POST-TENSIONED MASONRY STRUCTURES

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

2

VSL REPORT SERIES

PUBLISHED BY
VSL INTERNATIONAL LTD.
Berne, Switzerland

Contents

Author

H.R. Ganz, Dr. sc. techn., Civil Engineer ETH

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.

Prof. Dr. Bruno Thurlimann
Swiss Federal Institute of Technology
Zurich, Switzerland

1. Introduction

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

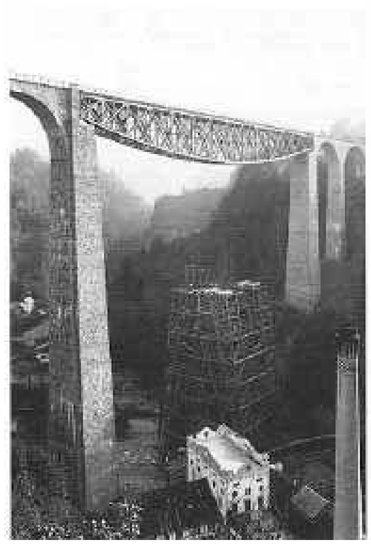
A brief historical review, [1], may help to understand why such reactions might be short-sighted or even wrong. Brick actually is the oldest man-made building material, invented almost ten thousand years ago. Its simplicity, strength and durability led to extensive use and gave it a dominant place in 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 twentiethcentury, 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.

Figure 1 Old Masonry Constructions

a) Tower in Siena, Italy

b) Arch in Ctesiphon, Iraq



c) Railway Viaduct, Switzerland

d) Monadnock Building, Chicago, USA

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

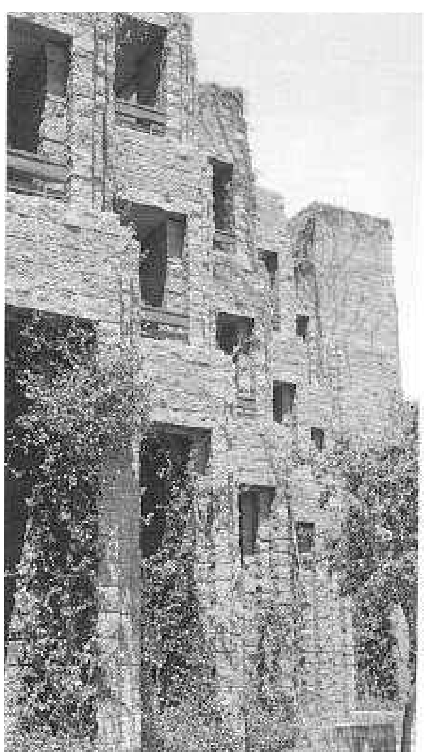
The major advantages of ancient and modern masonry have always been the overall availability of the raw materials, the easy and economical construction, and the natural beauty and durability. Thus, again, why post-tensioning masonry?

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 constrution, 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

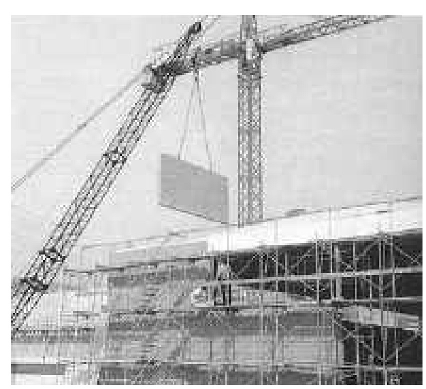


a) Hotel

b) Residential

c) High-Rise

d) Commuty Hall, Photograph courtesy of Consulting Engineers



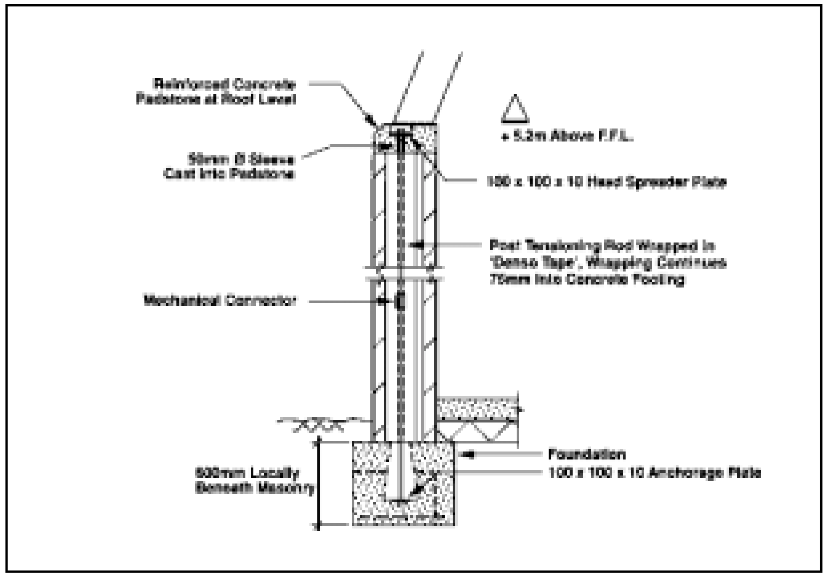
e) Prefabrication

Figure 3: Post-Tensioned Masonry Constructions

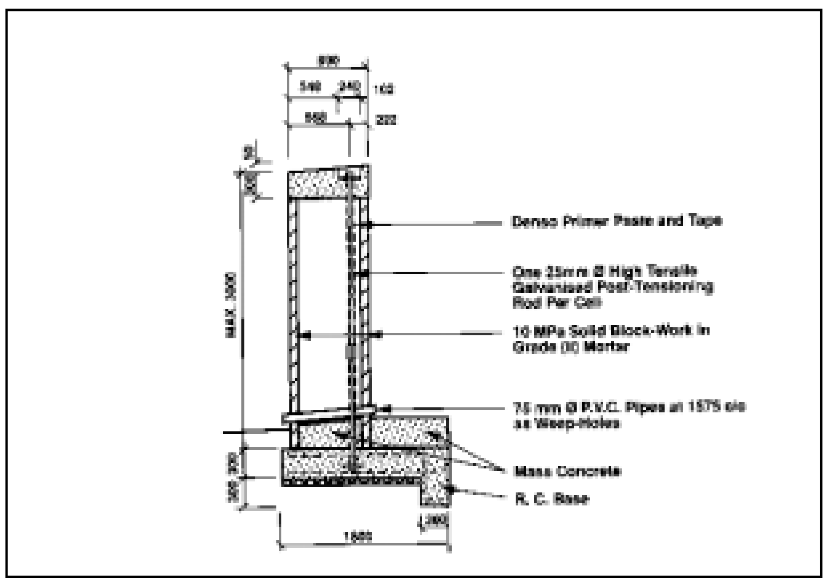
a)Salvation Army Hall, [2], Photograph courtesy of Curtins Consulting Engineers

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 M Pa.

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

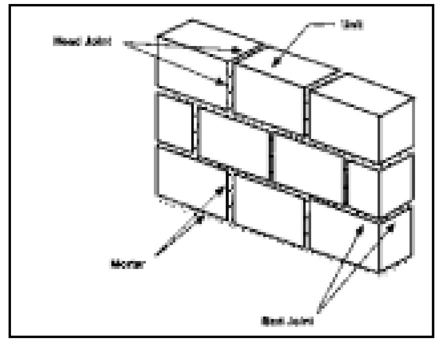


Figure 4: Components of Plain Masonry

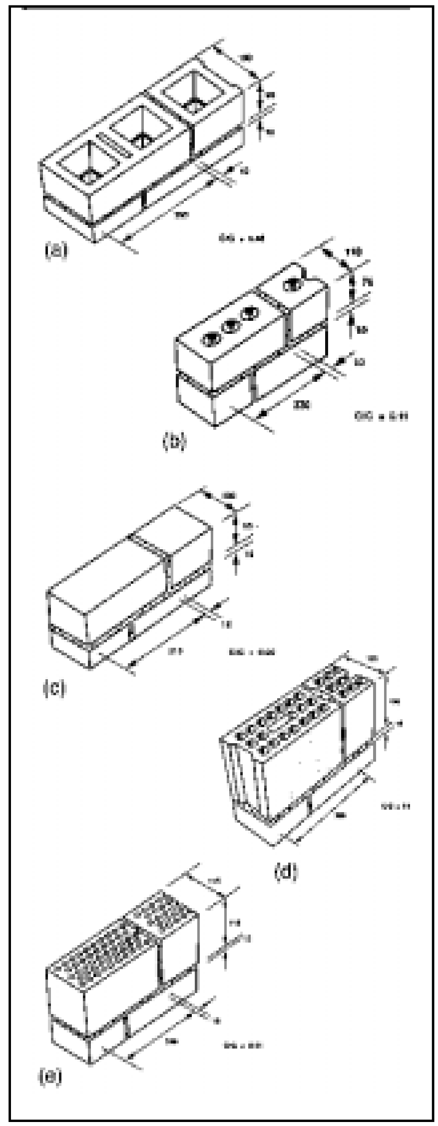


Figure 5: Typical Clay Bricks
a) Canada; b) Australia;
c) Great Britain; d) Switzerland;
e) Germany FR
Note: C/G = Core area to gross cross sectional area

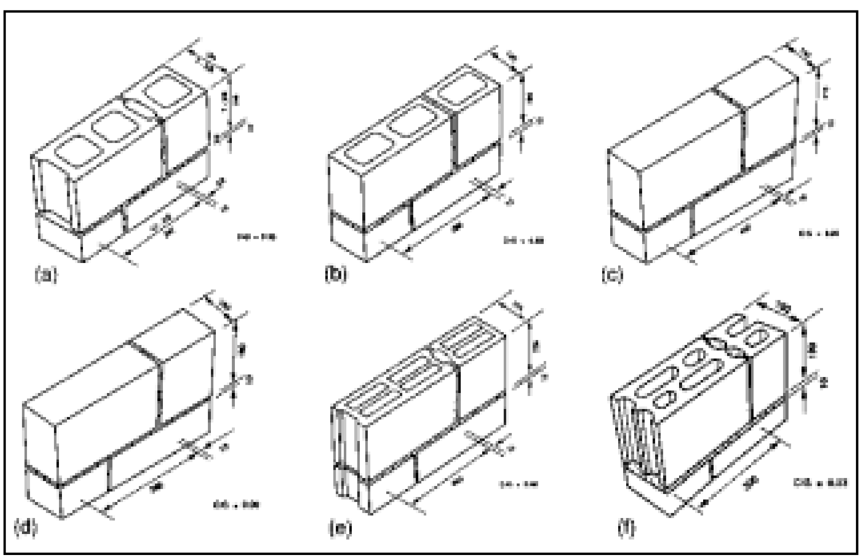


Figure 6: Typical Concrete Bricks and Blocks
a) United States;
b) Australia;
c) Great Britain / Australia;
d) Great BritainIAustralia
e) Germany FR;
f) Switzerland
Note: C/G = Core area to gross cross sectional area

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

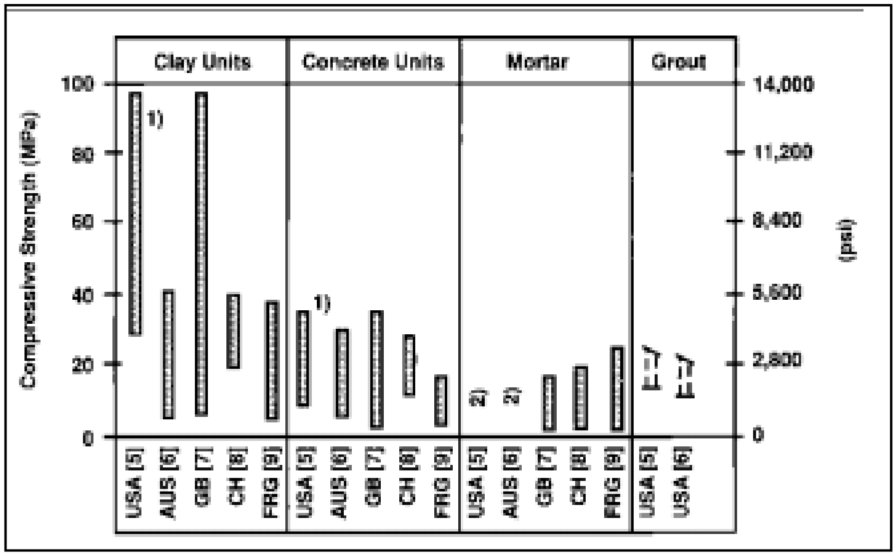


Figure 7: Strength Requirements for Units and Mortar, [5, 6, 7, 8, 9]
Note: 1) Based on net area, 50 to 75°l of gross; 2) Not specified

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 ≥ 300 kg/ m^3
3) Cement content (300-450) kg / m^3
4) Lime content 250 kg/ rn3, cement content 100 kg/ m^3
1 MPa = 140 psi

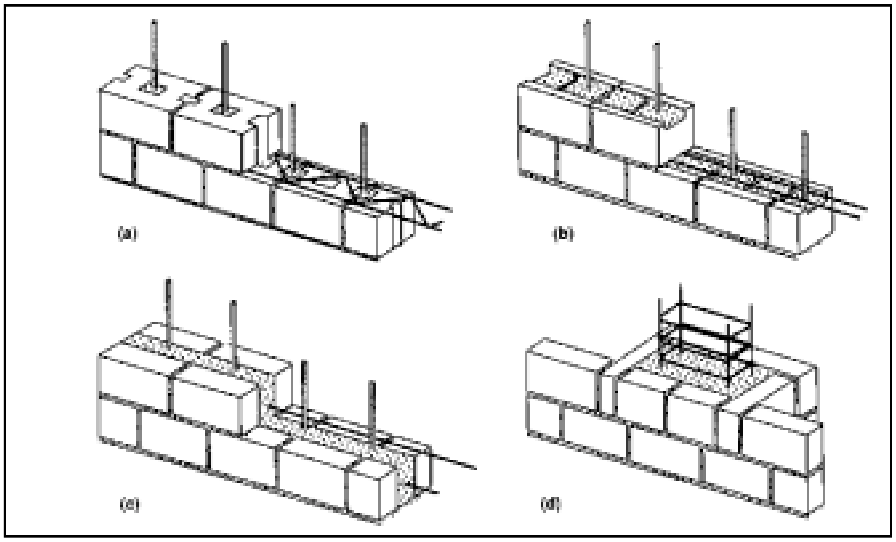


Figure 8: Typical Layout of Horizontal and Vertical Reinforcement in Walls
a) In cores and bed joint mortar; b) In cores and bed joint grooves;
c) In grouted cavities; d) In pockets

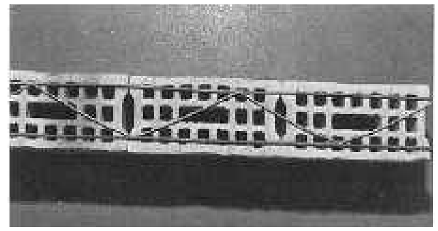


Fig. 9: Typical Bed Joint Reinforcement

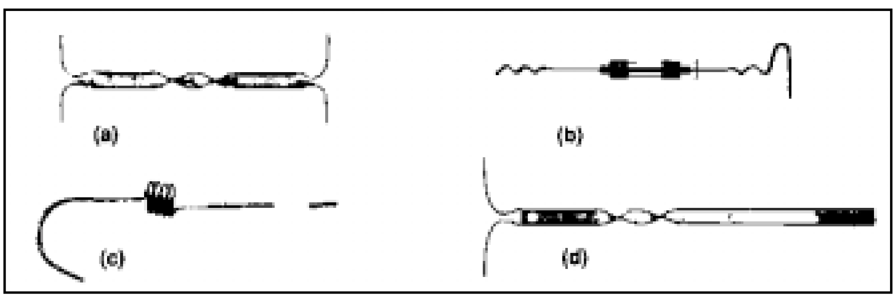


Figure 10: Ties
a), b): Both ends embedded in mortar;
c), d): One end embedded in mortar, other end thread and sleeve

Properties		Bar ϕ 36 mm hot rolled, cold-worked and tempered, ribbed	Strand ϕ13mm 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	210	250
	min	210	205
Min. diameter of curvature at max. allowable stress of 90% of yield	m	6.83	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.

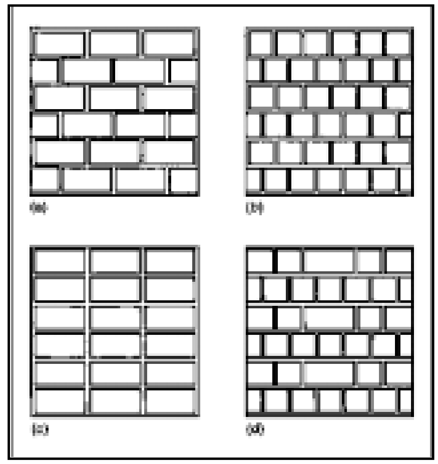


Figure 12: Typical Masonry Bonds
a) Running bond with stretchers;
b) Running bond with headers;
c) Stack bond;
d) Dutch bond

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.

