



POST-TENSIONED MASONRY STRUCTURES

Properties of Masonry
Design Considerations
Post-Tensioning System for
Masonry Structures
Applications

2

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Preface

Clay bricks were the first man-made artificial building material. They have been extensively used since the time of the Assyrians and Babylonians throughout all ages. Even today masonry - using bricks, concrete or calcium-silicate blocks - is weight-wise the second most important construction material after concrete. Recently reinforcing and prestressing systems have been introduced in order to improve the performance of masonry and extend its range of applicability.

However, analysis and design of masonry structures have not kept pace with the corresponding developments in the fields of steel and concrete structures. They have been governed for too long by tradition and dubious semi-empirical formulas.

Only in recent years attempts have been made to investigate masonry as a structural material like steel and concrete. Accordingly the same limit states of serviceability and ultimate strength are also applied for the design of masonry structures. However, it should be recognized that masonry is mostly used for minor structures or parts of structures for which these structural criteria will not govern the design. Hence a threelevel approach seems indicated:

Level 1: The cases not governed by structural criteria should be quickly identified by physically understandable criteria in the form of simple formulas.

Level 2: The structural system is relatively simple. The structural criteria do not impose restrictive conditions on the architectural design. In such cases simplified physical models should lead to simple design methods, design charts or simple computer programs.

Level 3: The structural system and the imposed loading cases are such that a detailed structural analysis and design are required. Hence a specification of the appropriate structural properties of masonry (stress-strain; moment-axial force-curvature; failure criterion under uni-axial, bi-axial and general loading) is necessary to perform such an analysis.

It should be recognized that the level 3 approach will be the rare exception such that masonry structures can be generally designed by simple and efficient methods.

Masonry is a building material with an excellent mix of architectural, physical, physiological and structural properties. Through the application of modern structural design methods and the use of reinforcing and prestressing systems it is evolving into a modern structural engineering material.

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1. Introduction

Post-tensioning masonry? Combining the most advanced techniques with an old building material almost forgotten in the education of civil engineers?

A brief historical review, [1], may help to understand why such reactions might be short-sighted or even wrong. Brick actually is the oldest man-made building material, invented almost ten thousand years ago. Its simplicity, strength and durability led to extensive use and gave it a dominant place in history alongside stone. Hand-shaped, sun-dried bricks, reinforced with such diverse materials as straw and dung were so effective that fired bricks did not appear until the third millennium B.C.. Some of the oldest bricks in the world were found at the site of ancient Jericho. Other important constructions include the Tower of Babel and the Temples at Ur. Perhaps the most important innovation in the evolution of masonry constructions was the development of masonry arches and domes. Such constructions found in Babylonia are believed to have been built around 1400 B.C.. Arches reached a high level of refinement under the Romans. During the Middle Ages the leading centres for brick construction were located in Europe, primarily in the Netherlands, the Northern parts of Germany and Italy, and in Central Asia.

With the Industrial Revolution, emphasis shifted to iron, steel and concrete construction. By the early twentieth century, the demand was for high-rise construction, and the technology of stone and masonry buildings had not kept pace with the developments of other structural systems. The Monadnock Building in Chicago (1891) is cited in the United States as the "last great building in the ancient tradition of masonry architecture". Its massive structure, 16 stories high, with stone and brick walls 1.8 m thick at the base, supported on immense footings, seemed to prove that the medium was not suited to the demands of a modern, industrialized society. Design of masonry was at that time purely empirical rather than rationally determined, and rapid advances in the concrete engineering quickly outpaced what was seen as an outmoded, inefficient, and uneconomical system. Some ancient and old masonry constructions are illustrated in Figure 1.

In 1920 economic difficulties in India convinced officials that alternatives to concrete and steel systems had to be

Figure 1 Old Masonry Constructions



a) Tower in Siena, Italy



b) Arch in Ctesiphon, Iraq



c) Railway Viaduct, Switzerland



d) Monadnock Building, Chicago, USA

found. Extensive research began into the performance of reinforced masonry walls, slabs, beams and columns. It was not until the 1940's, however, that European engineers and architects began serious studies of masonry bearing wall designs, almost 100 years after the same research had begun on concrete bearing walls. Switzerland introduced its first provisional masonry standard in 1943. In the United

States, the first engineered masonry building code was published in 1966. Continued research brought about refinements in testing methods and design procedures in the following decades and new types of masonry construction were explored including buildings up to twenty stories, Figure 2.

The major advantages of ancient and modern masonry have always been the

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overall availability of the raw materials, the easy and economical construction, and the natural beauty and durability. Thus, again, why post-tensioning masonry?

Masonry has a relatively large compressive strength but only a low tensile strength. Therefore, masonry has been used so far primarily as a construction material for vertical members subjected essentially to gravity loads. Apart from this principal action, however, in-plane shear and out-of-plane lateral loads as well as imposed deformations caused by deflections and volume changes of floor slabs may be applied to masonry walls. Small lateral loads and deformations may be resisted due to the weight of the walls. However, for larger lateral loads, walls with low axial loads exhibit a poor cracking behavior and a low strength. To overcome these disadvantages, masonry may be post-tensioned. Post-tensioning offers the possibility to actively introduce any desired

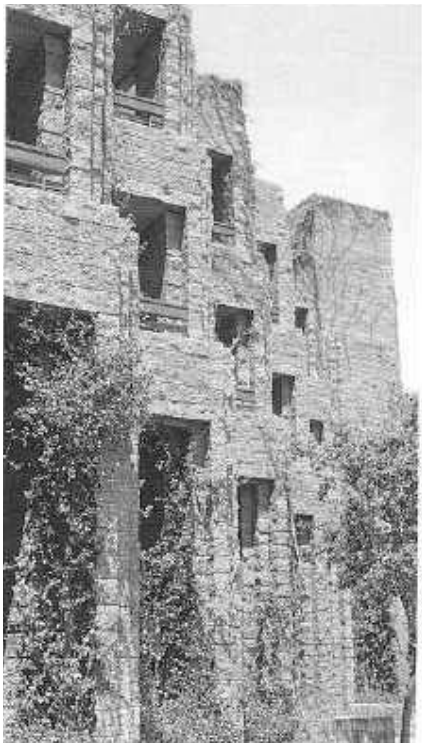
level of axial load in a wall to enhance strength, performance, and durability of masonry structures. The prestressing steel helps avoiding brittle tensile failure modes of masonry walls and offers major advantages for the connection of vertical and horizontal members in precast construction. Existing structures may be strengthened by prestressing to comply with recent code requirements for lateral loading; in particular, seismic areas.

As a matter-of-fact, the idea of post-tensioning of masonry is not new. In 1825 a post-tensioning method for tunnelling under the River Thames was utilized in England. The project involved the construction of vertical tube caissons of 15m diameter and 21 m height. The 0.75m thick brick walls were reinforced and posttensioned with 25mm diameter wrought iron rods. Since the 1960's research on, and a number of applications of, prestressed masonry have been reported primarily in England primarily

in England [2,3,4]. Applications include a prestressed masonry watertank, retaining walls, large walls in buildings and even road and railway bridge abutments, Figure 3.

The main purpose of this report is to contribute to a better understanding of the behaviour of masonry structures and thus, to help designers to transfer the post-tensioning technique, well-known in concrete construction, to structural masonry. After a brief overview on typical masonry components and construction details, important engineering properties of masonry are discussed and detailed design considerations for typical structural members are presented. Finally, the VSL System for post-tensioned masonry and its handling are illustrated together with recent applications. It is hoped that this report is able to highlight some potential of post-tensioned masonry yet to be exploited by innovative engineers, architects, and contractors.

Figure 2: Recent Masonry Constructions



a) Hotel



b) Residential



d) Commuty Hall, Photograph courtesy of Consulting Engineers



c) High-Rise



e) Prefabrication

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Figure 3: Post-Tensioned Masonry Constructions



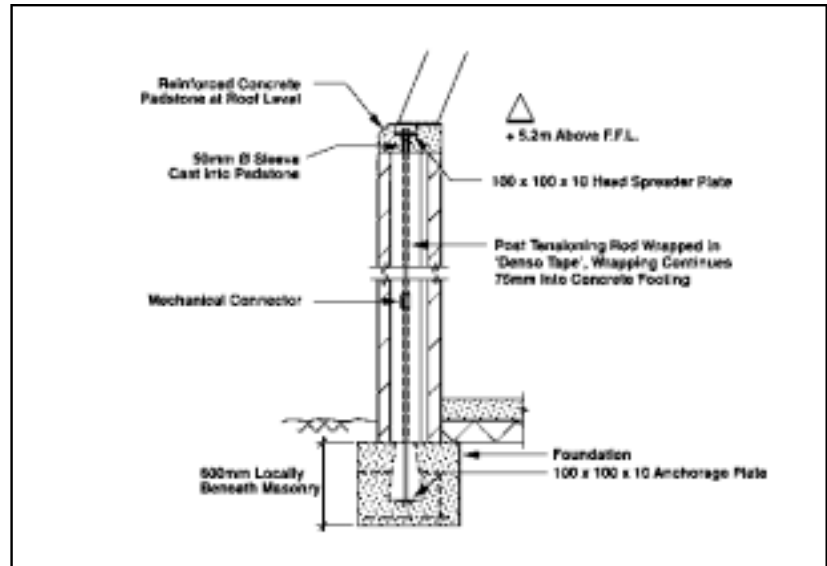
a) Salvation Army Hall, [2], Photograph courtesy of Curtins Consulting Engineers



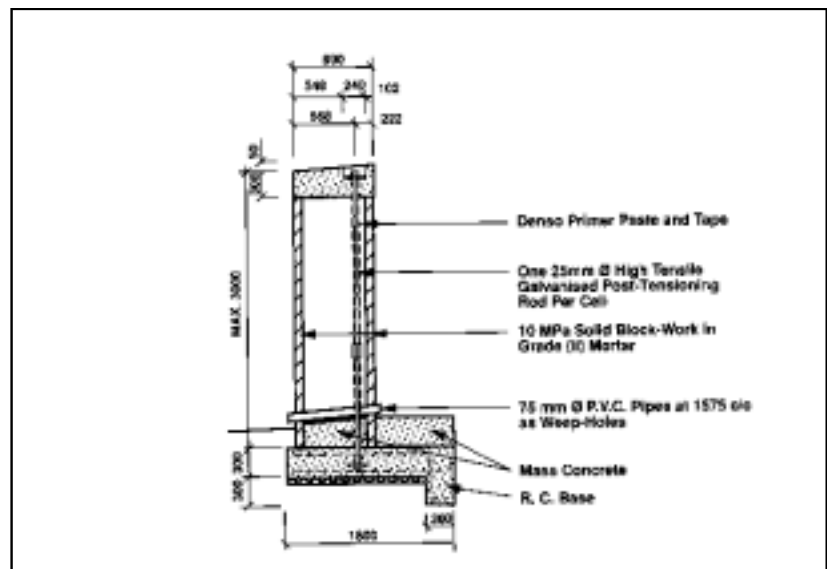
b) Wall Section Salvation Army Warrington, [2], Photograph courtesy of Curtins Consulting Engineers



e) Glinton-Northborough Bypass, [4], Courtesy of Cambridgeshire County Council and Armitage Brick Limited



c) Wall Section Orsborn Memorial Hall, [3], Courtesy of Curtins Consulting Engineers



d) Retaining Wall Section, [3], Courtesy of Curtins Consulting Engineers

2. Masonry Components and Construction

The most widely accepted definition of masonry is "an assemblage of small units joined with mortar", Figure 4. Horizontal and vertical joints are called bed and head or perpend joints, respectively.

Today, masonry units include not only stone and clay bricks, but a variety of other manufactured products such as concrete blocks, calcium-silicate bricks, structural clay tile, terra cotta veneer, etc, which are available in an almost unlimited number of sizes. To cover all of them would be far beyond the scope of this report. Therefore, only the most commonly used clay brick and concrete block units are considered in the following. Some typical units and available sizes are illustrated in Figures 5 and 6. Core patterns typically vary from manufacturer to manufacturer. Units without cores or with core areas up to 25% of the gross cross section are called solid units. Hollow units have core areas up to a maximum of about 50% of the gross area. Basically, units for wall thicknesses between 100mm and 250mm are available all over the world.

Masonry mortar typically is a mix of portland cement, hydraulic lime, sand and water. The mix proportions influence the strength of the mortar and its workability. Commonly used and specified mix proportions in the United States, [5],

Australia, [6], Great Britain, [7], Switzerland, [8], and the Federal Republic of Germany, [9], are summarized in Table 1 together with the minimum required compressive strength. A typical cement mortar has a mix proportion of cement: lime: sand by volume of 1: (0-¼): 3 and reaches a compressive strength of 15 to 20 MPa at 28 days. For a typical cement/lime mortar the corresponding values are 1:1:6 and approximately 5 MPa. Primarily in the United States and Australia, the cores of the units are often filled with grout to obtain grouted masonry. Typical grout mixes and strengths are also given in Table 1. Figure 7 illustrates the range of available compressive strength of masonry units, mortar and grout, according to National Standards [5,6,7,8,9]. Typically, unit strengths range from 5 to 40 MPa based on gross cross sectional area. Great Britain is well-known for its exceptionally high strength engineering clay bricks with compressive strengths up to and even beyond 100 MPa.

Reinforced masonry typically includes horizontal reinforcement laid in the bed joints or grouted cavities and/or vertical reinforcement placed in large cores, head joints or specially formed pockets, Figure 8. Normal reinforcing bars in common

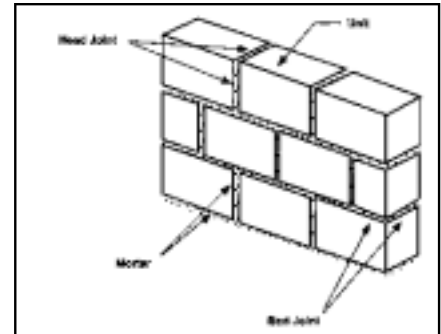


Figure 4: Components of Plain Masonry

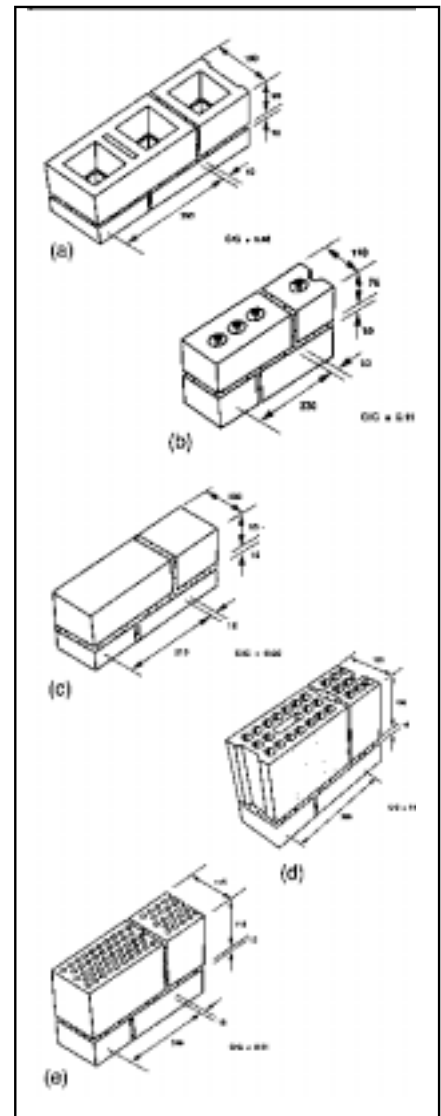


Figure 5: Typical Clay Bricks
a) Canada; b) Australia;
c) Great Britain; d) Switzerland;
e) Germany FR
Note: C/G = Core area to gross cross sectional area

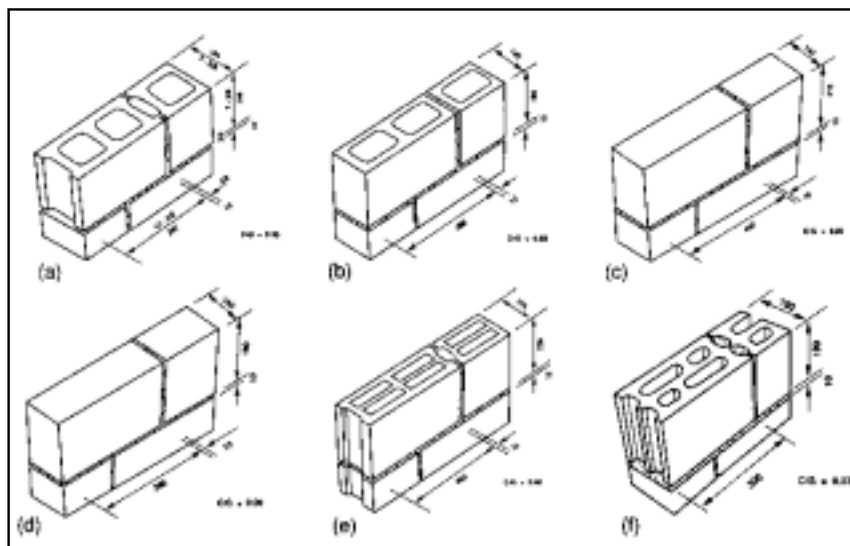


Figure 6: Typical Concrete Bricks and Blocks
a) United States;
b) Australia;
c) Great Britain / Australia;
d) Great Britain/Australia
e) Germany FR;
f) Switzerland
Note: C/G = Core area to gross cross sectional area

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grades can be used in general. However, special truss-type galvanized bed joint reinforcement is often preferred since it is easier to place while providing an improved corrosion protection. A typical bed joint reinforcement is presented in Figure 9.

In the applications of post-tensioned masonry to date, prestressing bars or strands were usually used. Some characteristics of these prestressing steels are summarized in Table 2. Bars typically show higher relaxation losses and much lower strength/weight ratios than strands.

Apart from these basic masonry components a large variety of metal accessories are available such as ties and anchors to connect individual wall leaves and to support them, respectively. Some typical ties are presented in Figure 10. They are made of stainless steel, in general.

Figure 11 illustrates typical masonry wall constructions. A solid wall may be constructed as a single leaf (wythe) wall, Figures 11 a and g, or may consist of multiple leaves which are connected with a mortar joint. This so-called collar joint may be either continuous over the wall height or staggered as shown in Figure 11 b with a maximum thickness of 25mm. Cavity walls consist of two single leaf walls, usually at least 50mm apart, and effectively tied together with wall ties. The space between the leaves may either be left as a continuous cavity, Figures 11c and e, filled with a non-loadbearing insulation material, Figure 11 d, or filled with grout, Figure 11f. For tall and/or heavily loaded walls, so-called diaphragm walls are commonly used in Great Britain, Figure 11 h. A diaphragm wall is a wide cavity wall where the two leaves are connected together by cross ribs of masonry. More complex diaphragm wall sections have been used. Typical floor slab systems using in-situ and precast concrete members, steel and timber joists together with possible connections to the walls are also illustrated in Figure 11.

