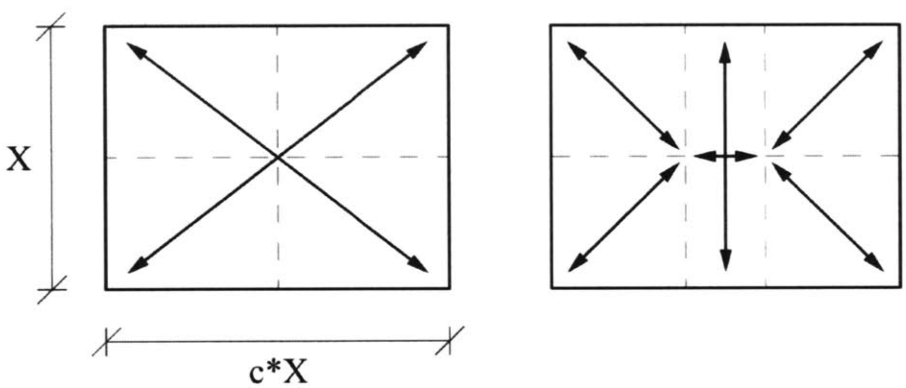
While the thrust line here is not contained within the thickness of the dome itself, domes of this shape can still stand – the hoop forces will certainly act to stabilize the thrust.* The

* For example, a triangular arch of any reasonable thickness will not stand, however a conical dome formed by revolving said triangle will stand because of the hoop forces, neglected here to generate conservative designs. See Lau (2008) for an extensive discussion of the role of hoop forces in dome stability.

compacted fill will also provide a load path for the compressive stresses, but since it is difficult to ensure "proper" behavior of the fill (and to avoid specifying additional material requirements) it is conservative to assume in design that for normal loading cases, the fill does not carry compressive stress.

2.2.3: Alternative shapes for domes

The results of §2.2.1 apply to square domes – however there is understandably significant demand for enclosing rectangular spaces. In order to determine the failure load for a rectangular dome, a load path and failure mechanism in equilibrium with external loads are again postulated and the required external loading to cause the failure is determined. First, the rectangular dome is again simplified in Figure 2.14(a) as two arches spanning from corner to corner of the rectangular plan.



(a) (b)

Figure 2.14: Possible stress paths for rectangular dome

Following the same procedure as for a square dome, the following expression is obtained for the failure load of a rectangular dome with dimensions X by cX, considering both the yielding of the steel rebars and the crushing of the bricks in the surface of the dome:

$$\sigma_0 = min\left(\frac{\alpha}{c^2}\frac{A_s F_y * z}{x^3}, \frac{t * (f_b * 144)}{\sqrt{\frac{X^2(1+c)}{4} + \frac{X^4(1+c)^2}{8z^2}}}\right) \tag{2.5}$$

As no load tests have been carried out on rectangular domes to date, it is impossible to verify the accuracy of Equation (2.5), however the trend it suggests is reasonable – as the aspect ratio increases, both the critical span from corner to corner and the load acting on that span increase, leading to a reduction in strength. However, as c increases it becomes more and more unlikely that the load is carried all the way to the corners and load being transferred to the ring beam in the transverse direction must be considered. Figure 2.14(b) shows an alternative load path for a rectangular dome displaying this behavior. The dome is split into three parts: the two ends acting as two halves of a square dome and thrusting out at the corners at 45°, and a rectangular section between them acting as a barrel shell and thrusting out perpendicular to the ring beam at its midspan. This will introduce bending stress into the rebars in the ring beam, and as mentioned in §2.1, the capacity of the ring beam to resist this bending stress (section modulus, S) becomes a parameter. The ring beam to be fixed at both ends and the thrust of the "barrel shell" section acts as a distributed load q at the midspan, generating a maximum moment of $M = q(c-1)X^2/16$. Assuming the rebars are distributed evenly in two rows a distance d apart, the section modulus S is $A_s d/2$.*

In this configuration, the rebars in the ring beam experience tensile stress ($\sigma_T = T/A$) and bending stress ($\sigma_B = M/S$). Setting the yield stress f_y equal to the sum of these two stresses, we obtain the following expression for the failure load do to the combined bending and tension stress in the reinforcing steel in the ring beam of a rectangular dome:

$$\sigma_0 = \frac{f_y * 144}{\dfrac{X^3}{16 * z * A_s} + \dfrac{X^4(c-1)}{128 * z * S}} \tag{2.6}$$

As expected, for $c = 1$ (square dome), the first term in Equation (2.5) is equivalent to (2.6). Figure 2.15 below compares the results of Equations (2.5) and (2.6) for an example rectangular dome with 4 10mm Ø reinforcing bars placed at a distance $d = 8$" apart. It is shown that Equation (2.6), taking the transverse bending of the ring beam into account will generally provide a more conservative (safe) value for the dome capacity.

* Since the ring beam is acting in tension, it is assumed the concrete is cracked and thus provides no bending stiffness.

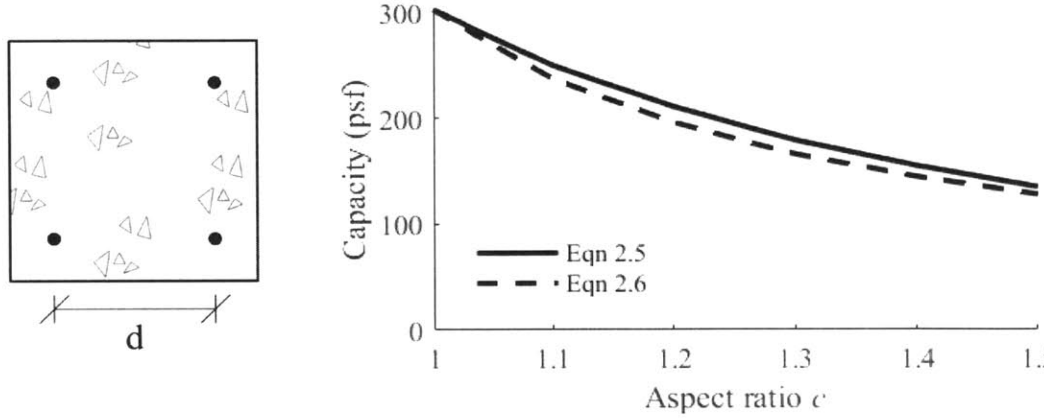


Figure 2.15: Comparison of results for Equations (2.5) and (2.6)

2.3: Performance under alternative load cases

2.3.1: Concentrated live loading

The parabolic shape described and analyzed in §2.2.2 is the optimal shape for an arch to resist its own self-weight. While this is the dominant load case for arches composed of relatively heavy masonry units, if they are to be used as a flooring system, these domes must be able to resist nonuniform live loading. For example, imagine a group of people standing at the quarter-point of an arch (again, the representative arch spanning from corner to corner of a square dome). The concentrated load (1.8 kN $\approx$ 400 lbs, concentrated live load requirement from IS 875) is added to the load line at the quarter point, and instead of the slope of the thrust line smoothly changing a kink appears at the point of this load application. P = 400 lbs represents a small load compared to the total self-weight of the dome – thus the thrust line remains relatively close to its original parabolic shape and is still contained within the thickness of the dome surface.

While in §2.2.2 we chose to disregard the fill above the surface of the dome when confirming the stability of a given shape, it is reasonable to rely on this material to carry the relatively small compressive forces generated by concentrated this live loading. Increasing the load shows that for any reasonable value of P a thrust line can be found within the fill and in fact, for a value of P which is infinite compared to the self-weight of the structure, two straight lines can be found within the fill carrying the load directly to the supports. Again, as in the case of uniform loading, it can be shown that the dominant failure mechanism is brought on by the spreading of the supports as the tension steel yields, not by the thrust line exiting the surface of the dome. The fill also acts to reduce the intensity of a concentrated load – a given load P will not act directly on the dome where it is applied but instead will be spread out over a larger area by the fill. Figure 2.16 below illustrates this fact – the addition of the fill for this example increases the capacity of the arch by almost 30%.

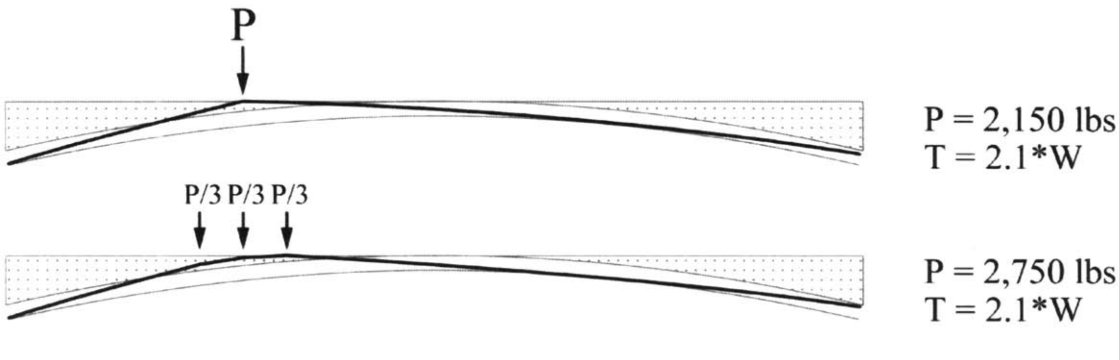


Figure 2.16: Effect of concentrated load on arch geometry, with and without spreading effect of fill

2.3.2: Lateral loading

While complete seismic analysis of these flat domes is beyond the scope of this thesis, it is important to understand generally their behavior under lateral loading in order to both direct future work and to ensure the safety of existing buildings of this typology. A first-order approximation of masonry behavior under lateral loading is a tilt test* – the masonry geometry is tilted, introducing a lateral component of force which is a percentage of the vertical force depending on the tilt angle, β. While this tilting reduces the magnitude of the vertical component of the force, the compressive stresses in the masonry units are low and the failure mechanism to be investigated is the instability brought on by lateral loading, so the parameter of interest is the relative values of horizontal and vertical acceleration, not their magnitudes. This ratio is:

$$\gamma = \frac{a_h}{a_v} = \tan\beta \tag{2.7}$$

The ratio of *H/V* is commonly used in seismic design codes as a design variable for lateral loading – a structure must be designed to resist a prescribed fraction of its self-weight applied laterally, depending on the seismic characteristics of the region and the structure itself. To affirm the stability of a masonry structure, a model can be tilted through an angle $\beta = \tan^{-1}(H/V)$ and the thrust line observed graphically – as established in §2.2.2, if it remains within the structural

* This method is adapted directly from Chapter 3 of DeJong (2009).

depth of the arch or dome the masonry structure is stable. The tilt test is modeled parametrically in Rhino/Grasshopper in order to observe the changing position of the thrust line as the arch is tilted in real-time. Figure 2.17(a) below shows an arch representing the diagonal span of a dome with $X = 10'$ and $z = 9''$ subjected to a tilt of $\beta = 21°$, which represents applying a horizontal force which is 36% of the total vertical load – the value prescribed for India's most seismic region, per IS 1893. Due to the flat geometry of the arch and the stabilizing effect of the fill, the arch easily retains its stability. Figure 2.17(b) shows an extreme situation: a horizontal force equivalent to the total vertical load on the arch. A thrust line can still be found within the depth of the arch system.