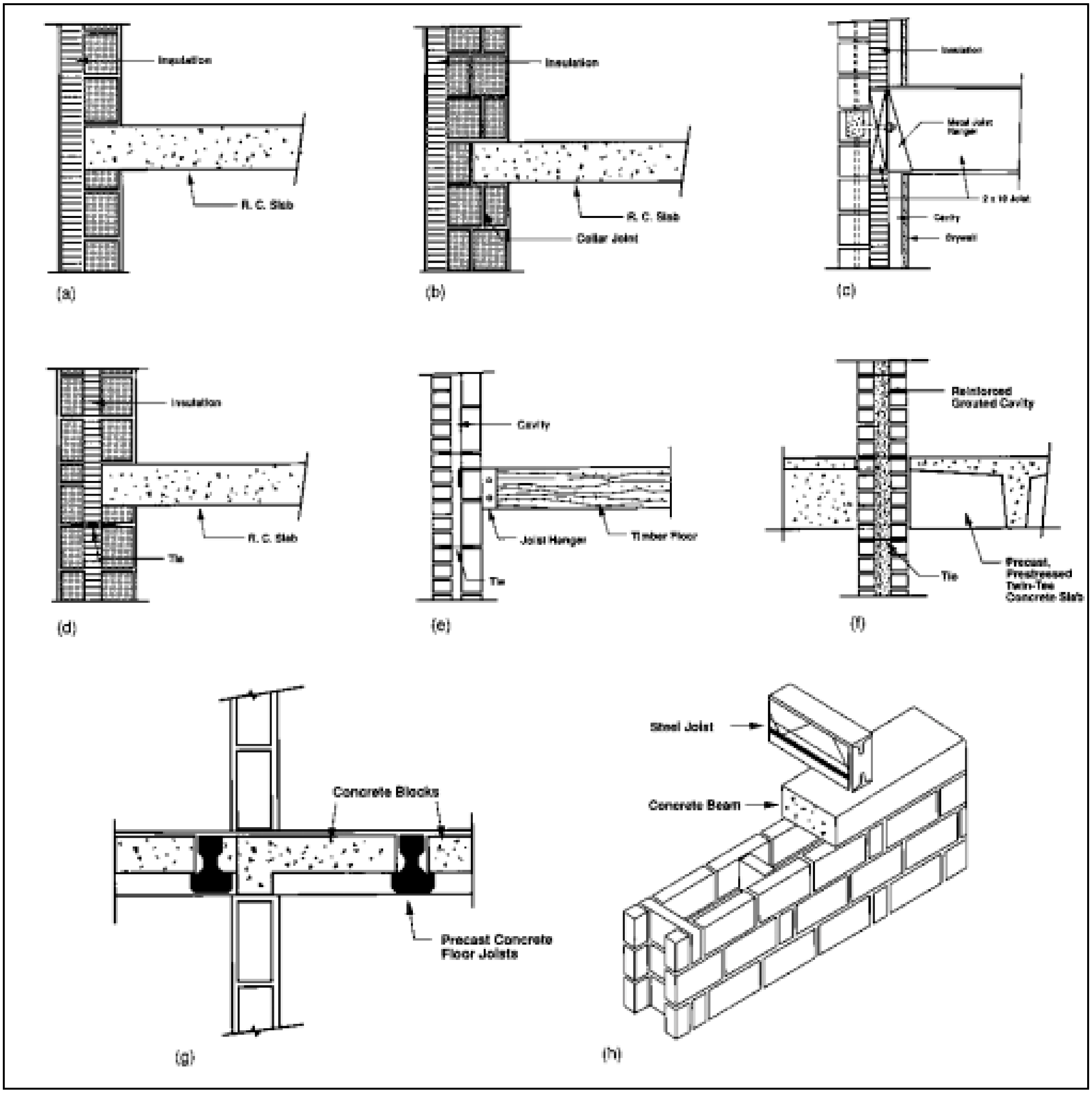


Figure 11: Typical Wall Sections and Connections to Floors / Roofs

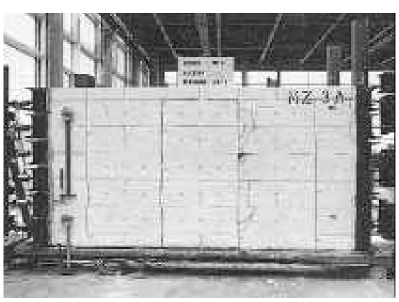
a) Single-leaf wall, concrete floor
b) Single-leaf bonded wall, concrete floor
c) Single-leaf wall, floor joists
d) Cavity wall, in-situ concrete floor
e) Cavity wall, timber floor
f) Cavity wall, precast concrete floor
g) Single-leaf wall, concrete block floor
h) Diaphragm wall, steel joist roof

3. Properties of Masonry

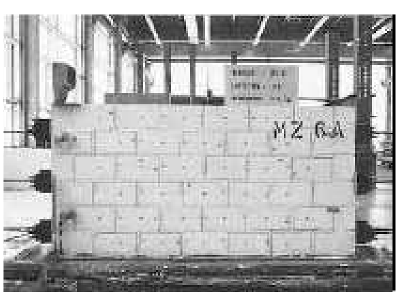
Fig. 14: Typical Failure Modes of Masonry

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

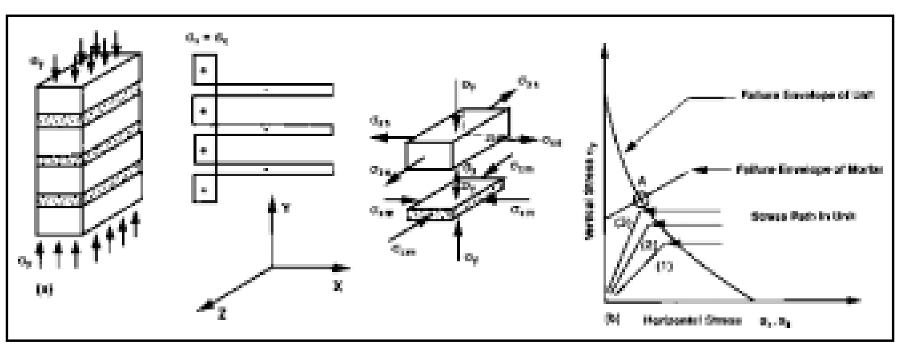


Fig. 13: Interaction of Units and Mortar Joints
a) Prism under uniaxial compression and stresses in unit and mortar;
b) Failure criterion for masonry

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 stressstrain 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

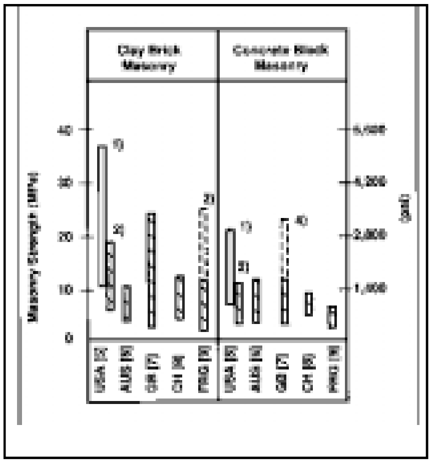


Fig. 16: Range of Masonry Compressive Strength according to National Standards, [5, 6, 7, 8, 9]
Note:
I) Based on net area;
2) Estimate for gross area based on netlgross = 0.5;
3) Special masonry;
4) For solid blocks only

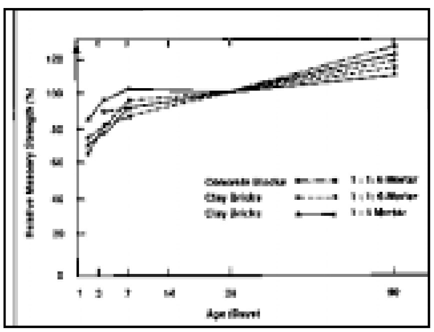
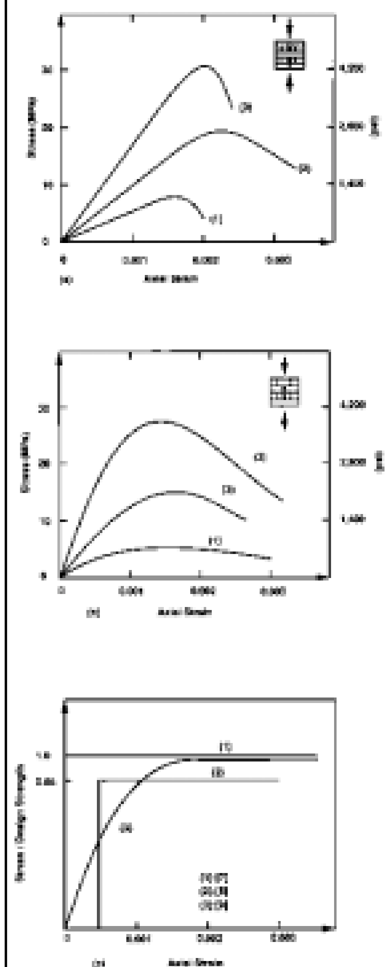


Figure 17: Effect of Age on Masonry Compressive Strength, [17]



Strength of components (MPa)

		Unit	Mortar	Grout	Ref.
(a)	(1)	32.0	12.1	-	[11]
	(2)	60.6	15.2	-	[14]
	(3)	107.0	21.5	-	[11]
(b)	(1)	11.8	5.8	-	[11]
	(2)	15.6	18.8	-	[13]
	(3)	21.0	11.4	27.8	[12]

all stresses/strengths based on gross area
1 MPa = 140 psi

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]

COUNTRY/ STANDARD	CLAY BRICK MASONRY Ex/fmx	CLAY BRICK MASONRY G/Ex	CONCRETE BLOCK MASONRY Ex/fmx	CONCRETE BLOCK MASONRY 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

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.

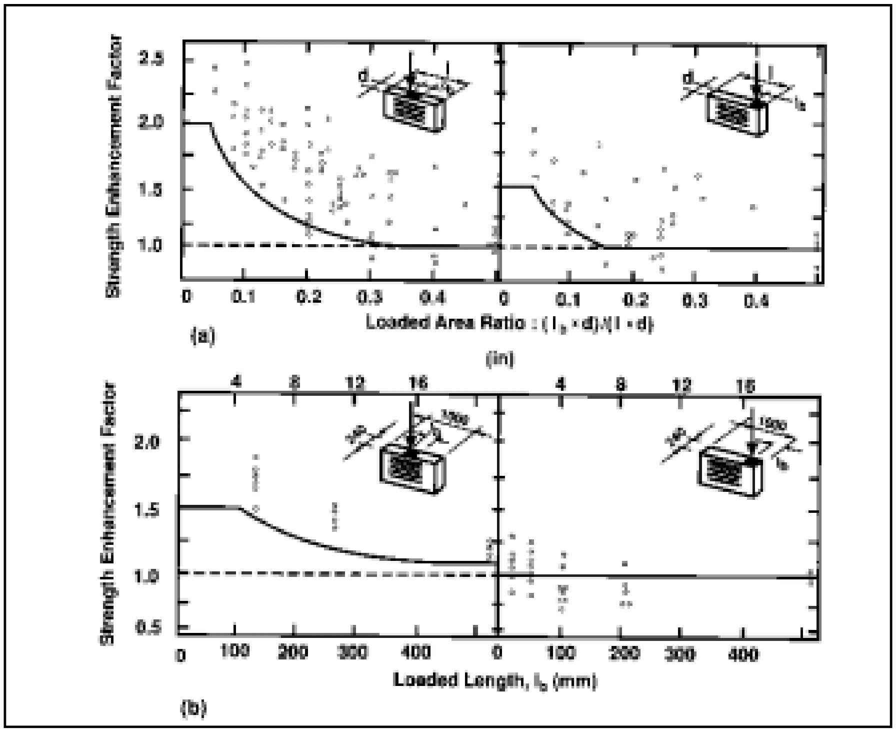


Figure 18: Bearing Strength of Masonry
a) Masonry with solid units, [*18*]*; b) Masonry with hollow units,*[*19*]

General uniaxial loading has been 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 θ=0° represents the uniaxial test described in Section 3. 2 with a compressive strength called fmx in the following. Prisms with loads applied under θ=90°, i.e. parallel to the bed joints, show lower strengths than fmx in general. In particular for hollow clay brick masonry, strengths as low as 0.40 fmx are obtained. For relatively small inclinations, say θ < 40°, splitting types of failure are observed with strengths as low as 0.40 to 0.50 fmx for clay and 0.60 fmx for concrete masonry. Except for grouted concrete masonry, even lower strengths are obtained for orientations 45° < θ < 75° when sliding failure along the joints is governing. The strength may drop as low as 0.10 to 0.15 fmx for clay and 0.35 fmx 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

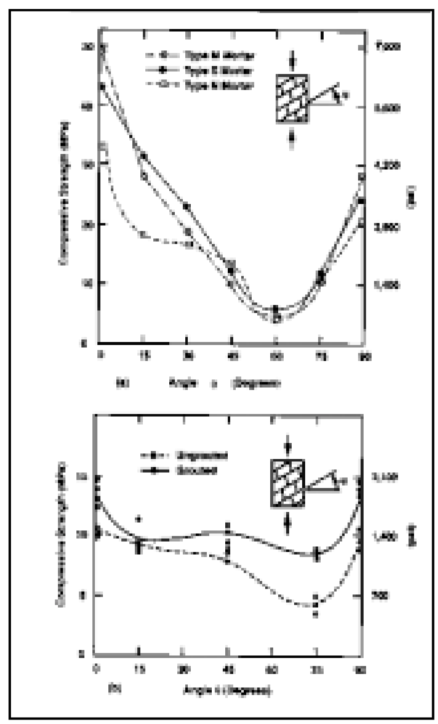


Fig. 19: Uniaxial Compressive Strength of Masonry
a) Clay brick masonry, [*20*]*;*
b) Concrete block masonry, [*21*]

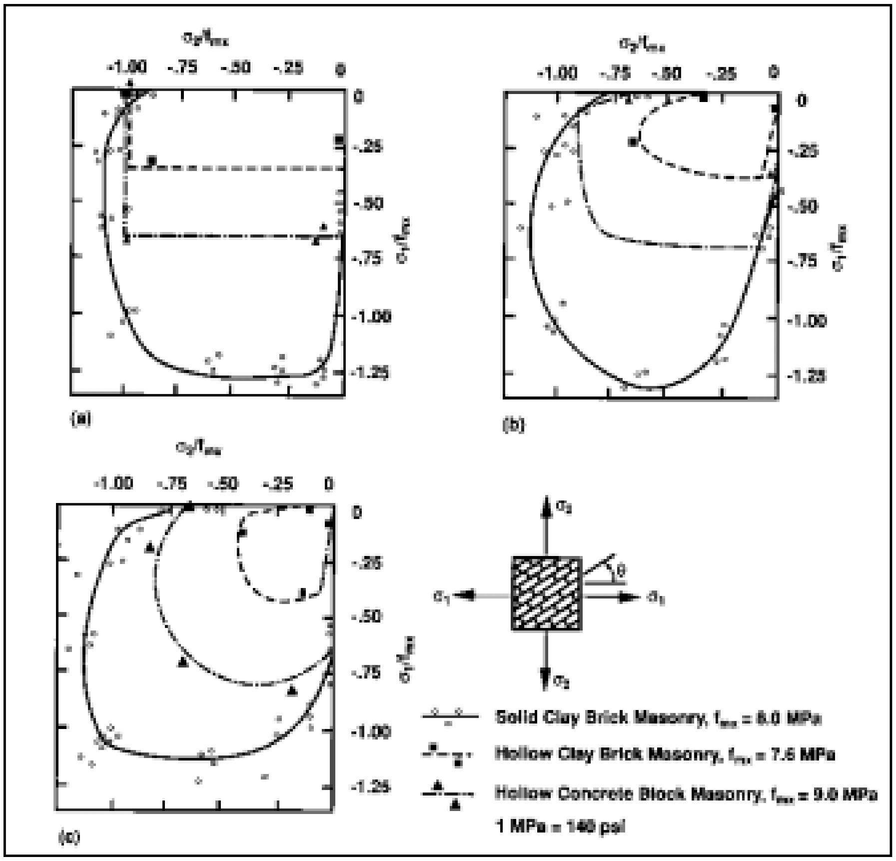


Fig. 20: Biaxial Compressive Strength of Masonry, (15, 22, 24]
a) Stresses parallel to joints θ = 0°;
b) Stresses under 22, 5° to joints, θ = 22. 5°; c) Stresses under 45° to joints, θ = 45°

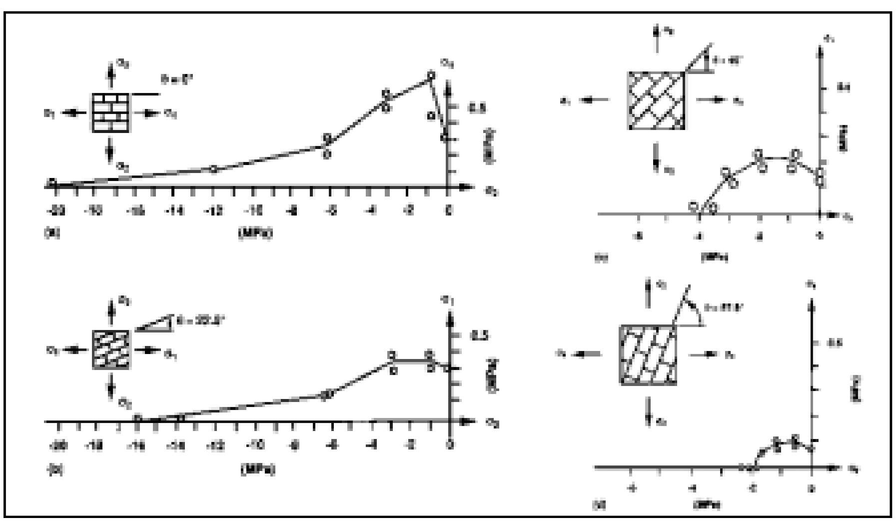


Fig. 21: Biaxial Tension-Compression Strength of Clay Brick Masonry with Solid Units, [23]
a) Stresses parallel to joints, θ = 0°;
b) Stresses under 22. 5° to joints, θ = 22. 5°
c) Stresses under 45° to joints, θ = 45°;
d) Stresses under 67 . 5° to joints, θ= 67.5° Note: 1 MPa = 140 psi

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 fmx. 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 θ = 22.5° and σ, and σ_2 for θ = 45°. 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 fmx. 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 fmx. Hollow concrete block masonry takes an intermediate position with, except for sliding failure, a minimum strength of approximately 0.7 fmx. 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 fmx. 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., fmx = uniaxial compressive strength for loads acting perpendicular to the bed joints, fmy = uniaxial compressive strength for loads acting parallel to the bed joints and tan φ

= 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 fmy except for loads perpendicular to the bed joints. As a consequence of neglecting the cohesion, no loads can be transferred for inclinations cp < θ < 90°. 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.