Masonry walls may be finished using plasters, rendering or painting. However, the use of unfinished walls with units of different texture and colour as well as different bond patterns has a wide aesthetic potential. Figure 12 illustrates just a small selection of possible bonds. The masonry units may be laid longitudinally or transversally to the wall plane as stretchers and headers, respectively, to

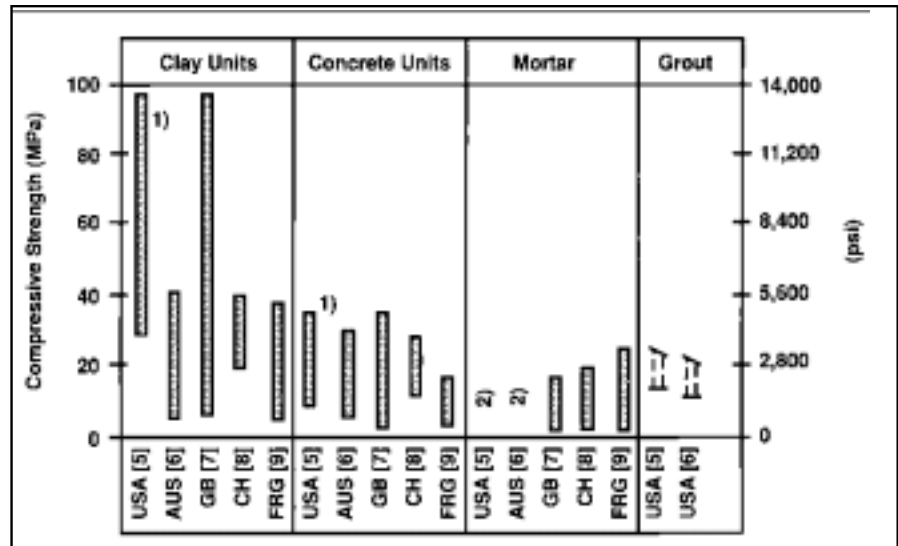


Figure 7: Strength Requirements for Units and Mortar, [5, 6, 7, 8, 9]

Note: 1) Based on net area, 50 to 75% of gross; 2) Not specified

COUNTRY STANDARD	TYPE	MORTAR & GROUT MIX PROPORTION BY VOLUME			MINIMUM COMPRESSIVE STRENGTH 1) (MPa)
		CEMENT	LIME	SAND	
USA [5]	M	1	1/4	(2 1/4 - 3)	Not specified
	S	1	>(1/4 - 1/2)	(2 1/4 - 3)	Not specified
	N	1	>(1/2 - 1 1/4)	(2 1/4 - 3)	Not specified
	O	1	>(1 1/4 - 2 1/2)	(2 1/4 - 3)	Not specified
	Grout	1	(0 - 1/10)	(2 1/4 - 3)	14.0
AUSTRALIA [6]	M4	1	(0 - 1/4)	3	Not specified
	M3	1	1	6	Not specified
	M2	1	2	9	Not specified
	M1	0	1	3	Not specified
	Grout ²⁾	1	0	(4 - 4 1/2)	12.0
GREAT BRITAIN [7]	(i)	1	(0 - 1/4)	3	16.0
	(ii)	1	1/2	(4 - 4 1/2)	6.5
	(iii)	1	1	(5 - 6)	3.6
	(iv)	1	2	(8 - 9)	1.5
SWITZERLAND [8]	C ³⁾	1	0	(3 - 3 1/2)	20.0
	V ⁴⁾	1	2 1/2	10	3.5
FEDERAL REPUBLIC OF GERMANY [9]	IIIa	1	0	4	25.0
	III	1	0	4	14.0
	IIa	1	1	6	7.0
	II	1	2	8	3.5

Table 1: Typical Mortar and Grout Mixes [5, 6, 7, 8, 9]

Note: 1) In laboratory testing

2) Cement content $\geq 300 \text{ kg/m}^3$

3) Cement content (300-450) kg/m^3

4) Lime content 250 kg/m^3 , cement content 100 kg/m^3

1 MPa = 140 psi

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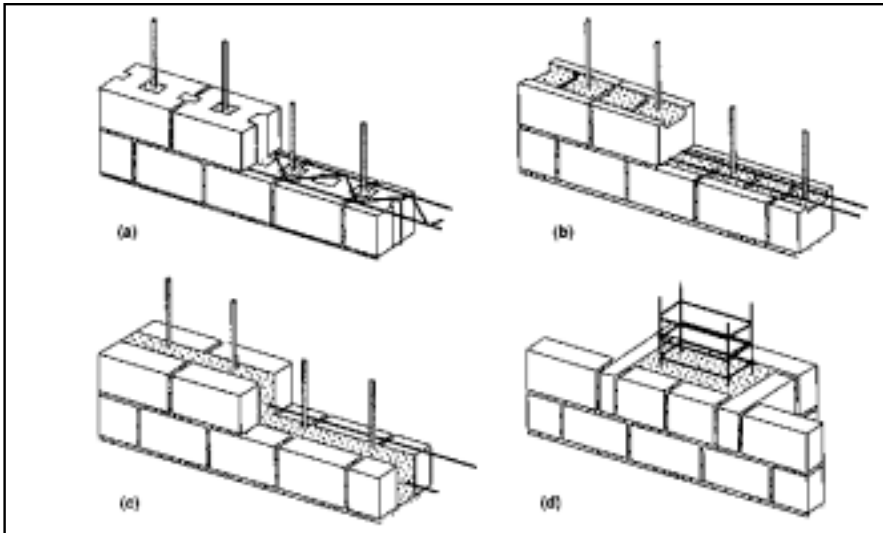


Figure 8: Typical Layout of Horizontal and Vertical Reinforcement in Walls
a) In cores and bed joint mortar; b) In cores and bed joint grooves;
c) In grouted cavities; d) In pockets

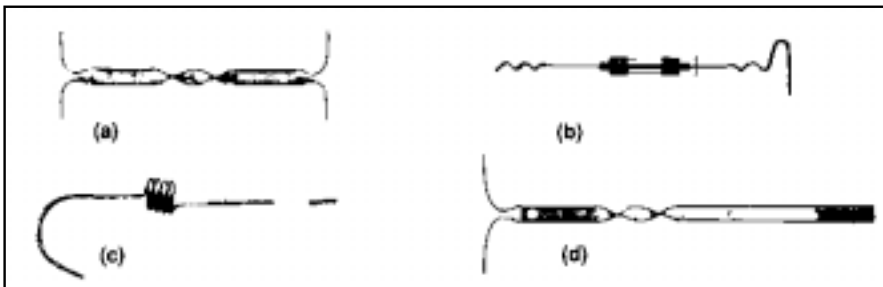


Figure 10: Ties
a), b): Both ends embedded in mortar;
c), d): One end embedded in mortar, other end thread and sleeve

Properties		Bar ϕ 36 mm hot rolled, cold-worked and tempered, ribbed	Strand ϕ 13mm cold-drawn stabilized
Ultimate tensile strength	MPa	1,230	1,770
Yield strength	MPa	1,080	1,570
Min. elongation at rupture	%	6	6
Relaxation at 70% of ultimate after 1,000 h at 20°C	%	3.3	2
Modulus of elasticity	MPa	$2.05 \cdot 10^5$	$1.95 \cdot 10^5$
Fatigue amplitude $2 \cdot 10^6$ load cycles at upper stress of 90% of yield	MPa max min	210 210	250 205
Min. diameter of curvature at max. allowable stress of 90% of yield	m	6.63	0.85
Friction coefficient		0.50	0.19

Table 2: Characteristics of Prestressing Steels (according to German Approval Documents)
Note: 1 MPa = 140 psi; 1 m = 3.3 ft.

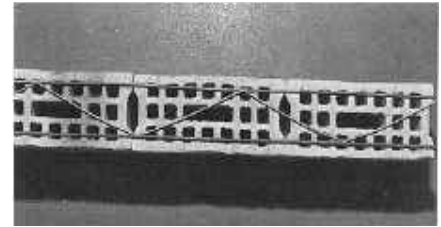


Fig. 9: Typical Bed Joint Reinforcement

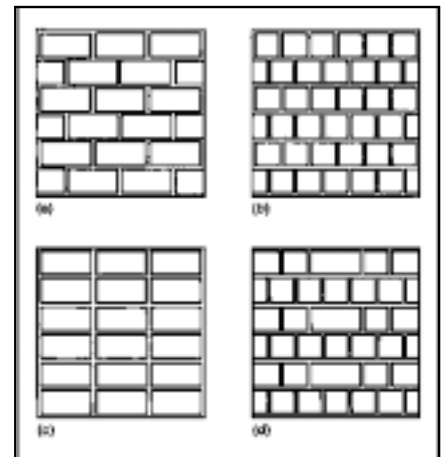


Figure 12: Typical Masonry Bonds
a) Running bond with stretchers;
b) Running bond with headers;
c) Stack bond;
d) Dutch bond

form the most common running bond for load bearing walls, i.e. walls which are primarily designed to carry an imposed vertical load in addition to their own weight, Figures 12a and b. Stack bond without overlap of the units in the head joint is not as effective as running bond and is, therefore, usually used for nonload bearing walls only, Figure 12c. Figure 12d depicts just one out of the large variety of available traditional and modern bonds. Veneers, i.e. non-load bearing facing walls, are a typical application of the potential offered by different bond patterns.

An important factor for any successful masonry construction is the protection of the masonry units from direct rain during storage and construction. Apart from the harmful effects of the enclosed humidity to the structure, soaked units develop much larger long term deformations and may show less strength than dry units.

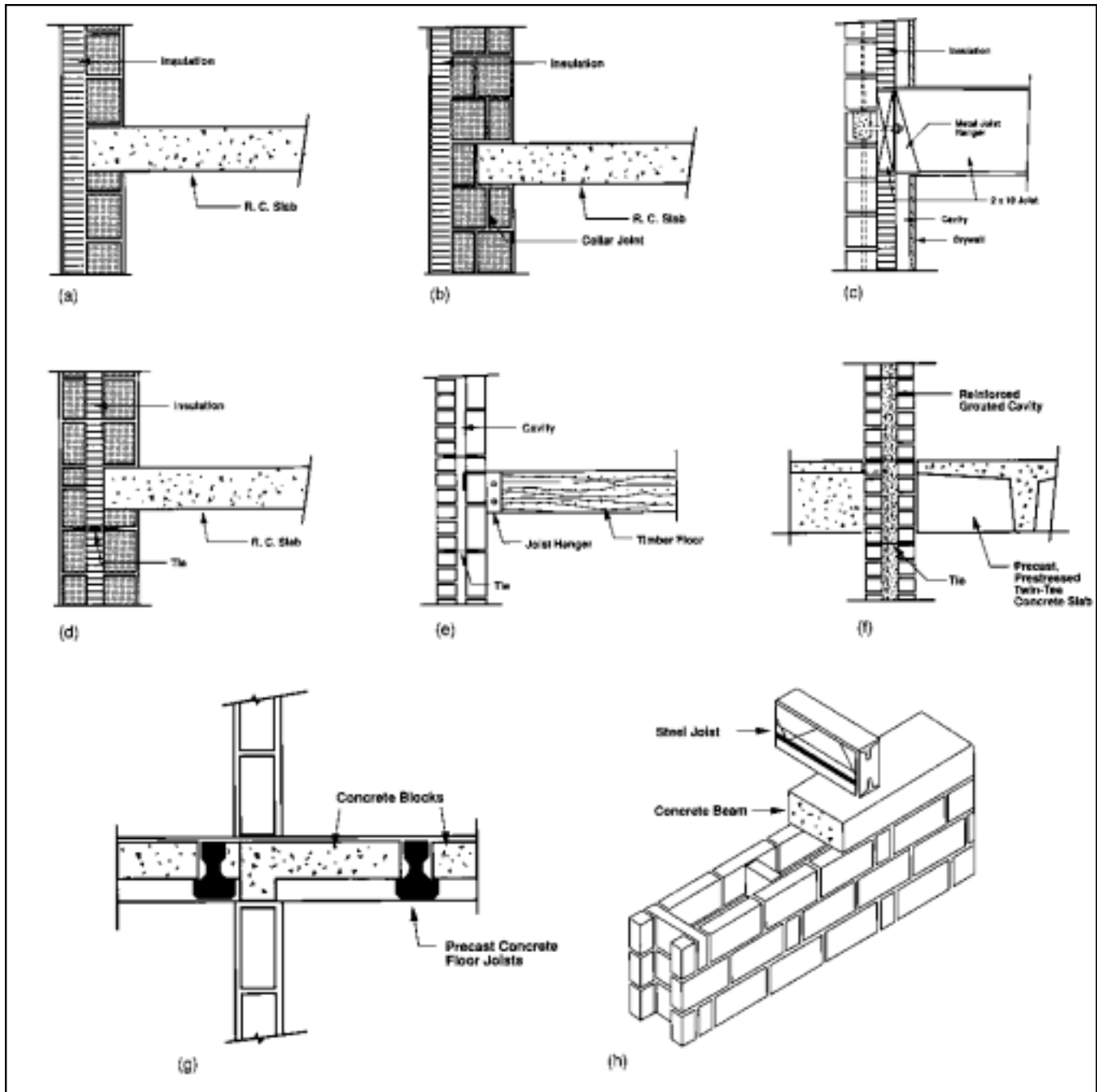


Figure 11: Typical Wall Sections and Connections to Floors / Roofs

- | | |
|--|---|
| a) Single-leaf wall, concrete floor | e) Cavity wall, timber floor |
| b) Single-leaf bonded wall, concrete floor | f) Cavity wall, precast concrete floor |
| c) Single-leaf wall, floor joists | g) Single-leaf wall, concrete block floor |
| d) Cavity wall, in-situ concrete floor | h) Diaphragm wall, steel joist roof |

3. Properties of Masonry

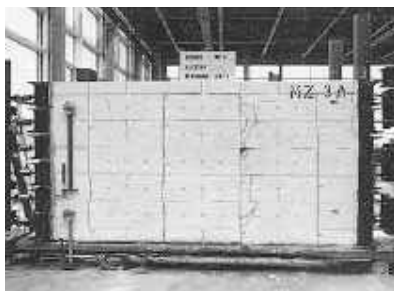
Fig. 14: Typical Failure Modes of Masonry



a) Compressive failure of units



b) Sliding failure along joints



c) Tensile failure of units



d) Tensile failure along joints

3.1 Introduction

Masonry is a rather complex composite material. The interaction of units and mortar joints has attracted the interest of many researchers. The interaction as presented by Hilsdorf. [10], is outlined in Figure 13. Due to different stress-strain characteristics, the mortar in the joints tends to have larger transverse strains than the masonry units under load. As differential deformations are prevented by bond between the materials, a uniaxial externally applied load introduces transverse stresses in the units and the mortar and thus, a multi-axial state of stress. Stresses in the units increase along Path (1) in Figure 13 leading to vertical cracks when this path intersects with the failure envelope of the units. Every crack results in a reduction of the transverse to vertical stress ratio, changing the stress path from (1) to (2), (3), etc.. Eventually, failure of the masonry occurs when the stress path reaches Point A in Figure 13 where the strength envelopes of units and mortar intersect. Based on such models and extensive experimental research, an almost unlimited number of equations were proposed trying to correlate masonry compressive strength with unit and mortar strengths. While such equations may be helpful to reduce testing expenses for brick and block manufacturers with a well-defined and limited set of parameters, the only reliable and general method of determining the masonry compressive strength is the testing of masonry prisms. However, this fact has not yet been universally recognized. Indeed, most national standards still base the masonry

compressive strength on unit and mortar strength.

In addition to the complex interaction of units and mortar, masonry shows an anisotropic behaviour both for deformations and for strength. The anisotropy results from the combined effects of the cores in the units and the mortar joints. While the effects of the head joints are somewhat mitigated by the staggering of the units laid in running bond, the bed joints are the plane of weakness in masonry. The anisotropic behaviour is reflected by the different failure modes of masonry encountered for general loading conditions. Figure 14. Under uniaxial compression perpendicular to the bed joints a splitting type of failure is usually observed in the units, Figure 14a. For relatively large shear stresses along the bed joints as for uniaxial compression under 45 degrees to the joints, a sliding type of failure develops along the joints in general, Figure 14b. Depending on the bond characteristics between units and mortar, different tensile failure modes will develop for axial tension parallel to the bed joints, either through head joints and units, Figure 14c, or through joints only, Figure 14d.

In the following sections, some important material properties for the design of clay brick and concrete block masonry are presented. Only masonry laid in running bond is considered. After illustrating the behaviour of masonry under uniaxial compression perpendicular to the bed joints, general biaxial compression and tension loadings are considered. Approaches of different national standards are presented where applicable. All stresses and strengths are based on the

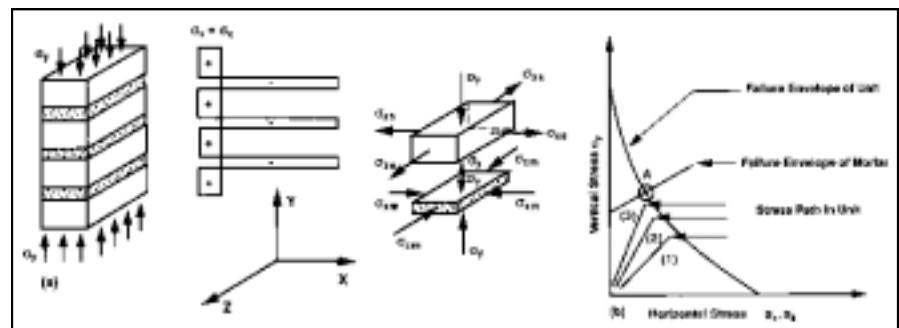


Fig. 13: Interaction of Units and Mortar Joints
a) Prism under uniaxial compression and stresses in unit and mortar;
b) Failure criterion for masonry

gross cross sectional area of the masonry elements because net area as used in the United States, [5], and Australia, [6], has no practical definition for general biaxial loading.

3.2 Uniaxial compression loading perpendicular to bed joints

The properties described below are typically obtained from tests on masonry prisms or small walls, three to six units high and one to four units wide. This is the basic masonry test specified in almost all national standards.

Figure 15 illustrates typical stress-strain characteristics of masonry of different strengths. Clay brick masonry shows a linear stress-strain characteristic almost up to ultimate. Strains at maximum stress are typically between 0.0015 and 0.002. Post peak strains up to and beyond 0.003 have been observed depending on the stiffness of the testing machine. Concrete block masonry shows a slightly more pronounced non-linear behaviour with similar strains at maximum stress. Obviously, stress-strain curves for masonry are similar to those of concrete. Therefore, the approaches used in national masonry standards are typically copies of the corresponding concrete codes.

Figure 16 summarizes masonry compressive strengths specified in National Standards, [5,6,7,8,9]. Typically, masonry compressive strengths range from 3 to 12 MPa. However, strengths up to 25 and 30 MPa comparable to the

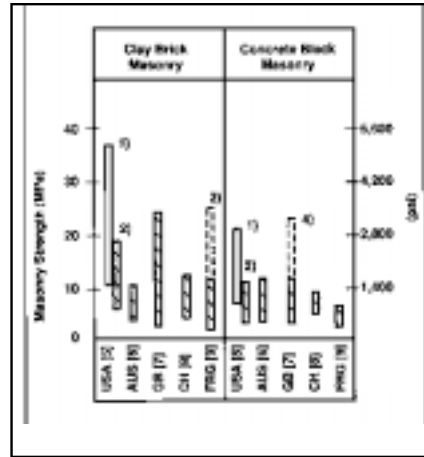


Fig. 16: Range of Masonry Compressive Strength according to National Standards, [5, 6, 7, 8, 9]

Note:

- 1) Based on net area;
- 2) Estimate for gross area based on $net/gross = 0.5$;
- 3) Special masonry;
- 4) For solid blocks only

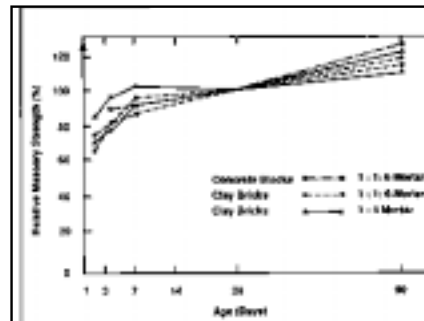


Figure 17: Effect of Age on Masonry Compressive Strength, [17]

COUNTRY/ STANDARD	CLAY BRICK MASONRY		CONCRETE BLOCK MASONRY	
	Ex/tmx	G/Ex	Ex/tmx	G/Ex
USA: [5]	750	0.4	750	0.4
Australia: [6]	1)	1)	1)	1)
Great Britain: [7]	900	1)	900	1)
Switzerland: [8]	850-1000	0.2-0.35	1250	0.4
Federal Republic of Germany: [9]	900-1200	1)	1000-1400	1)

Table 3: Elastic Properties of Masonry according to National Standards, [5,6,7,8,9]

Note: 1) not specified

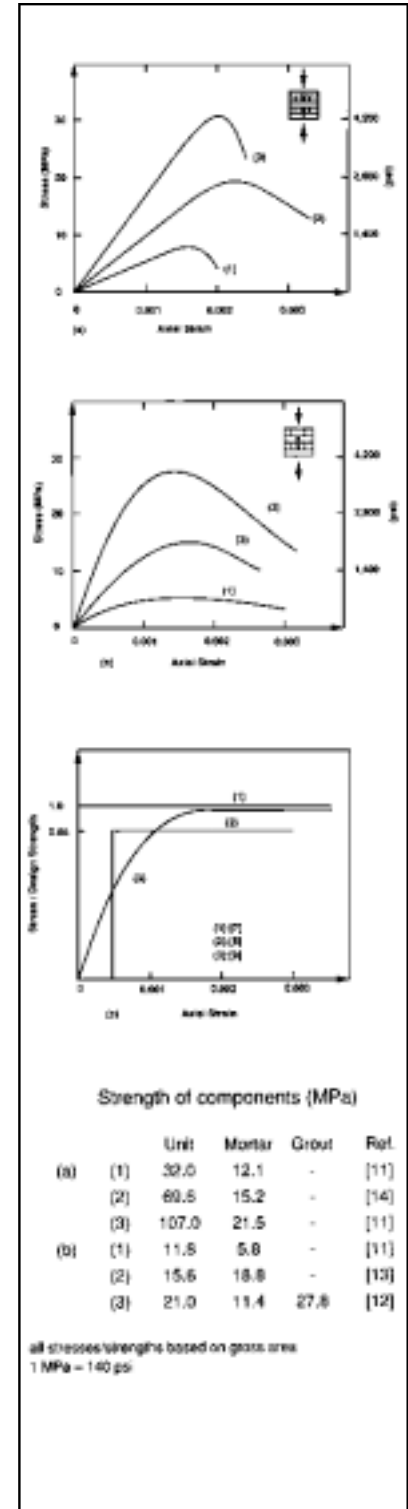


Fig. 15: Stress-Strain Characteristics of Masonry
a) Clay brick masonry, [11, 14];
b) Concrete block masonry, [11, 12, 13]
c) Code approaches, [5, 7, 9]

strength of normal concrete grades, may be obtained. Brick masonry seems to offer slightly higher strengths than concrete masonry.