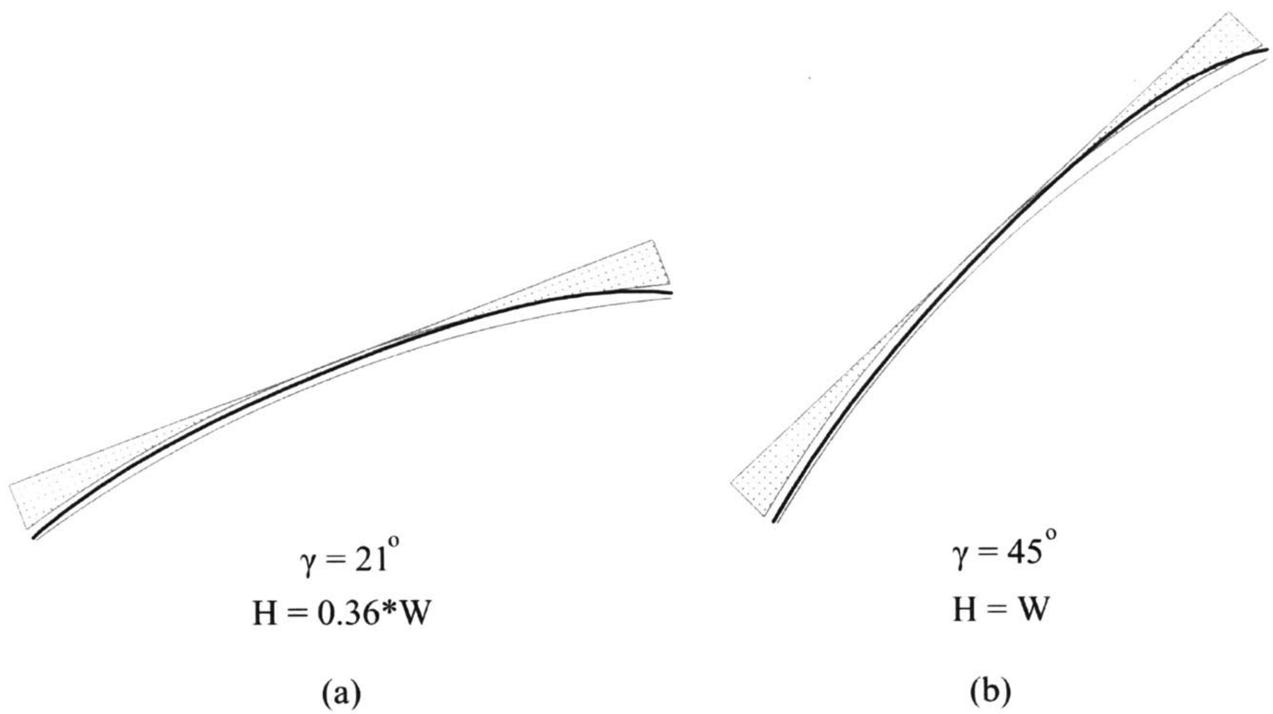


Figure 2.17: Arch with lateral force equivalent to (a) 0.36W and (b) W applied statically

These results show that this arch geometry can remain stable even with the application of a significant lateral load. However, the graphical analysis assumes purely static loading and ignores potential changes in the support conditions. Masonry arches, especially shallow ones, are highly susceptible to support displacement. In an earthquake, depending on the supporting structure, it is possible that there will be a spreading of the supports of the arch spanning the dome from corner to corner. For the flat, thin domes which are the subject of this thesis, the critical support displacement is on the order of 0.5% to 1.5% of the span *D*.* While these values are generated for two-dimensional loading and are therefore conservative, they are still low and will control the failure of the dome under lateral loading. For example, a dome spanning a 12' square plan with a rise of 9" will collapse if there is a differential displacement of only ¾ of an inch across the diagonal. As this failure mechanism is sudden (collapse due to hinging forming in the dome), it could easily lead to loss of life during a seismic event. For this reason, until further seismic testing and characterization of the supporting structure (usually concrete ring beam on load-bearing masonry walls) are performed, it is not recommended to construct these domes in regions with significant seismic activity.

The critical displacement values also serve to give reasonable upper limits on the span of these flat domes. As discussed in §2.2, as the dome is loaded the thrust is resisted by tension in the reinforcing steel in the concrete ring beam. This tension will cause the steel bars to extend elastically, introducing a small displacement across the span of the dome. In order to fully realize the strength of the rebars, the elastic strain must be less than the critical strain which will cause failure. For a given span, rise, and thickness there is a corresponding critical span increase (strain) which cause collapse – Figure 2.18 below shows these values compared with the yield strain of the steel reinforcement. The thickness *t* is taken to be 1 brick (3.5").

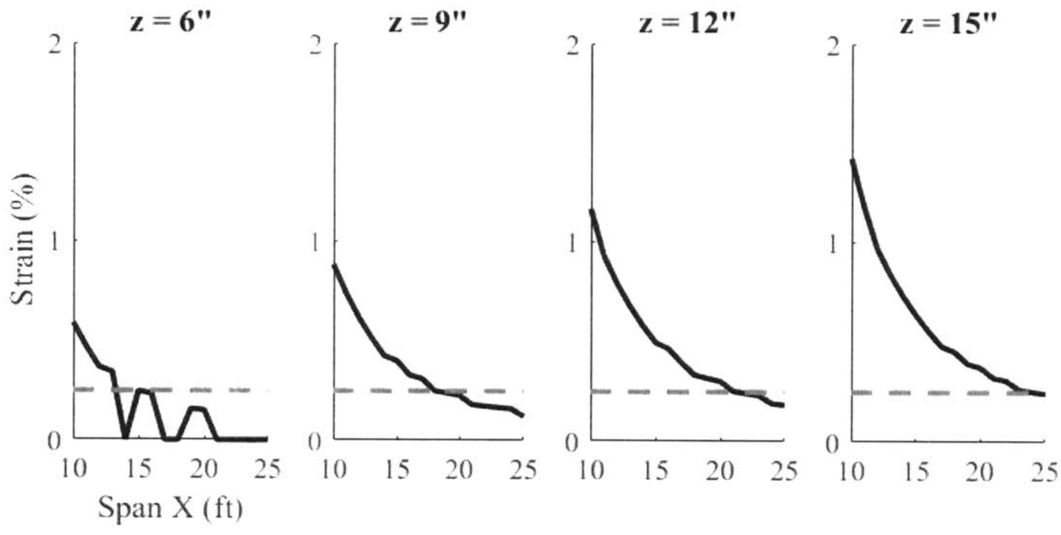


Figure 2.18: Comparison of yield strain of reinforcement (dashed line) with critical strain to cause dome collapse (solid line)

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* These results are generated using MATLAB code adopted from ArchSpread.m found in Appendix A of Ochsendorf (2002).

2.4: Chapter Summary

This chapter has developed expressions for the load capacity of square and rectangular domes under uniform loading and affirmed their stability under concentrated and lateral load cases. Equilibrium methods are first used to determine a failure load, σ_0 which will yield the tension steel in the ring beam or crush the bricks in the surface of the dome. Graphical analysis is then performed to determine the required shape of cross-sections of the dome such that the compressive network representing the flow of stress within the dome is always contained within the thickness of the masonry units used. Graphical analysis is also performed to study the response of the domes to concentrated and lateral load cases and the stabilizing influence of a compressed earth fill above the dome is observed. These two analyses are combined to show that for a well-shaped dome, the governing failure method will be the formation of a hinging mechanism due to spreading of the supports caused by yielding of tension steel in the ring beam. This result is also applicable to lateral loading – failure under seismic action will be induced not by the application of horizontal acceleration to the dome itself, but to differential displacement of the support conditions (ring beam).

Chapter 3: Construction

The previous chapter presents a method for determining the load capacity of a shallow masonry dome. While the values obtained are useful to develop a design method for the usage of these domes, it is equally important that proper construction procedure is followed in order for the theoretical strength to be realized in the field. This is especially important in the developing world, where construction administration (inspection of construction sites, reviewing of structural detailing) is not prevalent (and often nonexistent). In this chapter, the construction process for the structural elements of the dome is reviewed and critical details are discussed. Refer to §1.2.4 and Appendix B for a description of the existing dome construction process – left largely un-modified to encourage adoption by local builders.

3.1: Earthen formwork

As discussed in §2.2.2, it is imperative that the dome be shaped such that the compressive load path can always travel through the surface of the dome. The form of a masonry structure is ensured during construction by two elements: formwork, a load-bearing structure which bears the weight of the structure while under construction, and guidework, a system of non-load-bearing geometrical guides which describe the form of the masonry structure to be laid. Since much of the recent work done on masonry construction in the developing world is done in Africa (see §1.2.2), where timber is scarce, methods which do not require formwork (tile and Mexican vaulting) are the subject of most recent research. On the other side of the spectrum, research in developing new masonry structures is often concerned with generating daring, structurally expressive new forms which require complex wooden or even 3D-printed form and guidework to realize.

The domes that are the subject of this thesis occupy neither of these spaces – due to the prevalence of RCC slab construction in India, access to scaffolding is not an issue (wooden or metal scaffolds can be obtained for 10-25 Rs/sf or reused from a previous project). Traditionally, earthen mounds are used as load-bearing formwork during construction – thus the problem becomes how to use guides to generate the proper shape for these low-tech forms. Another advantage in using earthen formwork is that reliance on external consultants is limited – for example, the use of tile vaulting requires extremely high tolerance in the early stages of construction in order to achieve the proper form. A goal of this thesis is to generate design guidelines widely usable by local architects and builders: it is thus beneficial to reduce reliance on masonry vaulting experts who are generally concentrated in the US and Europe.

In order to ensure proper shaping of the dome, we consider again the critical span – diagonally from corner to corner across the square or rectangular dome plan. If these sections are parabolas, the surface of the dome will be stable under the design loads. Two methods for generating the parabolic shape are described below: depending on the local conditions and available materials, either can be used.

Elastica method

If an axial load is applied to an elastic rod with some resistance to bending (*EI*), the resulting shape is sinusoidal and is a reasonably good approximation of the required parabola.* In order to generate the shape of a dome from the design parameters (X, z), the initial length (L) of the rod (usually a small PVC pipe or steel rebar) must be determined. Equation (3.1) gives the L needed to span the diagonal D of a dome with rise at the center z.†

$$L = \sqrt{\frac{D^2}{4} + 4z^2} + \frac{D^2}{8z}\sinh^{-1}\left(\frac{4z}{D}\right) \tag{3.1}$$

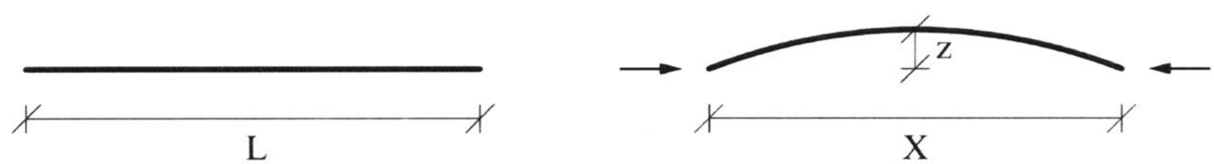

Figure 3.1: Elastica method for generating dome guidework

Circular arc method

As shown in Fig. 2.9, for the geometries in question (very shallow domes), a circular arc is also a reasonable approximation of a parabola. A rigid section of a circular arc can thus be used as guidework for the formation of the earthen formwork. A circular arc can be easily constructed by fixing a center point and tracing the arc between points distance D apart using a rod of a determined radius R. The resulting line can either be used to cut guidework from wood

* Maximum error of 5.6% of the rise, about ½" for a dome with $z = 12$"

† For square domes, $D = \sqrt{2}X$, and for rectangular domes $D = X\sqrt{1 + c^2}$

or generate a rigid guide in another way. Equation (3.2) below gives the radius R used to generate a circular arc spanning the diagonal D.

$$R = \frac{\frac{D^2}{4} + z^2}{2z} \tag{3.2}$$

Regardless of method, two of these guides are generated and placed so that they span corner to corner. The easiest way to complete the dome shape after the guides have been set is to simply use string to generate straight lines between the guides at regular intervals. This shape is known as the cloister dome, and like a groin vault (Fig. 3.2(b) below) is formed from the intersection of two vaults (here, parabolic vaults). The surface of the groin vault is the envelope of this intersection visible from above, while the cloister dome is the same envelope viewed from below. Apart from being easily constructible, the creases on the diagonals of this shape also serve to attract more of the compressive forces in the surface of the dome to the corners – avoiding placing the ring beam in bending. The disadvantage of the cloister dome is that every location only has single curvature – reducing the ability of the dome to resist bending induced by a concentrated load. This is mitigated by the stabilizing effect of the fill above the dome (see §2.3.1) and by the effect of the intersecting arches.