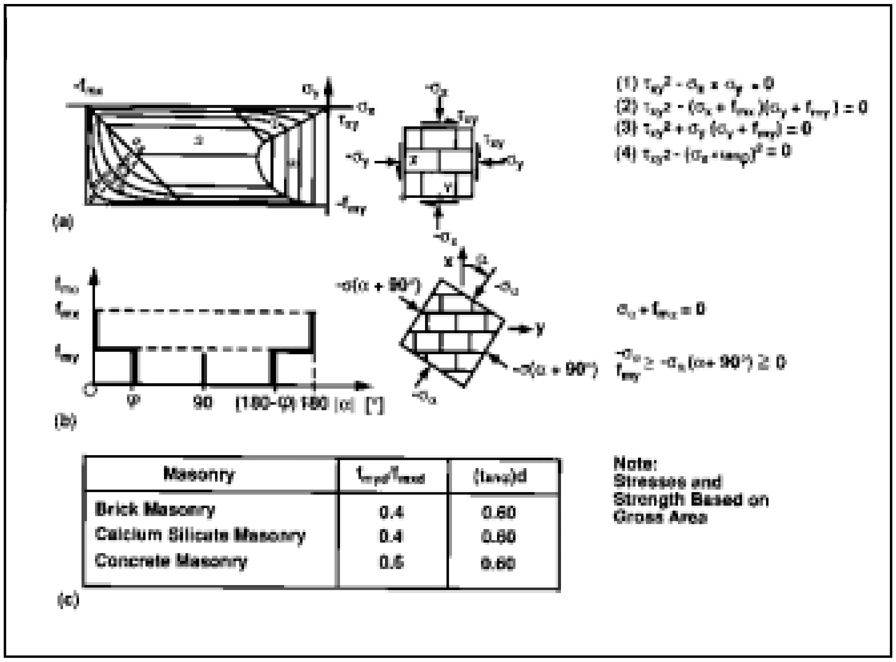


Fig. 22: SIA 177/2 Approach for the Biaxial Compressive Strength of Masonry, [8]
a) General failure surface;
b) Uniaxial and simplified biaxial strength;
c) Design strength parameters

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.

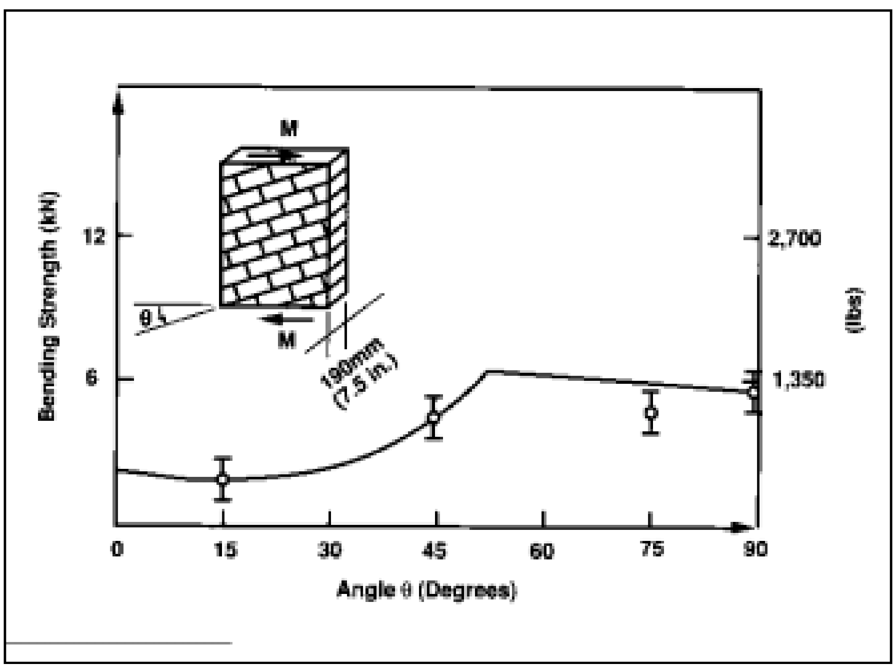


Fig. 23: Uniaxial Bending Strength of Ungrouted Concrete Masonry, [34]

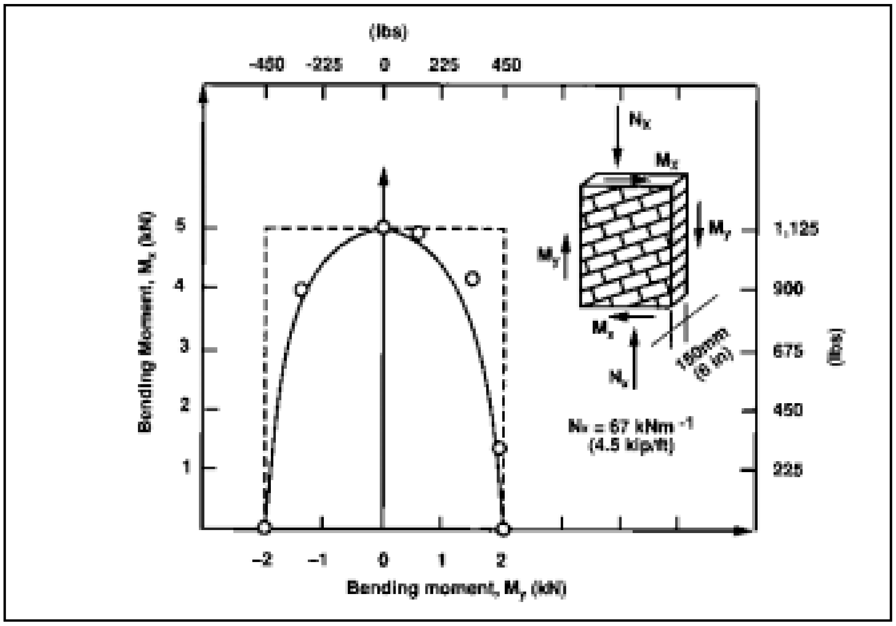


Fig. 24: Biaxial Bending Strength of Clay Brick Masonry, [36]

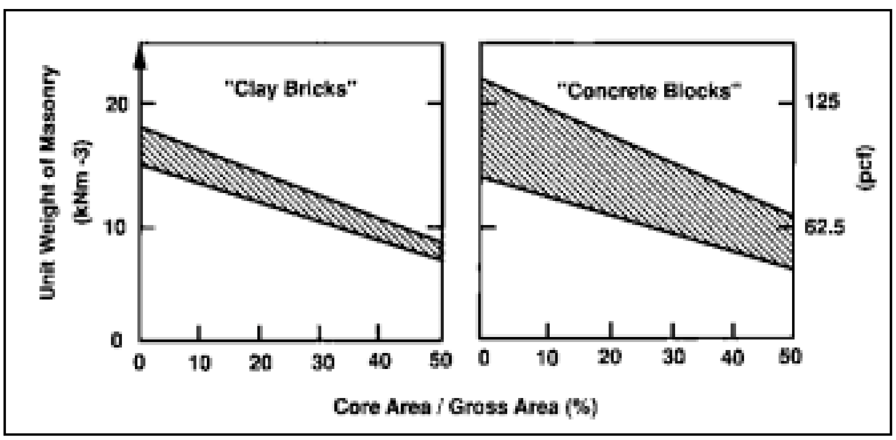


Fig. 25: Unit Weight of Masonry

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

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

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.

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 statical 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.

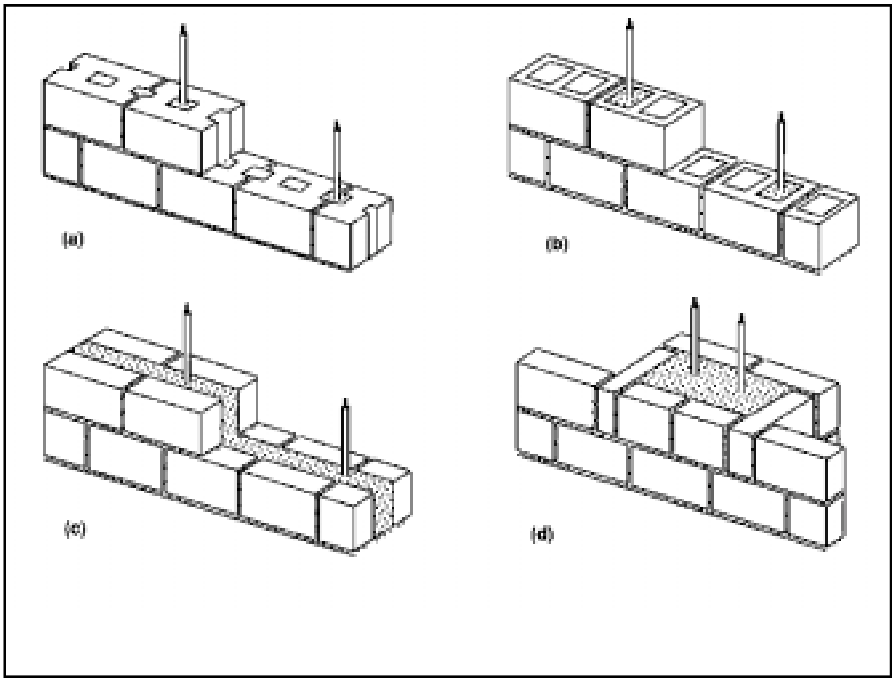


Fig. 26: Layout of Vertical Tendons in Masonry Walls
a) In cores and head joints; b) In cores only; c) In cavities; d) In pockets

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 similarily 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 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

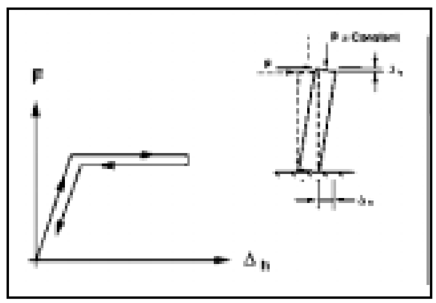


Fig. 27: Energy Dissipation in Unreinforced Masonry

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, Py, in general. For unbonded posttensioning, the tendon force will increase from service up to ultimate load level depending on

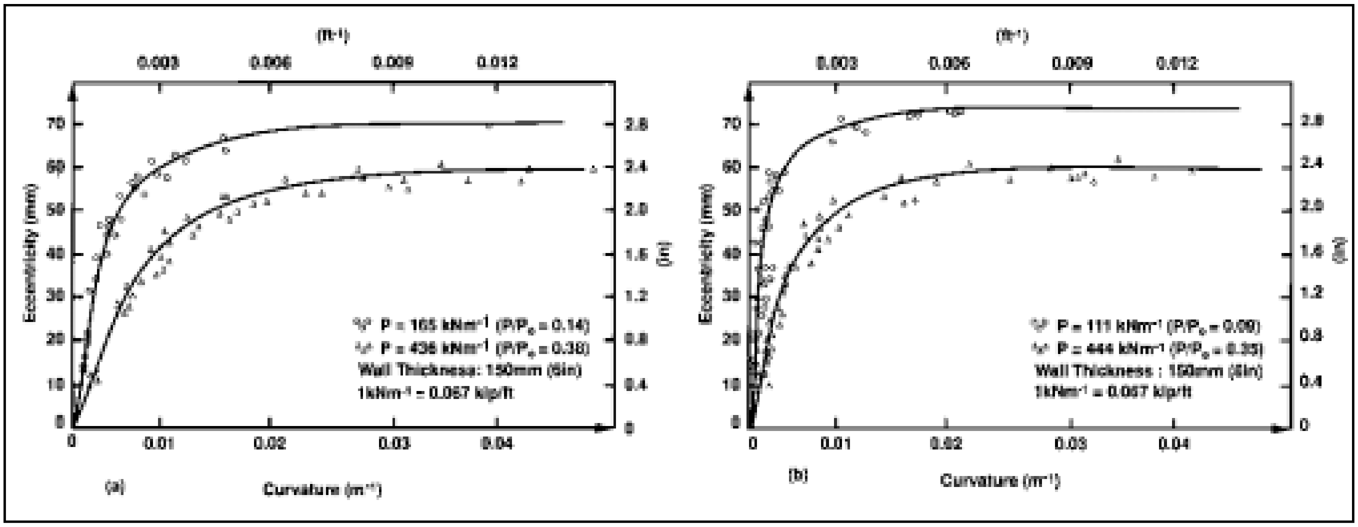


Fig. 28: Eccentricity - Curvature Curves
a) Clay brick masonry, [42]; b) Ungrouted concrete block masonry, [46]

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 bygrouting 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

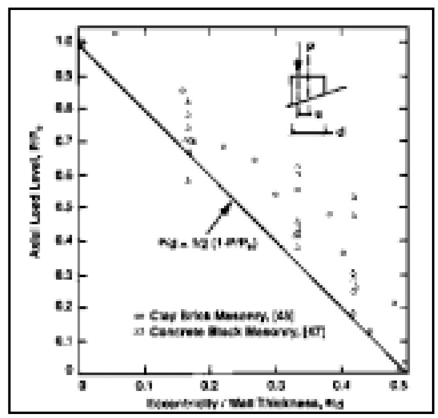


Fig. 29: Interaction of Axial Load and Bending, [45, 47]

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 momentcurvature curves for clay and concrete masonry wall sections at axial load levels of approximately 10% and 35% of the axial resistance, Po, 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 axialload. Almost bi-linear elastic-perfectly plastic, moment-curvature curves