The modulus of elasticity of masonry is given as a multiple of the masonry strength by many standards. Typically, that factor ranges from 750 to 1250 for both clay brick and concrete block masonry, Table 3. Most standards suggest a fixed ratio of shear modulus to modulus of elasticity of 0.4 as for concrete. However, investigations in Switzerland showed that actual ratios may be as low as 0.2 for hollow clay brick masonry, [15,16].

The development of masonry strength with age is illustrated in Figure 17. After seven days typically 80 to 90% of the strength at 28 days is reached. Masonry strength further increases at higher ages by 10 to 20% up to 90 days. There seem to be no basic differences between clay and concrete masonry.

3.3 General in-plane loading

The properties described below are typically not or only superficially addressed by national standards. However, they have a major impact on the behaviour of masonry walls when considering general loading conditions such as combined shear and axial loads or introduction of concentrated loads.

Bearing strength of masonry under local compression is illustrated in Figure 18. The strength enhancement factor given in Figure 18 is the ratio between the experimentally observed ultimate bearing pressure and the uniaxial compressive strength of masonry. Local loading at the end of a wall gives much smaller enhancement factors than central loading. The use of hollow units seems to further reduce the enhancement compared with solid units. Maximum enhancement factors of 1.5 and 2.0 are recommended for masonry with solid units in [18]. However, for masonry with hollow units, factors below unity have been reported for loads applied near the wall end and maximum enhancement factors of 1.5 are reached for central loading only for loaded lengths of approximately half a brick length. Thus, enhancement factors should be applied carefully depending on loading conditions and masonry type.

General uniaxial loading has been

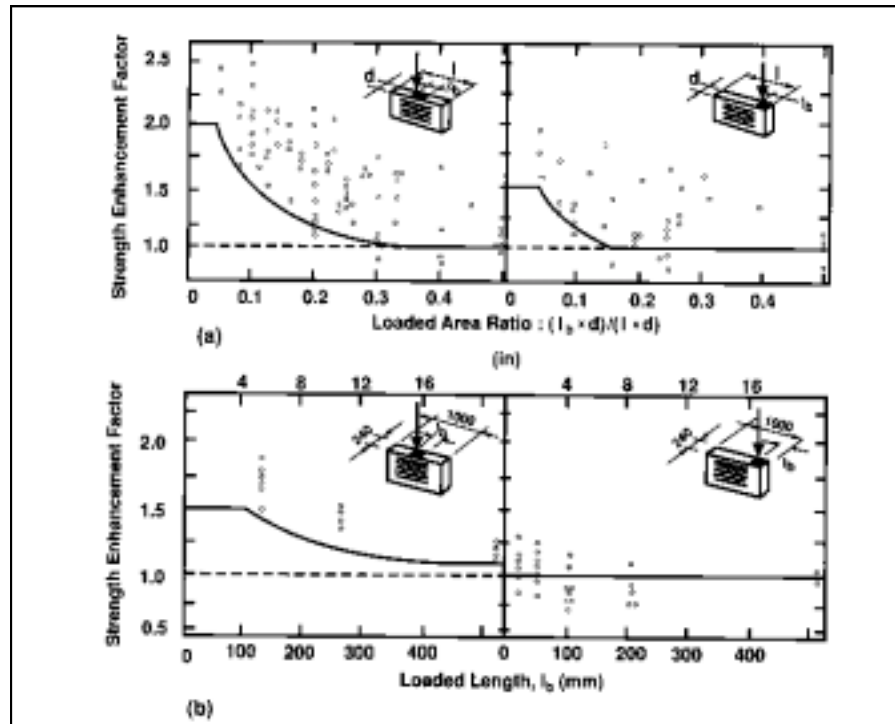


Figure 18: Bearing Strength of Masonry
a) Masonry with solid units, [18]; b) Masonry with hollow units, [19]

investigated primarily in Canada. Figure 19 illustrates the strength of masonry for uniaxial loading under different orientations, θ , with respect to the bed joints. A value of $\theta=0^\circ$ represents the uniaxial test described in Section 3.2 with a compressive strength called f_{mx} in the following. Prisms with loads applied under $\theta=90^\circ$, i.e. parallel to the bed joints, show lower strengths than f_{mx} in general. In particular for hollow clay brick masonry, strengths as low as 0.40 f_{mx} are obtained. For relatively small inclinations, say $\theta < 40^\circ$, splitting types of failure are observed with strengths as low as 0.40 to 0.50 f_{mx} for clay and 0.60 f_{mx} for concrete masonry. Except for grouted concrete masonry, even lower strengths are obtained for orientations $45^\circ < \theta < 75^\circ$ when sliding failure along the joints is governing. The strength may drop as low as 0.10 to 0.15 f_{mx} for clay and 0.35 f_{mx} for concrete masonry.

The biaxial strength of masonry has been investigated both experimentally and theoretically in Australia, Great Britain and Central Europe. In general, the test reports [15, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32] present principal stresses at

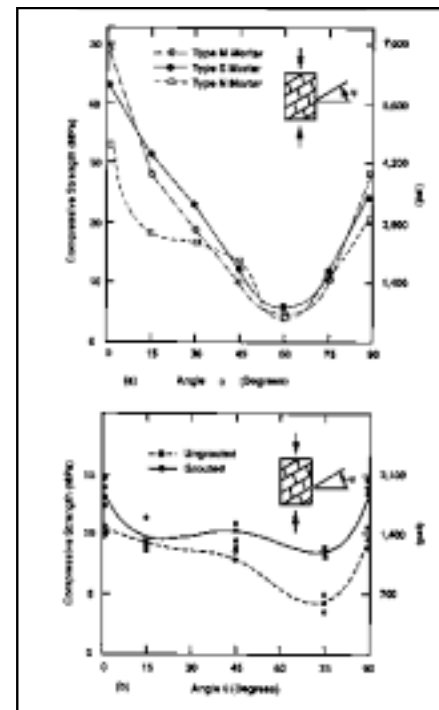


Fig. 19: Uniaxial Compressive Strength of Masonry
a) Clay brick masonry, [20];
b) Concrete block masonry, [21]

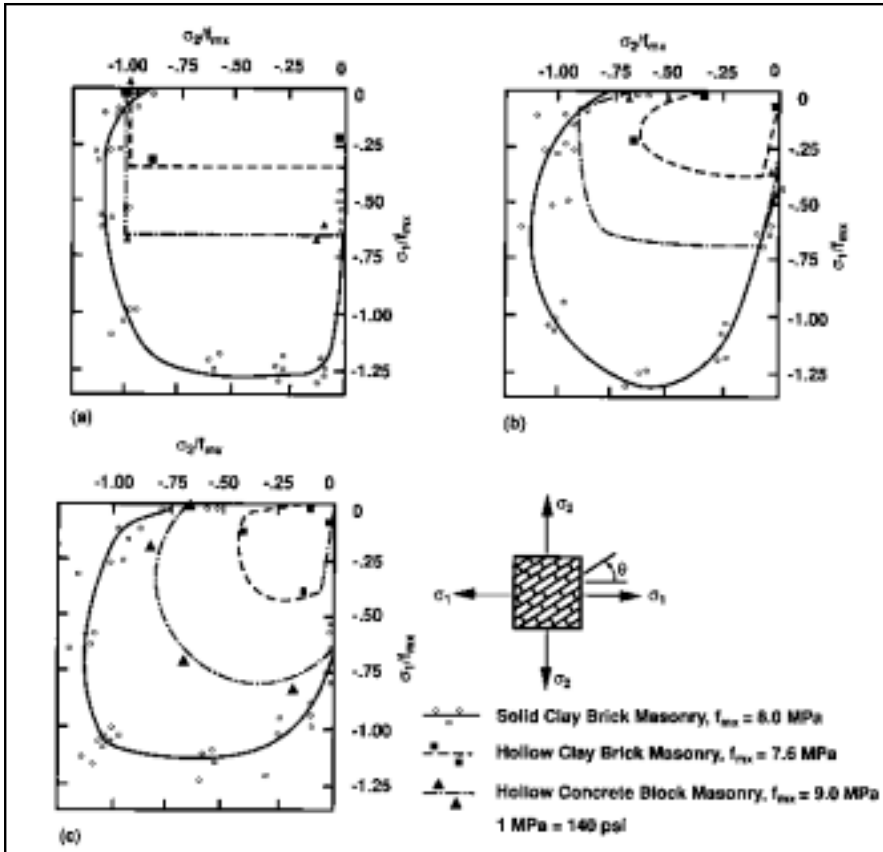


Fig. 20: Biaxial Compressive Strength of Masonry, (15, 22, 24]

- a) Stresses parallel to joints $\theta = 0^\circ$;
 b) Stresses under 22.5° to joints, $\theta = 22.5^\circ$; c) Stresses under 45° to joints, $\theta = 45^\circ$

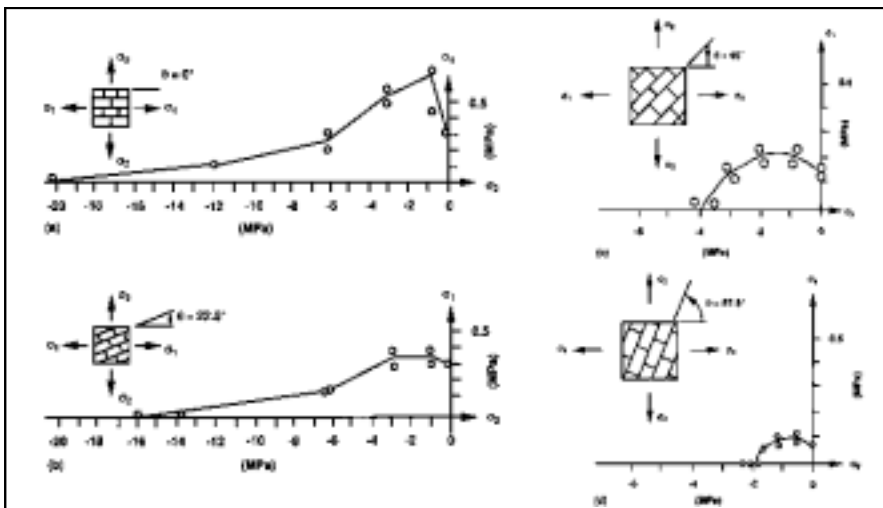


Fig. 21: Biaxial Tension-Compression Strength of Clay Brick Masonry with Solid Units, [23]

- a) Stresses parallel to joints, $\theta = 0^\circ$;
 b) Stresses under 22.5° to joints, $\theta = 22.5^\circ$;
 c) Stresses under 45° to joints, $\theta = 45^\circ$;
 d) Stresses under 67.5° to joints, $\theta = 67.5^\circ$ Note: 1 MPa = 140 psi

failure of the test specimens for different joint orientations.

Figure 20 summarizes the results of biaxial compression tests carried out on clay brick masonry made of solid and hollow units and ungrouted hollow concrete block masonry. The principal stresses at failure have been divided by the uniaxial compressive strength f_{mx} . Sliding failures along the joints were observed for uniaxial loading and/or moderate biaxial loading only and are therefore represented by points lying on or near the axes σ_1 for $\theta = 22.5^\circ$ and σ_1 and σ_2 for $\theta = 45^\circ$. As already noted in Figure 19 very low strengths are obtained for that failure mode, especially for hollow clay brick masonry. Except for sliding type of failure, solid clay brick masonry shows an almost isotropic behaviour with strengths close to or even in excess of f_{mx} . On the other hand, hollow clay brick masonry shows an exceptionally high degree of anisotropy. These types of brick seem to have been optimized solely to carry loads perpendicularly to the bed joints. For general biaxial loadings the strength only rarely exceeds 0.4 f_{mx} . Hollow concrete block masonry takes an intermediate position with, except for sliding failure, a minimum strength of approximately 0.7 f_{mx} . As already noted in connection with Figure 19, grouted concrete masonry is expected to show a nearly isotropic behaviour similar to solid brick masonry. Sliding failure along the joints is prevented by the grout, in general.

Figure 21 gives a similar presentation of the biaxial tension-compression strength of clay brick masonry with solid units. The maximum tensile strength was observed under a small axial compression applied perpendicularly to the bed joints. For this favourable loading condition, the tensile strength was only 3.5% of the compressive strength f_{mx} . Even smaller ratios were reported in [15].

The Swiss Standard, SIA 177/2, [8], is the only code which addresses the complete biaxial strength of masonry. Its approach is summarized in Figure 22 and has been further discussed in [33]. Basically, the strength of masonry is defined by three parameters, i.e., f_{mx} = uniaxial compressive strength for loads acting perpendicular to the bed joints, f_{my} = uniaxial compressive strength for loads acting parallel to the bed joints and $\tan \phi$

= coefficient of friction in the bed joints. The tensile strength of masonry as well as the cohesion in the bed joints are neglected. Figure 22a gives the strength for general biaxial loading as a function of normal and shear stresses in the joints. Failure of masonry is defined by four equations and is graphically represented by a three-dimensional surface. Stress combinations outside the surface are not possible and any combination in the interior of the surface does not introduce failure. The corresponding uniaxial strength is illustrated in Figure 22b. In general, the uniaxial strength of masonry is limited by f_{my} except for loads perpendicular to the bed joints. As a consequence of neglecting the cohesion, no loads can be transferred for inclinations $\alpha < \theta < 90^\circ$. This uniaxial strength according to Figure 22b may conservatively be used as a simplified biaxial strength. Figure 22c gives the parameters for design, i.e. already including strength reduction factors.

3.4 Flexural loading

For non-load bearing walls, the flexural strength of masonry is limited by its tensile strength and thus, is rather low and shows a brittle behaviour in general.

Uniaxial flexural loading has been investigated in Scandinavia and Canada [34,35]. Figure 23 illustrates the uniaxial bending strength of hollow, ungrouted concrete block masonry for moments applied under different orientations to the joints. The lowest strength typically is observed for bending stresses perpendicular to the bed joints. In general, two to five times larger capacities are obtained for bending stresses parallel to bed joints.

Biaxial flexural tests have been carried out in Switzerland and Australia [36,37]. Full scale tests have been presented in [36]. A typical moment interaction is illustrated in Figure 24 for a relatively low axial load. The general shape of the interaction seems not to be influenced by the axial load level. Figure 24 clearly demonstrates that for plain masonry an interaction between principal moments does exist. As a consequence, procedures developed for the design of reinforced concrete without such an interaction cannot simply be transferred to plain masonry.

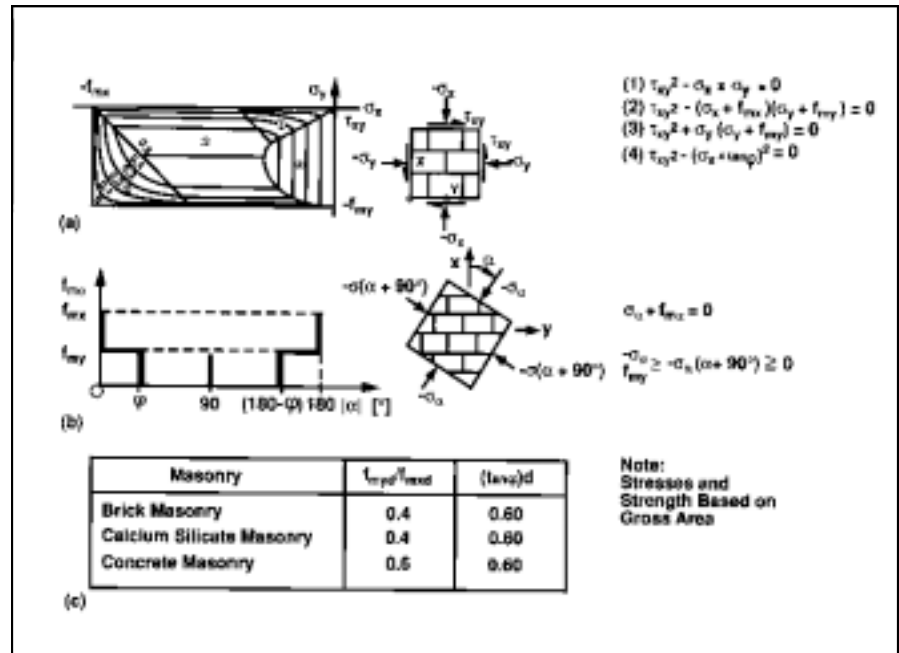


Fig. 22: SIA 177/2 Approach for the Biaxial Compressive Strength of Masonry, [8]

- a) General failure surface;
- b) Uniaxial and simplified biaxial strength;
- c) Design strength parameters

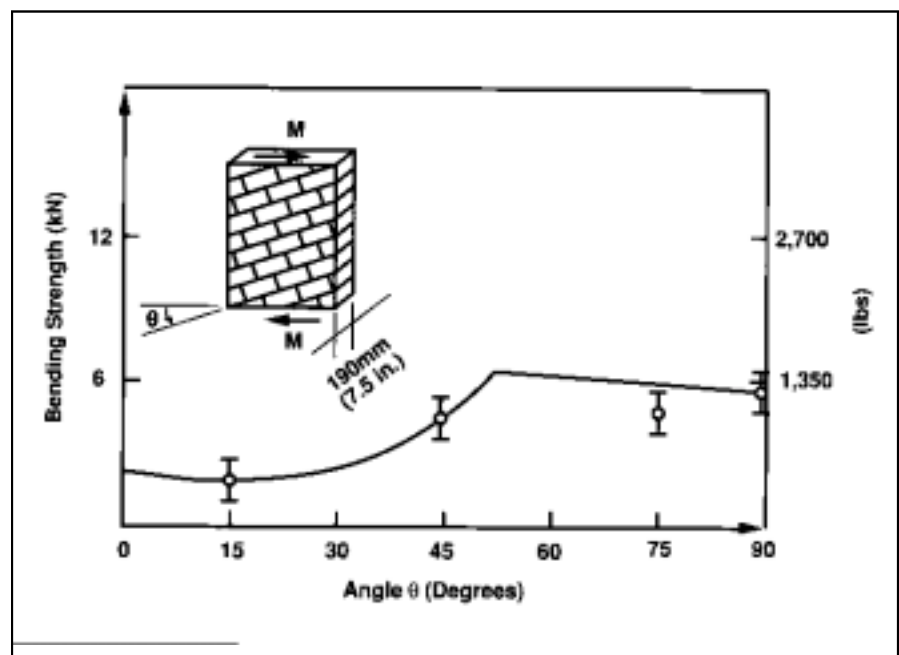


Fig. 23: Uniaxial Bending Strength of Ungrouted Concrete Masonry, [34]