(a) (b) (c)

Figure 3.2: Intersection of parabolic vaults (a), groin vault (b), cloister dome (c)

As reduction in height is directly detrimental to the dome's strength capacity, it is important that any material used as formwork must be not undergo settlement – for example, scaffolding must be placed and secured such that it will not deform, and earthen layers used for formwork should be compacted at regular intervals.* Per the Hunnarshala documentation of

* No less than 10cm, per personal communication with Lara Davis in March of 2016, masonry expert with ETH/AVEI.

current construction practices, bricks are generally laid with a loose spacing and thin, workable mortar is poured over them in order to fill the cracks. It is recommended that this practice is changed to laying the bricks in a regular rectangular pattern while laying mortar between each individual course. Loose brick spacing coupled with the wet mortar required to infiltrate that space will lead to shrinkage cracking in the mortar and the reduction of the ability of the dome to act as a monolithic surface.

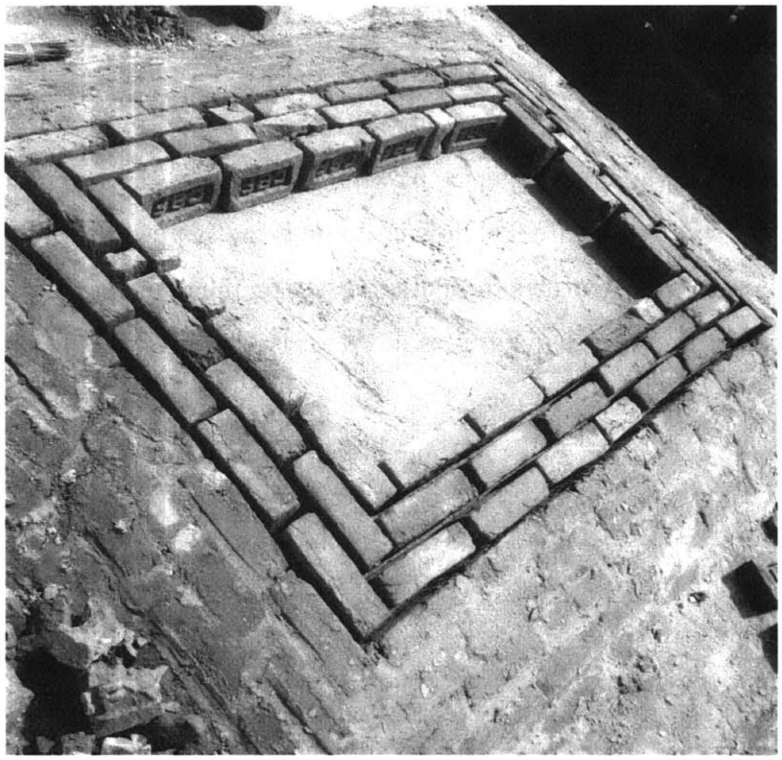

Figure 3.3: Suggested arrangement of brick courses for flat dome

3.2: Design of ring beam

The concrete ring beam (specifically, the rebar inside it) resists the thrust generated by the shape of the dome and its design is equally important in ensuring the dome's safety and stability. It is important to note that any amount of "required steel" referenced in this thesis refers to an area of steel required to resist the thrust of the steel in tension – NOT flexural reinforcement. While a concrete beam which spans a space can also be used as a ring beam for a shallow dome (for example, a long rectangular space subdivided into two squares which are each spanned by a square dome), any required area of steel must be included as reinforcement in addition to any flexural reinforcement designed in accordance with a recognized design code (IS 456, ACI 318, etc.). When placing the tension steel, it is recommended to locate the rebars as far apart in the horizontal direction as possible to resist any horizontal thrust from the dome. Relevant requirements from these codes regarding rebar placement and general detailing are compiled in the design guide in Appendix A. The most important of these is ensuring that there is proper concrete cover (1.5" or 40mm) over any steel reinforcement – corrosion of reinforcement due to poor cover is the leading cause of reinforced concrete failure in India today.* Design codes also give required minimum clear spacing between bars – these requirements ensure sufficient bonding between the concrete and the reinforcing bars, ensuring the beam as a whole can develop the full strength in tension of the rebars.

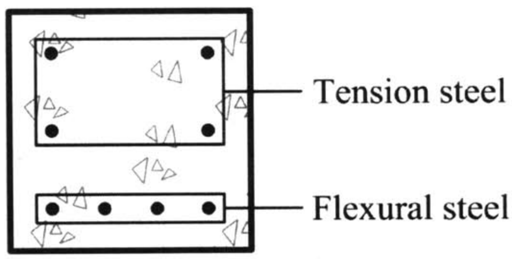

Figure 3.4: Suggested arrangement of reinforcing steel in ring beam section

Another critical detail in the casting of the ring beam is the arrangement of reinforcement at the corners and joints. Again, requirements for overlap lengths and dimensions of hooks are compiled and presented in the design guide in Appendix A. In general, at corners and all intersections (X, T), an overlap bar must be provided for a splice length L_s (see requirements in Appendix A) in order to ensure complete transfer of tensile force. This detail is crucial – since the controlling mechanism for dome failure is brought on by yielding of the tension steel, it is imperative that this capacity can be developed in all rebars. For larger ring beams, it is recommended to avoid using bars which are hooked at the ends (see Fig. 3.4 below) to facilitate the assembly of a rebar cage. For smaller beams with only 2 to 4 bars, using hooks is acceptable.

* Personal communication with Lara Davis, March 2016.

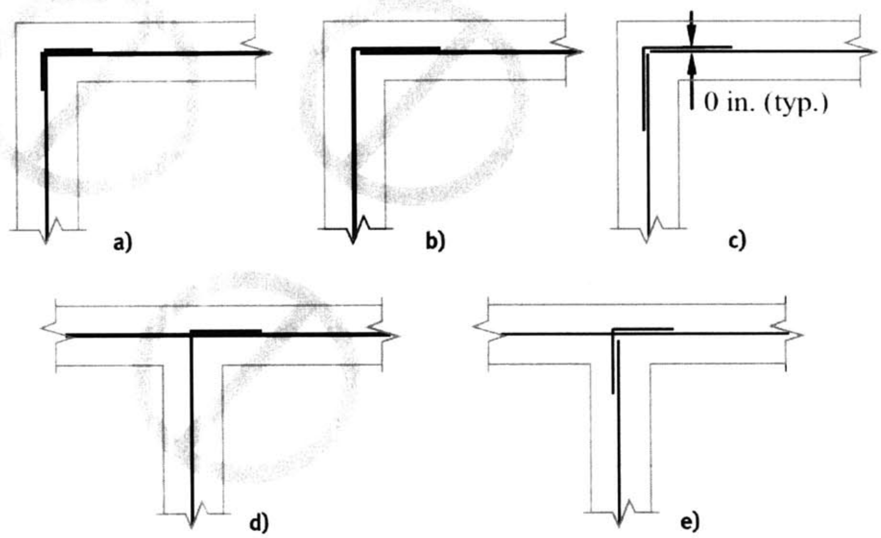

Figure 3.5: Recommended corner detailing, from CRSI (2009)

3.3: Use of design guide

The design guide presented as Appendix A to this thesis consists of three parts:

- Review of construction process with general best practices for masonry construction
- Collected requirements for design of ring beam per IS 456, ACI 318
- Design tables

The first two sections are fairly self-explanatory – the first gives qualitative requirements for safe, sound masonry domes and the second should be followed closely when placing the steel reinforcement. Each design table corresponds to an available strength of bricks from 500 to 1000 psi (3.5-7 MPa) and an available strength of steel reinforcement, either 36 or 72 ksi (25 or 50 MPa). For a desired span*, required rebar quantities are given. The design tables use a safety factor of 2 against yielding of the rebars and 4 against crushing of the bricks. α= 8 is used (see §2.2.1). The ring beam is required to have at least 4 rebars (to provide reasonable resistance against transverse bending) and no more than 12 (to avoid crowding). The design guide also

* Currently, design tables are currently generated only for square domes. For rectangular domes, a good first approximation is to multiply the required steel area for a square dome with a dimension X of the short dimension of a rectangular dome by c^2, and then use Eqn (2.6) to determine the capacity of the dome once S is calculated from the chosen rebar layout.

contains 2 sample calculations illustrating the use of the design tables and the equations for dome capacity developed in §2.2.1 and §2.2.3.

3.4: Material efficiency

The typical spanning system in northern India is, as discussed in §1.2.3, a two-way concrete slab. Typical reinforcement for the spans in the range considered in this thesis is 8 or 10mm Ø bars spaced at 6 inches on center.* Using the design tables in Appendix B, we can obtain the required amounts of steel for domes of various heights, and compare with the total weight of steel in a concrete slab. Figure 3.6 shows this comparison graphically – it is possible to achieve up to 60% savings in steel weight for domes with lower span and higher rise. In general, usage of domes results in a reduction of total material usage compared with casting a concrete slab, but an increase in required amount of labor, and the introduction of a new construction typology. Altogether, using flat domes in place of a concrete slab promotes local material usage (bricks with the required strength to be used in flat domes can be produced anywhere in India) and reduces quantities of materials which are not produced locally and must be imported (steel, cement for use in concrete).

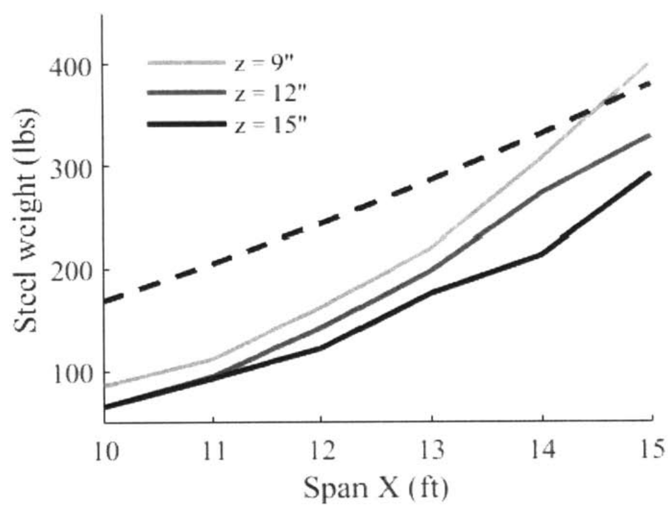

Figure 3.6: Comparison of steel weight for concrete slab (dashed line) and flat domes

* Per personal communication with architects and contractors in the Muzzafarnagar region.

3.5: Chapter Summary

This chapter has discussed the construction process of the two major components of a flat dome: the surface of the dome which carries loads through its form and compression in the masonry units, and the concrete ring beam which resists the inclined thrust at the base through tension in its steel reinforcement. Methods for realizing the theoretical capacities of these two elements in the field are discussed. The cloister dome shape is recommended for three reasons:

- Guidework is generated easily using elastica or circular arc methods.
- It facilitates proper laying of brick courses, minimizing mortar quantities.
- Ribs along the diagonal will direct thrust to corners of plan, avoiding transverse loads on the ring beam.