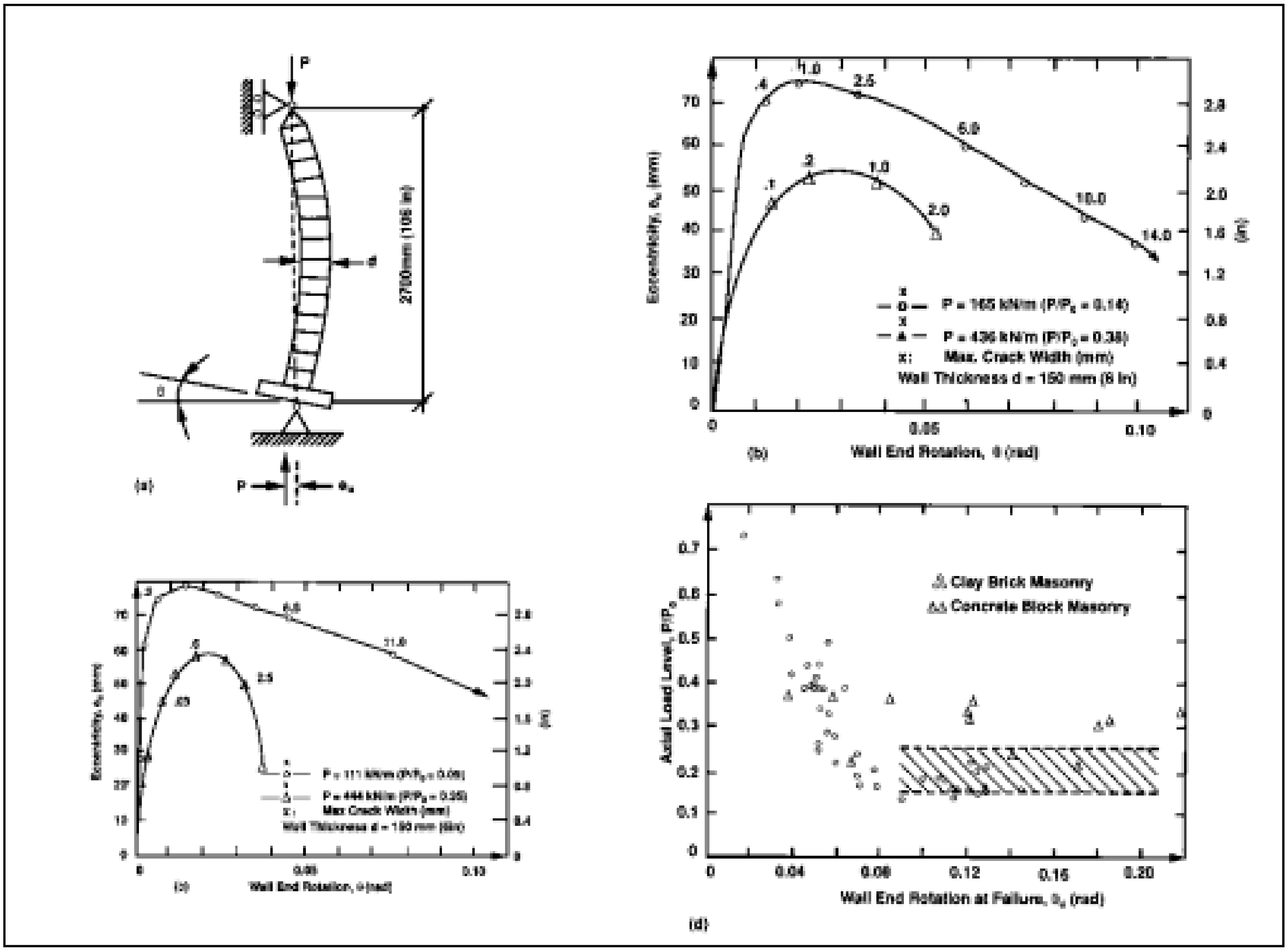


Fig. 30: Response of Plain Masonry Walls to Axial Load and Bending
a) System; b) Eccentricity - wall end rotation of clay brick masonry walls, (42];
c) Eccentricity - wall end rotation of concrete block masonry walls, [46];
d) Wall end rotation at failure, [44, 46]

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 funtion of the axial load level, P/Po. 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, eu , i.e. wall end moment, and wall end rotation, A. 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-ofplane 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:

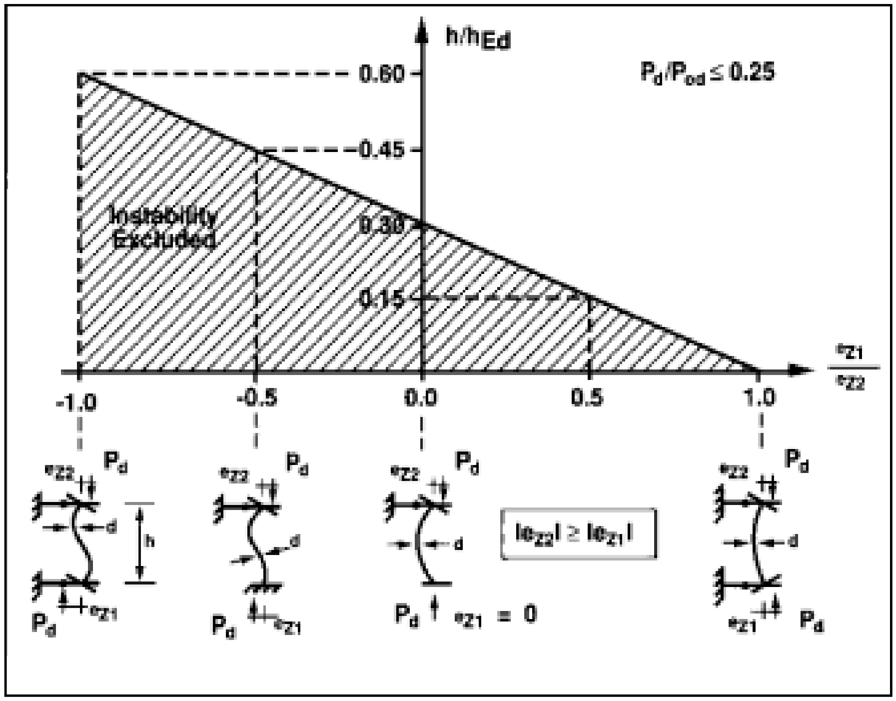


Fig 31: SIA 177/2 Approach for Walls Subjected to Axial Load and Bending, [8]
Note:

Pod	=	*fmxd x d*	*: Design Resistance*
hEd	=	$\pi \sqrt{Byd / Pd}$	*: Buckling Length*
Byd	=	$Emd \times I \times \sqrt{1 - \frac{Pd}{2\,Pod}}$	*: Bending Stiffness*

Emd: Modulus of elasticity of masonry; I: Moment of inertia of gross cross-section

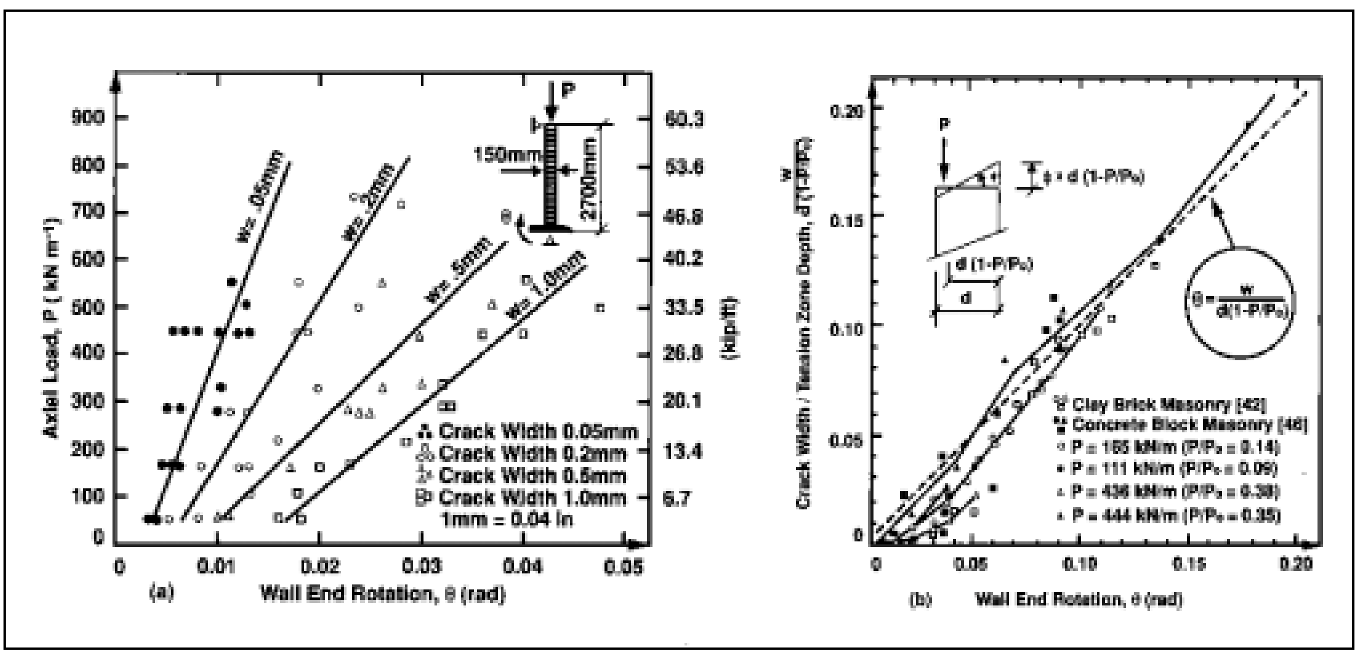


Fig. 32: Cracking Behaviour of Masonry Walls
a) Influence of axial load on crack width, [44]; b) Development of crack width with wall end rotation, [42, 46]

1. For axial load levels due to the combined effects of gravity loads and prestressing not exceeding 25% of the design resist ance, a wall will behave in a ductile man ner 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/Po), 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.

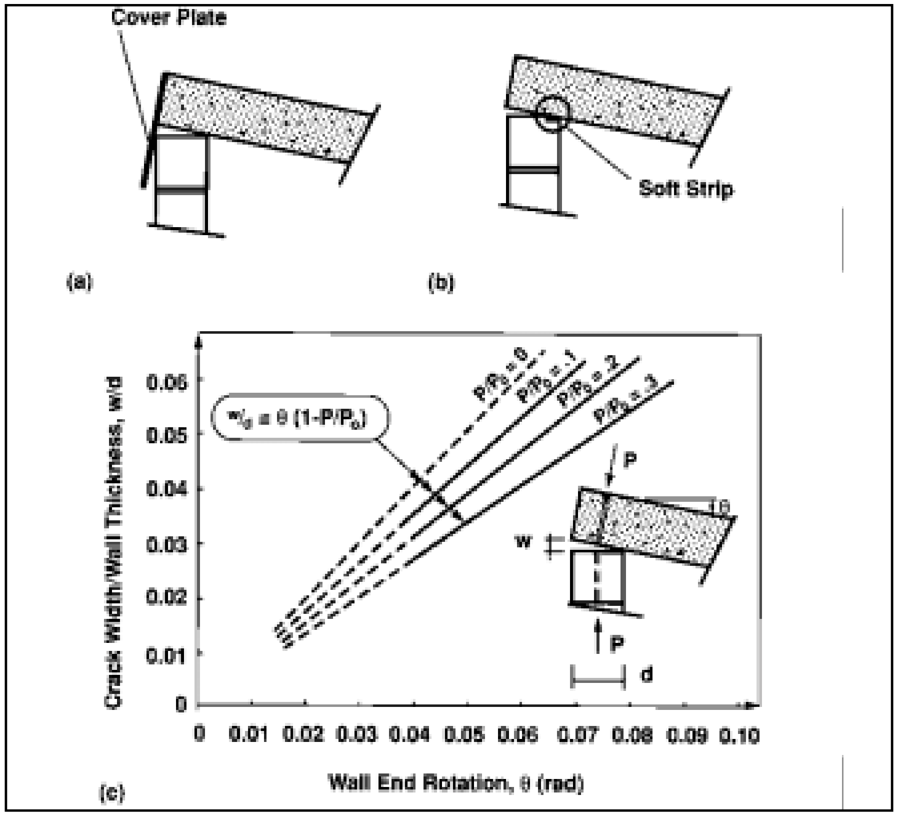


Fig. 33: Mitigation of Bending Cracks a) Protect crack; b) Reduce wall thickness; c) Post-tensioning of wall

4.3 Walls subjected to out-ofplane 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,

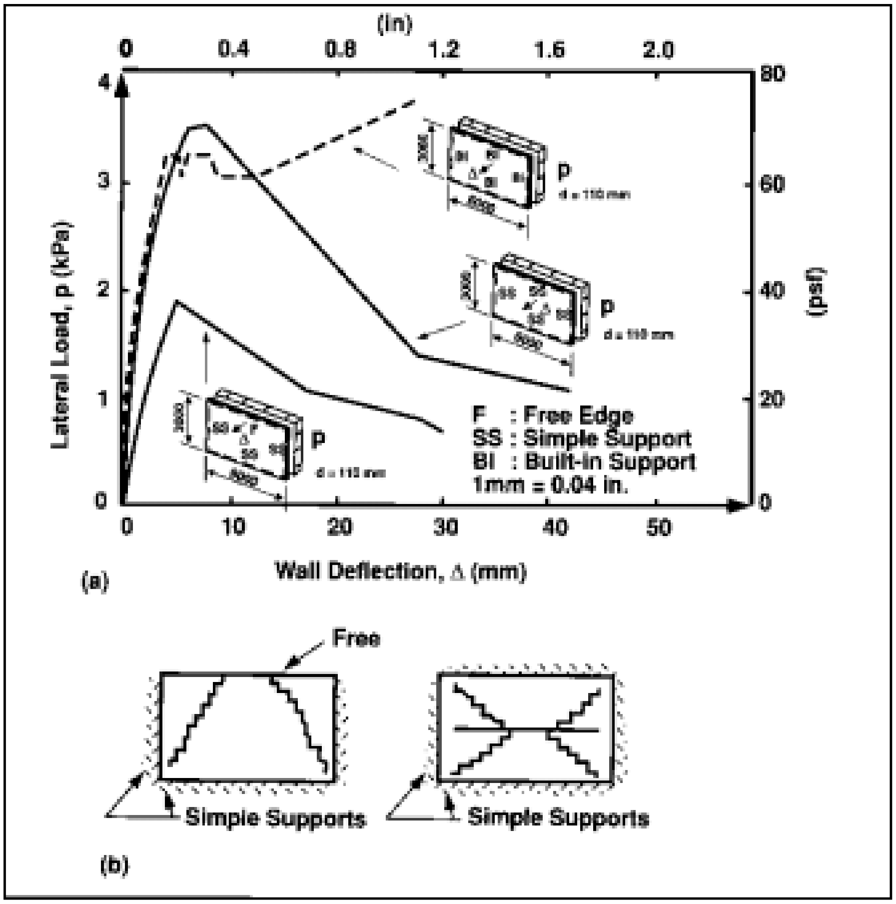


Fig. 34: Response of Plain Masonry Walls to Out-of-Plane Lateral Loads, [48]
a) Typical load-deflection curves; b) Typical crack pattern

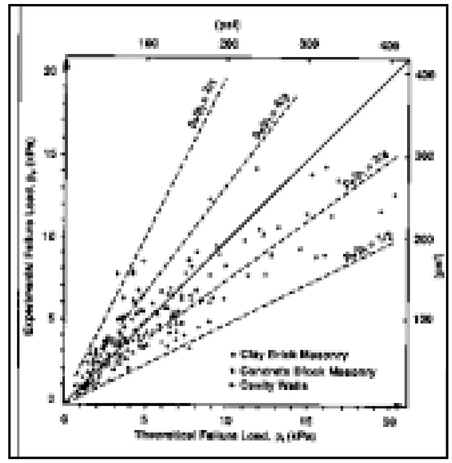


Fig. 35: Theoretical Lateral Failure Load vs. Experimental Load, [49]

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.

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, pd=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 Peff/pd 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 Peff = 150kN reduce the spacing of 2.15 m found in the graph to 1.34 m according to the following relationship:

a≤ 2.15 x 1.0/1.5 x 150/160 = 1.34m

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 momentcurvature 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.

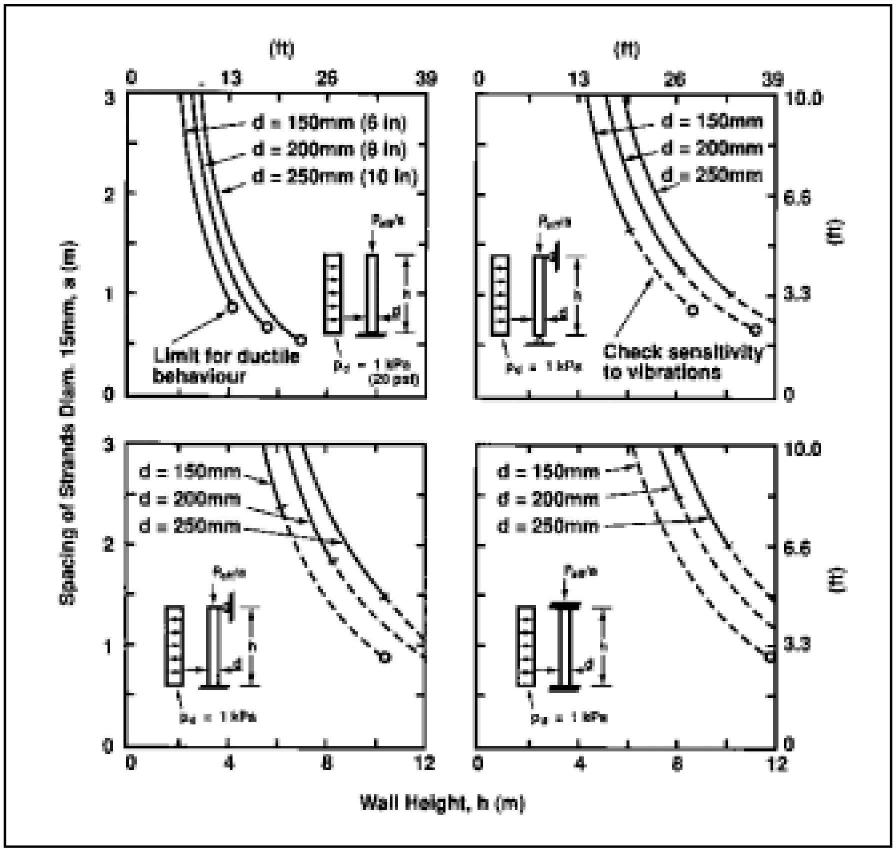


Fig. 36: Proposal for the Design of Vertically Post-Tensioned Walls
Note: Peff = 160 kN (36 kips), effective force per strand
Pd = 1 kPa (20 psf), design lateral load (factored)
Strength reduction factor for Peff: 1.2

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 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/Po.