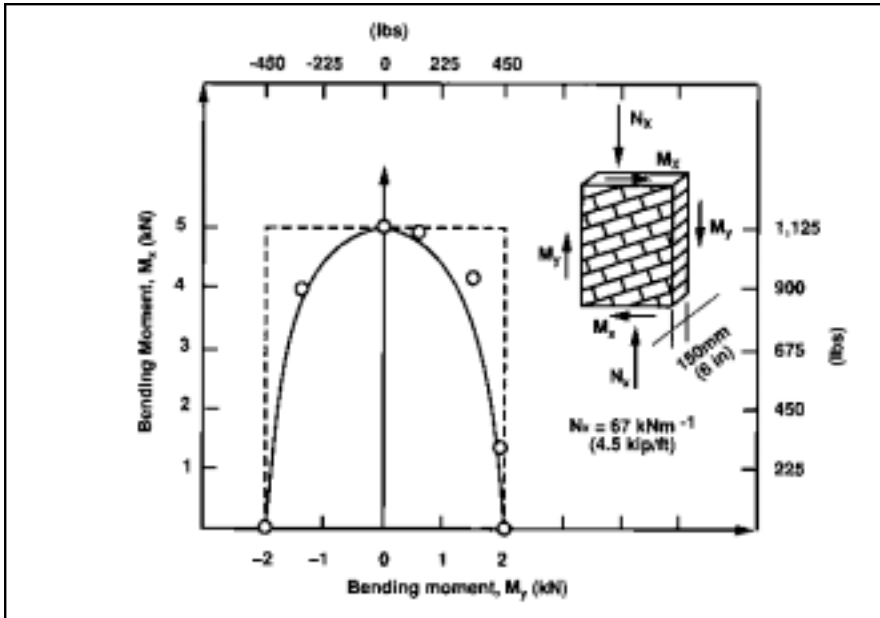


Fig. 24: Biaxial Bending Strength of Clay Brick Masonry, [36]

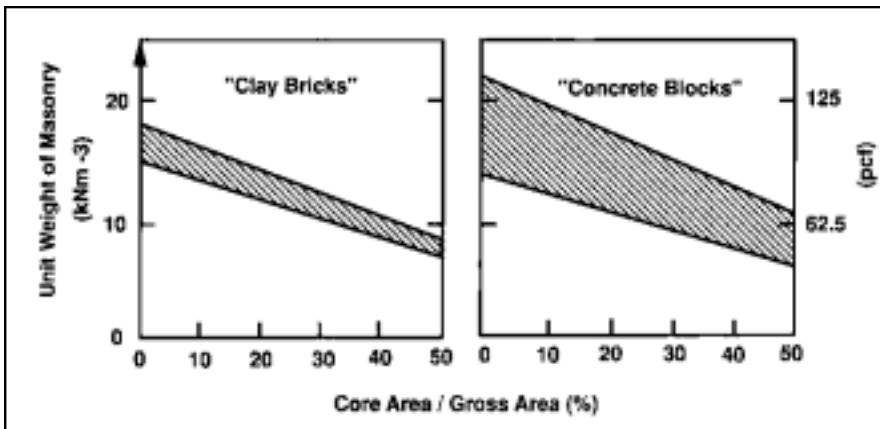


Fig. 25: Unit Weight of Masonry

3.5 Unit weight of masonry

Due to the large variety of raw materials and net/gross area ratios no unique unit weight can be given for clay and concrete masonry. Figure 25 indicates the range of unit weights that may be expected in practice.

3.6 Temperature, creep and shrinkage deformations

Clay brick masonry is well known for its low values of volume changes. The coefficient of thermal expansion is only about 60% of the value of concrete. Shrinkage shortening is usually compensated by expansion due to increase in humidity and final creep deformations have the same order of magnitude as the elastic deformations. The corresponding values for concrete masonry are similar to those of concrete. Table 4 summarizes a proposal given in [38]. It should be noted that these values are heavily influenced by the initial water content of the masonry units and therefore, care should be taken for proper storage and protection of the units during construction.

TYPE OF MASONRY	CLAY BRICK	CONCRETE BLOCK
• Coefficient of thermal expansion	$(5 \text{ to } 7) \times 10^{-6}/^{\circ}\text{C}$	$(8 \text{ to } 12) \times 10^{-6}/^{\circ}\text{C}$
• Final shrinkage strain	$(0.4 \text{ to } -0.2) \times 10^{-3}$	$(-0.1 \text{ to } -0.6) \times 10^{-3}$
• Final creep coefficient	$(0.5 \text{ to } 1.5)$	$(1.5 \text{ to } 2.5)$

Table 4: Volume Change Characteristics of Masonry, [38]
Note: Positive shrinkage value means elongation.

4. Design Considerations

4.1 General

The major advantages offered by post-tensioning have been outlined in Chapter 1. Specifics about the behaviour of masonry walls subjected to axial loads and imposed deformations, out-of-plane lateral loads and shear loads will be given in this chapter. Finally, some aspects of detailing of post-tensioned masonry walls will be presented.

The layout of the prestressing steel in masonry walls is similar to that of reinforcing bars and is illustrated in Figure 26. Some inherent properties of masonry and the statcal system of most walls clearly favour the vertical axis of a wall to be the direction best suited for the placing of the tendons. For walls made of hollow units the tendons may be placed in relatively large cores at the centre of the wall. For applications with solid units, cavities and special pockets may be formed by masonry leaves to place the tendons. Thus, the tendons basically are straight at the centre of the wall or at a constant eccentricity. For special applications such as ties at floor levels, tendons might be placed horizontally if units with special grooves are used.

While for grouted masonry constructions a bonded post-tensioning system might be used similar to concrete constructions, an unbonded system using monostrands offers major advantages in ungrouted masonry constructions both for constructability and durability reasons. Monostrands do not require grouting, thus eliminating a whole step in the construction of a wall. Furthermore, they provide an excellent double corrosion protection of the prestressing steel made up by grease and plastic duct. Some basic differences in the behaviour of structural elements post-tensioned with bonded and unbonded tendons have been addressed elsewhere, [39]. In particular, masonry post-tensioned with unbonded tendons may show a similarly low energy dissipation as unreinforced masonry under lateral load reversal. Due to the lack of yielding of reinforcement, in general, almost the entire energy introduced in an element by the lateral load is stored in an increase of potential energy of gravity loads and/or in elastic deformations of the tendons and thus, will be regained upon unloading, see Figure 27.

As for any structural element, masonry walls have to be checked for serviceability

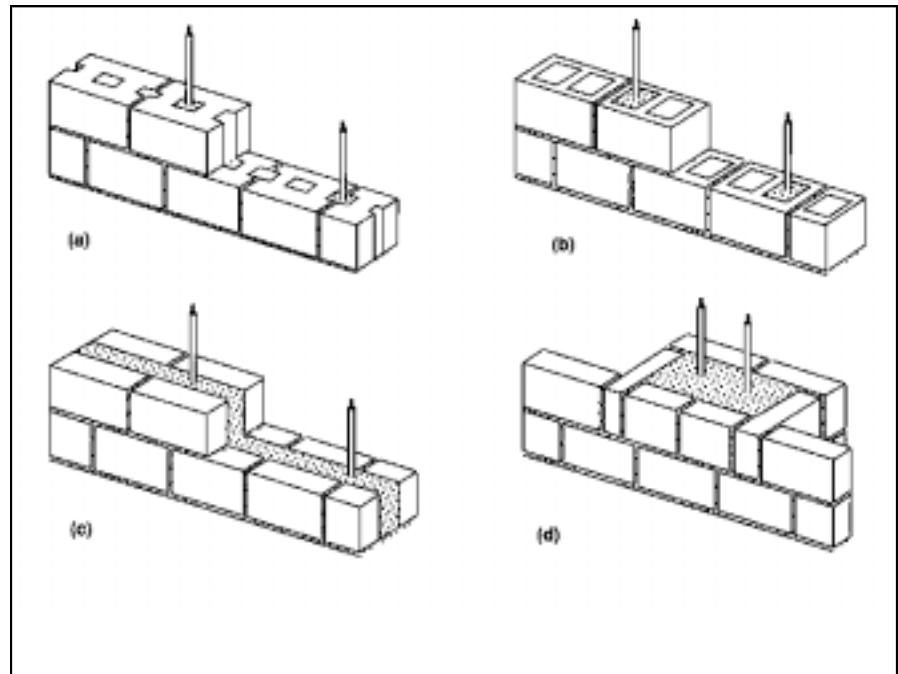


Fig. 26: Layout of Vertical Tendons in Masonry Walls
a) In cores and head joints; b) In cores only; c) In cavities; d) In pockets

and ultimate limit state requirements. Due to the relatively small span to depth ratios and the small out-of-plane transverse loads applied to masonry walls, the governing criterion at the serviceability limit state is the limitation of crack widths, in general. Under permanent actions, cracking should either be avoided or maximum crack widths should be limited, depending on the exposure conditions and on the requirements regarding visibility of the cracks. In [8], wall deformations are limited such as to have nominally crack-free masonry for severe exposure and high requirements and a nominal crack width of approximately 0.2mm (0.008in) for members protected from direct rain and normal requirements. Under short term transient loads, a more liberal attitude towards crack width might be adopted.

For serviceability limit state checks, it is generally agreed to consider posttensioning as an externally applied action using effective tendon forces. Thus, for straight tendons placed at the centre of a wall, the only action to be considered is the axial load introduced at the anchorages of the tendon. At the ultimate limit state, the required strength shall be provided without considering the tensile strength of masonry. Although this principle is well accepted in concrete codes, a number of national

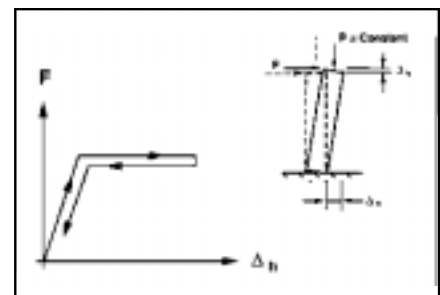


Fig. 27: Energy Dissipation in Unreinforced Masonry

national masonry standards still allow one to consider flexural tensile stresses and even small axial tension for the design of walls for out-of-plane lateral loads and shear. For ultimate limit state checks, post-tensioning should be considered as a resistance and thus, proper strength reduction factors have to be applied to the tendon force. For bonded posttensioning, the tendon force at ultimate may be determined in the critical sections following the same principles as in prestressed concrete design, i.e. assuming rigid bond and plane sections. Thus, the tendons will reach their yield force, P_y , in general. For unbonded posttensioning, the tendon force will increase from service up to ultimate load level depending on

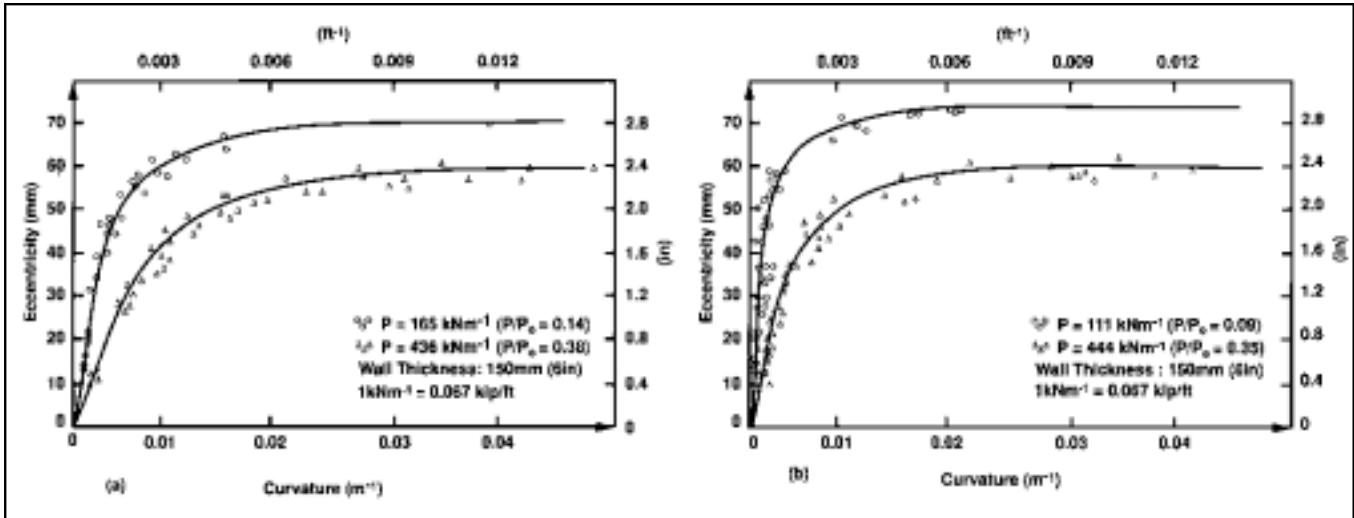


Fig. 28: Eccentricity - Curvature Curves
a) Clay brick masonry, [42]; b) UngROUTED concrete block masonry, [46]

This tendon force increase may be estimated by applying rigid body mechanisms similar to those used in the design of post-tensioned slabs with unbonded tendons, [40]. Typically, a nominal failure characterized by a maximum deflection of about two percent of the slab span is assumed and the resulting elongation of the tendon is then determined from geometry. For a simply supported beam with a plastic hinge at midspan, such a deflection corresponds to a rotation of eight percent in the hinge. Although such large rotations can be achieved in masonry walls under low axial loads as well, any possible tendon force increase beyond the effective force is often neglected in the design at ultimate limit state. Although, from a theoretical point of view, the post-tensioning should be considered as a resistance, in practice it may be easier to introduce it as a known action at ultimate limit state as well if the strength reduction factor is properly taken into account. Design considerations developed for plain masonry may then directly be applied to post-tensioned masonry.

Unlike bonded tendons, monostrands placed in ducts are not continuously guided and thus may obtain displacements transversely to the plane of the wall. Such displacements may reduce the effective depth of the post-tensioning and introduce second order effects in the wall. Such effects can easily be controlled by limiting the size of the ducts

and/or by grouting the cores around the duct/monostrand. Provided that such measures have been taken, stability failure of masonry walls due to the introduction of prestressing forces must not be considered, in general.

4.2 Walls subjected to axial load

To carry gravity loads is one of the primary functions of masonry walls. However, typical wall-floor slab connections such as illustrated in Figure 11, introduce eccentricities of the axial loads into the walls and as a consequence may introduce cracks running in a bed joint parallel to the slab plane. To reduce or even eliminate such eccentricities, various types of bearings have been proposed between wall and slab. While such bearings may be helpful to control cracking, they greatly reduce the redundancy of the wall-slab frame system, and thus may not be appropriate for ultimate conditions. So, a reasonable compromise between requirements at service and at ultimate conditions has to be found.

The strength of axially loaded walls has been investigated throughout the world. However, the interaction of walls subjected to axial loads and floor slabs was first addressed by Sahlin, [41], and has been further studied both experimentally and theoretically at the ETH in Zurich, [33,42,43,44,45,46]. An axial load was applied at the beginning of a test and held

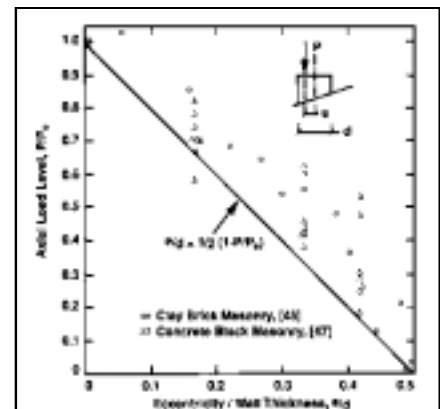


Fig. 29: Interaction of Axial Load and Bending, [45, 47]

constant during the entire length of the test. Then, a rotation was introduced at the lower wall end through a concrete slab and increased in a deformation controlled manner up to failure of the wall. This test set-up allowed for the investigation of the response of axially loaded walls well beyond the peak eccentricity.

Figure 28 illustrates observed moment-curvature curves for clay and concrete masonry wall sections at axial load levels of approximately 10% and 35% of the axial resistance, P_0 , respectively. For constant axial load it is convenient to replace the moment by the eccentricity of the axial load, simply defined as the ratio of moment to axial load. Almost bi-linear elastic-perfectly plastic, moment-curvature curves

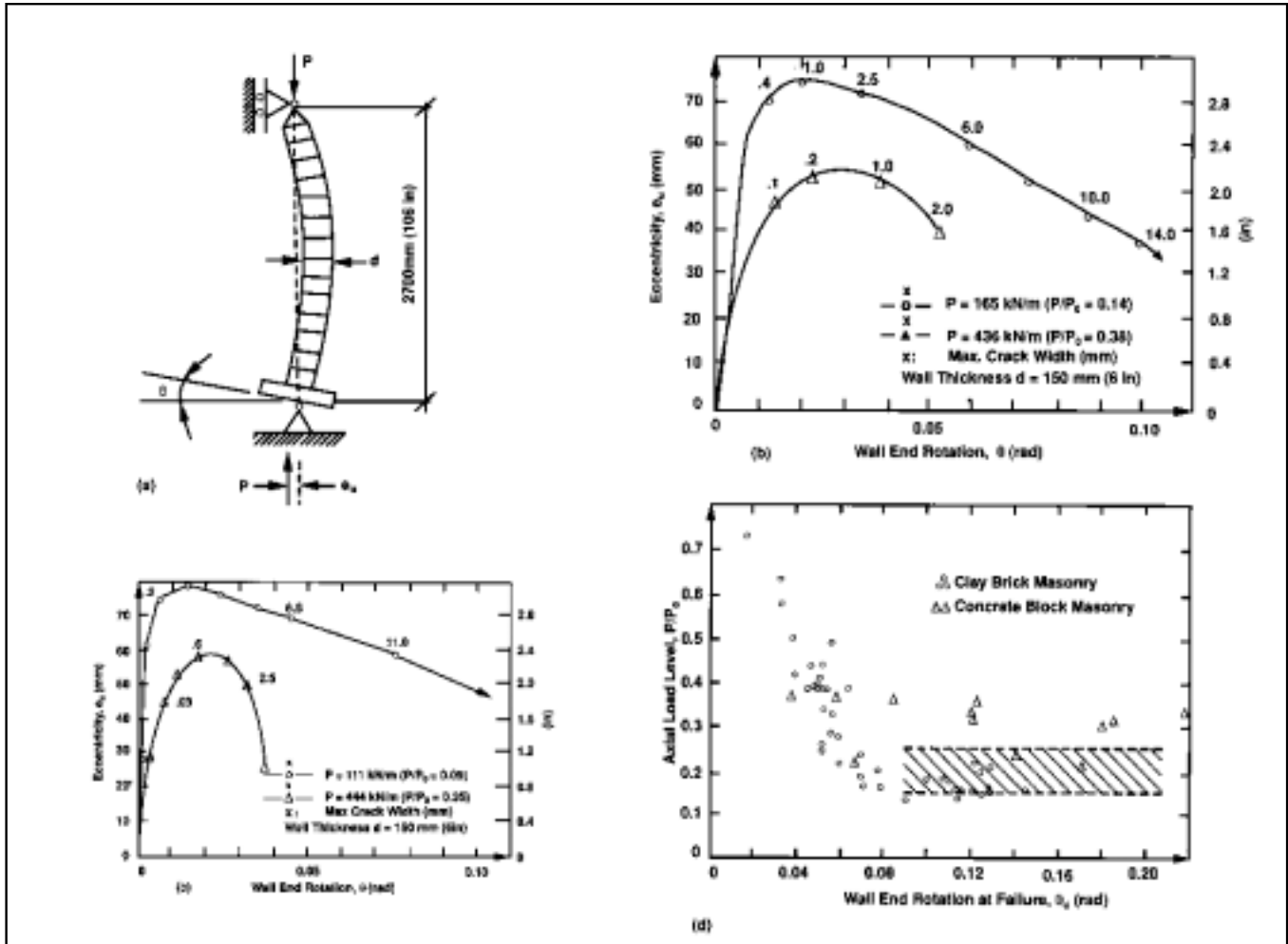


Fig. 30: Response of Plain Masonry Walls to Axial Load and Bending
 a) System; b) Eccentricity - wall end rotation of clay brick masonry walls, [42];
 c) Eccentricity - wall end rotation of concrete block masonry walls, [46];
 d) Wall end rotation at failure, [44, 46]

ture curves are obtained for small axial loads. For larger axial loads, the curves become more and more non-linear. Whereas the strength of the section is strongly influenced by the axial load level, the initial bending stiffness is almost constant for the practical range of axial load levels. No basic differences have been detected between clay and concrete masonry. The ultimate bending strength of axially loaded sections is illustrated in Figure 29. The ratio of maximum eccentricity to wall thickness is presented as a function of the axial load level, P/P_0 . Results shown include tests of full scale clay masonry walls with constant axial load, [45], as well as on grouted and ungrouted concrete masonry prisms with constant eccentricity, [47].