Discussion of proper detailing for the concrete ring beam is also given. It is important that requirements in relevant building codes are met so that the strength of the ring beam in tension can be fully developed. An explanation of the contents of the design guide in Appendix B is given, and it is shown that these flat domes generally use less material than a typical concrete slab for reasonable spans.

Chapter 4: Conclusion

4.1: Summary of findings

This thesis set out to perform design calculations on a non-engineered spanning system – shallow, square or rectangular masonry domes – in order to ensure their safety and stability. Due to the highly indeterminate nature of masonry structures, elastic analysis cannot accurately describe their behavior. Equilibrium methods and plastic theory are used to determine a collapse mechanism and associated load. The capacity of the masonry units in compression and the reinforced concrete ring beam in tension are considered for uniform, concentrated, and lateral load cases.

Graphical methods are used to ensure the stability of these domes for all three load cases. It is determined that for a well-shaped dome, the failure mode is likely to be collapse induced by the yielding of the steel reinforcement causing the dome supports to spread without a corresponding increase in thrust. In general, the thickness of one brick laid on its side (t = 3.5" or 90mm) as well as a layer of compacted fill above the surface of the dome is sufficient to ensure stability in the case of asymmetrical or concentrated loading. While graphical methods can ensure the stability of most flat dome geometries under significant lateral loading, the critical factor for resistance to a seismic event is the supporting mechanism of the dome – small relative displacements of the supports can cause collapse.

The construction method of these domes is also reviewed. Simple methods to generate guidework to shape load-bearing earthen formwork into a cloister dome shape are presented. While there are infinite possible ways to shape an earthen mound into a form which will produce a stable, compression-only dome, the cloister dome shape – see Fig. 3.2(c) – is both generated easily and will generate stable forms.

These domes are determined to be safe and cost-effective compared to a two-way reinforced concrete slab for spans on the order of 10 to 15 feet (3 – 5 meters) with a rise of 9 to 15 inches (25 – 40 cm). This range of spans is governed by three factors:

- Spreading of supports: as the tension steel is loaded, it will deform elastically. It is desirable that the critical support displacement to cause collapse is not reached before the reinforcing steel reaches its full capacity. See Fig. 2.18: for a given z and t, there is a critical span which should not be exceeded for which the dome will form a collapse mechanism before the steel reinforcement reaches its yield point.

- Capacity of materials: As the span increases, so does the compression in the surface of the dome and the tension in the reinforcing steel. Depending on strengths of available materials and geometrical constraints, there are upper bounds on the ability of the bricks in the surface of the dome and the steel rebars in the ring beam to resist these applied loads. The design tables in Appendix A show these limits – the usage of a dome which is only one layer of bricks thick limits the span such that there is a sufficient factor of safety against brick crushing. The size of the ring beam and requirements for concrete cover and clear spacing between rebars limit the area of steel within the ring beam.
- Material economy: While there is obviously a significant reduction of usage of concrete for these shallow masonry domes compared to a concrete slab, the conservative design method developed here does not result in a reduction of total quantity of reinforcing steel and will always incur additional labor costs. While the exact nature of these tradeoffs depends significantly on local material availability, Figure 3.6 illustrates a "breakeven" points for total volume of reinforcing steel.

4.2: Future work

There are several further opportunities to continue the study of these domes in order to further assert their level of performance. Full-scale testing, while costly, provides an unparalleled opportunity to confirm or refute the assumptions made in this thesis on the structural behavior of these domes.

4.2.1: Future testing regime

Instrumentation of ring beam: For essentially all calculations performed, it is assumed that the reinforced concrete ring beam is acting purely in tension. For any future testing, placing strain gages on the inside and outside surfaces of the ring beam will prove whether or not this assumption is accurate: the magnitudes of these two values will allow the state of stress in the ring beam to be determined.

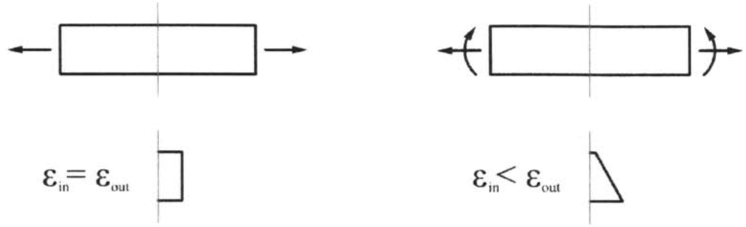

Figure 4.1: Instrumentation to determine state of stress in ring beam

Testing of rectangular domes: Equations (2.5) and (2.6) give two estimates of the capacity of a rectangular dome, but since no testing has been performed on this geometry their validity is not certain. Depending on the relative values of c, X, and z, thin shells can act in many different ways (for example, long cylindrical vaults can act like deep beams). Further analysis using membrane theory combined with full-scale testing could further characterize the behavior of flat, rectangular domes.

Seismic behavior: As discussed in §2.3.2, the critical element in the lateral resistance of these domes is the supporting structure. It is unlikely that unreinforced masonry walls will provide the required rigidity to prevent collapse due to differential motion of the supports in a seismic event. The application of confined masonry – see Porst (2015) – to the supporting structure could be relevant in providing the necessary resistance and is an avenue of future investigation. Further analysis of the surface of the dome such as performing dynamic testing on partial or full-scale models will be useful in demonstrating their capacity (or lack thereof) to resist seismic loading.

4.2.2: Material characterization

While the strength of the bricks used in the surface of the dome and the reinforcing steel has been parametrized and incorporated into the methodology of this thesis, the behavior of the mortar used between bricks is not considered in detail. Further investigation into the relationship of mortar strength (both absolute and relative to brick strength) and behavior of the masonry could provide information useful for the implementation of these domes in the field. A study of mortar mixes will also allow for a more accurate comparison of total material usage when compared to a concrete slab flooring system.

4.2.3: Continuation of 3-dimensional analysis

As discussed in the literature review of this thesis, the behavior of three-dimensional masonry structures is complicated and computationally intensive. While the goals of this thesis were to focus on simple calculations applicable to low-tech design in the developing world, further high-tech analysis could serve to refine the design guidelines generated here. The significant open question is the behavior of the stress path as discussed in §2.2.1: how can the highly indeterminate flow of compressive stresses within the surface of these domes be analyzed in a way which generates methods usable for *safe design*. While this thesis performs a reasonably accurate analysis of this behavior, future work in determining an more precise value of the load capacity factor α and testing the validity of the expressions for load capacity developed in §2.2.1 is welcomed.

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Appendix A: Design Guide

This document is intended to provide guidance for architects and builders who wish to use these flat domes as a roofing system. This guide has been produced as the result of a master's thesis and has not been reviewed or approved by any relevant building authority. It is recommended that a licensed engineer review all drawings and calculations before construction begins.

Required materials

- Masonry units with a minimum strength of 3.5 MPa (500 psi)
- Steel reinforcement with a yield strength of at least 240 MPa (36 ksi)
- Cement/mortar: mortar is recommended to be of equivalent strength of masonry units used
- Formwork/concrete to cast ring beam, minimum grade M20 (f'_c = 3000 psi)

Geometry determination

- Partition space into domes: maximum feasible span = 16', maximum c (ratio of length to width) = 1.33
- After determining strength of available materials (f_b, f_y) consult appropriate design table, select z and determine required amount of steel rebars in ring beam. **IF RING BEAM DOES NOT REST ON LOAD-BEARING WALLS, THIS REQUIRED STEEL IS ***IN ADDITION*** TO REQUIRED FLEXURAL REINFORCEMENT**

Construction process

- Build foundations, supporting masonry walls: Domes are extremely susceptible to support displacement, foundations and walls must be constructed per existing specification.
- Cast ring beam: RCC ring beam must contain required flexural and tension steel. Design must be done by licensed engineer or architect per relevant local design code (ACI 318, IS 456). See following section for relevant requirements for concrete cover, overlap length, etc.
- Assemble scaffolding: Any scaffolding (centering, shuttering) may be used as long as settlement is minimized.

- Lay earthen formwork: Use Equation (3.1) or (3.2) to get *L* or *R* for given geometry and generate curves across diagonal. Check form by ensuring that quarter-point of span has height of ¾ * *z*. Compact earth every 10cm to ensure minimal settlement.
- Lay bricks: It is recommended that mortar is laid between each course, as opposed to pouring method presented in Hunnarshala documentation.

Rebar detailing requirements

All requirements are taken from the ACI (American Concrete Institute) structural design code (ACI 318-11). Relevant sections in this code are given in parentheses, while relevant sections from the Indian Standard code of practice for concrete design [IS 456] are given in square brackets. There are no significant discrepancies between the two codes which preclude either use for the design of these concrete ring beams.

- Minimum spacing of bars: a minimum clear spacing between bars (including at locations of splices) of 1" (25mm) is recommended. (7.6.1) [26.3.2]
- Minimum clear cover: a minimum of 1.5" (~40mm) of clear concrete cover is required outside all steel reinforcement. (7.7.1) [26.4]
- Splice length: For these ring beams, a splice length of $50*d_b$ is recommended for all bars. See §3.2 for recommended detailing at beam intersections. (12.2, 12.14, 12.16) [26.2.5]

Example calculations

EX 1) Design the ring beam for a dome spanning a rectangular space 14' by 11'. You have access to high-strength steel rebars (f_y = 72 ksi) and high-quality red clay bricks (f_b = 1000 psi, t = 3.5"). Use a safety factor of 2 against steel yielding and 4 against bricks crushing, and α = 8.

1) Select *z*. While this can be governed by architectural constraints, here we have a large span, so we select *z* = 15".
2) Get design loads. $\sigma_r = DL_{bricks} + DL_{fill} + LL = 40 + \left(\frac{1}{3} * \frac{9}{12}\right) * 150 + 40 = 117.5psf$
3) Use Equation (2.5) to get a first estimate for the required amount of steel. (2.5) can be rearranged to:

$$A_{s,req} = SF_{steel} * \frac{\sigma_r * X^3 * c^2}{\alpha * z * f_y}$$

4) Plugging in values:

   σ_0 = 117.5 psf

   X = 11'

   a = 14/11 = 1.27

   z = 9/12 = 0.75'

   f_y = 72,000 psi (we do not convert to feet to obtain our result, A_s, in square inches)

   We obtain $A_{s,req}$ = 0.94 in^2. Preliminarily select 6 10mm Ø bars (A_s = 6 * 0.175 = 1.05 in^2)

5) Design ring beam: Set the width of the ring beam to be 9" (the width of the masonry walls below). Including space for overlaps, the required height of the beam is 7.5": 1.5" clear cover on each side, 3 bundles of bars (for overlaps) at 0.8" (20mm), and 2 clear distances of 1" between each group of bars. Since all bars can be fit in one row, d is equal to the width of the ring beam minus the clear spacing on either side (9 – 2*1.5 = 6"). we can calculate S as:

$$S = \frac{A_s d}{2} = \frac{1.05 * 6}{2} = 3.15\ in^3$$

6) Check with Equation (2.6):

$$\sigma_n = \frac{\sigma_0}{SF_{steel}} = \frac{f_y * 144\ / SF_{steel}}{\frac{X^3}{a * z * A_s} + \frac{X^4(a-1)}{128 * z * S}} = \frac{72000 * 144\ /2}{\frac{(11 * 12)^3}{8 * 15 * 1.4} + \frac{(11 * 12)^4(1.27-1)}{128 * 9 * 3.15}}$$

$$= 162\ psf > \sigma_r \qquad OK$$

7) Check crushing of bricks, using Equation (2.5)

$$\sigma_n = \frac{\sigma_0}{SF_{bricks}} = \frac{t * (f_b * 144)/SF_{bricks}}{\sqrt{\frac{X^2(1+a)}{4} + \frac{X^4(1+a)^2}{8z^2}}}$$

$$= \frac{3.5 * (1000 * 144)/4}{\sqrt{\frac{(11 * 12)^2(1+1.27)}{4} + \frac{(11 * 12)^4(1+1.27)^2}{8 * 15^2}}} = 134 psf$$

$$\sigma_n > \sigma_r \qquad OK$$

8) Thus, our design is sufficient. It is observed that the bricks crushing is the governing mode of failure – this often occurs at large spans due to the large thrusts introduced by the high *D/z* ratio and the high factor of safety used.