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

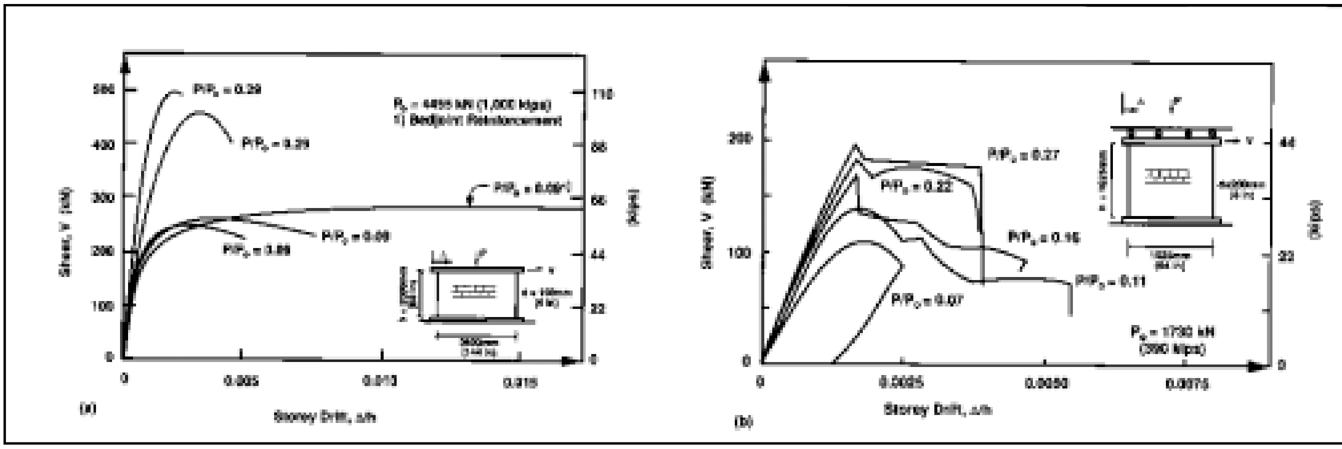


Fig. 37: Response of Plain Masonry Shear Walls a) ETH Tests, [55]; b) NBS Tests, [58]

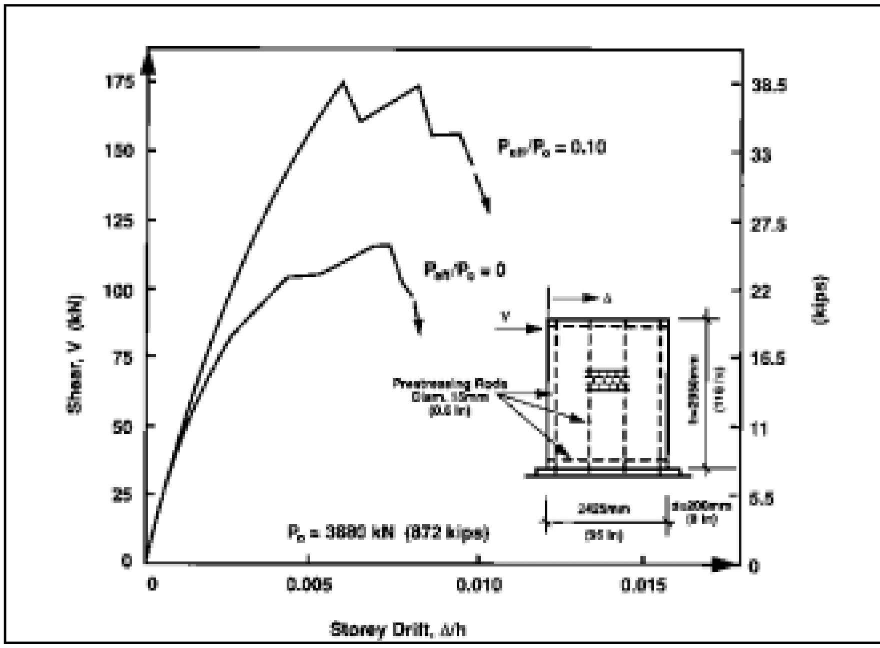


Fig. 38: Response of Reinforced and Prestressed Walls, [63]

of approximately10% 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 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 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 rathe 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 ETHtests 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

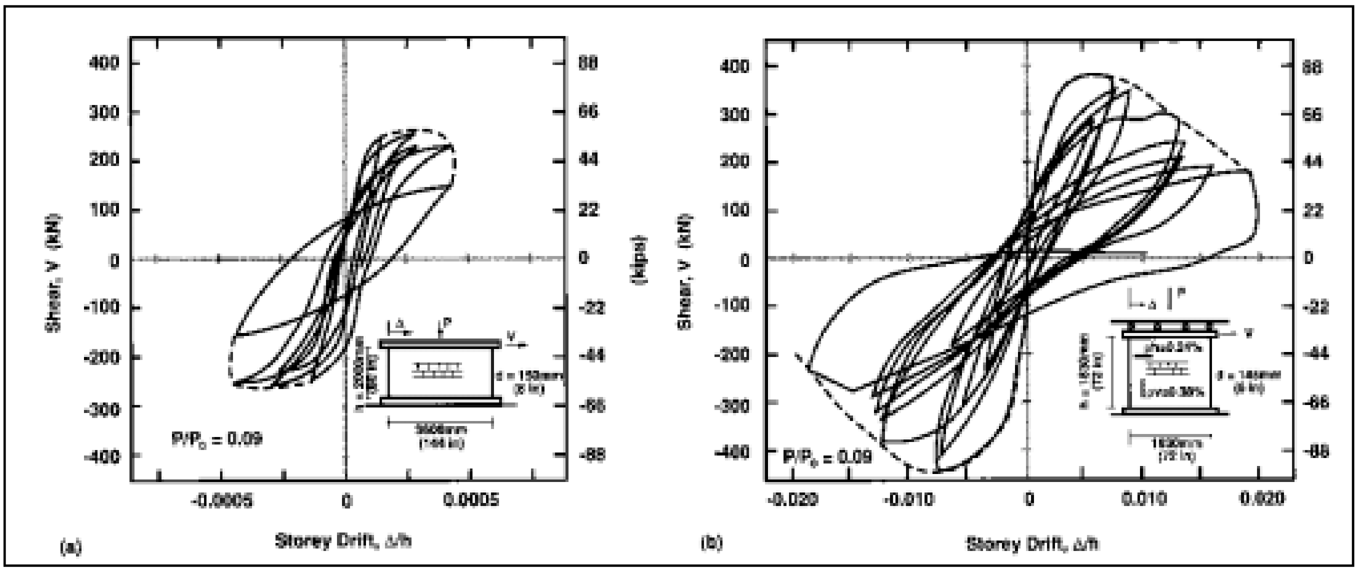


Fig. 39: Response of Shear Walls to Cyclic Loading a) Plain brick masonry shear wall, [55]; b) Reinforced concrete masonry shear wall, [64]

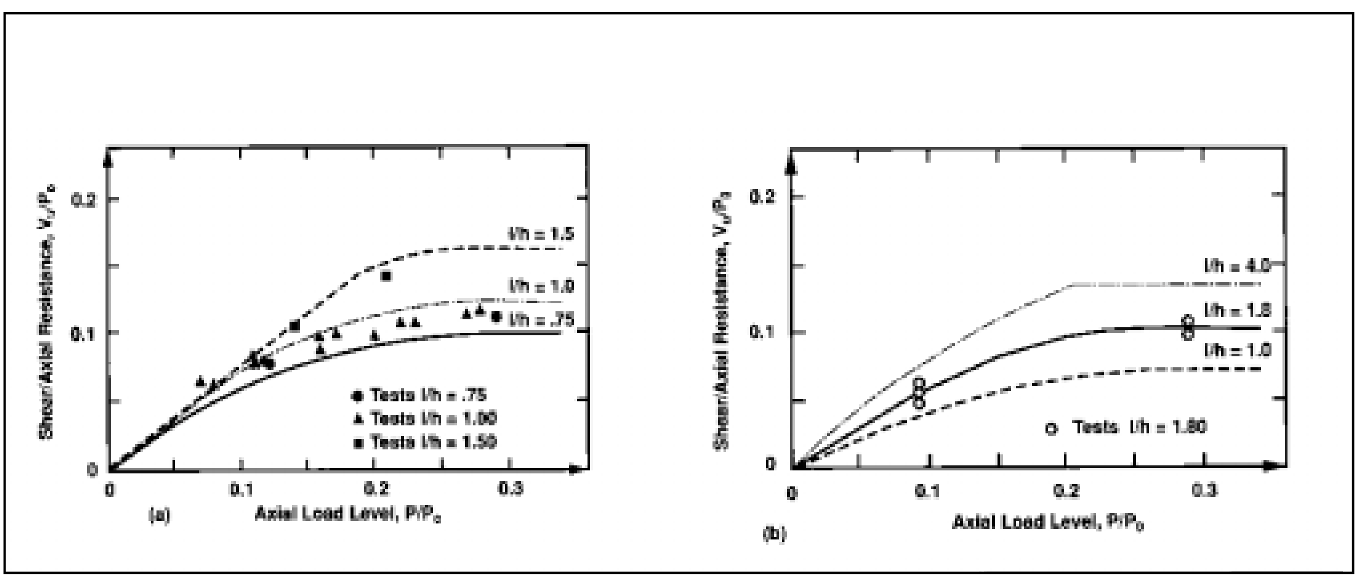


Fig. 40: Strength of Shear Walls a) Interaction of shear strength and axial load for N8S tests, [58]; b) Interaction of shear strength and axial load for ETH tests, [55]

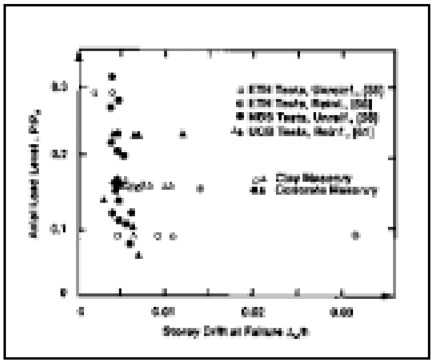


Fig. 41: Ultimate Deflection of Shear Walls, [55, 58, 61]

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 simple statics, Figure 43b.

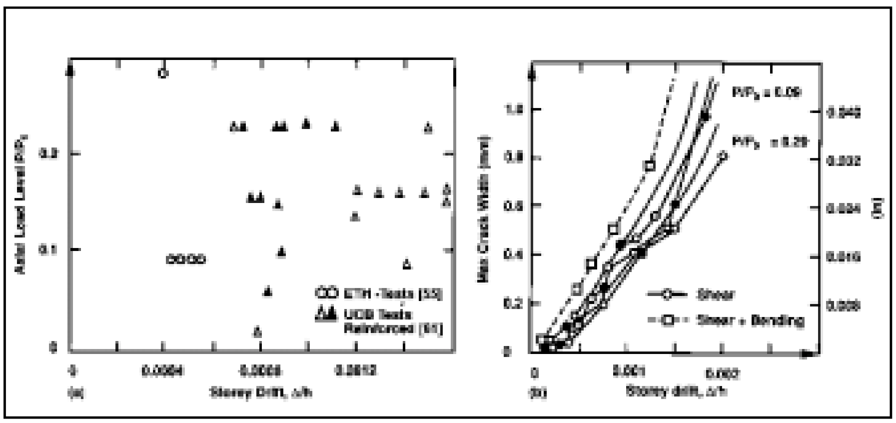


Fig. 42: Cracking Behaviour of Shear Walls
a) Influence of axial load on crack initiation, [55,61]; b) Development of crack width with storey drift, [55]

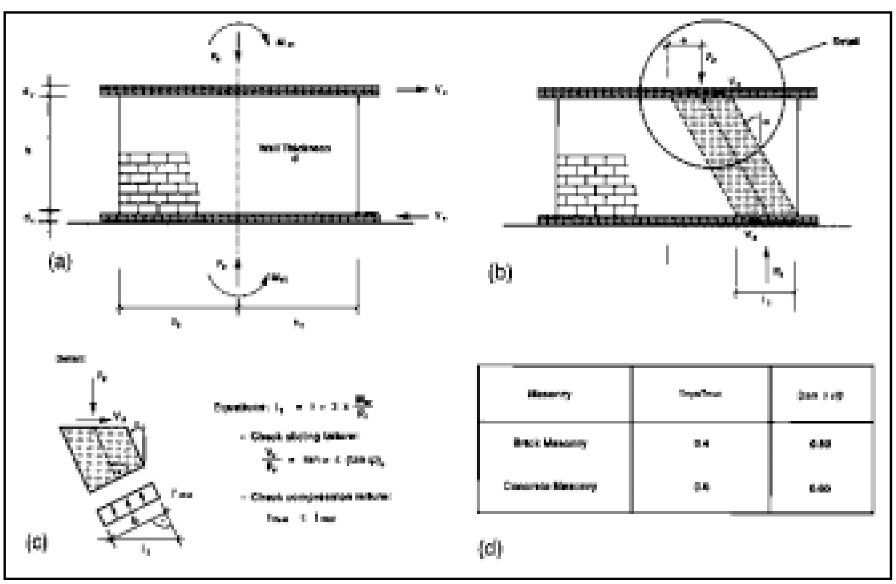


Fig. 43: SIA 177/2 Approach for Shear Walls, [8]
a) System and notations; b) Assumed stress field; c) Check of strength; d) Masonry strength properties

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,fmα, do not exceed the angle of friction, φ, and the compressive strength, fmy, respectively.

The two limiting criteria have been summarized in Figure 43c. Figure 43d gives the masonry properties for design where fmxd = 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 Vpd. 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 a 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, (fmxd-fmyd), [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

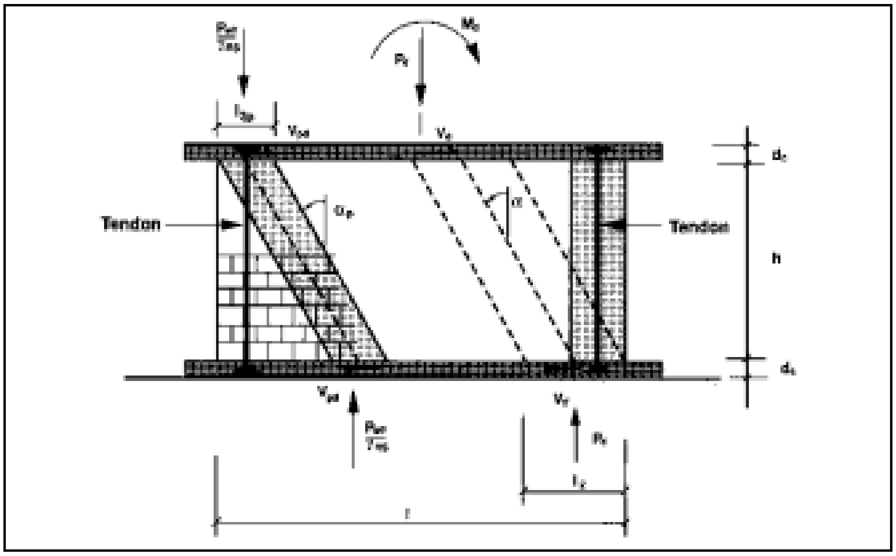


Fig. 44: Proposal for the Design of Vertically Post-Tensioned Shear Walls
Note: Peff = Effective tendon force under design loads.
γ_{RS} *= 1.2 - Strength reduction factor for reinforcement.*