Assuming a rectangular stress block, such as the ones presented in Figure 15c, over a nominally solid cross section of masonry results in the linear relationship between axial load and eccentricity shown in Figure 29. Obviously, such a linear relationship is conservative. One reason for the underestimation of the actual resistance by the linear relationship lies in the fact that hollow units have material concentrated along the edges whereas the above mentioned curve assumes a uniform distribution of material over the section.

Figure 30 gives a summary of the observed behaviour of the wall-slab system. Notations are introduced in Figure 30a. It should be noted that rotations, θ , may not only be caused by deflections but also by in-plane

deformations of slab due to creep, shrinkage and temperature variations. Figures 30b, and c illustrate the relationship of wall end eccentricity, e_u , i.e. wall end moment, and wall end rotation, θ . The behaviour is highly nonlinear due to the combined effects of material non-linearity as shown in Figure 28 and second order effects due to out-of-plane wall deflections. Up to rotations close to maximum eccentricity an almost linear behaviour with either uncracked sections or cracked sections with only small crack widths is observed. For rotations beyond maximum eccentricity, second order effects result in a reduction of the applied wall end moment and crack widths increase rapidly due to a localization of the applied rotation to only a small number of cracks, often just one crack,

in bed joints. Again, no basic difference is noted between clay and concrete masonry. Figure 30d gives a summary of wall end rotations at failure of the wall. For axial loads below 15% to 25% of the axial capacity, very large rotations can be imposed on masonry walls without causing failure. The walls behave almost perfectly plastic and much larger rotations than 0.08 as assumed in Section 4.1 may be obtained. For larger axial loads the rotations at failure rapidly decrease to relatively small values.

Based on the above investigations, a relatively simple procedure has been proposed in [8] to check axially loaded walls at ultimate limit state. For axial load levels not exceeding 25% of the wall design resistance only instability failure has to be checked. Instability failure is excluded as long as the ratio of actual wall height to Euler buckling length, h/hEd , falls below the straight line given in Figure 31 for given wall end eccentricities. Note that Index d indicates that loads and material properties are to be taken at design level, i.e. they include load and strength reduction factors. The following conclusions can be drawn for post-tensioned walls:

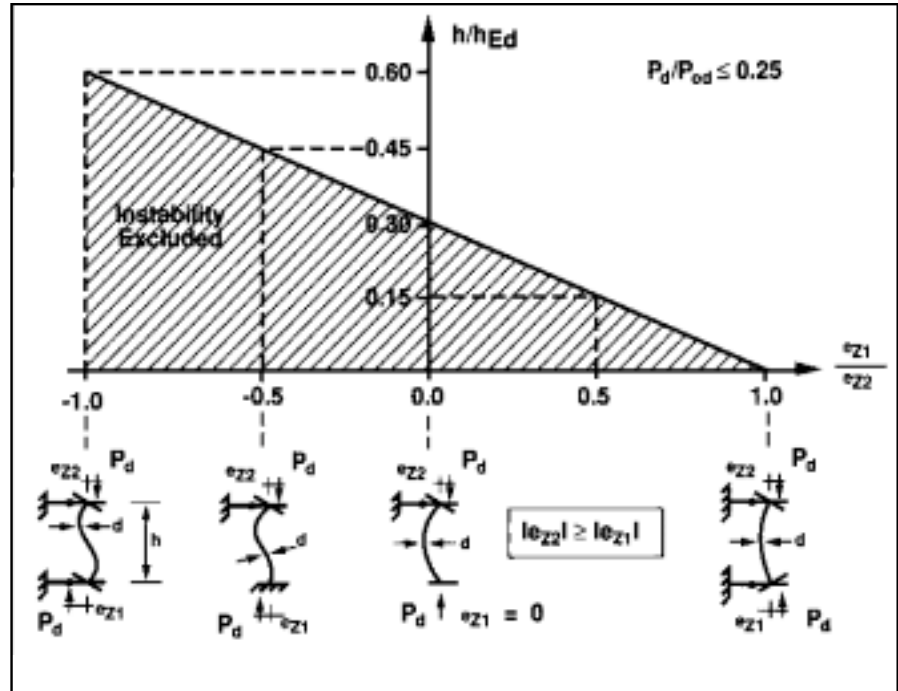


Fig 31: SIA 177/2 Approach for Walls Subjected to Axial Load and Bending, [8]

Note:

$$\begin{aligned} P_{od} &= \frac{f_{mx} d \times d}{\pi \sqrt{Byd/Pd}} & : \text{Design Resistance} \\ hEd &= \frac{\pi \sqrt{Byd/Pd}}{Emd \times I \times \sqrt{1 - \frac{Pd}{2 P_{od}}}} & : \text{Buckling Length} \\ Byd &= & : \text{Bending Stiffness} \end{aligned}$$

Emd : Modulus of elasticity of masonry;

I : Moment of inertia of gross cross-section

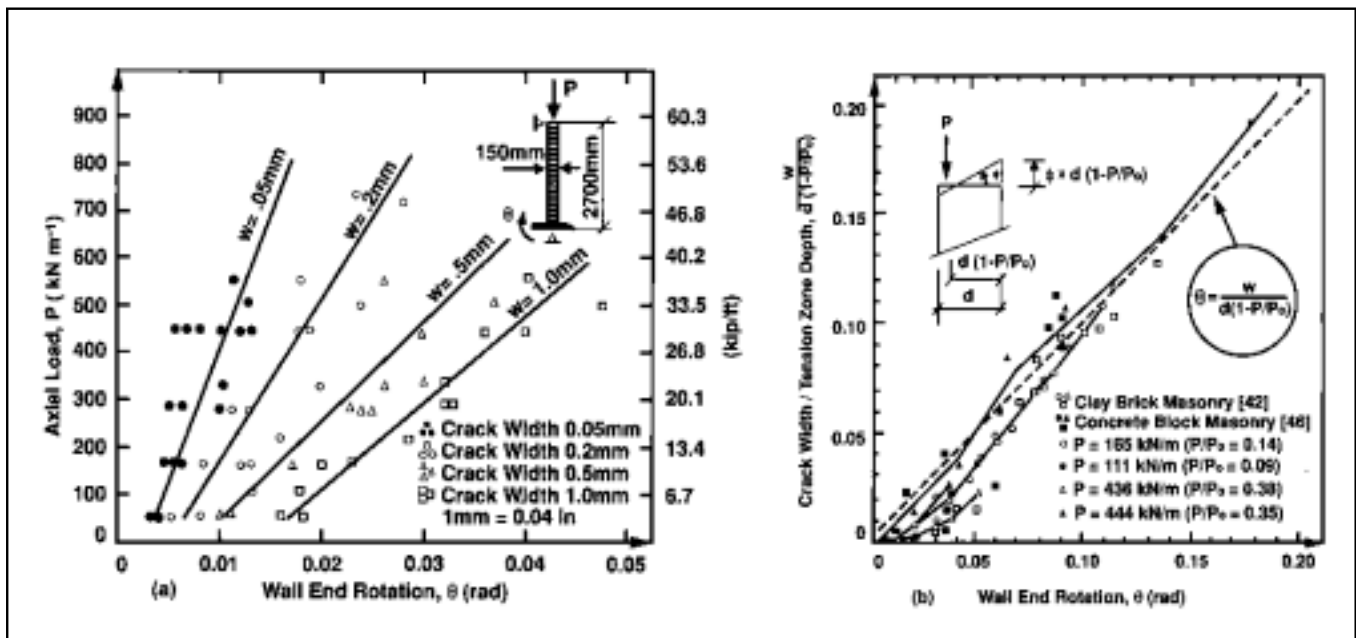


Fig. 32: Cracking Behaviour of Masonry Walls

a) Influence of axial load on crack width, [44]; b) Development of crack width with wall end rotation, [42, 46]

1. For axial load levels due to the combined effects of gravity loads and prestressing not exceeding 25% of the design resistance, a wall will behave in a ductile manner and material failure can be excluded.
2. As prestressing does not contribute to second order effects, instability failure may be checked for the effects of gravity loads only according to Figure 31. The effects of prestressing shall be considered in the computation of the bending stiffness.

Thus, for relatively low axial loads, crack widths generally are the governing design criteria. Figure 32 gives some further aspects of the cracking behaviour of axially loaded walls. Figure 32a shows the influence of axial load on crack width for a given wall end rotation. Obviously, any increase in axial load either caused by additional gravity loads or prestressing will reduce the maximum crack width. In Figure 32b, the development of maximum crack width is presented and the ratio of crack width to tension zone depth, w/d ($1-P/P_0$), is compared with the applied rotation, θ . Crack widths equal to the product of applied rotation times tension zone depth are represented by the straight line at 45 degrees in Figure 32b. For small rotations, the actual crack widths fall well below the product of rotation times tension zone depth. However, for large rotations actual crack widths group closely around the straight line, indicating that the total imposed wall end rotation is concentrated in one major crack.

Based on the observed cracking behaviour, some recommendations for the mitigation of horizontal cracks due to imposed wall end rotations are given in Figure 33. If the location of the crack is known or deliberately forced to a known place, it simply may be covered by some plate as indicated in Figure 33a to prevent water entering the wall or avoid visibility. Reducing the wall thickness over the full height or only by using a soft strip below the slab will help to reduce crack width, Figure 33b. Often a combination of a reduction of wall thickness with an increase of axial load by introducing prestressing will yield optimum benefits both for crack width and for strength of the wall, Figure 33c.

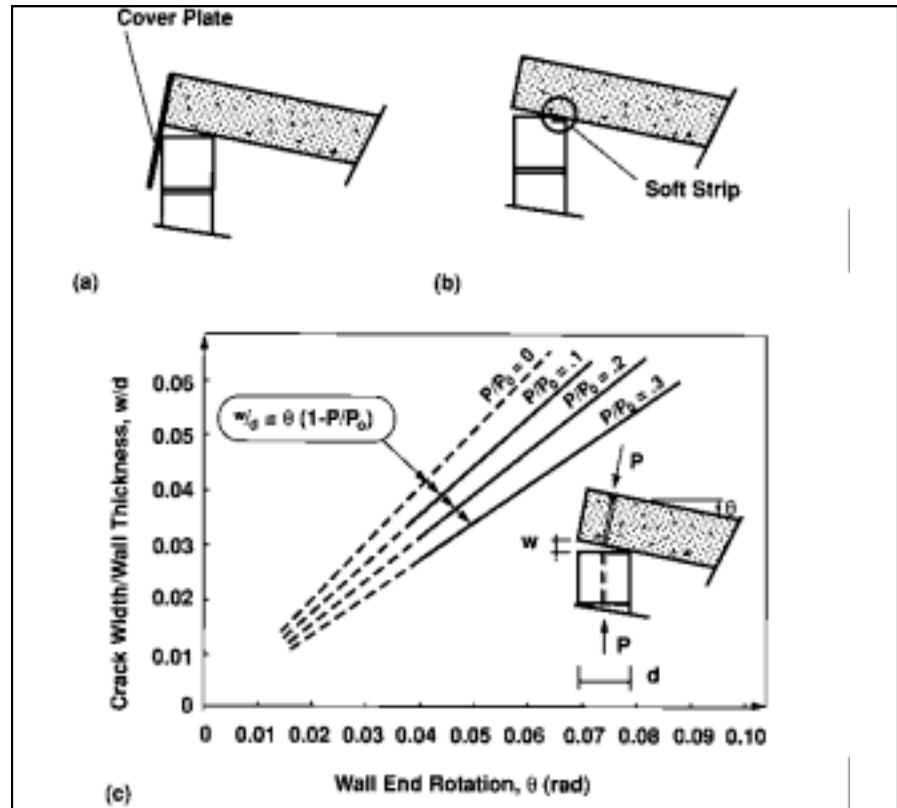


Fig. 33: Mitigation of Bending Cracks a) Protect crack; b) Reduce wall thickness; c) Post-tensioning of wall

4.3 Walls subjected to out-of-plane lateral load

Basically, the information given in Section 4.2 is equally valid for walls subjected to out-of-plane lateral loads. However, there exists one basic difference. Let us, for a moment, neglect second order effects which is a reasonable assumption under low axial loads. For walls subjected to axial loads and imposed rotations, first order moments are maximum at the joints. For ultimate conditions, a viable equilibrium solution is to set them equal to zero and to design the floor slab for simple supports. Thus, the walls would not require any bending strength and the whole problem would reduce to one of service conditions. However, for lateral loads directly applied to the wall obviously some bending strength is required for any

equilibrium solution. In addition, maximum moments will shift from the joints towards mid height of the wall with an increase in wall deflections and thus an increase in second order effects.

The strengths of unreinforced non-load bearing walls and walls with low axial loads depend primarily on the tensile strength of masonry which has been shown to be low in Chapter 3. For statically determinate systems ultimate load is identical to cracking load. For statically indeterminate wall systems, some redistribution of moments after initial cracking is possible. Figure 34 illustrates typical load deflection curves for walls simply supported on three or four sides, [48]. While walls supported on four sides show a reasonably high cracking and ultimate load, walls supported on three sides exhibit a low cracking load and a small ultimate strength in general,

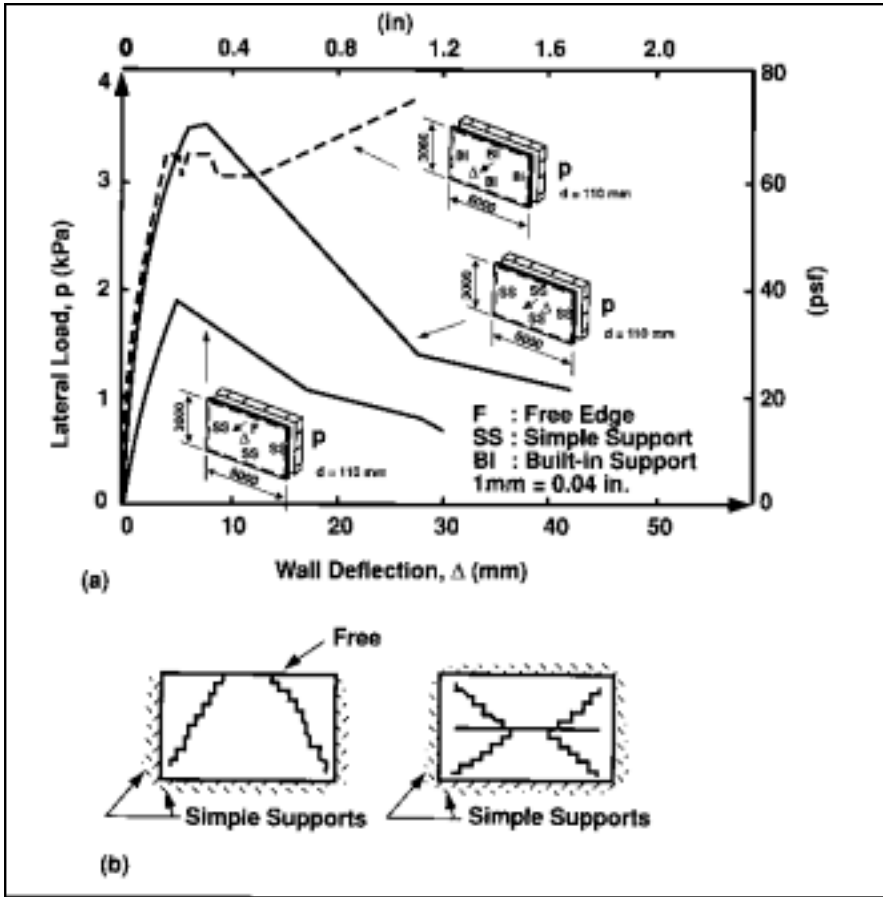


Fig. 34: Response of Plain Masonry Walls to Out-of-Plane Lateral Loads, [48]
a) Typical load-deflection curves; b) Typical crack pattern

Figure 34a. Without arching action developing between built-in supports, a major drop in the wall resistance is observed after exceeding peak load. Typical crack patterns are presented in Figure 34b. In practice, door and window openings divide walls supported on four sides into an assemblage of strips and panels supported on two or three sides only.

A considerable amount of research into walls subjected to out-of-plane lateral loads has been carried out in Great Britain, Australia, and Scandinavia. This research has recently been reviewed, [49]. Basically, three different approaches have been proposed for the design of unreinforced walls at ultimate limit state, i.e. approaches based on empirical methods, elastic methods, and yield line methods. The latter has found the most widespread application due to its practical advantages in the design of walls of

different shapes and with different loads, and has been introduced in the British Masonry Standard, [7]. Reference [49] also contains a comparison between failure loads calculated according to yield line theory and actual strength values, Figure 35.

Although the average ratio of predicted and measured failure load is close to unity, there is a large scatter with extreme ratios as low and high as 0.5 and 2.0, respectively. Under such circumstances a typical engineering approach would be to avoid the solution of the problem altogether and to provide the required strength by a reliable and well known method, i.e. to reinforce and/or prestress the wall. Based on the information presented in Section 4.2, the bending strength of post-tensioned masonry sections can easily and reliably be calculated and the strength of vertically spanned wall strips predicted based on equilibrium solutions.

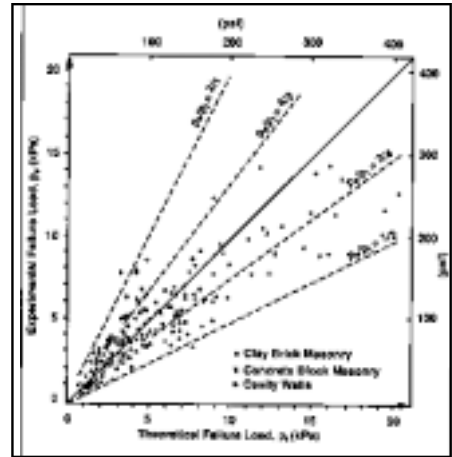


Fig. 35: Theoretical Lateral Failure Load vs. Experimental Load, [49]

Such an approach has been used to develop the diagrams presented in Figure 36 for the design of non-load bearing, post-tensioned, vertically spanned wall strips. The diagrams give the required minimum spacing of 15mm diameter monostrands as a function of the wall height for a given design out-of-plane lateral load, $p_d=1$ kPa. Four different support systems and three wall thicknesses have been considered. An effective prestressing force of 160 kN (36 kips) per strand has been used with a strength reduction factor of 1.2. The internal lever arm has been fixed to three eighths of the wall thickness for the centric tendon for axial load levels not exceeding 25% of the axial capacity, [8]. Therefore, the strand spacing becomes proportional to the factor P_{eff}/p_d and different prestressing forces and lateral loads can easily be considered. For example, for a simply supported wall, 6m high and 200mm thick, a lateral load of 1.5 kPa and tendon force of $P_{eff} = 150$ kN reduce the spacing of 2.15 m found in the graph to 1.34 m according to the following relationship:

$$a \leq 2.15 \times 1.0/1.5 \times 150/160 = 1.34 \text{ m}$$

A reduction of the ductility of masonry walls due to prestressing is to be avoided and can be controlled by limiting the axial load due to the combined design effects of gravity loads and prestress to 25% of the design capacity of the wall, as presented in Section 4.2. Such a limit has been introduced in the graphs in Figure 36 and minimum strand spacings of

approximately 0.9m, 0.7m and 0.5m have been found for wall thicknesses of 150mm, 200mm, and 250mm, respectively. A strand spacing of 0.5m is a reasonable lower limit in practice, and thus a reduction of the ductility of masonry walls due to prestressing is avoided.

Except for cantilever walls the minimum strand spacing is reached only for wall slenderness ratios, h/d , much larger than 40. For such large slenderness ratios a check of the sensitivity of the wall to vibrations is recommended and therefore the corresponding parts of the curves have been shown dashed. The bending stiffness can be estimated from Figure 31.

The presence of gravity loads in load bearing walls reduces the required amount of prestressing, i.e., it increases the tendon spacing. On the other hand, second order effects will be introduced in the wall which are not present in prestressed non-load bearing walls. For large axial gravity loads, a rigorous analysis of the wall for the combined effects of axial and lateral loads is recommended. Efficient numerical procedures using column deflection curves have been presented by different authors, [50,51], and may be applied to masonry wall systems using moment-curvature curves proposed in [8].