EX 2) Use the design tables to design a dome with rise z = 9" to cover a 10' square plan. You have access to high-strength rebar, but only low-strength compressed earth (f_b = 500 psi) bricks.

1) Given f_y = 72 ksi and f_b = 500 psi, we consult the appropriate design table.

z = 9"

X (ft)	cks sufficie	#6	#8	#10	#12
6	OK	3	2	1	1
7	OK	5	3	2	2
8	OK	7	4	3	2
9	OK	10	6	4	3
10	OK	13	7	5	4
11	OK	17	10	6	5
12	NG	22	13	8	6
13	NG	28	16	10	7
14	NG	35	20	13	9
15	NG	42	24	16	11
16	NG	51	29	19	13

2) For a z = 9" and X = 10', the design table gives the option of choosing between 8, 10, and 12mm Ø bars. We select 4 12mm Ø bars to avoid crowding in the ring beam.

Fy = 36ksi Fb = 500psi

z = 6"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	8	5	3	2
7	OK	12	7	5	3
8	OK	18	10	7	5
9	OK	25	14	9	7
10	NG	34	19	12	9
11	NG	45	25	16	12
12	NG	58	33	21	15
13	NG	74	42	27	19
14	NG	92	52	33	23
15	NG	113	64	41	29
16	NG	137	77	50	35

z = 9"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	6	4	2	2
7	OK	9	5	4	3
8	OK	13	8	5	4
9	OK	19	11	7	5
10	OK	25	14	9	7
11	OK	34	19	12	9
12	NG	43	25	16	11
13	NG	55	31	20	14
14	NG	69	39	25	18
15	NG	84	48	31	21
16	NG	102	58	37	26

z = 12"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	5	3	2	2
7	OK	8	4	3	2
8	OK	11	6	4	3
9	OK	16	9	6	4
10	OK	21	12	8	6
11	OK	28	16	10	7
12	OK	36	21	13	9
13	NG	46	26	17	12
14	NG	57	32	21	15
15	NG	70	40	26	18
16	NG	85	48	31	22

z =15"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	4	3	2	1
7	OK	7	4	3	2
8	OK	10	6	4	3
9	OK	14	8	5	4
10	OK	19	11	7	5
11	OK	25	14	9	7
12	OK	32	18	12	8
13	OK	40	23	15	10
14	NG	50	28	18	13
15	NG	61	35	22	16
16	NG	74	42	27	19

Fy = 36 ksi Fb = 700 psi

z = 6"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	8	5	3	2
7	OK	12	7	5	3
8	OK	18	10	7	5
9	OK	25	14	9	7
10	OK	34	19	12	9
11	OK	45	25	16	12
12	NG	58	33	21	15
13	NG	74	42	27	19
14	NG	92	52	33	23
15	NG	113	64	41	29
16	NG	137	77	50	35

z = 9"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	6	4	2	2
7	OK	9	5	4	3
8	OK	13	8	5	4
9	OK	19	11	7	5
10	OK	25	14	9	7
11	OK	34	19	12	9
12	NG	43	25	16	11
13	NG	55	31	20	14
14	NG	69	39	25	18
15	NG	84	48	31	21
16	NG	102	58	37	26

z = 12"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	5	3	2	2
7	OK	8	4	3	2
8	OK	11	6	4	3
9	OK	16	9	6	4
10	OK	21	12	8	6
11	OK	28	16	10	7
12	OK	36	21	13	9
13	NG	46	26	17	12
14	NG	57	32	21	15
15	NG	70	40	26	18
16	NG	85	48	31	22

z =15"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	4	3	2	1
7	OK	7	4	3	2
8	OK	10	6	4	3
9	OK	14	8	5	4
10	OK	19	11	7	5
11	OK	25	14	9	7
12	OK	32	18	12	8
13	OK	40	23	15	10
14	NG	50	28	18	13
15	NG	61	35	22	16
16	NG	74	42	27	19

Fy = 36 ksi Fb = 1000 psi

z = 6"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	8	5	3	*2*
7	OK	*12*	7	5	*3*
8	OK	*18*	10	7	5
9	OK	*25*	*14*	9	7
10	OK	*34*	*19*	*12*	9
11	OK	*45*	*25*	*16*	*12*
12	OK	*58*	*33*	*21*	*15*
13	OK	*74*	*42*	*27*	*19*
14	OK	*92*	*52*	*33*	*23*
15	NG	*113*	*64*	*41*	*29*
16	NG	*137*	*77*	*50*	*35*

z = 9"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	6	4	2	*2*
7	OK	9	5	4	*3*
8	OK	*13*	8	5	4
9	OK	*19*	*11*	7	5
10	OK	*25*	*14*	9	7
11	OK	*34*	*19*	*12*	9
12	OK	*43*	*25*	*16*	*11*
13	OK	*55*	*31*	*20*	*14*
14	OK	*69*	*39*	*25*	*18*
15	OK	*84*	*48*	*31*	*21*
16	OK	*102*	*58*	*37*	*26*

z = 12"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	5	*3*	*2*	*2*
7	OK	8	4	*3*	*2*
8	OK	*11*	6	4	*3*
9	OK	*16*	9	6	4
10	OK	*21*	*12*	8	6
11	OK	*28*	*16*	10	7
12	OK	*36*	*21*	*13*	9
13	OK	*46*	*26*	*17*	*12*
14	OK	*57*	*32*	*21*	*15*
15	OK	*70*	*40*	*26*	*18*
16	OK	*85*	*48*	*31*	*22*

z =15"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	4	*3*	*2*	*1*
7	OK	7	4	*3*	*2*
8	OK	10	6	4	*3*
9	OK	*14*	8	5	4
10	OK	*19*	*11*	7	5
11	OK	*25*	*14*	9	7
12	OK	*32*	*18*	*12*	8
13	OK	*40*	*23*	*15*	10
14	OK	*50*	*28*	*18*	*13*
15	OK	*61*	*35*	*22*	*16*
16	OK	*74*	*42*	*27*	*19*

Fy = 72 ksi Fb = 500 psi

z = 6"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	4	3	2	1
7	OK	6	4	3	2
8	OK	9	5	4	3
9	OK	13	7	5	4
10	NG	17	10	6	5
11	NG	23	13	8	6
12	NG	29	17	11	8
13	NG	37	21	14	10
14	NG	46	26	17	12
15	NG	57	32	21	15
16	NG	69	39	25	18

z = 9"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	3	2	1	1
7	OK	5	3	2	2
8	OK	7	4	3	2
9	OK	10	6	4	3
10	OK	13	7	5	4
11	OK	17	10	6	5
12	NG	22	13	8	6
13	NG	28	16	10	7
14	NG	35	20	13	9
15	NG	42	24	16	11
16	NG	51	29	19	13

z = 12"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	3	2	1	1
7	OK	4	2	2	1
8	OK	6	3	2	2
9	OK	8	5	3	2
10	OK	11	6	4	3
11	OK	14	8	5	4
12	OK	18	11	7	5
13	NG	23	13	9	6
14	NG	29	16	11	8
15	NG	35	20	13	9
16	NG	43	24	16	11

z =15"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	2	2	1	1
7	OK	4	2	2	1
8	OK	5	3	2	2
9	OK	7	4	3	2
10	OK	10	6	4	3
11	OK	13	7	5	4
12	OK	16	9	6	4
13	OK	20	12	8	5
14	NG	25	14	9	7
15	NG	31	18	11	8
16	NG	37	21	14	10

Fy = 72 ksi Fb = 700 psi

z = 6"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	4	3	2	1
7	OK	6	4	3	2
8	OK	9	5	4	3
9	OK	13	7	5	4
10	OK	17	10	6	5
11	OK	23	13	8	6
12	OK	29	17	11	8
13	OK	37	21	14	10
14	OK	46	26	17	12
15	NG	57	32	21	15
16	NG	69	39	25	18

z = 9"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	3	2	1	1
7	OK	5	3	2	2
8	OK	7	4	3	2
9	OK	10	6	4	3
10	OK	13	7	5	4
11	OK	17	10	6	5
12	OK	22	13	8	6
13	OK	28	16	10	7
14	NG	35	20	13	9
15	NG	42	24	16	11
16	NG	51	29	19	13

z = 12"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	3	2	1	1
7	OK	4	2	2	1
8	OK	6	3	2	2
9	OK	8	5	3	2
10	OK	11	6	4	3
11	OK	14	8	5	4
12	OK	18	11	7	5
13	OK	23	13	9	6
14	OK	29	16	11	8
15	NG	35	20	13	9
16	NG	43	24	16	11

z =15"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	2	2	1	1
7	OK	4	2	2	1
8	OK	5	3	2	2
9	OK	7	4	3	2
10	OK	10	6	4	3
11	OK	13	7	5	4
12	OK	16	9	6	4
13	OK	20	12	8	5
14	OK	25	14	9	7
15	OK	31	18	11	8
16	NG	37	21	14	10

Fy = 72 ksi Fb = 700 psi

z = 6"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	4	*3*	*2*	*1*
7	OK	6	4	*3*	*2*
8	OK	9	5	4	*3*
9	OK	*13*	7	5	4
10	OK	*17*	10	6	5
11	OK	*23*	*13*	8	6
12	OK	*29*	*17*	*11*	8
13	OK	*37*	*21*	*14*	10
14	OK	*46*	*26*	*17*	*12*
15	NG	*57*	*32*	*21*	*15*
16	NG	*69*	*39*	*25*	*18*

z = 9"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	*3*	*2*	*1*	*1*
7	OK	5	*3*	*2*	*2*
8	OK	7	4	*3*	*2*
9	OK	10	6	4	*3*
10	OK	*13*	7	5	4
11	OK	*17*	10	6	5
12	OK	*22*	*13*	8	6
13	OK	*28*	*16*	10	7
14	OK	*35*	*20*	*13*	9
15	OK	*42*	*24*	*16*	*11*
16	OK	*51*	*29*	*19*	*13*

z = 12"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	*3*	*2*	*1*	*1*
7	OK	4	*2*	*2*	*1*
8	OK	6	*3*	*2*	*2*
9	OK	8	5	*3*	*2*
10	OK	*11*	6	4	*3*
11	OK	*14*	8	5	4
12	OK	*18*	*11*	7	5
13	OK	*23*	*13*	9	6
14	OK	*29*	*16*	*11*	8
15	OK	*35*	*20*	*13*	9
16	OK	*43*	*24*	*16*	*11*

z =15"

X (ft)	Bricks sufficient?	#6	#8	#10	#12
6	OK	*2*	*2*	*1*	*1*
7	OK	4	*2*	*2*	*1*
8	OK	5	*3*	*2*	*2*
9	OK	7	4	*3*	*2*
10	OK	10	6	4	*3*
11	OK	*13*	7	5	4
12	OK	*16*	9	6	4
13	OK	*20*	*12*	8	5
14	OK	*25*	*14*	9	7
15	OK	*31*	*18*	*11*	8
16	OK	*37*	*21*	*14*	10

Appendix B: Hunnarshala Foundation documentation

Brick domes of Ninnah's house

A doccumentation of the shallow brick dome making technique precticed in and around Muzaffarnagar.