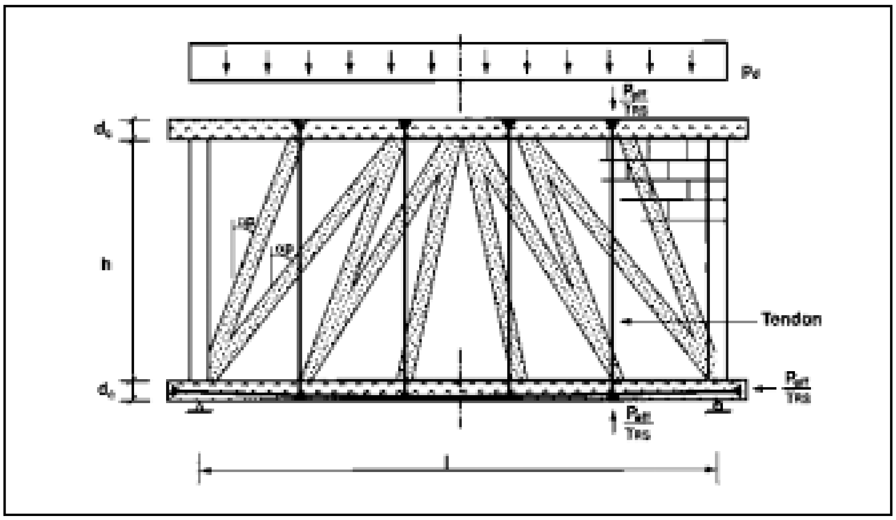


Fig. 45: Post-Tensioned Deep Beam

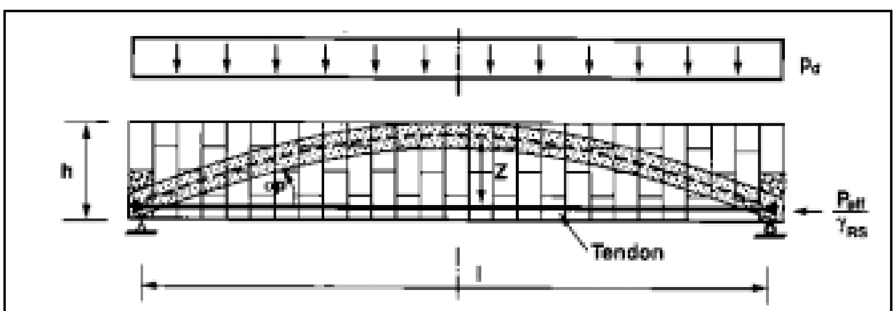


Fig. 46: Post-Tensioned Beam

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, fm, 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 bf 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 tranfer. 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	-0.4x10-3	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)

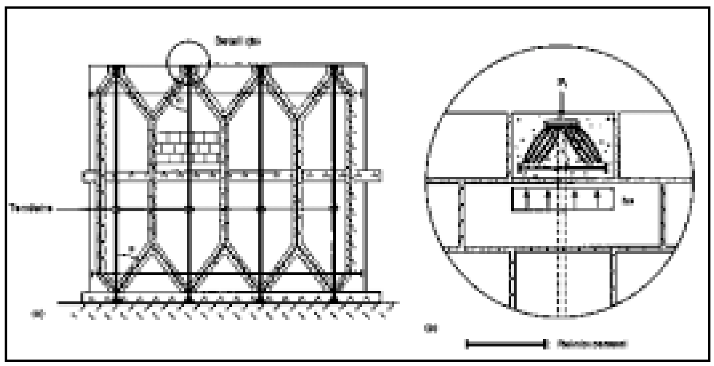


Fig. 47: Introduction of Prestressing Forces into Masonry Walls
(a) Global effects in wall; b) Local effects near anchorage

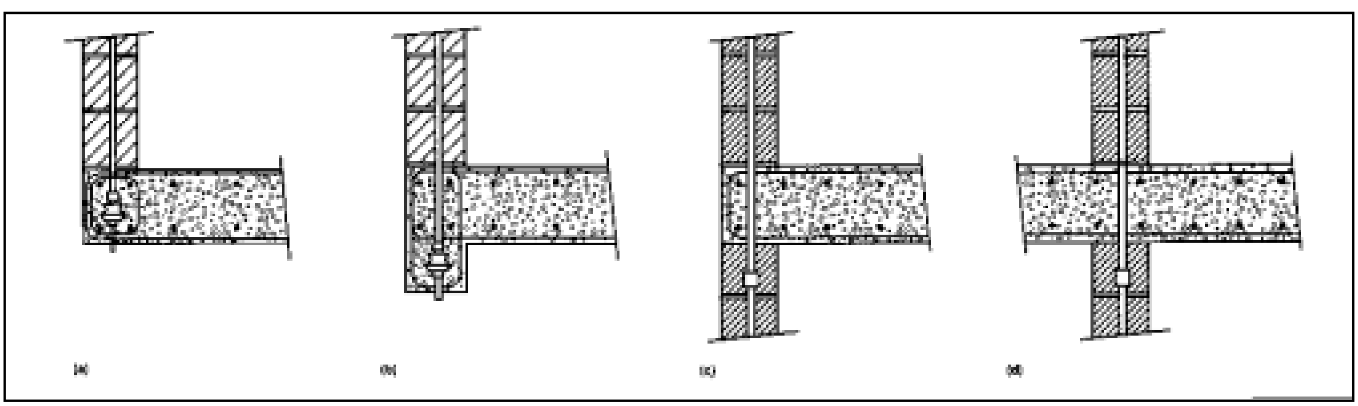


Fig. 48: Wall-Slab Connections
a) Without moment transfer; b), c), d) With moment transfer

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

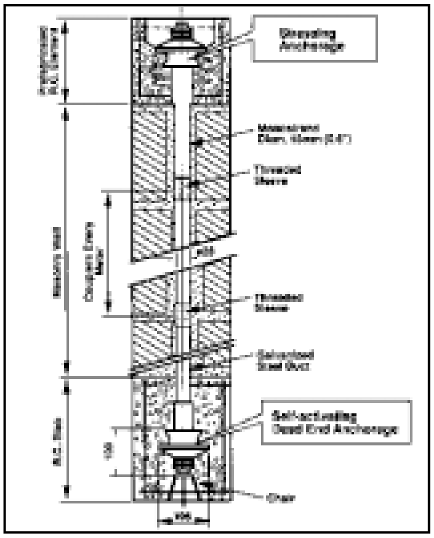


Fig. 49: VSL System for Post-Tensioned Masonry

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 e 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.

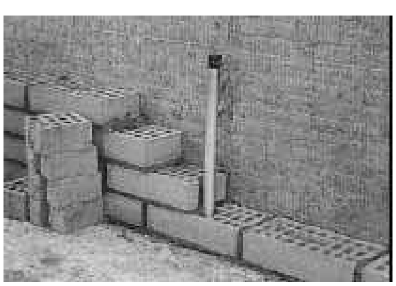
After the masonry reaches the specified strength, prestressing may com

Fig. 50: Practical Handling of the VSL System

a) Placing of dead-end anchorages

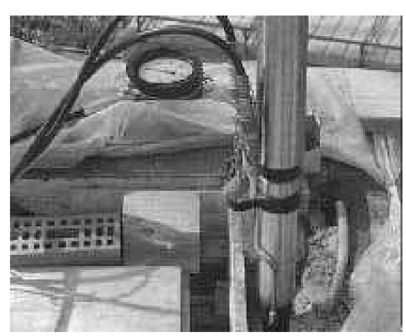
c) a Adding new duct segment

e) Introduction of monostrand



b) Start of wall construction

d) Placement of precast element



f) Stressing monostand

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

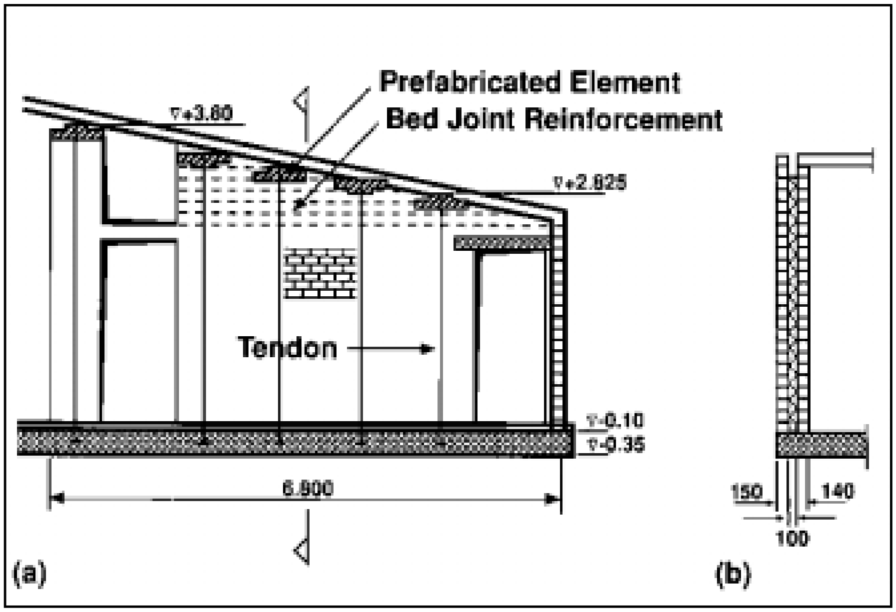


Fig. 51: Kindergarten, Zurich
a) Wall dimensions and tendon layout
b) Wall under construction
Note: Dimensions in mm (1mm=0.04in)
Elevations in m (1m=3.3ft)

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

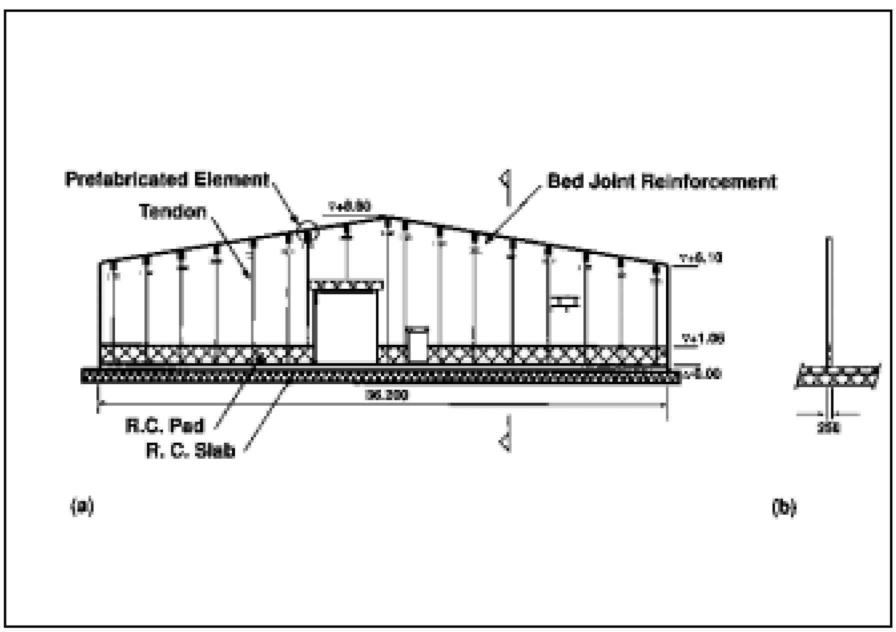


Fig. 52: Factory, Regensdorf
a) Wall dimensions and tendon layout
b) Wall under construction

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

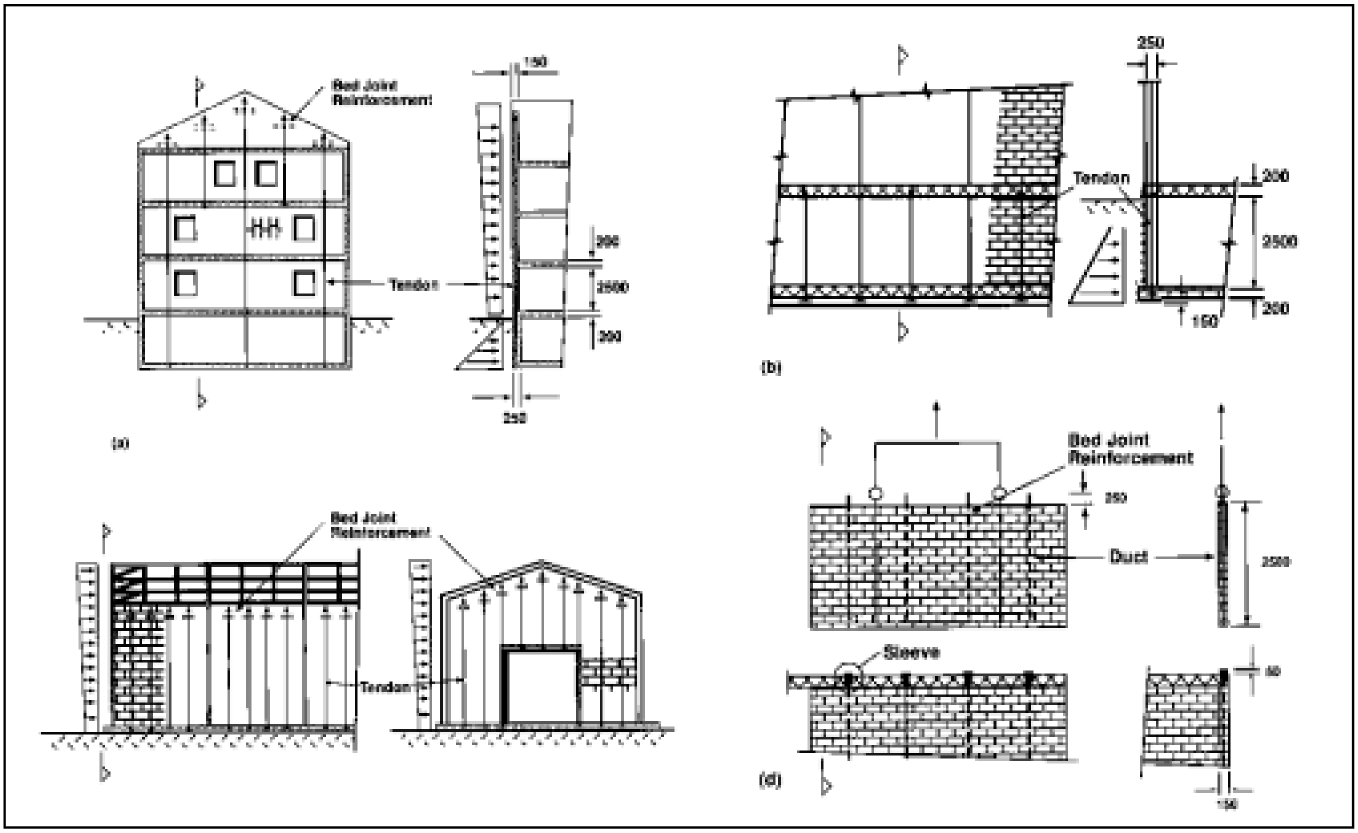


Fig. 53: Typical Future Applications of Post-Tensioned Masonry for New Structures
a) Residential building; b) Basement wall; c) Infilled frames; d) Prefabricated walls
Note: Dimensions in mm (1mm=0.04in)