Walls prestressed and/or reinforced vertically and horizontally may be considered using the strip method [52]. To avoid problems at service conditions, cracking load may be estimated using elastic plate theory.

4.4 Walls subjected to in-plane shear loads

A considerable amount of research has been devoted to the behaviour of masonry shear walls. While unreinforced shear walls have been investigated, [53,54,55,56,57,58], in Europe and the Eastern United States, primarily New Zealand and the Western United States have focused on the behaviour of reinforced masonry shear walls under seismic loading [59,60,61,62]. Similar to the difference in materials, different test setups have been used. In Europe, walls have been tested under applied axial and shear loads without restraining the deflections at the top of the wall. In the United States, walls have

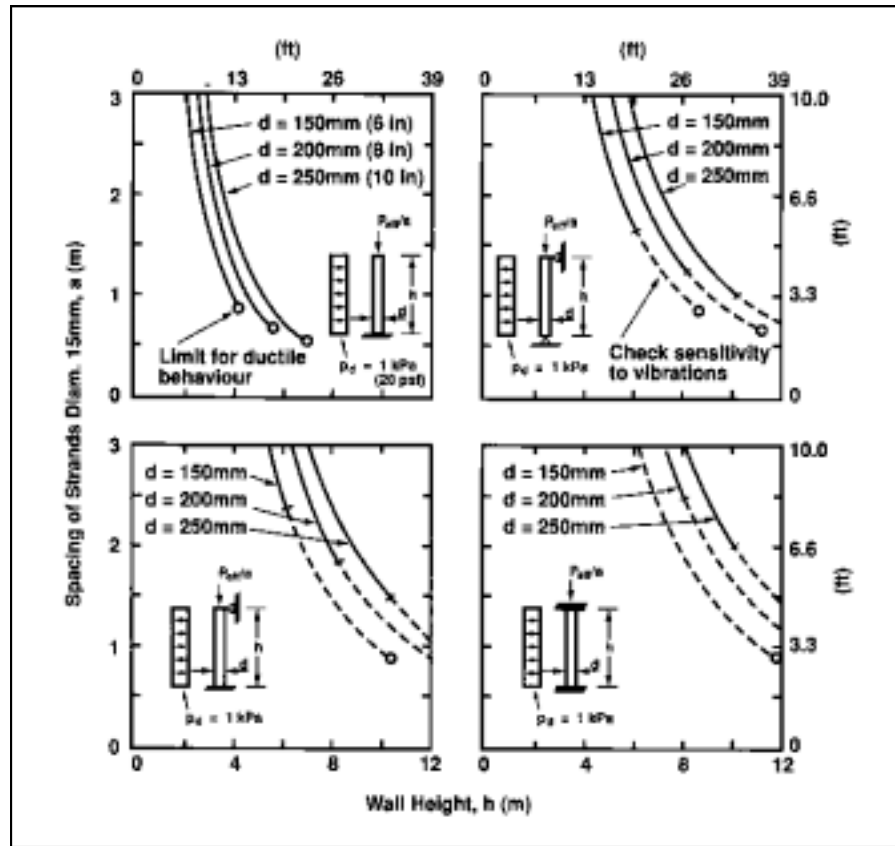


Fig. 36: Proposal for the Design of Vertically Post-Tensioned Walls
Note: $P_{eff} = 160 \text{ kN}$ (36 kips), effective force per strand
 $P_d = 1 \text{ kPa}$ (20 psf), design lateral load (factored)
Strength reduction factor for P_{eff} : 1.2

been subjected to axial and shear load with in-plane rotations at the top of the walls being prevented by strong spreader beams, thus applying a deformation-controlled bending moment opposing the effect of the shear load. Axial loads are kept constant in both test procedures.

Figure 37 presents load-deflection curves for unreinforced walls tested in the two different test set-ups. Figure 37a illustrates the response of walls with length to height of 1.8 tested at ETH Zurich, [55]. Smooth load-deformation curves were observed with a gradual onset of cracking at or above approximately 50% or the ultimate shear load. Compared with unreinforced walls, the one wall with well-anchored bed joint reinforcement showed a major enhancement in ductility. The initial stiffness does not seem

to be influenced by the axial load level, P/P_o .

In the NBS-tests, [58], cracking initiated typically at/or close to peak load, followed by a drop in load, Figure 37b. The drop in strength of unreinforced masonry is comparable to the behaviour of underreinforced concrete members and reduces as the axial load increases. Again, the initial stiffness of the square walls was independent of the axial load level. In the investigated range of axial loads the shear strength increased with axial load.

Figure 38 compares load-deflection curves of reinforced and prestressed shear walls, [63]. Both walls had four vertical prestressing bars with a diameter of 15mm and two horizontal bars at top and bottom of the wall. In one test, the vertical bars were prestressed to introduce an axial load level

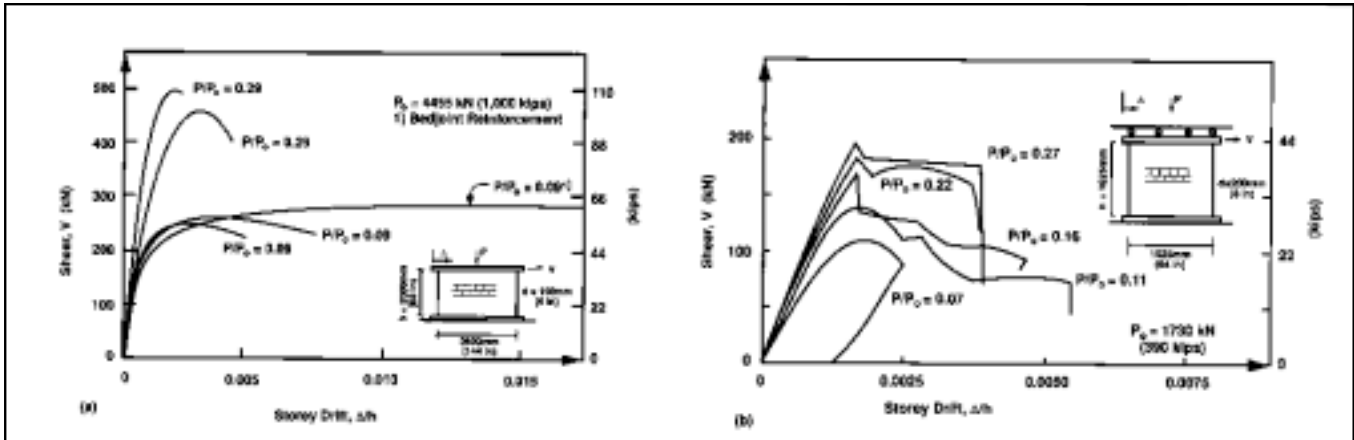


Fig. 37: Response of Plain Masonry Shear Walls a) ETH Tests, [55]; b) NBS Tests, [58]

of approximately 10% of the wall capacity. The wall was not grouted. In the second test the bars were not stressed but all cores subsequently grouted. The prestressed wall showed a stiffness approximately 35% larger than the non-prestressed wall and a considerably higher shear capacity. Although grouted, the strength of the prestressing bars could obviously not be exploited in the non-prestressed wall.

Under cyclic loading, shear walls show a less favourable load deflection characteristic than for monotonic loading, Figure 39. For deflections exceeding peak shear strength a pronounced drop in load is observed, in general, due to deterioration of masonry. The energy dissipation per loading cycle, represented by the area enclosed by the load-deflection curve is rather low for unreinforced walls, see Figure 39a, [55]. However, it can be considerably increased by using bonded reinforcement, see Figure 39b, [64].

In design, the shear strength of masonry walls is usually expressed in the form of a linear Coulomb-type failure criterion using average shear and normal stresses on bed joint and disregarding the effects of wall geometry and boundary conditions. The two parameters of the Coulomb criterion, a pseudo cohesion or bond and a friction coefficient, typically give a conservative lower bound on test results obtained from a large number of shear wall tests of varying geometry, masonry strength and testing procedure. While such an approach is extremely simple it lacks any rational basis. Recently, a general approach for the

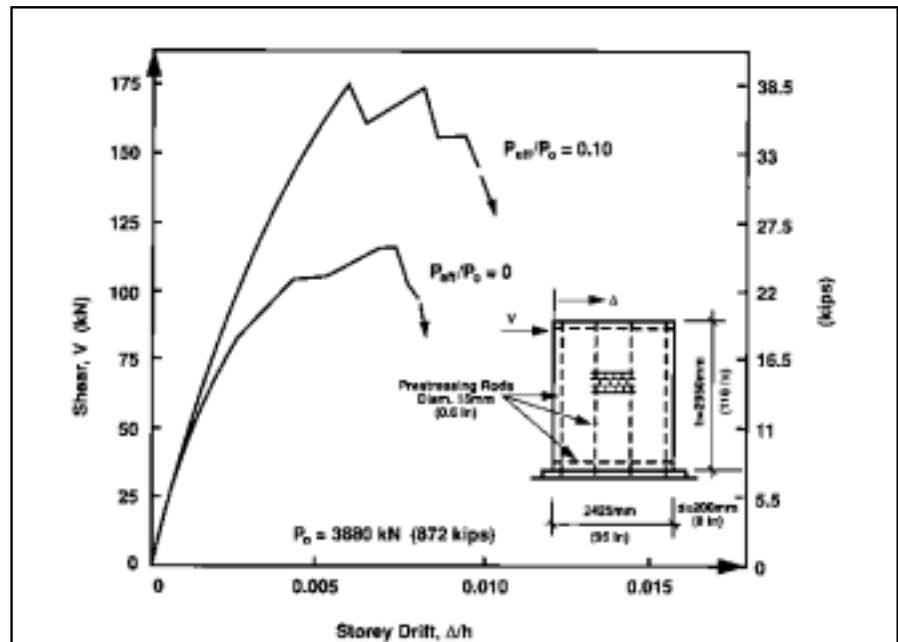


Fig. 38: Response of Reinforced and Prestressed Walls, [63]

design of masonry shear walls based on limit analysis using stress fields and biaxial failure criteria of masonry has been proposed, [25,65]. This proposal forms the basis for the design of shear walls in the new Swiss Masonry Standard SIA 177/2, [8]. That approach has been compared with test results in Figure 40. Figure 40a shows results of NBS-tests on ungrouted hollow concrete masonry walls with different length to height ratios [58], together with the theoretical interaction curves. The results of ETH-tests on hollow brick masonry walls, [55], are compared with the theoretical

interaction for different aspect ratios in Figure 40b. A fair agreement has been obtained for ratios of compressive strengths parallel and perpendicular to the bed joint of 0.6 and 0.35 for concrete and brick masonry, respectively, and a friction coefficient of 0.75. These parameters have been introduced and defined in Chapter 3.

In general, the interaction curves consist of an ascending non-linear branch which depends primarily on the friction coefficient and the wall aspect ratio. At an axial load level of approximately 20 to 25% of the axial resistance, a plateau is reached. Its level is a

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function of the aspect ratio and the ratio of compressive strength parallel/perpendicular of bed joint. At axial loads exceeding 50 to 70% of the wall resistance the shear strength reduces, depending on the strength ratio parallel/perpendicular to bed joint. However, such high axial loads are not reached in practice.

Figure 41 presents a summary of deflections of shear walls at failure measured in ETH-, NBS- and UCB-tests, [55,58,61]. Unlike the wall rotations presented in Figure 31, the storey drift at failure (defined as the ratio of top deflection and wall height) seems

to be fairly independent of the axial load level. Typically, unreinforced masonry shear walls fail at storey drifts between 0.004 and 0.006. Values up to 0.010 are obtained for reinforced walls. The wellanchored bed joint reinforcement used in the ETH-tests seems to increase the deflection capacity considerably. Storey drifts exceeding 0.030 have been measured under these conditions.

As described earlier, cracking in shear walls under applied loads initiates at loads above 50% of ultimate, in general. Due to the rather high stiffness of shear walls the onset of

cracking corresponds to very small deflections. Storey drifts as low as 0.0004 have been observed in the ETH-tests with unrestrained deflections at the top of the wall for hollow brick masonry, Figure 42a. In the UCB-tests on walls with rotational restraints at the top, values around 0.0008 and 0.0012 have been measured for grouted reinforced concrete and brick masonry, respectively. As for the deflections at failure, onset of cracking seems to be fairly independent of the axial load level.

Figure 42b illustrates the development of maximum crack widths in shear walls

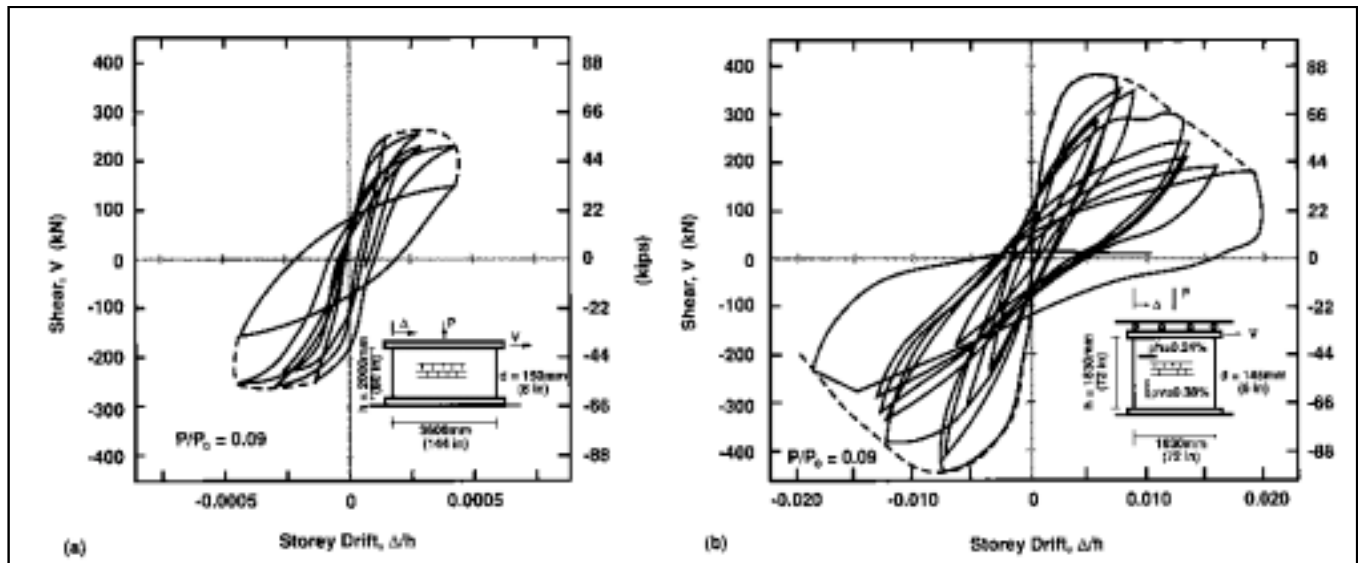


Fig. 39: Response of Shear Walls to Cyclic Loading a) Plain brick masonry shear wall, [55]; b) Reinforced concrete masonry shear wall, [64]

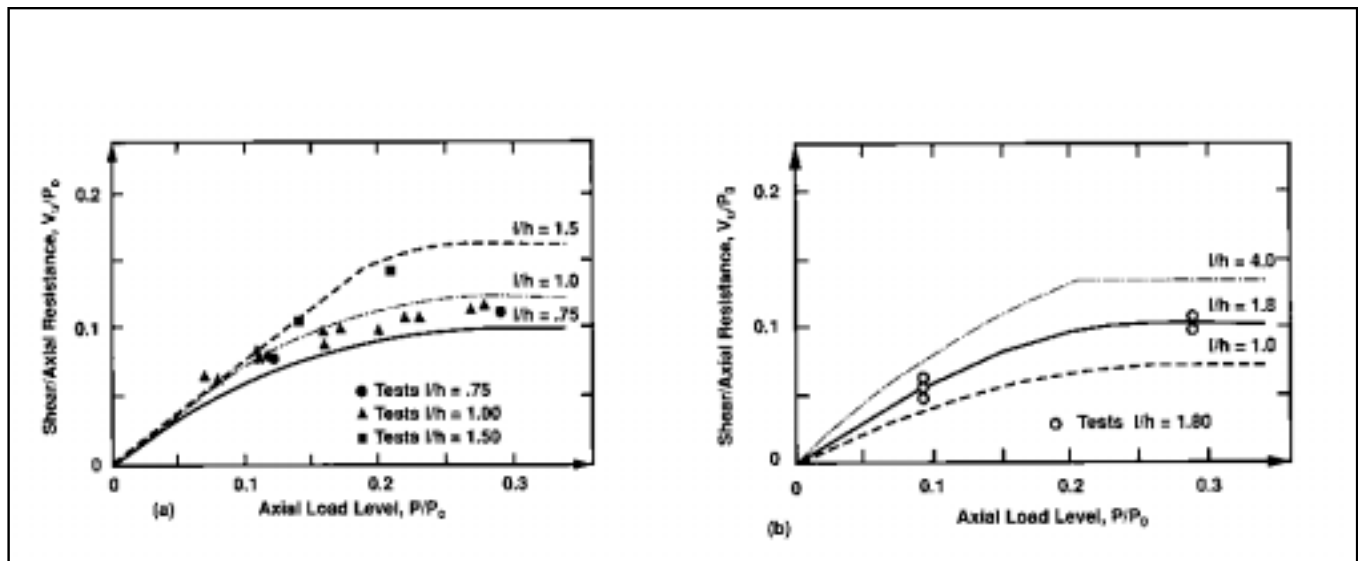


Fig. 40: Strength of Shear Walls a) Interaction of shear strength and axial load for NBS tests, [58]; b) Interaction of shear strength and axial load for ETH tests, [55]

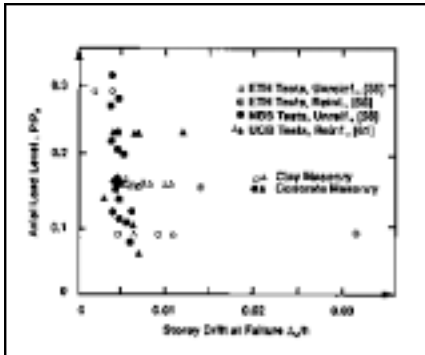


Fig. 41: Ultimate Deflection of Shear Walls, [55, 58, 61]

measured in ETH-tests. Again, the influence of axial load is negligible. However, moments applied in addition to shear on top of a wall may introduce bending cracks exceeding the width of shear cracks considerably.

Based on the above mentioned theoretical and experimental investigations, a relatively simple approach for the design of shear walls has been introduced in [8]. Figure 43 gives a summary of the main considerations. Starting from the factored axial load, shear, and moment, Figure 43a, and replacing the moment by an eccentricity of the axial load, the resultant thrust in the wall is known from

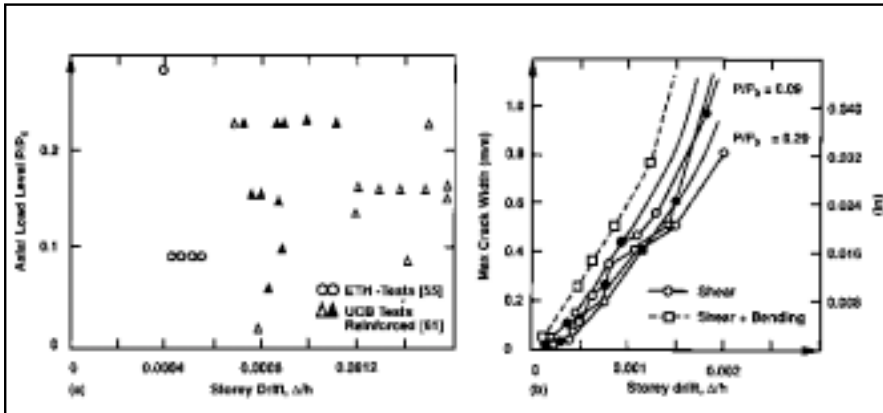


Fig. 42: Cracking Behaviour of Shear Walls
a) Influence of axial load on crack initiation, [55,61]; b) Development of crack width with storey drift, [55]

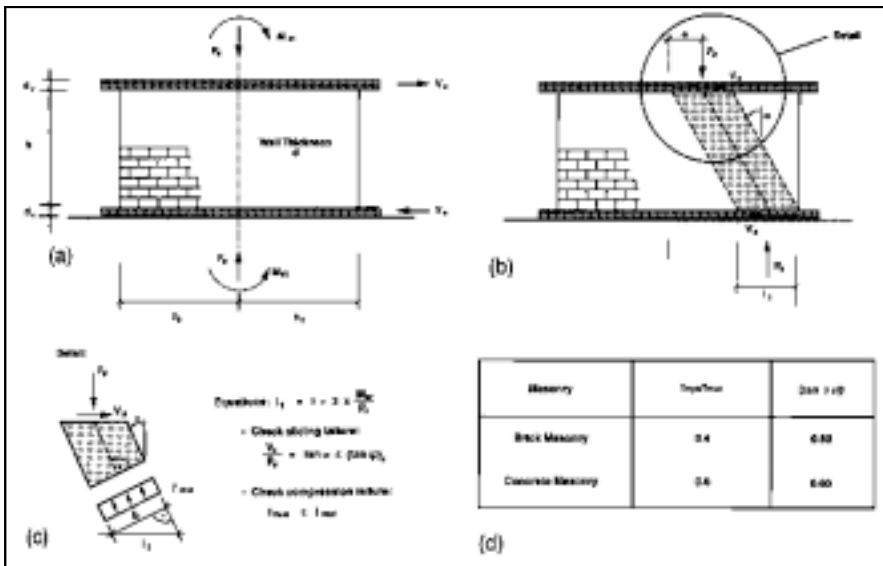


Fig. 43: SIA 177/2 Approach for Shear Walls, [8]
a) System and notations; b) Assumed stress field; c) Check of strength;
d) Masonry strength properties

simple statics, Figure 43b.