Abandoned house in kutba village of muzaffarnagar.(Post riot). Photo/Nipun Prabhakar

CONTENTS

THE CONTEXT

Muzaffar Nagar is a town located in the state of Uttar Pradesh. It is surrounded by the historic cities of karnal, panipat and kurukshetra. It was named During the reign of Mughal Emperor Shah Jahan.The region is situated in the fertile Gangetic plains with abundant water resources. It lies on what is called the Sugar Belt of Western Uttar Pradesh. The region is one of the important sugar-cane producing regions in the world.

Role of hunnarshala

Hunnarshala was invited by an umbrella organisation called Joint Citizen's Initiative formed by Sanathkada, Vanagana, Sadbhavna Trust, and some independent citizens in response to the communal riots in Muzaffarnagar which displaced thousands of families. People from 9 worst affected villages in riots were given compensation by the Government. Some families have bought land from the compensation money. Hunnarshala and JCI are working together to rehabilitate these families. Hunnarshala is building houses while JCI working towards taking steps for a more holistic rehabilitation. It includes working for healthcare, sanitation, education, spreading awareness about various government schemes for the people etc.

While exploring the local building technologies, hunnarshala came across the the technique of roofing with bricks by making shallow domes.

Bricks as the major building material

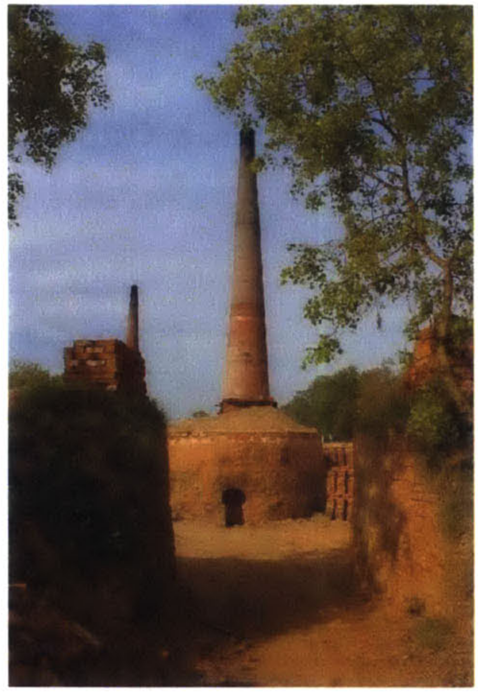
Bricks are the major building material in almost all the stages of construction in the region starting from the foundation to the roof.

One of the important features of the region are the brick kilns. It is the hub of production of excellent quality bricks. This is because of the availability of good quality sandy-silty soil and water resources. Apart from agriculture, brick kilns are also the major source of occupation for the marginalised population.

The shallow brick domes:

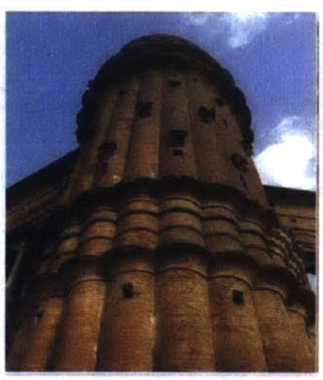
Shallow brick domes are one of the roofing styles found in the area. They are called shallow because the depth of dome is just around 7-9 inches depending on the span. Kutba village in muzaffarnagar has many such examples.

The following is a documentation of the process of the construction of domes in one of the houses of the rehabilitation project.



OLD MUGHAL SARAI Gate, Gharonda

Hemispherical dome



built in 1632 , during the reign of emperor Shah Jahan. The mughal sarai served as a rest-house for travelers. This three storied high structure. structure is the entrance gate of the sarai. It is said that the bricks of the sarai were used as aggregate for the construction of the railway line.

flat dome at the entrance

HOUSE PLAN

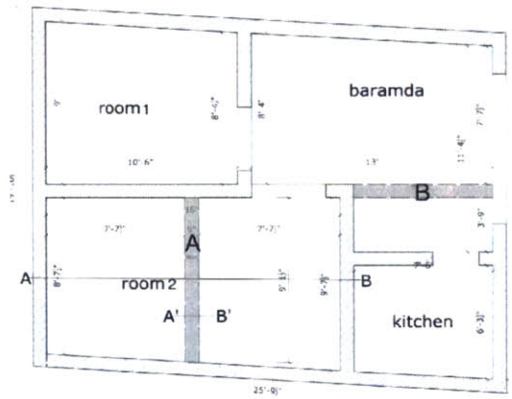
The house located in Aryapuri village in Kairana, Shamli District is part of the Rehabilitation project of the displaced victims of 2013 Muzaffarnagar Riots. It is the first house in the locality to opt for the Brick domed roof.

Ninnah is the proud owner of the house. He works as a "Feri wala". Feri walas are street vendors who go to different regions across the country and sell a particular item they picked from the lot. Most of the time it is cotton cloth. He will be staying in the house with his wife and two kids.

As visible, the plot is not a rectangle but a parallelogram. There is a skew of about 2'7".

There are four potential spaces for the brick dome roof.
The spaces are,
Room 1,
Room 2,
baramda
kitchen.

Room 2 is larger and the length by breadth ratio is more so it is divided into two parts by casting a beam (A) in the centre. It is done because the height of the dome is directly proportional to the span of the room. So if a single dome is made, the dome's height will mismatch with those of other rooms. The beam (B) is cast to eliminate the "L" shape formation in the Baramda and kitchen. This is because square and rectangle shaped rooms are preferred for domed roofs.



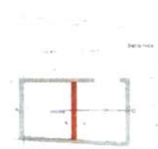
PLAN NOT TO BE SCALED

CASTING OF BEAM

THE SIZE OF THE ROOM IS LARGE (16' * 8'7" * 16'1" * 9'7") SO IT IS DIVIDED INTO TWO EQUAL PARTS BY CASTING A BEAM IN THE CENTRE OF THE ROOM

1. MS I SECTIONS (GIRDERS) ARE PLACED INTO THE GROOVES MADE IN THE WALL

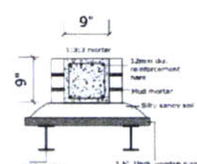
2. WOODEN PLANKS ARE PLACED ON THE GIRDERS, SANDY SILTY SOIL IS LAID ON THE PLANKS.



KEY PLAN

3. BRICK MASONRY IS DONE AS FORMWORK. CLAY IS USED AS MORTAR.

4. REINFORCEMENT IS PLACED AND CONCRETE IS POURED



SECTION OF BEAM

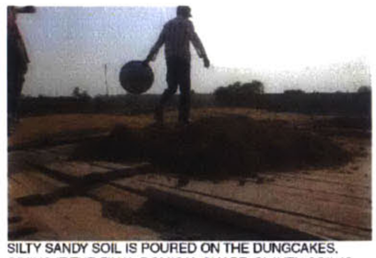
SHUTTERING AND FORM-WORK

IT IS THE MOST IMPORTANT PART OF THE PROCESS. AND CONSUMES THE MAXIMUM TIME.

SHUTTERING IS DONE. PLANKS ARE PLACED ON THE SAME LEVEL OF THE BEAM.

DUNG CAKES ARE USED TO GIVE THE BASIC DOME SHAPE. THE MAIN ADVANTAGES OF USING THEM ARE THEIR LIGHT WEIGHT AND RE-USIBILITY.

THE GAPS IN THE PLANKS ARE FILLED BY BUSHES.



SILTY SANDY SOIL IS POURED ON THE DUNGCAKES. GIVING IT THE FINAL DOMICAL SHAPE. CLAYEY SOIL IS NOT USED AS IT IS HARD TO SPREAD AND HEAVY.

SHUTTERING AND FORM-WORK

GIVING FORM TO THE DOME

1. SOIL IS PUT ON THE DUNGCAKES.
2. THE PILE IS GIVEN A DOMICAL FORM.
3. IT IS FURTHER BEATEN WITH A PLANK.
4. THE FORM IS FURTHER FINISHED BY SHAVING IT WITH A LONG PLANK.
5. SEVEN INCH HEIGHT OF THE DOME IS MEASURED USING "SOOT" OR THREAD AND BRICKS.

THE SOIL, CALLED "ROSHANI MITTI" IS USED TO MAKE THE FORMWORK IT CONTAINS GOOD AMOUNT OF SAND AND SILT AND LESS AMOUNT OF CLAY.

TOOLS USED

FORMWORK FOR DOME: TOOLS USED

MAKING OF THE CENTRAL MOTIF

1.

2.

3.

4.

1 THE CENTRE IS MARKED BY INTERSECTING THE DIAGONALS

3. HOOK IS INSERTED.

6. SOIL DOUGH IS GIVEN A CIRCULAR FORM.

9. THE GEOMETRIC PATTERN IS MARKED AND READY TO BE CUT.

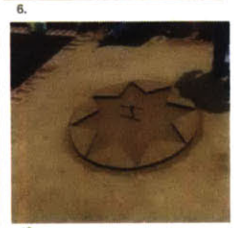
5.

6.

7.

8.

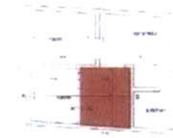
9.



10.

PLACEMENT OF BRICKS

THE BRICKS ARE PLACED OVER THE RECESSED MOTIF. THE RECESSION WILL BE FILLED BY CONCRETE.

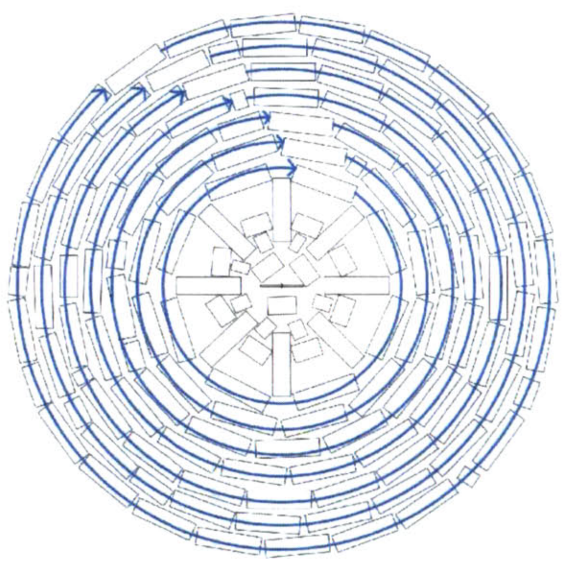


KEY PLAN

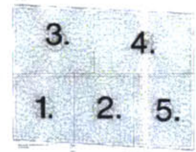
OTHER MOTIF DESIGNS

PLACEMENT OF BRICKS

THE BRICK BATS SHOULD BE PLACED IN ONE SINGLE ROTATION. THE ARROWS IN THE DIAGRAM REPRESENTS THE LAYING PATTERN OF THE BRICKS.



PLACEMENT OF BRICKS



KEY PLAN

IN NINAH'S CASE, DOME 1,2 AND 3 WERE FIRST CASTED TOGETHER. DOME NUMBER 4 AND 5 WERE BUILT LATER IN THE FOLLLOWING DAYS.

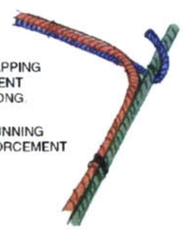
TERMINOLOGY OF BRICKS ACCORDING TO THEIR QUALITY.

QUALLITY	GRADE	NAME
BEST	FIRST	AWWAL
	SECOND	DOYAM
	THIRD	CHATKA
	FOURTH	TADSA
BAD	FIFTH	PILLA

REINFORCEMENT

TRADITIONALLY ONE SINGLE 12MM REINFORCEMENT BAR IS RUN THROUGHOUT THE CORNERS. THE BARS ARE JOINED TOGETHER USING HOOKS AT THE ENDS.
IN THIS PARTICULAR ROOF, ALL EDGES ARE OVERLAPPED WITH 10MM REINFORCEMENT BARS.