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

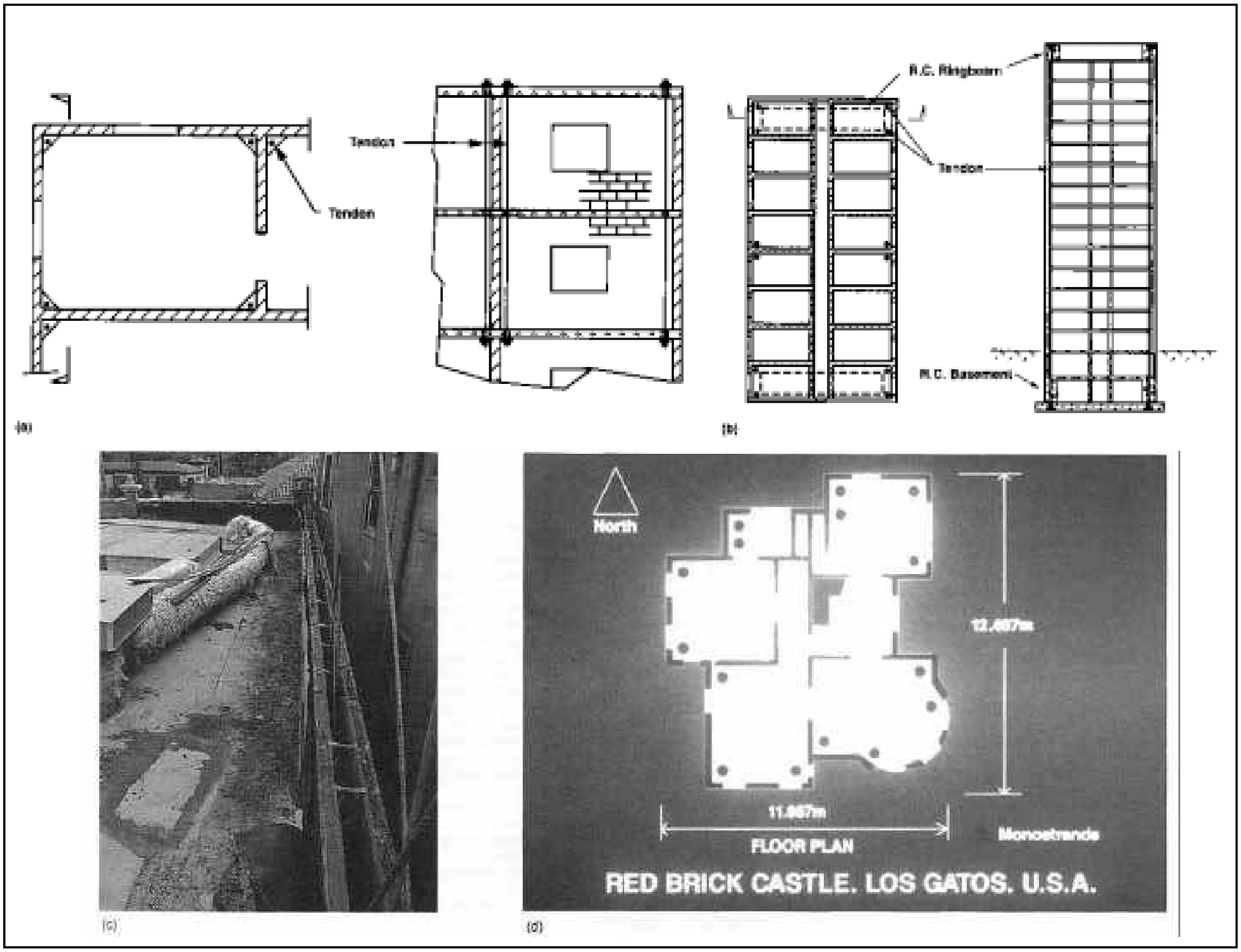
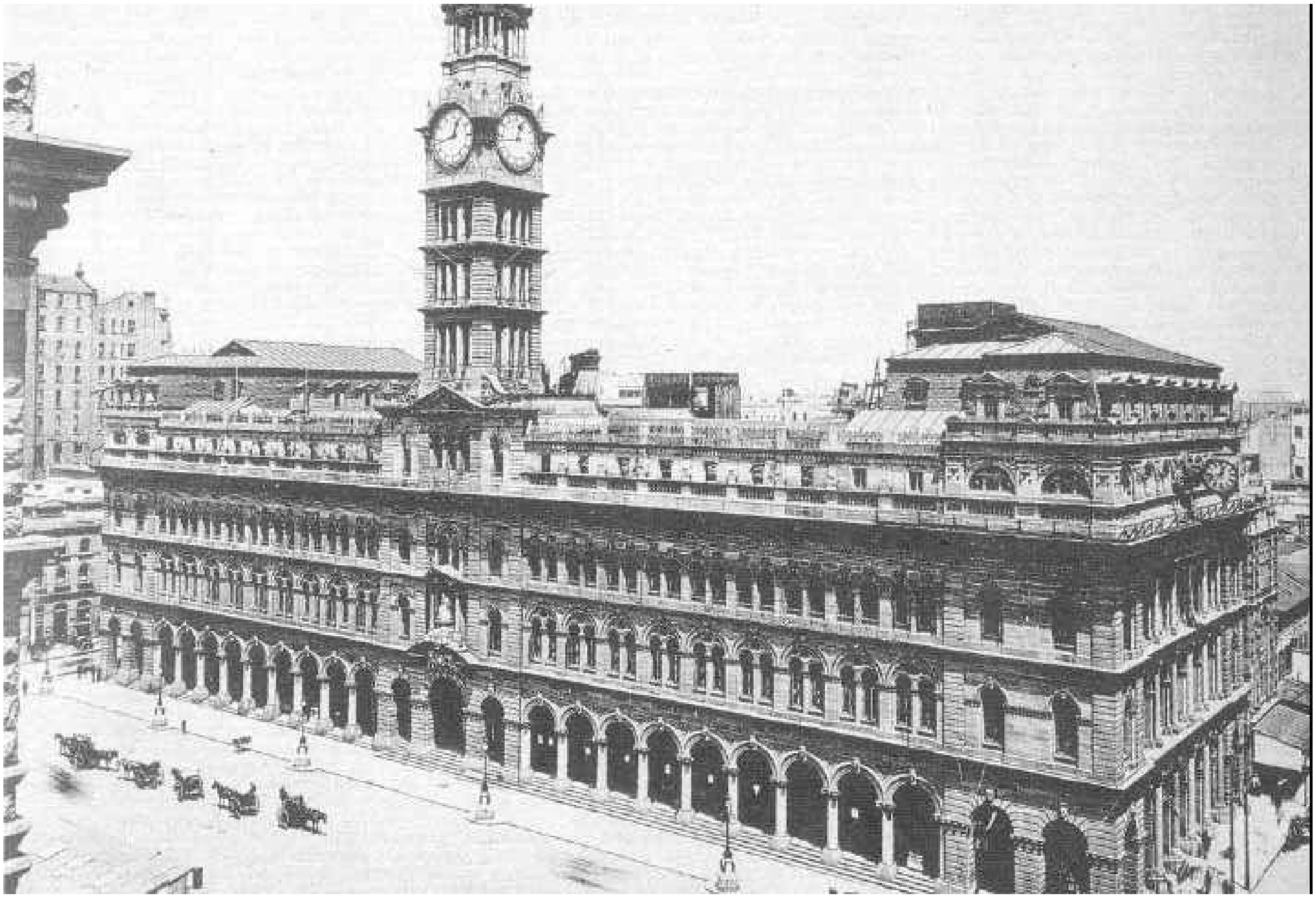


Fig. 54: Strengthening of Existing Masonry Structures
a) Individual walls, plan and section; b) Entire buildings, plan and section; c) Strengthening of a cavity wall with prestressing, Los Gatos, California; d) Proposal for strengthening of Brick Castle, Los Gatos, California

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.



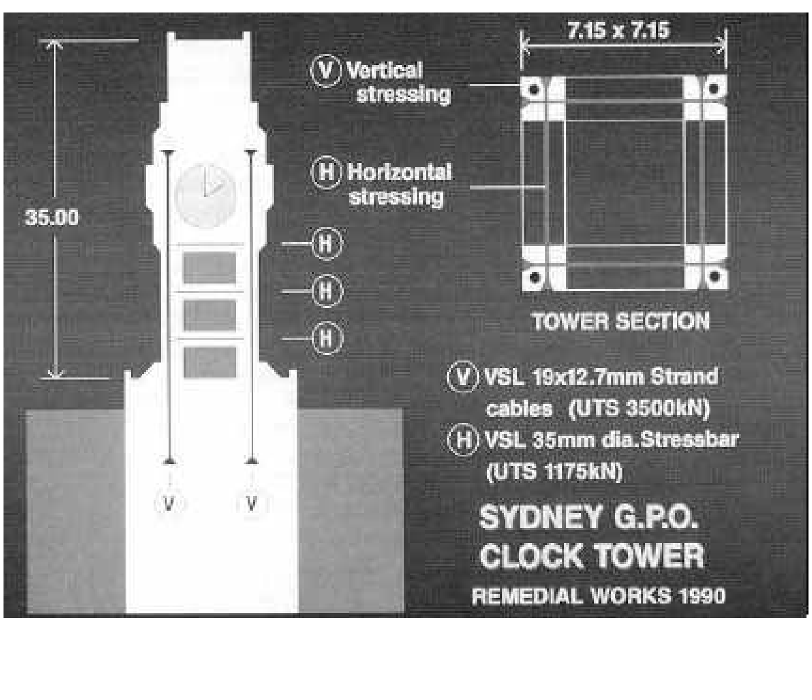


Fig. 55: Strengthening of GPO Tower, Sydney
a) General Post Office Building.
b) Tendon layout in tower, courtesy of McBean & Crisp, Pty, Ltd.

6. References

[1] Beall C., "Masonry design and detailing", Prentice-Hall International, Inc., London, 1984, 491 pp.

[2] "Reinforced and prestressed masonry" Proceedings, Institution of Civil Engineers, Thomas Telford Ltd., London, 1982, 163 pp.

[3] Practical design of masonry structures" Proceedings, Institution of Civil Engineers, Thomas Telford Ltd., London, 1986, 384 pp.

[4] Bell S.E., "Development of prestressed clay brickwork in the United Kingdom", Proceedings, 5th Canadian Masonry Symposium, University of British Columbia, Vancouver, B.C., Canada, 1989, pp. 155-163.

[5] Uniform Building Code, Chapter 24, "Masonry", 1988 Edition, International Conference of Building Officials, Whittier, California, United States of America, 1988, pp. 180-226.

[6] AS 3700 : "Masonry in buildings", 1988. Standards Association of Australia, Sydney, New South Wales, 45 pp.

[7] BS 5628: Part 1: "Use of Masonry", 1978, 43 pp. BS 5628: Part 2: "Use of Masonry", 1985, 44 pp. BS 5628: Part 3: "Use of Masonry", 1985, 100 pp. British Standard Institution, London.

[8] SIA 177: "Mauerwerk",(Masonry), 1980, 64 pp. SIA V 177/2: "Bemessung von Mauerwerkswanden" (Design of Masonry Walls), Empfehlung, 1989, 33 pp. Schweizerischer Ingenieur and ArchitektenVerein, Zurich, (available in German, French, Italian, English).

[9] DIN 1053, Teil 1: "Mauerwerk",(Masonry), 1987, 30 pp. DIN 1053, Teil 2: "Mauerwerk",(Masonry), 1984, 13 pp. DIN 1053, Teil 3: "Mauerwerk",(Masonry), 1987, 10 pp. Normenausschuss Bauwesen (Na Bau), Deutsches Institut fur Normung e.V., Berlin.

[10] Hilsdorf, H.K., "An Investigation into the failure mechanism of brick masonry loaded in axial compression", Proceedings 1st Int. Brick/Block Masonry Conference, Austin, Texas, 1967, ed. F.B. Johnson, Gulf Publishing Comp., Houston, Texas, 1969, pp. 34-41.

[11] Schneider H., "Tragfahigheit and Verformungsmodul von Mauerwerk mit neuen Mortelgruppen", (Strength and stiff ness of masonry with new mortar mixes) Schriftenreihe des Otto-Graf -Institutes, Heft 74, Stuttgart, 1979, 138 pp.

[12] Priestley M.J.N., Elder D.M., "StressStrain curves for unconfined and confined concrete masonry", ACI Journal, Proceedings V80, No 3, May/ June 1983, pp. 192-201.

[13] Wong H.E., Drysdale R.G., "Compression characteristics of concrete block masonry prisms", ASTM special technical publication 871, American Society for Testing & Materials, 1985, pp. 167-177.

[14] Powell B., Hodgkinson H.R., "The deter mination of stress/strain relationship of brickwork", Proceedings, 4th International Brick Masonry Conference, Brugge, Belgium, 1976, Section 2a, Paper 5, 5 pp.

[15] Ganz, H.R., Thurlimann B., "Versuche Ober die Festigkeit von zweiachsig beanspruchtem Mauerwerk", (Tests on the biaxial strength of masonry), Institut fur Baustatik and Konstruktion, ETH Zurich, Bericht Nr. 7502-3, Birkhauser Verlag, Basel, Boston 1982, 61 pp.

[16] Guggisberg, R., Thurlimann, B., "Versuche zur Festlegung der Rechenwerte von Mauerwerksfestigkeiten", (Experimental determination of design values for mason ry), Institut fur Baustatik and Konstruktion, ETH Zurich, Bericht Nr. 7502-5, Birkhauser Verlag, Basel, Boston, 1987, 96 pp.

[17] Schubert P., Wilmes K., "Einfluss des Prufalters auf die Mauerwerkdruckfestigkeit and die Druckfestigkeit des Mbrtels im Mauerwerk", (Influence of testing age on masonry and mortar compressive strengths), MauerwerkKalender 1987, Ernst & Sohn, Berlin, 1987, pp. 489-496.

[18] Page A.W., Hendry A.W., "Design rules for concentrated loads on masonry", The Structural Engineer, Vol 66, No 17/6, September 1988, pp. 273-281.

[19]Kirtschig K., Kasten D., "Teilflachenbelastung bei Mauerwerk," (Concentrated trated loads on masonry), MauerwerkKalender 1981, Ernst & Sohn, Berlin, 1981, pp. 161-175.

[20] Hamid A.A., Drysdale R.G., "Behaviour of brick masonry under combined shear and compression loading", Proceedings 2nd Canadian Masonry Symposium, Ottawa, Canada, 1980, pp. 57-64.

[21] Hamid A.A., Drysdale R.G., "Concrete masonry under combined shear and com pression along the mortar joints", ACI Journal, Proceedings V77, No 5, September-October 1980, pp. 314-320.

[22] Page A.W., "The biaxial compressive strength of brick masonry", Proceedings Institution of Civil Engineers, Part 2, 71, 1981, pp. 893-906.

[23] Samarasinghe W., Hendry A.W., "Strength of brickwork under biaxial,tensile and compressive stress", Proceedings 7th International Symposium on Load-Bearing Brickwork, London, 1980, Proceedings British Ceramic Society, No. 30, 1982, pp. 129-140.

[24] Lurati F., Graf H., Thurlimann B., "Versuche zur Festlegung der Rechenwerte fur Festigkeiten von Zementstein Mauerwerk", (Experimental determination of design values for concrete masonry), Institut fur Baustatik and Konstruktion, ETH Zurich, Bericht Nr. 8401-2, Birkhauser Verlag, Basel, Boston, expected end of 1989.

[25] Ganz H.R., "Mauerwerksscheiben unter Normalkraft and Schub", (Masonry walls subject to normal force and shear), Institut fur Baustatik and Konstruktion, ETH Zurich, Bericht Nr. 148, Birkhauser Verlag Basel, Boston, 1985, 133 pp.

[26] Ganz H.R., "Failure criteria for mason ry", Proceedings 5th Canadian Masonry Symposium, University of British Columbia, Vancouver, B.C., Canada, 1989, pp. 65-76.

[27] Muller H., "Untersuchungen zum Tragverhalten von querkraftbeanspruch tem Mauerwerk", (Investigations on the Behaviour of Masonry Subjected to Shear), Dissertation, Technische Hochschule, Darmstadt,

1974, 139 pp.
[28] Mann W., Muller H., "Bruchkriterien fur querkraftbeanspruchtes Mauerwerk and ihre Anwendung auf gemauerte Windscheiben", (Failure criteria for mason ry and their application to masonry shear walls), Bericht, Technische Hochschule Darmstadt, 1977, 80 pp.
[29] Mann W., Muller H., "Failure of shearstressed masonry and enlarged theory, tests and application to shear walls" Proceedings 7th International Symposium on Load-Bearing Brickwork, London, 1980, Proceedings British Ceramic Society, No. 30, 1982, pp.223-235.
[30] Hofmann P., Stockl S., "Versuche zum Verformungs-and Bruchverhalten von schubbeanspruchtem Mauerwerk" (Tests on the behaviour of masonry subjected to shear), Lehrstuhl fur Massivbau, Institut fur Bauingenieurwesen III., Technische Universitat Munchen, 1983, 48 pp.
[31] Dialer, C., "Weiterfuhrende Versuche zum Tragverhalten von schubbeanspruch tem Mauerwerk mit Vergleich von Ergebnissen aus der Literatur", (Additional tests on the behaviour of masonry subjec ted to shear and their comparison with other published test results), Lehrstuhl fur Massivbau, Institut fur Bauingenieurwesen III, Technische Universitat Munchen, 1984, 83 pp.
[32] Dialer C., "Simple spring model for the deformation of bricks in masonry under normal and shear stresses", Proceedings 5th Canadian Masonry Symposium, University of British Columbia, Vancouver, B.C., Canada, 1989, pp. 671-679.
[33] Ganz H.R., Guggisberg R, Schwartz J., Thurlimann B., "Contributions to the design of masonry walls", Institut fur Baustatik and Konstruktion, ETH Zurich, Bericht Nr. 168, Birkhauser Verlag Basel, Boston, 1989, 37 pp.
[34] Gazzola E.A., Drysdale R.G., "A component failure criterion for blockwork in flexure", Proceedings of "Advances in Analysis of Structural Masonry", Structures
Congress 86, American Society for Civil Engineers, New York, 1986, pp. 134-153.
[35] Losberg A., Johansson S., "Sideway pressure on masonry walls of brickwork", Proceedings International Symposium on Bearing Walls, (CIB), Warsaw, 1969 International Council for Building Research, Chalmers University of Technology, Publication 69:5, 8 pp.
[36] Thurlimann, B., Guggisberg, R., "Failure criterion for laterally loaded masonry walls: Experimental Investigations", Proceedings 8th International Brick/Block Masonry Conference, Dublin, Ireland, 1988, pp. 699706.
[37] Baker L.R., "A failure criterion for brick work in bi-axial bending", Proceedings 5th International Brick Masonry Conference, Washington, D.C., U.S.A., 1979, Session II, Paper 7, pp. 71-78.
[38] Schubert P., Wesche K., "Verformung and Rissesicherheit von Mauerwerk", (Deformations and cracking behaviour of masonry), MauerwerkKalender Ernst & Sohn, Berlin, 1984, pp. 85-98.
[39] "External post-tensioning", VSL International Ltd., Berne, Switzerland, 1988, 25 pp .
[40]"Post-tensioned slabs", VSL International, Ltd., Berne, Switzerland, 1981 / 1985, 41 pp.
[41] Sahlin, S., "Structural Masonry", Prentice-Hall, Englewood Cliffs, N.J., 1971, 290 pp.
[42] Furler, R., Thurlimann, B., "Versuche Ober die Rotationsfahigkeit von Backsteinmauerwerk", (Tests on clay brick masonry walls subjected to imposed wall end rotations), Institut fur Baustatik and Konstruktion, ETH Zurich, Bericht Nr. 7502-1, Birkhauser Verlag Basel, Boston, 1977, 95 pp.
[43] Furler, R., Thurlimann, B., "Versuche Ober die Rotationsfahigkeit von Kalksandsteinmauerwerk", (Tests on calcium silicate brick masonry walls subjected to imposed wall end rotations), Institut fur Baustatik and Konstruktion, ETH Zurich, Bericht Nr. 7502-2, Birkauser Verlag Basel, Boston, 1980, 44 pp .
[44] Furler, R., Thurlimann, B., "Strength of brick walls under enforced end rotations", Institut fur Baustatik and Konstruktion, ETH Zurich, Bericht Nr. 89, Birkhauser Verlag Basel, Boston, 1979, 13 pp.
[45] Furler, R., "Tragverhalten von Mauerwerkswanden unter Druck and Biegung", (Behaviour of masonry walls subjected to axial load and bending), Institut fur Baustatik and Konstruktion, ETH Zurich, Bericht Nr. 109, Birkhauser Verlag Basel, Boston, 1981, 142 pp.
[46] Schwartz, J., Thurlimann, B., "Versuche Ober die Rotationsfahigkeit von Zementsteinmauerwerk", (Tests on concrete block masonry walls subjected to imposed wall end rotations), Institut fur Baustatik and Konstruktion, ETH Zurich, Bericht, Nr. 8401-1, Birkhauser Verlag Basel, Boston, 1986, 114 pp.
[47] Drysdale, R.G., Hamid, A.A., "Capacity of concrete block masonry prisms under eccentric compressive loading", ACI Journal, Proceedings V80, No 2, March April 1983, pp. 102108.
[48] Lawrence, S.J., "Behaviour of brick masonry walls under lateral loading", Thesis Ph.D., School of Civil Engineering,University of New South Wales, 1983, 597pp.
[49] Haseltine, B.A., Tutt, J.N., "Implications of research on design recommendations", The Structural Engineer, Vol 64A, No 11/4, November 1986, pp. 341-350.
[50] Chen, W.F., Lui E.M., "Structural Stability", Elsevier, New York, 1987, 490 pp.
[51] Thurlimann, B., Schwartz, J., "Design of masonry walls and reinforced concrete columns with column deflection-curves", IABSE Proceedings, P108/87, Zurich, 1987,pp.17-24.
[52] Hillerborg, A., "Strip method of design", Viewpoint Publication, Cement and Concrete Association, Wexham Springs, England, 1974, 256 pp.
[53] Hendry, A.W., Sinha, B.P., "Shear tests on full-scale single-storey