Considering a uniaxial stress field symmetrically about the line of thrust and bound completely within the wall, an equilibrium stress distribution within the wall is obtained. According to the simplified uniaxial strength introduced in Figure 22, the inclined stress field will not fail as long as the inclination, α , and the principal stress, $f_m \alpha$, do not exceed the angle of friction, ϕ , and the compressive strength, f_{my} , respectively.

The two limiting criteria have been summarized in Figure 43c. Figure 43d gives the masonry properties for design where f_{mxd} = uniaxial compressive strength for loads applied perpendicularly to bed joints reduced by a strength reduction factor of 2.0. This approach has been described in some more detail in [66].

The concept described above may easily be applied to prestressed walls if the effects of the prestressing are considered as actions, Figure 44. For each prestressing tendon or tendon group a new inclined stress field may be introduced, based on the same principles as described above, contributing to the shear strength of the wall with an amount of V_{pd} . When the stress fields of tendons and applied axial loads interfere, they should be considered as one field with the combined action of prestressing and axial load. In line with the comments made at the beginning of this chapter, the effective prestress should be reduced by a strength reduction factor.

Shear forces typically may act in either direction for wind and seismic loads and therefore, tendons will be arranged symmetrically to the wall axis, in general. Obviously, tendons close to the compression face of the shear wall cannot contribute to the shear strength because the inclination α vanishes. More important, however, they do not reduce the resistance of the inclined stress field as long as a vertical stress field can be found for those tendons with stresses not exceeding the value, $(f_{mxd} - f_{myd})$, [25]. This criterion may be used to determine a minimum edge distance of a tendon.

Of course, some major benefits for the control of flexural cracks in shear walls may be expected due to prestressing similar to those described in Section 4.2.

Other masonry members subjected to shear such as beams can be treated similarly. Figure 45 gives a proposal for

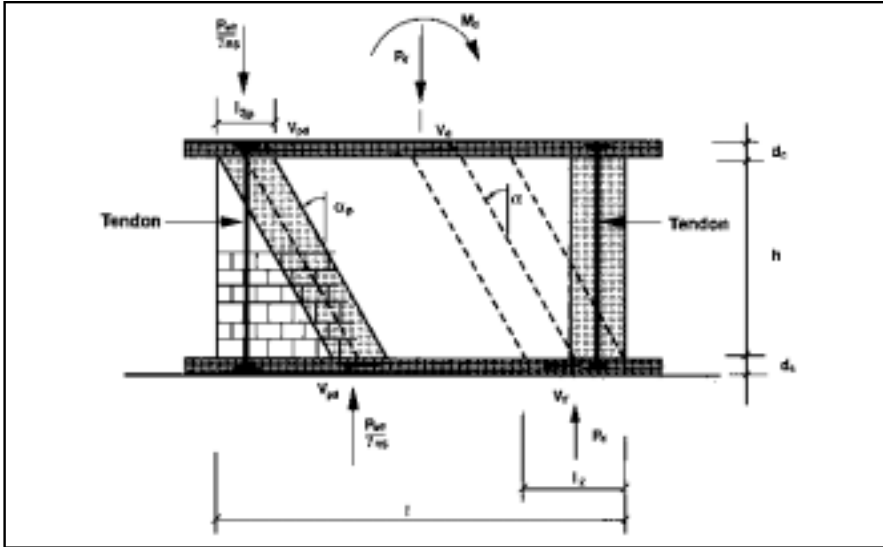


Fig. 44: Proposal for the Design of Vertically Post-Tensioned Shear Walls

Note: P_{eff} = Effective tendon force under design loads.

$\gamma_{RS} = 1.2$ - Strength reduction factor for reinforcement.

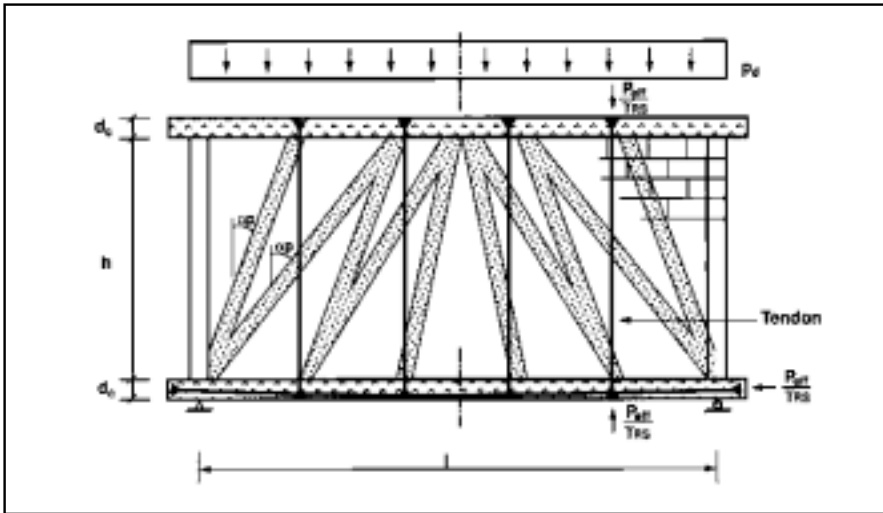


Fig. 45: Post-Tensioned Deep Beam

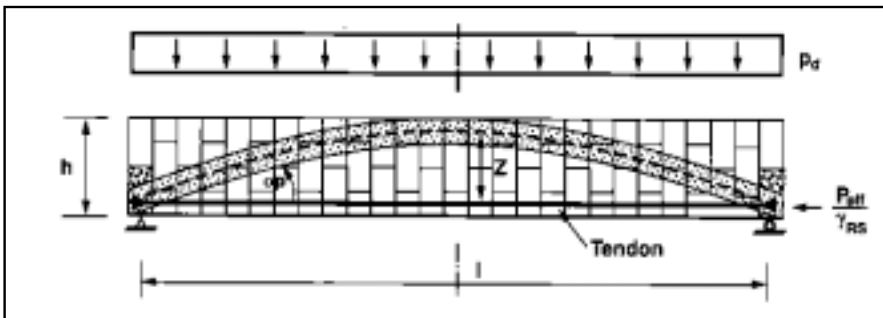


Fig. 46: Post-Tensioned Beam

the layout of tendons and for the stress fields in deep masonry beams. With a well detailed layout of tendons placed in the wall and the supporting concrete member, a hybrid girder may be formed for the transfer of gravity loads. The vertical tendons act as shear reinforcement and together with the horizontal tendons they will effectively control and limit cracking along the masonry joints.

For larger span to depth ratios the maximum stresses gradually change from the vertical to the horizontal direction. As a consequence, the masonry units should be rotated accordingly to have their strong axis parallel or nearly parallel to the maximum stress. For a beam such as the one shown in Figure 46, again the same principles can be applied as for post-tensioned shear walls.

4.5 Miscellaneous

Two problems in the design of posttensioned masonry have not yet been addressed, i.e. the introduction of the prestressing force into the masonry and the loss of prestress due to creep and shrinkage of masonry and relaxation of the prestressing steel.

Either continuous or individual concrete elements are best suited for the introduction of the prestressing forces. They serve the dual purpose of protecting the prestressing anchorage from corrosion and distributing the anchorage force to a section sufficiently large to avoid local failure in the masonry. As for any anchorage zone, bursting forces have to be resisted by an adequate reinforcement for local effects behind the anchorage and global effects in the wall. Strut and tie models such as the one shown in Figure 47a may be helpful for the design of the reinforcement for global effects in the wall. Bed joint reinforcement may be used to take the horizontal forces in the plane of the wall. However, such reinforcement should not be placed in the joint directly below the anchorage element. The design for local effects in the concrete element can follow the principles used in posttensioned concrete design. The stresses, f_m , at the transition from concrete to masonry, Figure 47b, should not exceed the masonry compressive strength adjusted by a strength reduction factor. Although some enhancement in the bearing strength under local loads has been found one may opt to neglect it and, on the

on the other hand, assume a load factor of one for the prestressing force.

A possible moment transfer from walls to floor slabs and vice versa depends on the geometry and detailing of the joint. If prestressing anchorages cannot be placed at a sufficient depth in the concrete member no moment transfer should be considered, Figure 48a. Connections which allow a transfer of moments are illustrated in Figure 48b,c,d.

Loss of prestress in prestressed masonry still seems to be a concern for many researchers, [67]. Losses exceeding 40% of the initial prestress have been assumed for the design of concrete blockwork walls prestressed with Macalloy high tensile steel bars, [68]. Based on the volume changes of masonry

presented in Table 4, the loss of prestress has been calculated for clay brick and concrete block masonry posttensioned with prestressing strands to a maximum recommended load level of 25% of the wall resistance, Table 5. An upper limit of the final loss due to creep, shrinkage and relaxation of 7% and 18% has been found for clay and concrete masonry, respectively.

The key to these relatively low losses compared with the value given in [68] lies in the use of high strength prestressing strand rather than prestressing bar. This considerably reduces the percentage loss due to creep and shrinkage of masonry.

Volume changes due to moisture may be larger than for shrinkage in clay brick masonry, Table 4. The corresponding expansion has been

mentioned in the literature to cause spalling and cracking in confined masonry walls such as infilled frames. Such problems must not be expected in unconfined but prestressed masonry walls because the corresponding tendon force increase is only in the order of 6% and 10% for strand and bar systems, respectively.

In the calculation of the effective tendon forces, losses due to wedge and nut seating have to be considered. For relatively long tendons, such losses can be compensated completely by overstressing the tendons at transfer. However, for strand systems with tendon lengths below approximately 10m (33ft.) this could only be achieved with special measures.

	Clay Brick Masonry		Concrete Block Masonry	
	Assumed Values	Associated Losses	Assumed Values	Associated Losses
Shrinkage	0	0	-0.4×10^{-3}	7%
Creep	1.0	4%	2.0	8%
Relaxation	3%	3%	3%	3%
Total	-	7%	-	18%

Table 5: Loss of Prestress

Note: Initial stress in masonry: 2 MPa (280 psi)
Initial stress in strand: 1250 MPa (175 ksi)
Modulus of masonry: 8 GPa (1,120 ksi)
Modulus of strand: 195 GPa (27,300 ksi)

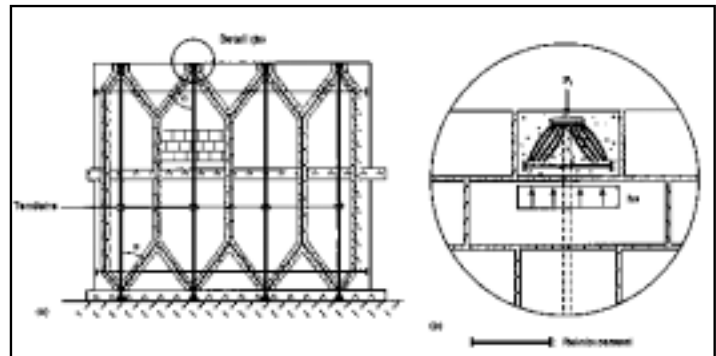


Fig. 47: Introduction of Prestressing Forces into Masonry Walls
(a) Global effects in wall; (b) Local effects near anchorage

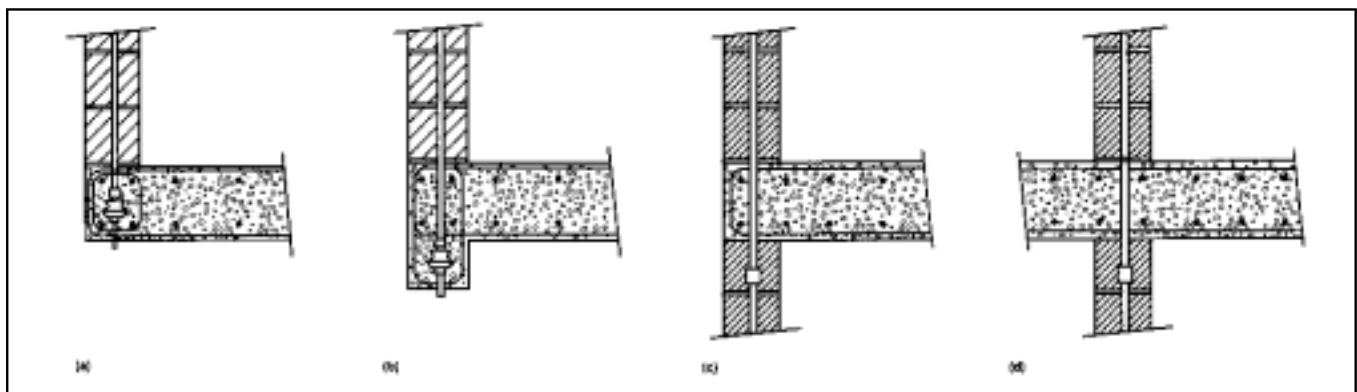


Fig. 48: Wall-Slab Connections

a) Without moment transfer; b), c), d) With moment transfer

5. The VSL Post-Tensioning System for Masonry and Its First Applications

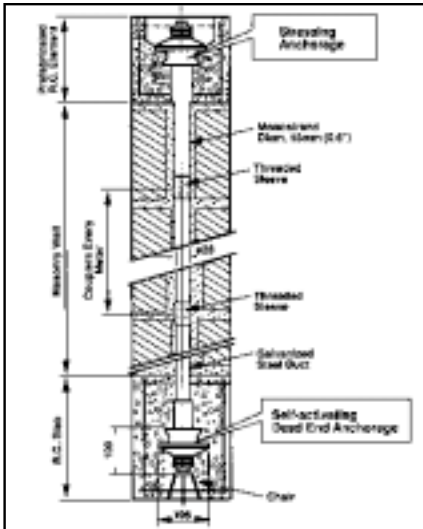


Fig. 49: VSL System for Post-Tensioned Masonry

5.1 VSL Post-Tensioning System for Masonry

The VSL System for masonry is an unbonded system. It utilizes monostrands, i.e. high strength steel strands that are greased and coated with extruded plastic for maximum corrosion protection. A solid and durable duct around the monostrand tendon provides a third layer of protection. The system is easy to use in the field, eliminating the multiple couplings of prestressing bars and providing a highly superior tension capacity per weight of prestressing steel.

A typical VSL masonry tendon is illustrated in Figure 49. It consists of a self-activating dead-end anchorage, a stressing anchorage placed in a prefabricated concrete element, the 15mm (0.6") diameter monostrand, and a

galvanized steel or plastic duct.

Construction of a wall post-tensioned with VSL masonry tendons is illustrated in Figure 50. The dead-end anchorage at the lower end of the tendon is cast inside an in-situ concrete bearing pad. After casting, wall construction begins and duct segments are threaded to the anchorage or previously placed duct segments, according to the progress of construction. This procedure allows the bricks to be laid easily because they need only be threaded a small number of lifts over the duct segment. When the final wall height is reached, the final duct segment is cut to the required length and a prefabricated concrete element containing the stressing anchorage and a sleeve for the duct is placed on top of the wall.

After the masonry reaches the specified strength, prestressing may com

Fig. 50: Practical Handling of the VSL System



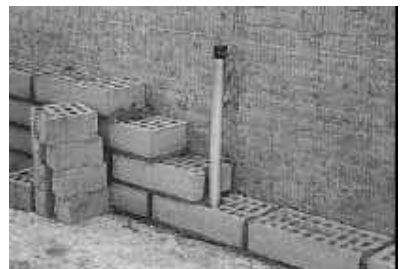
a) Placing of dead-end anchorages



c) Adding new duct segment



e) Introduction of monostrand



b) Start of wall construction



d) Placement of precast element



f) Stressing monostrand

mence. First, the monostrands are fed through the stressing anchorage and duct into the self-activating dead-end anchorage. The tendons are subsequently stressed to a maximum of 75% of their tensile strength.

The VSL System includes accessories such as pre-assembled chairs at the deadend anchorage and caps on top of each duct segment to ease the placing of the anchorage in the formwork of the concrete bearing pad and avoid dirt falling down the duct, respectively.

Although the application of the VSL masonry tendons is not restricted by masonry compressive strength, its benefits can be best exploited if a masonry strength of at least 8 MPa (1,200 psi) based on gross cross section is specified and a cement mortar is used.

The common application of the system with the tendons running in the centre of a single leaf wall is greatly facilitated if masonry units with specially formed cores are used. Therefore, special masonry units are available in Switzerland where the complete system including units is marketed jointly by VSL International, Ltd. and Zurcher Ziegeleien a supplier of building materials and brick manufacturer.

5.2 Recent Applications

The VSL Post-Tensioned Masonry System has been successfully employed for two recent applications in Switzerland. The first was for two brick cavity walls of a Kindergarten in Zurich, where the interior leaves were post-tensioned to provide the required strength to resist out-of-plane lateral wind loads, Figure 51. The interior clay brick leaves are 140mm thick and up to 4m high, with large window openings. The walls are laterally supported on top by a steel frame in the roof. Five monostrand tendons were used for each wall. The dead-end anchorages were placed in a 250mm thick floor slab. The stressing anchorages were placed in prefabricated concrete elements whose height had to be kept to an absolute minimum of 130mm to avoid visibility in the interior of the room. Common truss-type bed joint reinforcement was placed below the elements to take bursting forces. Each tendon was stressed to 180kN, i.e. 70% of ultimate.