- 10MM OVERLAPPING REINFORCEMENT BAR, 6FEET LONG.
- STRAIGHT RUNNING 12MM REINFORCEMENT BAR

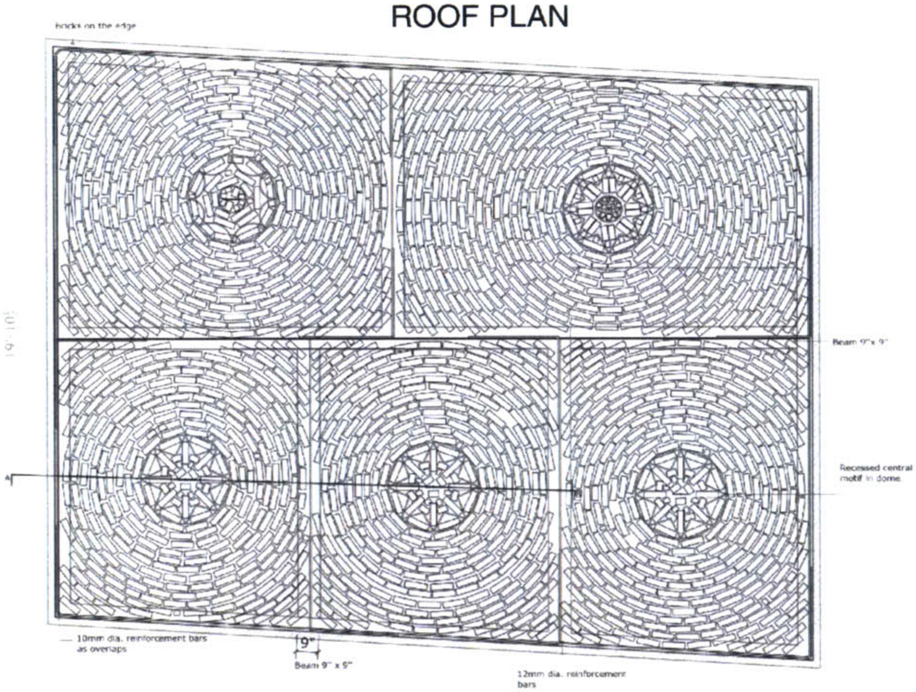


AT T JUNCTION WITH OVERLAPPING BAR

TRADITIONALLY, NO OVERLAPPS ARE GIVEN.

BARS AT L JUNCTION

ROOF PLAN



CASTING OF CONCRETE

AFTER THE BRICKS ARE LAID, CONCRETE OF COMPOSITION 1:3:3 OF CEMENT, DUST AND COURSE AGGREGATE RESPECTIVELY IS POURED. THE THICKNESS OF THE CONCRETE IS ONE INCH.

SWEET LADDU IS MIXED INSIDE THE FIRST MIXTURE AS A SIGN OF CELEBRATION.

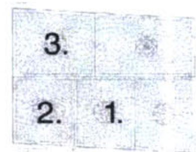
CONCRETE IS MANUALLY PUT BETWEEN THE BRICKS AND SMALL GAPS.

CASTING OF CONCRETE

POURING CONCRETE FROM TOP TO BOTTEM OF THE DOME WILL OVERTURN THE BRICKS, SO IT IS ADVISED TO POUR IT FROM BOTTOM TO TOP.

ARROWS REPRESENT THE DIRECTION OF CASTING CONCRETE

DOMES 1,2 AND 3 AFTER THE CONCRETE IS POURED.



OPENING OF SHUTTERING

THE PLANKS ARE REMOVED FIRST.

DUNG CAKES USED IN FORMWORK ARE REUSABLE.

AFTER THE SHUTTERING IS REMOVED, THE DOME IS CLEANED AND ANY SOIL IS SCRAPED OFF

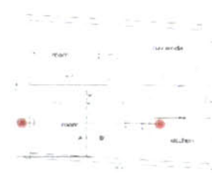
AFTER THE CLEANING IS DONE.

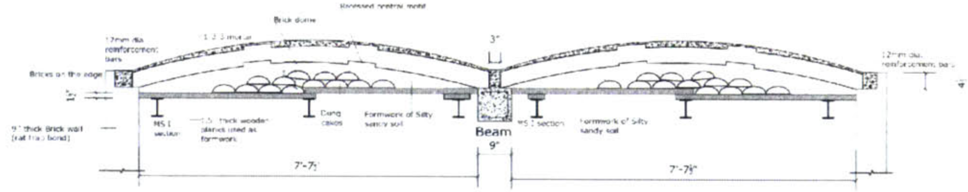
DOME WHEN THE SHUTTERING IS REMOVED IN 10 DAYS AFTER CASTING.

DOME WHEN THE SHUTTERING IS REMOVED IN 19 DAYS AFTER CASTING..

SECTION OF FORMWORK

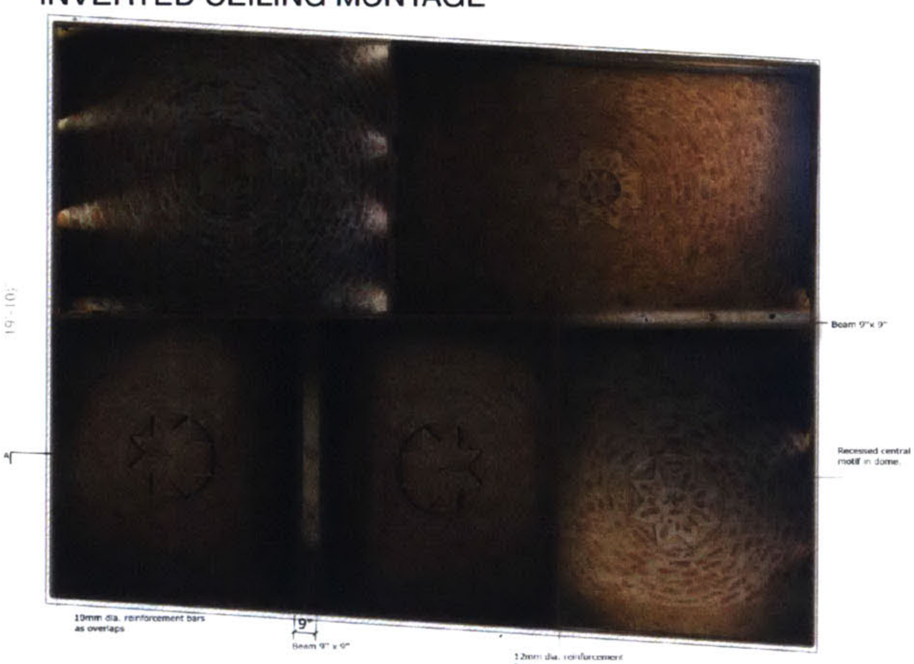
HOUSE PLAN





Section AB (Shuttering Details)

INVERTED CEILING MONTAGE



TOOLS USED

COSTING

THE COST OF THE BRICK DOME ROOF IS ABOUT 81 RUPEES PER SQUARE FEET. WHEREAS THE COST OF A NORMAL RCC ROOF IS ABOUT 135 RUPEES PER SQUARE FEET.

Estimate For a 500 Sq Ft brick dome roof

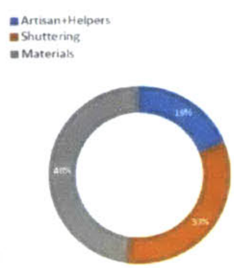
Note: the cost calculations are done including the casting of beams and excluding naranji's mandays

Item	Quantity	Unit	rate (in Rupees)	Amount	Total amount
Shuttering and formwork					
Takhta(planks, 4' x 3")	550	No	0.2	110	
I section Girder (12', 16',13')	19	No	4	76	
Balli (wooden logs)	25	No	1	25	
total rent of material per day				211	
total rent of material for 19 days	19	days	211	4009	
Mud	1	trolly	500	500	
shuttering transportation	2	trolly	500	1000	
Mason	2	days	500	1000	
Helper	6	days	300	1800	
dung cakes	100	units	1	100	
total shuttering				8409	8409
Making of dome					
Bricks	2000	no.	4.6	9200	
Cement	16	Bags	305	4880	
Coarse aggregate	45.55	quintal	78	3552.9	
dust	39.65	quintal	83	3290.95	
Reinforcement (12mm)	81.3	Kg	40	3252	
Reinforcement (10mm)	9.07	kg	40	362.8	
Reinforcement (8mm)	16.6	kg		697	
Reinforcement (6mm)	2.25	kg		101	
binding wire	0.95	kg		62	
hooks	2.4	kg		144	
weighing cost				40	
beam binding labor cost				355	
helper	16	days	300	4800	
mason	6	days	300	1800	
total cost of making of dome				32537.65	32537.65

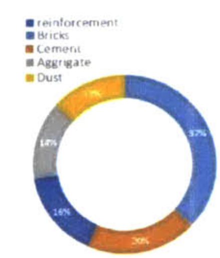
*THE RATES TAKEN IN THE COSTING ARE AREA SPECIFIC. THE RATES TAKEN HERE ARE OF MUZAFFARNAGAR AREA.

BRICK DOME ROOF

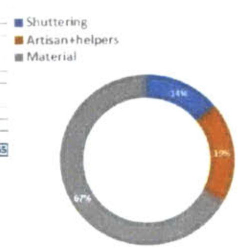
TOTAL COST



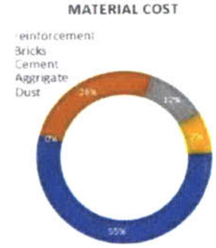
MATERIAL COST



RCC ROOF



MATERIAL COST



OTHER EXAMPLES:

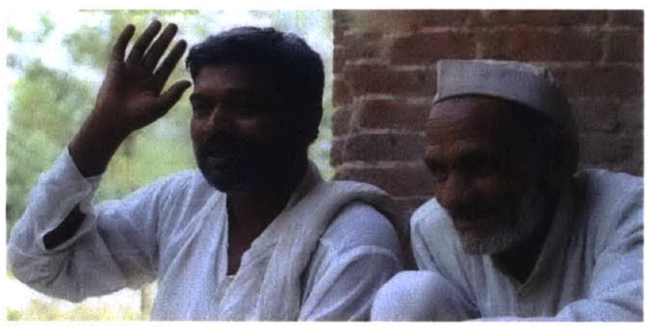
AFTER THE CEILING IS PLASTERED AND PAINTED. AT KUTBA VILLAGE, MUZAFFARNAGAR

NAWAB'S HOUSE IN KUTBA.

FOUND IN KUTBA

Conversation with the artisans.

Nawab is the artisan who built the 3 brick domes at Ninnah's house. Born and brought up in the Kutba village of muzaffarnagar, he belongs to a community of blacksmiths. He dropped out of school after he completed the eighth standard when he was 16 to work with his father as a blacksmith, he then worked as woodworker. Later he learnt the art of masonry from his Chacha, his Ustaad (master). After working with the Hunnarshala team and discussing other techniques of construction, especially earth, Nawab is looking forward to learn the technique of CSEB blocks and use it as a building material in Muzaffarnagar. The following text is the conversation with Nawab and his Ustaad specifically on the art of Brick domes.



Nawab and his Ustaad in a conversation.

Q: When was the first time you made the brick dome? How did you learn it?

Ustaad: About 20 years ago I went to *Berna* village. There I saw a Brick dome in a house. I was very impressed by it. I couldn't find the mason who made it so I decided to make it myself in my village, Kutba. The first dome was not very successful the second time was also not convincing but the third one succeeded. It is still standing there in the village. Later I taught the skill to Nawab and another student who belongs to the *kumbhar* community.