brickwork structures subjected to precom pression", Civil Engineering and Public Works Review, 66, December 1971, pp . 1339-1344 .

[54] Haller, P ., "Der Widerstand von Mauerwerk aus Backstein and Kalksandstein gegen Schubkrafte", (The strength of clay and calcium silicate brick masonry subjected to shear), Bericht, Verband Schweizerischer Ziegel- and Steinfabrikanten, Zurich, 1980, 44 pp.

[55] Ganz, H.R., Thurlimann, B., "Mauerwerksscheiben unter Normalkraft and Schub", (Tests on masonry walls subjected to axial load and shear), Institut fur Baustatik and Konstruktion, ETH Zurich, Bericht Nr. 7502-4, Birkhauser Verlag Basel, Boston, 1985, 102 pp .

[56] Woodward, K., Rankin, F., "Influence of vertical compressive stress on shear resistance of concrete block masonry walls", Report No. NBSIR 84-2929, National Bureau of Standards, Washington D.C., U.S.A., 1984, 53 pp.

[57] Woodward, K., Rankin, F., "Influence of aspect ratio on shear resistance of concrete block masonry walls", Report No. NBSIR 84-2993, National Bureau of Standards, Washington, D.C., U .S .A., 1985, 65 pp.

[58] Woodward, K., Rankin, F., "Influence of block and mortar strength on shear resistance of concrete block masonry walls", Report No. NBSIR 85-3143, National Bureau of Standards, Washington, D.C., U.S.A., 1985, 73 PP.

[59] Priestley, MJN., Bridgeman D.O., "Seismic resistance of brick masonry walls", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol 7, No 4, 1974, pp. 167-187.

[60] Preistley, M.J.N., "Seismic resistance or reinforced concrete masonry shear walls with high steel percentages", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 10, No. 1, 1977, pp. 1-16.

[61] Sveinsson, B.I., McNiven, H.D., Sucuoglu, H., "Cyclic loading tests of masonry single piers - Volume 4: Additional tests with height to width ratio of 1 ", Earthquake Engineering Research Center, College of Engineering, University of California, Report No. UCB/ERRC-85/15, Berkeley, California, 1985, 163 pp.

[62] Shing, P.B., Schuller, M., Klamerus, E., Hoskere, V.S., Noland, J.L., "Design and analysis of reinforced masonry shear walls", Proceedings 5th Canadian Masonry Symposium, University of British Columbia, Vancouver, B.C., Canada, 1989, pp. 291-300.

[63] Page, A.W., Huizer, A., "Racking tests on reinforced and prestressed hollow clay masonry walls", Proceedings 8th International Brick/Block Masonry Conference, Dublin, Ireland, 1988, pp. 538-547.

[64] Hart, G.C., Hong, W.K., "Modeling the performance of unconfined concrete masonry flexural walls", Proceedings 5th Canadian Masonry Symposium, University of British Columbia, Vancouver, B.C., Canada, 1989, pp. 283-290.

[65] Ganz, H.R., Thurlimann, B., "Shear design of masonry walls", Proceedings of "New Analysis Techniques for Structural Masonry", Structures Congress 85, American Society for Civil Engineers, Chicago, Illinois, 1985, pp. 56-70.

[66] Ganz, H.R., Thurlimann, B., "Design of masonry walls under normal force and shear", Proceedings 8th International Brick/Block Masonry Conference, Dublin, Ireland, 1988, pp. 14471457.

[67] Shrive, N.G., "Post-tensioned masonry-status & prospects", Proceedings 1988 Annual Conference of the Canadian Society for Civil Engineers, Calgary, Alberta, Canada, 1988, pp. 679-696.

[68] Phipps, M.E., Montague T.I., "The design of prestressed concrete blockwork diaphragm walls", Aggregate Concrete Block Association (ACBA), Leicester, United Kingdom, 21 pp.

[69] Ganz, H.R., "Vorgespanntes Mauerwerk", (Post-tensioned masonry), Schweizer Ingenieur and Architekt, Vol 108, No 8/90, 1990, pp. 177-182.

CONFERENCES:

Proceedings of the International Brick/ Block Masonry Conference:

1 st: Austin, Texas, USA, 1969

2nd: Stoke-on-Trent, GB, 1971

3rd: Essen, FRG, 1973

4th: Brugge, Belgium, 1976

5th: Washington D.C., USA, 1979

6th: Rome, Italy, 1982

7th: Melbourne, Australia, 1985

8th: Dublin, Ireland, 1988

Proceedings of the Canadian Masonry Conference:

1 st: Calgary, Alberta, 1976

2nd: Ottawa, Ontario, 1980

3rd: Edmonton, Alberta, 1983

4th: Fredericton, N.B., 1986

5th: Vancouver, B.C., 1989

Proceedings of the North American Masonry Conference:

1st: Boulder, Colorado, 1978

2nd: College Park, Maryland, 1982

3rd: Arlington, Texas, 1985

4th: Los Angeles, California, 1987

5th: Urbana - Champaign, Illinois, 1990

VSL International Ltd.

P.O. Box 7124, 3001 Beme/Switzerland, Tel 41-31-66 42 22, Telex 911755 vsl ch, Fax 41-31-66 42 50

VSL OPERATING UNITS, SUBSIDIARIES AND LICENSEES

Australia - New Zealand

Australia
VSL Prestressing (Aust.) Pty. Ltd.
P.O.Box 102
Pennant Hills, NSW 2120
6 Pioneer Avenue
Thornleigh, NSW 2120
Tel 61-2-484-5944
Tlx AA 25891
Fax 61-2-875-3894

VSL Prestressing (Austj Pty. Ltd.
P.O. Box 171
Albion, Old. 4010
Tel 61-7-265-6400
Telex AA 44703
Fax 61-7-265-7534

VSL Prestressing (Aust.) Pty. Ltd.
P.O. Box 398
1 Summit Road
Noble Park, Vic. 3174
Tel 61-3-795-0366
Tlx AA 34028
Fax 61-3-795-0547

New Zealand
Precision Precasting (Wgtn.) Ltd.
Main Road South
Private Bag, Otaki
Tel 64-694-8126 Otaki
Tlx aalensn nz 32069
Fax 64-694-8344

USA West - Canada

USA
VSL Corporation
1671 Dell Avenue
Campbell, CA 95008
Tel 1-408-866-6777
Tlx 821 059
Fax 1-408-374-4113

VSL Western
1077 Dell Avenue
Campbell, CA 95008
Tel 1-408-866-5000
Fax 1-408-379-6205

VSL Corporation
10810 Talbert
Fountain Valley, CA 92708
Tel 1-714-964-6330

VSL Corporation
91-313 Kauhi Street
Ewa Beach, HI 96707
Tel 1-808-682-2811

VSL Corporation
4208 198th Street, S.W.
Lynnwood, WA 98036
Tel 1-206-771-3088
Fax 1-206-672-3020

Canada
Canadian BBB (1980) Inc.
P.O. Box 37
Agincourt, Ontario
M'S 3134
Tel 1-416-291-1618
T1x 065-25190 bbrcda

USA East
VSL Eastern
P.O. Box 866
8006 Haute Court
Springfield
VA 22150-0886
Tel1-703-451-4300
Fax 1-703-451-0862

VSL Corporation
5555 Oakbrook Parkway
Suite 530
Norcross, GA 30093
Tel1-404-446-3000
Fax 1-404-242-7493

VSL Corporation
7223 N.W. 46th Street
Miami, FL 33166-6490
Tel 1-305-592-5075
Fax 1-305-592-5629

VSL Corporation
11925 12th Avenue South
P.O. Box 1228
Burnsville
MN 55337-0228
Tel 1-612-894-6350
Fax 1-612-894-5708

VSL Corporation
1414 Post and Paddock
Grand Prairie, TX 75050
Tel 1-214-647-0200
Fax 1-214-641-1192

VSL Corporation
608 Garrison Street
UnitV, Lakewood
CO 80125
Tel1-303-239-6655
Fax 1-303-239-6623

Far East
VSL Far East Pte. Ltd.
151 Chin Swee Road
#11-01/10
Manhattan House
Singapore 0316
Tel 65-235-7548
Fax 65-733-8642

Brunei
VSL Systems (B) Sdn. End.
P.O. Box 33
Bandar Seri Begawan 2600
Tel 673-2-29153/21827
Fax 673-2-21954

Hong Kong
VSL Engineers (HK) Ltd.
20/F., East Town Bldg.
41 Lockhart Road
P.O. Box 20056
Wanchai/Hong Kong
Tel 852-5-201-600

Europe - Middle East - Africa - South America

Switzerland
VSL International AG
Konizstrasse 74
Postfach 7124
3001 Bern
Tel 41-31-66 42 22
Tlx 91 1 755 vsl ch
Fax 41-31-66 42 50

VSL International AG
Bernstrasse 9
3421 Lyssach
Tel 41-34-47 99 11
Tlx 914 131 vsl ch
Fax 41-34-45 43 22

Austria
Sonderbau GesmbH
Sechshauser Strasse 83
1150 Vienna
Tel 43-1-812-0280
Tlx 134 027 sobau a
Fax 43-1-812-0280/33

Brazil
Rudloff-VSL Industrial Ltda.
Rua Dr. E. Th. Santana, 158
Barra Funda
Sao Paulo / CEP 01140
Tel 55-11-826 0455
T1x 113 1121 rudf br
Fax 55-11-826 62 66

Chile
Sistemas Especiales de Construccion SA
Josue Smith Solar 434
Santiago 9
Tel 56-2-231-1205/232-2608
Fax 56-2-231-1205

France
VSL France s.a r.l.
154, rue du Vieux-Pont-du-Sevres
92100 Boulogne-Billancourt
Tel 33-1-4621-4942
Tlx 200 687 vslpari f
Fax 33-1-4761-0558

Indonesia
PT VSL Indonesia
Jalan Bendungan Hilir Raya
Kav. 36A Blok B No. 3
Tromol Pos 3609/JKT
Jakarta 10210
Tel 62-21-586 190
Tlx 45396 vslind is
Fax 62-21-581-217

Japan
VSL Japan Corporation
Tachibana Shinjuku Bldg. 4F
2-26, 3-chome, Nishi-Shinjuku, Shinjuku-ku
Tokyo 160
Tel 81-3-346-8913
Fax 81-3-345-9153

Taisei Corporation
Engineering & Construction
P.O. Box 4001
Shinjuku Center Building
Tokyo 163
Tel 81-3-348-1111
Tlx 232-2424 taisei j
Fax 81-3-343-4046

Germany
SUSPA Spannbeton GmbH
Max-Planck-Ring 1
P.O. Box 3033
4018 Langenfeld/Rhld
Tel49-2173-79020
Tlx 172173412 suspalf
Fax 49-2173-790-220

Greece
EKGE S/A
75, El. Venizelou
(Ex.Vas.Sophias) Avenue
11521 Athens
Tel 30-1-724-8312
Tlx 216-064 tev gr
Fax 30-1-724-1746

Italy
PRECO S.r.l.
Via Olona 12
20123 Milan
Tel 39-2-4818 031
Tlx 335-392 prebet i
Fax 39-2-284 4137

Netherlands
Civielco B.V.
P.O. Box 751
Rhijnhofweg 9
2300 AT Leiden
Tel31-71-768-900
Fax 31-71-720-886

Norway
VSL Norge A/S
P.O. Box 173
4001 Stavanger
Tel 47-4-563-701
Tlx 33054
Fax 47-4-570-052

Peru
Pretensado VSL del Peru SA
Avenida Principal 190
Santa Catalina, Lima 13
Tel 51-14-718-347
Tlx 20 198 be inca
Fax 51-14-717-400

Japan
Taiyo Kogyo Co., Ltd.
8-4, 4-chome, Kigawa-higashi, Yodogawa-ku
Osaka 532
Tel 81-6-306-3055
Tlx 523-3818 taiyo j
Fax 81-6-306-1755

Korea
VSL Korea Co., Ltd.
5/F, Yang Jae Building
261 Yangjae-Dong
Seocho-Gu
Seoul
Tel 82-2-574-8200
Tlx vslkor k 28786
Fax 82-2-577-0098

Malaysia
VSL Engineers (M) Sdn. Bhd.
39 B Jalan Alor
50200 Kuala Lumpur
Tel 60-3-2424711
Fax 60-3-242-9397

Portugal
Prequipe Pre-Esforoqo
Equipamento a Montagens, SA
Av. da Republica, 47
1000 Lisboa
Tel 351-1-770-730
Tlx 63708 elfp p
Fax 351-1-771-791

Saudi Arabia
VSL International Ltd
Nada Village
P.O. Box 3886
Jeddah 21481
Tel 966-2-691-8810 ext.146
Tlx 602 432 siemdv sj
Fax 966-2-691-8810 ext.167

South Africa
SteeledaleSystems (Pty.) Ltd.
P.O. Box 1210
Johannesburg 2000
Tel 27-11-613-7741 /9
Tlx 426 847 sa
Fax 27-11-613-7404

Sweden
Intemordisk Spannarmering AB
Vendevagen 89
18225 Danderyd
Tel 46-8-753 02 50
Tlx 11524 skanska s
Fax 46-8-7557126

Turkey
Yapi Sistemleri Insaat ve Sanayii A.S.
Balmumcu, Arzu Sokak
No. 5 Daire 3
80700 Besiktas - Istanbul
Tel 90-1-174-09 54
Tlx 39552 ypss tr
Fax 90-1-174-23 08

United Kingdom
Balvac Whitley Moran Ltd.
P.O. Box 4,
Ashcroft Road, Kirkby
Liverpool L33 7ZS
Tel44-51-549 2121
Fax 44-51-549 1436

Singapore
VSL Singapore Pte. Ltd.
151 Chin Swee Road
#11-01/10
Manhattan House
Singapore 0316
Tel 65-235-7077/9
Fax 65-733-8642

Taiwan
VSL System (Taiwan) Ltd.
803 Tun Hwa South Road, 2/F.
Taipei 10673 R.O.C.
Tel 886-2-738-8837
Fax 886-2-736-2595

Thailand
VSL (Thailand) Co., Ltd.
138/1 PhanSak Bldg.
Suite 201
Petchburi Road, Phayathai
Bangkok 10400
Tel 66-2-215-9498
Tlx 81055 cnc cord th
Fax 66-2-215-9490