The second project, a 250mm thick fire

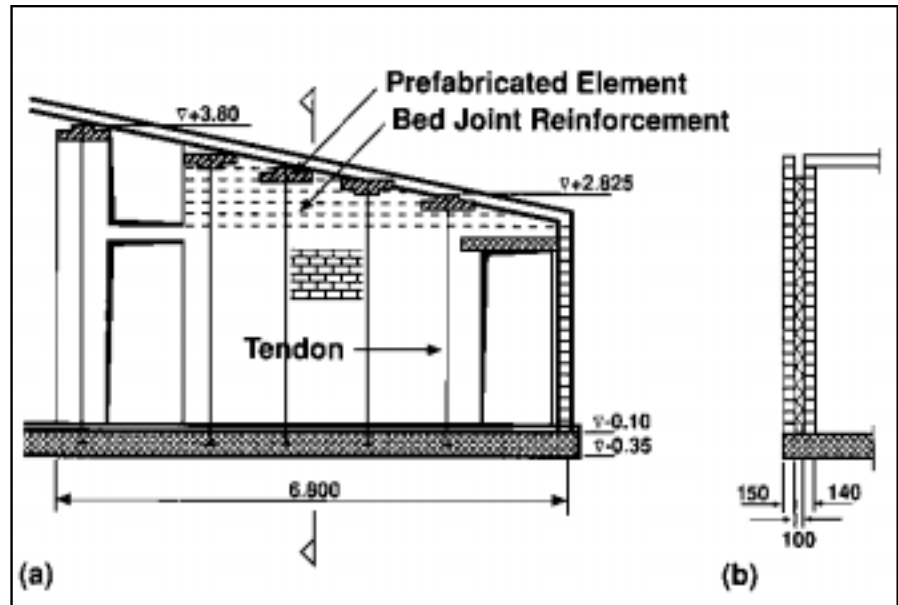


Fig. 51: Kindergarten, Zurich

a) Wall dimensions

and tendon layout

b) Wall under construction

Note: Dimensions in mm (1mm=0.04in)
Elevations in m (1m=3.3ft)

Note:

Owner: Hochbauinspektorat, City of Zurich, Zurich

Structural Eng.: A. Urech, Zurich

Architect: U. Rufenacht, Zurich

Contractor: Schwager AG, Zurich

Bricks: Zurcher Ziegeleien, Zurich

Post-Tensioning: VSL International AG, Lyssach

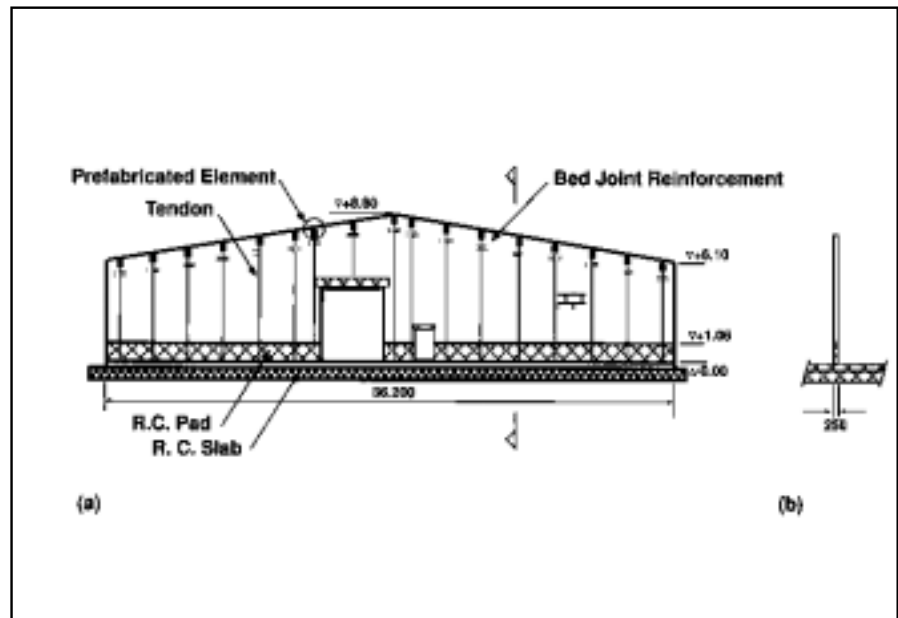


Fig. 52: Factory, Regensdorf

a) Wall dimensions

and tendon layout

b) Wall under construction

Note: Dimensions in mm (1 mm=0.04in)
Elevations in m (1m=3.3ft)

Note:

Owner: Biber Papier AG Regensdorf

Structural Eng.: A. Urech, Zurich

Architect: AIV Architekten - Ingenieure -
Verwaltungen

A G, Zurich

Contractor: Fietz & Leuthold AG, Wallisellen

Bricks: Hard AG, Volketswil

Post-Tensioning: VSL International AG, Lyssach

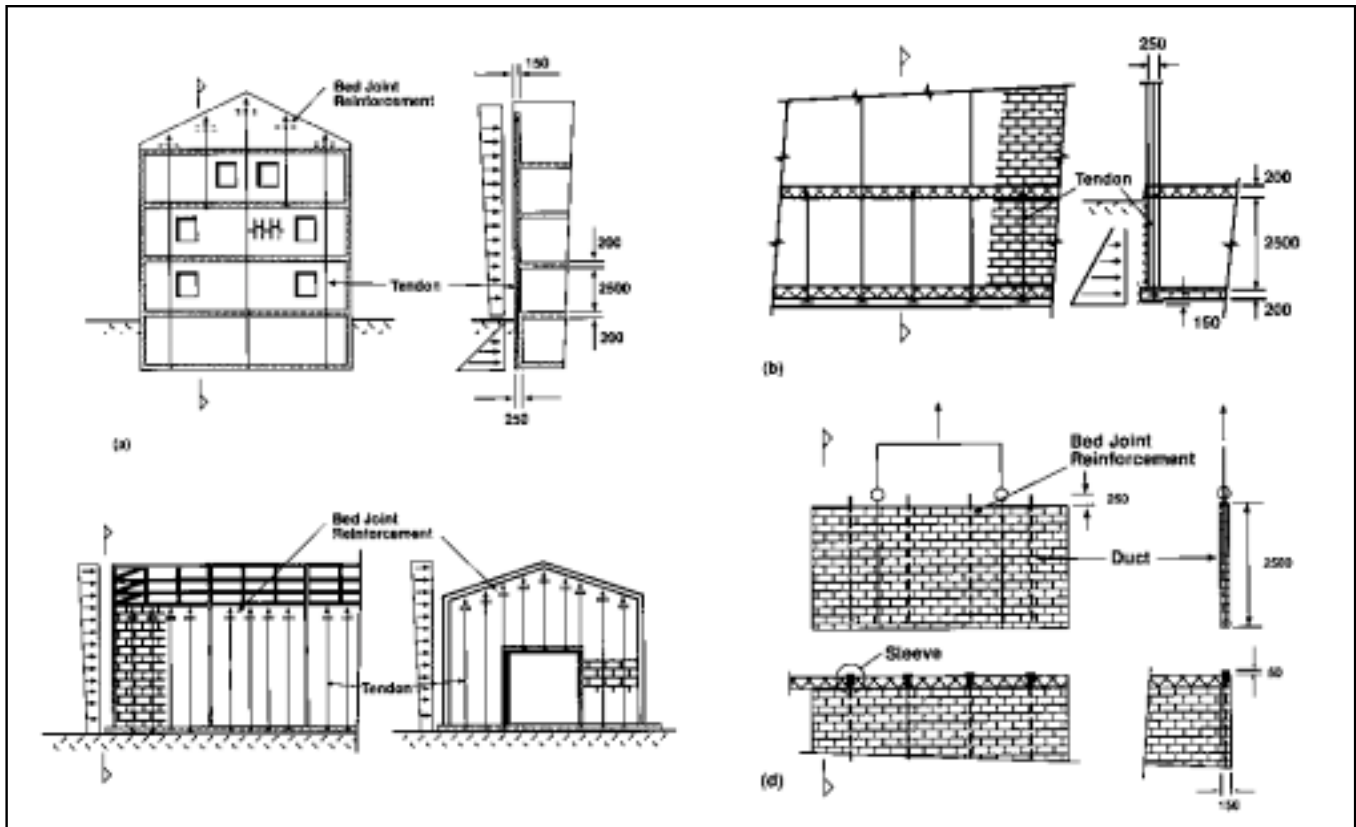


Fig. 53: Typical Future Applications of Post-Tensioned Masonry for New Structures
a) Residential building; b) Basement wall; c) Infilled frames; d) Prefabricated walls
Note: Dimensions in mm (1mm=0.04in)

proof wall in a factory near Zurich, 36m long and up to 8.8m high, was posttensioned with a total of 17 tendons, Figure 52. The wall was designed to withstand a wind velocity of 21 ms⁻¹ as a cantilever. Calcium silicate bricks were utilized in running bond with headers. The dead-end anchorages were placed in a 1 m high cast-in-place concrete pad beneath the masonry. The concrete pad was connected to an already existing floor slab by anchors. Stressing anchorages were placed in prefabricated concrete cubes with a side length of 250mm. Below each anchorage two layers of bed joint reinforcement were placed.

A minimum wedge seating of 4 mm has been considered in both projects. Due to the minimum dimensions of the precast elements in the first project, preliminary tests were carried out in the VSL laboratory to verify the safety for the introduction of the prestressing force into the masonry under conditions similar to the actual site. In the second project, the dimensions of the elements were chosen such as to limit the

bearing stresses under a maximum jacking force of 200kN (75% of ultimate) to 40% of the uniaxial masonry strength. This value was considered to provide a sufficiently large safety margin against local failure even for a probable early stressing at seven days. Therefore, no special tests were specified prior to stressing of the tendons.

5.3 Future Applications

Post-tensioning offers a new potential to innovative engineers and architects for the revival of masonry as a structural material. Plenty of types of constructions, by far not limited to those presented in Figure 3 and 53, are feasible at costs competitive with reinforced concrete structures, [4,69].

Figure 53 illustrates a selection of some of the most straight forward applications of post-tensioned masonry. In residential and office buildings primarily walls in the upper storeys, would benefit from post-tensioning both for strength and in-service performance,

Figure 53a. At lower storeys, gravity loads will reduce the required amount of post-tensioning, in general. Basement walls, subjected to out-of-plane lateral earth pressure, are another application in residential buildings, Figure 53b. Post-tensioned masonry may be used to infill large frames in industrial buildings, Figure 53c. Apart from cast-in-place construction, posttensioning offers benefits to prefabricated walls during transport and erection and could be used to effectively connect the walls to cast-in-place elements, Figure 53d. Tilt-up masonry walls and sound walls seem to be other potential applications. Possible applications of posttensioning in load bearing deep wall beams and beams have been mentioned in Section 4.4.

Lots of masonry constructions were built at a time when people were not yet as concerned as today about the strength for lateral wind and seismic loads. Such constructions, either individual walls or entire buildings, can be strengthened by

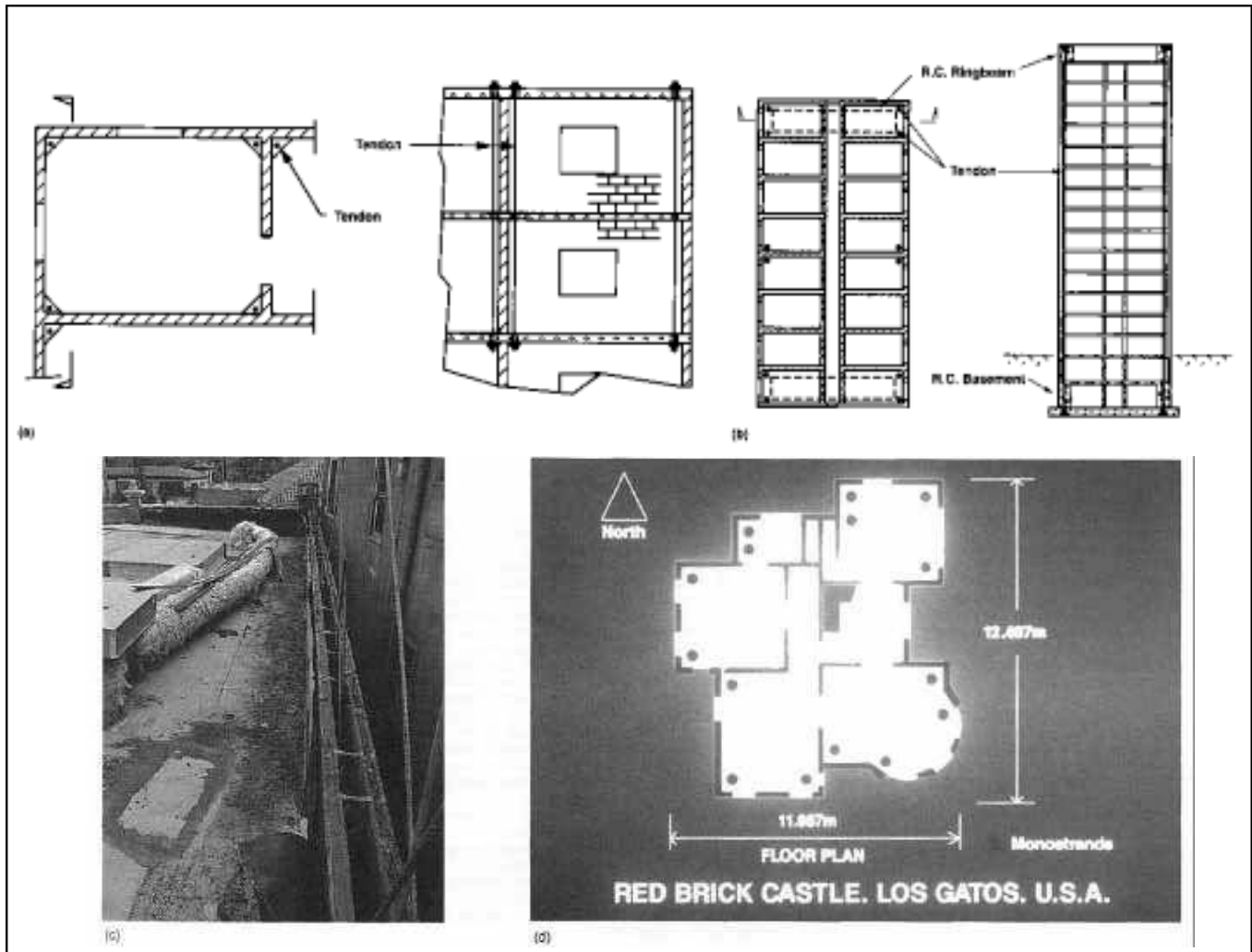


Fig. 54: Strengthening of Existing Masonry Structures

a) Individual walls, plan and section; b) Entire buildings, plan and section; c) Strengthening of a cavity wall with prestressing, Los Gatos, California; d) Proposal for strengthening of Brick Castle, Los Gatos, California

post-tensioning to comply with the most recent requirements, Figure 54. While for individual walls, Figure 54a, the tendons described in this report could be used, buildings would require external posttensioning tendons, Figure 54b, [39]. Techniques similar to those described in this report have been used by VSL Corporation for retrofitting masonry buildings damaged in the October 1989 Loma Prieta Earthquake, Figures 54 c and d. For such applications, the axial resistance of the walls has to be carefully evaluated and the introduction of the tendon forces needs special details such as concrete blocks,

spreader beams or walls.

Presently, the general Post Office in Sydney, a more than one hundred year old sandstone masonry building, is undergoing a massive restoration both inside and out. As part of this restoration, the GPO Tower will be strengthened with four vertical post-tensioning tendons, 19 diameter 0.5" strands each, and a number of horizontal prestressing bars diameter 35mm at floor levels, Figure 55. The vertical tendons will be placed in holes diameter 100mm drilled from the top through the sandstone columns at the corner of the tower. Special steel chairs will be used to anchor the tendons and

spread the anchorage forces of 1,771 kN (400 kips). The anchorages of the unbonded tendons allow for monitoring and adjustment of the tendon forces to compensate volume changes of the sandstone, if necessary. The entire restoration is expected to take five years and installation and stressing of the tendons is scheduled for 1990.

Both for new and strengthening of existing constructions the same final comment applies: The hardware is ready, applications are only limited by the imagination of the experts involved.

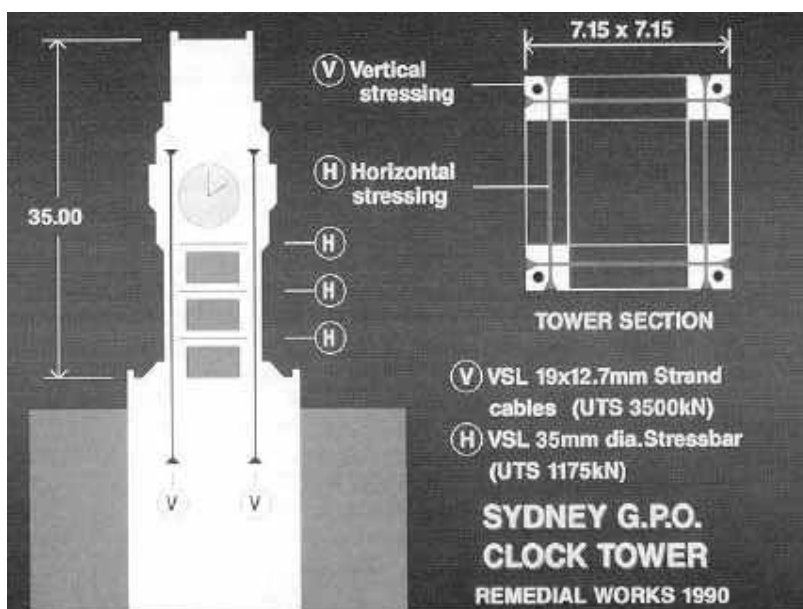
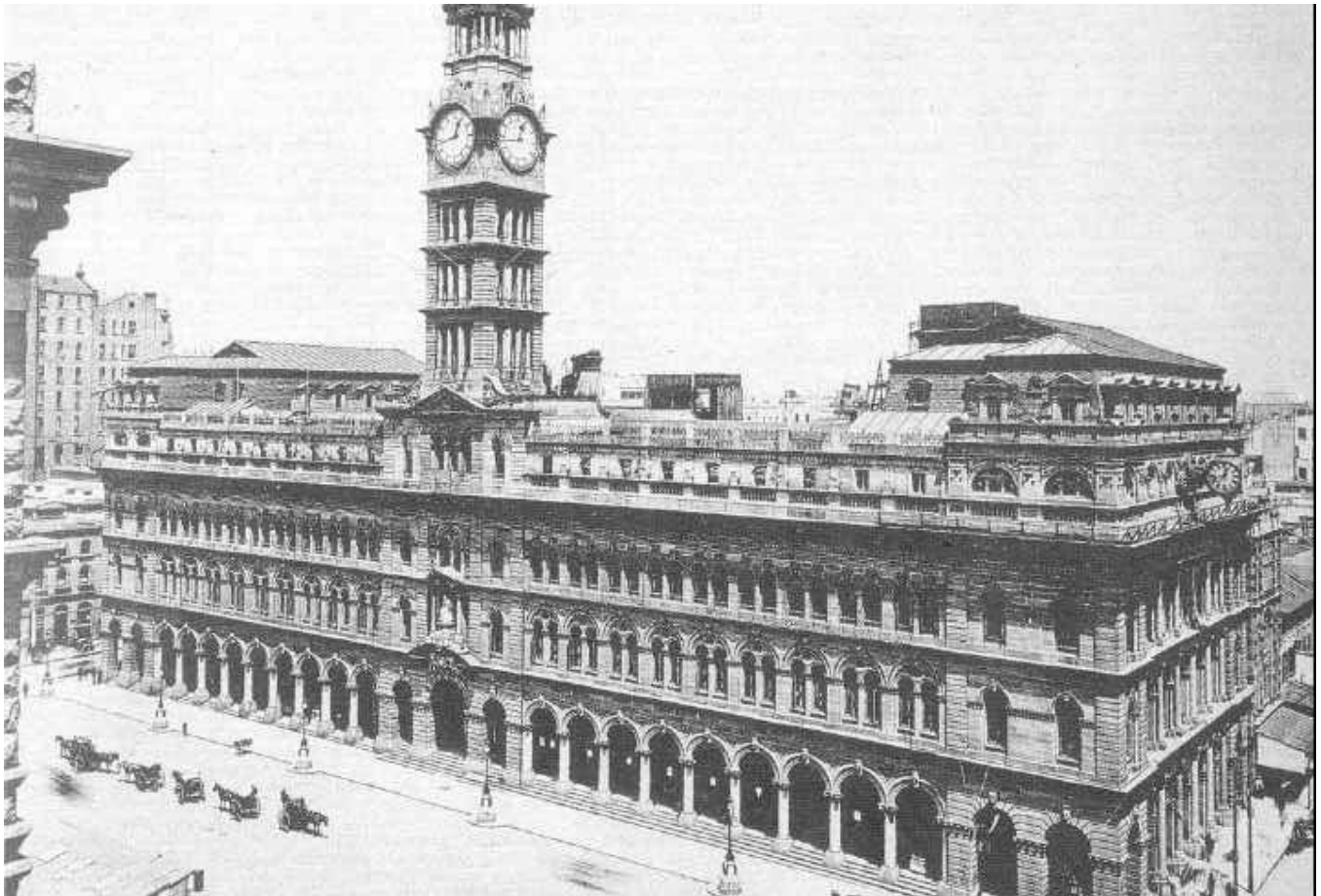


Fig. 55: Strengthening of GPO Tower, Sydney
a) General Post Office Building.
b) Tendon layout in tower, courtesy of McBean & Crisp, Pty, Ltd.

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