Q: Did you do any modification in the structure/aesthetics or just repeated what you saw?

Ustaad: Yes I did. The dome that I saw was not strong enough. Masons used to put a bar from below; it looked ugly so I added 12mm reinforcement bars on the walls at the perimeter of the roof. The bars were hooked together. It made the roof stronger.

Q: How did the people accept it when you first started to build the domes and staircase using arches in your village?

Ustaad: People saw live examples and hence they trusted me. One time, I was sitting and drinking tea with the house owner after his arched staircase was built. We were sitting in the Veranda. Suddenly his buffalo climbed up the stairs and went on the roof. The house owner was shocked, like he had a small heart attack. I assured him that the

Conversation with the artisans.

staircase was strong enough and will take up the load. And it survived.

Q: Why do you prefer domes to other roofing styles?

Nawab: Yes, I think the Domes are the strongest of all the roofs. They evenly distribute the load and are able to withstand even if one of the walls of the room collapses. It happened in my own house. One of the walls collapsed but the dome is still intact. Due to this property it does not need to have thicker walls hence the cost is saved. Even if we leave a 3-4 foot hole in the center of the dome, it will not break. In vaults the load only gets distributed to two walls and it fails at times. So we have to make thicker stronger walls. Even in flat roofs the load is not properly distributed and I don't consider it strong enough.

Q: What are the advantages of brick domed roofs over RCC roofs?

Nawab: RCC slabs are temporary. The life depends on the life of steel bars. As soon as the reinforcement bars corrode, the roof is gone. In domes there is no such thing. It will remain strong as long as it is in compression and the key is well set.

Q: You learnt dome making along with your *guru-bhai* (the other student from the same guru.). Over the years after practicing and getting experience in the art, is there any difference between your construction style and your *guru-bhai's*?

Nawab: Every Artisan has a different style of working which he develops after experience. I always use Dust with cement while making the concrete for the dome. And my central motif of the dome is always 3 feet in diameter. That's my signature style.

Q: And what made you do so?

Nawab: I think cement is not a permanent material. It will lose its strength after some years and will wither like sand. So when I use dust, even after the cement withers off the particles in dust will be in tension and hold the bricks together in the dome. My *guru-bhai* uses sand in place of dust.

Q: The techniques and form of domes or even other structures keep on changing. Modifications keep on happening. Like maybe it has evolved because the quality of bricks changed and new materials like cement got introduced. What are your thoughts on this?

Nawab: Each Artisan develops his own style over a period of time to create his own identity. If you go to my village, as soon as you look at a dome the people can tell which artisan built it. People can easily identify the domes that I built by looking at the design and the size of the central motif.

Q: After you make the dome, how do you level it from the top?

Nawab: We can do three things; either we can put earth on it followed by a layer of gutka (thin brick tiles). Or we can put fine aggregate and do flooring on it. The best thing would be using the residue of the over burnt bricks from the kilns. It is light in weight and porous.

Q: one common criticism of domes which lead many people to take other

Conversation with the artisans.

roofing options is that domes are prone to leakage. in your experience how do you counter this argument?

Nawab: Out of all the domes I have built, only one client has complained about leakage. This is because when I make a dome I always cover the dome up. But certain people want to be *kanjoos* (miserly). This particular client felt that by leaving the masonry exposed he could be a little thrifty. Well it backfired on him!

Ustaad: Obviously the dome will be held by the mortar! Magic won't hold the bricks together, mortar will. You cannot be a miser and expect good work.

Q: You said that you started making shallow domes for last 18 years, what were the techniques used before that?

Ustaad: Yes, Before that there were plank and joist roofs, there were girder and stone sill roofs, vaults with mud mortar, Proper hemispherical domes, back then there were arches too. I have expertise in arches and the arched staircases as well.

Q: When you were young, how were the homes built? Was it brick construction or mud construction or something else?

Ustaad: Earlier poor people used to make sun-dried bricks by themselves. They made square adobe bricks from a wooden mould. At times when it rained heavily, many such houses collapsed. Then over the time their wealth increased so they started making homes with brick and mud mortar. The roofs were made of plank and joists, followed by a layer of soil and brick tiles. After the introduction of cement, some people used cement plaster on the outer wall just to make it safe. Now everyone is lusting for RCC roofs and cement mortar houses.

Q: So do you feel bricks are better than earth to build homes?

Ustaad: Now everyone likes brick homes. The old houses were *kaccha*, that era is gone. But they used to be cooler than the brick houses.

Q: Now people prefer RCC roofs. Do they still give to importance to the domed roofs?

Nawab: We only make domes at the places where people like it and admire it. We never force anyone.

Q: What do you want to do after this project?

Nawab: I am interested in learning about CSEB blocks. I will go to Bhuj and Bangalore to learn that. Then I will come back and make the Block machine myself. After making the blocks, I will build my home with it. If that is satisfactory, I will start using it in to make other buildings too.

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Shallow Dome Experiment

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INTRODUCTION

The region covering Haryana and western U.P. is a fertile landscape. The area is mostly flat; the plains have been formed by alluvial deposits from the numerous big and small rivers that flow through it. The primary building material in this area is brick: It is used in every part of the building from the foundation to the roof. A long tradition of working with brick, making arches, domes, carving with bricks, etc. exists in the artisans of the region.

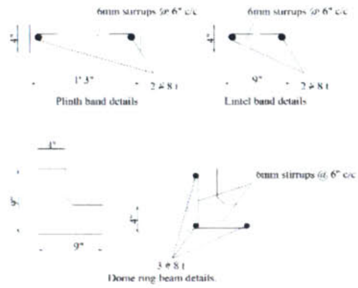
One of the interesting aspects of this tradition is the practice of making shallow brick masonry domes. The depth of a dome spanning 10 to 12 feet is only 7 to 9 inches making it ideal for intermediate floors as well as roof tops. While working with communities displaced in 2013 from muzaffarnagar, hunnarshala documented these domes and has been working on technology validation and upgradation.

Shallow Dome Built In Muzaffarnagar

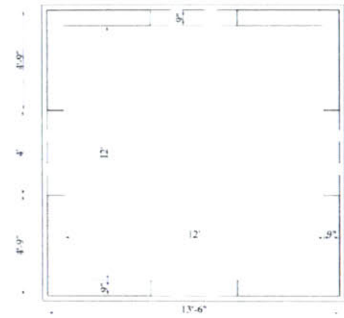
Shallow Dome Built At Hunnarshala

TECHNICAL SPECIFICATIONS

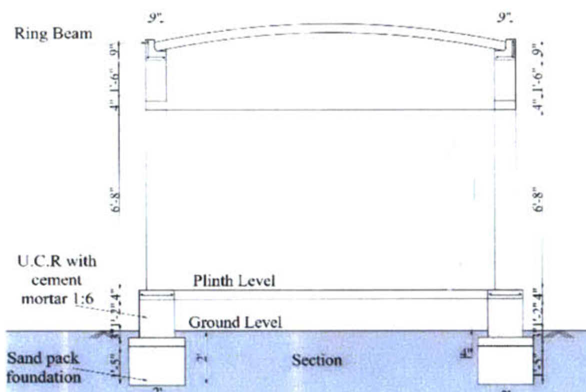
- Traditional shallow domes have a ring beam with a single 12 mm bar that holds the dome. In this test a beam with 'L' crossection was designed to hold the dome and counteract the outwards thrust.
- The rise of the dome was 5.5 inches.
- The grade of concrete used was m20.
- The ring beam was designed for a live load of 2Kn/m.

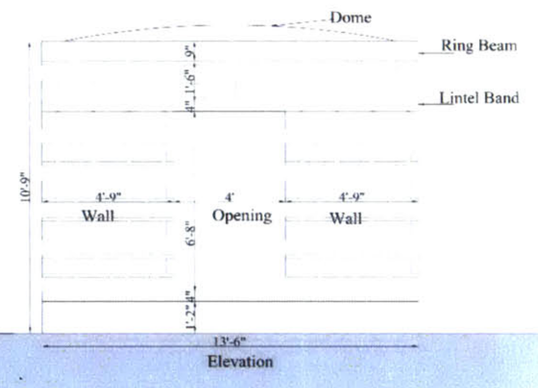


Details



Key Plan





CONSTRUCTION PROCESS

A load bearing structure was built. Uncoursed rubble was filled up to plinth band.

Sandstone masonry wall was built on top.Lintel band with 2 bars of 8 mm and stirrups of 6 mm was made

The shuttering is prepared for the 'L' beam after a 2 more masonry courses. This beam is designed to have 3 bars of 8 mm and stirrups of 6 mm each.

Flat steel plates were used for shuttering that held the "dholda" or mould which was made of sandy clayey earth.

To make the mold sandy clayey soil was used layers of sand was put and compacted as it continued.

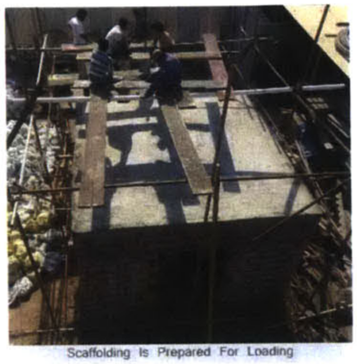
Wooden planks are used to hammer the soil softly into shape. Water is sprinkled and compacted with wooden hammers again.

C.S.E.B blocks are placed from the top of the dome and the courses follow in a concentric manner to the edge.

About 1 to 1/2 an inch gap was kept and the concrete was filled. It is poured from the edge towards the centre. Concrete is cured for a day.

LOADING

- This test was carried out to understand the behaviour of the shallow dome with relation to ring beam under static loading.
- Sand bags weighing 30 kgs each are prepared for loading.
- Before loading a boundary of 1 and a half feet from the edge of the structure was marked. The sand bags were placed inside this boundary to ensure only the dome was loaded and not the walls.
- There were 4 gauges placed to measure the vertical deflection.
- The measurements were taken after every 20 bags approximately.
- The bags were loaded in a symmetricallly to ensure loading was uniform. Readings were takne carefully once it was loaded.



Scaffolding Is Prepared For Loading

Loading Done Symmetrically

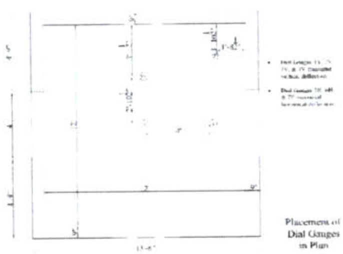
Cracks Appearing Near The Corners

Moment When Dome Broke Down

Structure After The Loading Test

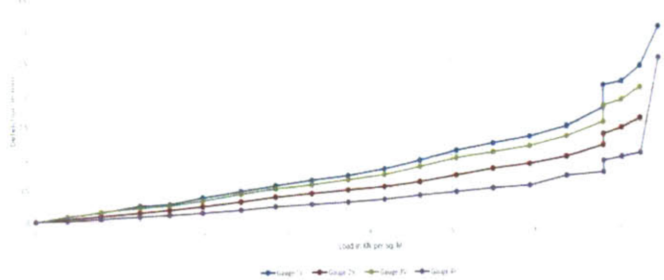
OBSERVATIONS

- The ring beam was designed for the live load of 2 KN/ meter sq., up to 6 KN/ meter sq. the rise in deflection is gradual but increases suddenly after that. The dome collapsed at the load of 7.4 KN / meter sq.
- Separation cracks were observed in the stone masonry above the lintel level.
- Cracks in the dome started close to the corners and progressed diagonally towards the center before collapse.



Plan Indicating Location Of The Gauge

Separation Cracks Above The Lintel Level



Vertical Deflection VS Load per Sq. M

Overturned Beam