1

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS
STANDARD REFERENCE MATERIAL 1010a
(ANSI and ISO TEST CHART No. 2)

National Library
of Canada

Bibliothèque nationale
du Canada

Canadian Theses Service

Services des thèses canadiennes

Ottawa, Canada
K1A 0N4

CANADIAN THESES

THÈSES CANADIENNES

NOTICE

The quality of this microfiche is heavily dependent upon the quality of the original thesis submitted for microfilming. Every effort has been made to ensure the highest quality of reproduction possible.

If pages are missing, contact the university which granted the degree.

Some pages may have indistinct print especially if the original pages were typed with a poor typewriter ribbon or if the university sent us an inferior photocopy.

Previously copyrighted materials (journal articles, published tests, etc.) are not filmed.

Reproduction in full or in part of this film is governed by the Canadian Copyright Act, R.S.C. 1970, c. C-30.

AVIS

La qualité de cette microfiche dépend grandement de la qualité de la thèse soumise au microfilmage. Nous avons tout fait pour assurer une qualité supérieure de reproduction.

S'il manque des pages, veuillez communiquer avec l'université qui a conféré le grade.

La qualité d'impression de certaines pages peut laisser à désirer, surtout si les pages originales ont été dactylographiées à l'aide d'un ruban usé ou si l'université nous a fait parvenir une photocopie de qualité inférieure.

Les documents qui font déjà l'objet d'un droit d'auteur (articles de revue, examens publiés, etc.) ne sont pas microfilmés.

La reproduction, même partielle, de ce microfilm est soumise à la Loi canadienne sur le droit d'auteur, SRC 1970, c. C-30.

**THIS DISSERTATION
HAS BEEN MICROFILMED
EXACTLY AS RECEIVED**

**LA THÈSE A ÉTÉ
MICROFILMÉE TELLE QUE
NOUS L'AVONS REÇUE**

Canada

Permission has been granted to the National Library of Canada to microfilm this thesis and to lend or sell copies of the film.

The author (copyright owner) has reserved other publication rights, and neither the thesis nor extensive extracts from it may be printed or otherwise reproduced without his/her written permission.

L'autorisation a été accordée à la Bibliothèque nationale du Canada de microfilmer cette thèse et de prêter ou de vendre des exemplaires du film.

L'auteur (titulaire du droit d'auteur) se réserve les autres droits de publication; ni la thèse ni de longs extraits de celle-ci ne doivent être imprimés ou autrement reproduits sans son autorisation écrite.

ISBN 0-315-29856-1

LIMIT STATES DESIGN OF MASONRY:

**AN ASSESSMENT OF PROPOSED NEW
CANADIAN BUILDING CODE REQUIREMENTS**

By

V. Charles Fenton, P.Eng.
Bachelor of Applied Science
University of British Columbia

1954

A Thesis submitted to the Faculty of Graduate
Studies in partial fulfilment of the require-
ments for the Degree of Master of Engineering

Department of Civil Engineering

Carleton University

© Ottawa, 1983

The undersigned recommend to the Faculty of Graduate
Studies and Research acceptance of the thesis

**LIMIT STATES DESIGN OF MASONRY:
AN ASSESSMENT OF PROPOSED NEW
CANADIAN BUILDING CODE REQUIREMENTS**

submitted by V. Charles Fenton, B.A.Sc., P.Eng.

in partial fulfilment of the requirements for
the degree of Master of Engineering

Thesis Supervisor

Chairman, Department of Civil Engineering

Carleton University

December 13, 1983

ABSTRACT

This study investigates a number of methods that have been proposed for masonry strength prediction for use in design.

A model section capable of describing in a simple manner the geometry for any hollow, partially grouted or solid masonry element with or without reinforcement was developed.

Resistance expressions based on this model section were derived for use in the computer programs and design problems.

The background theory for the moment magnifier method confirming its applicability to beam columns with transverse loads was found in the literature and the pertinent clauses extracted and included for information.

The methods were applied to a number of tested walls to confirm the validity of the derivations and consistency of the methods.

A number of typical structural masonry design problems were done to assess the relative ease of applicability of the methods and to clarify the potential of the methods for design aids.

A number of computer programs were written to perform the calculations using the various methods and providing numeric or graphic output of the iterative procedures. These were written in HP Basic for a Hewlett-Packard 85A desktop portable computer. Listings have been included in the appendix to assist in the development of future design aids and programs.

A wall test series has been suggested to fill an apparent gap in the available data.

ACKNOWLEDGEMENTS

The author would like to thank his research director, Professor Gary Suter, for helping to develop the concept and form of this paper and for the guidance provided during the work.

The work done by Professor Carl Turkstra and his Task Group is very much appreciated and it is hoped that this paper will assist him in this important project.

The author would especially like to thank his longtime friend and partner Professor John Adjeleian for the encouragement to begin this long but extremely rewarding project.

CONTENTS

	Page
Abstract	i
Acknowledgements	ii
Table of Contents	iii
List of Figures	v
Notation	vi
1. Introduction	1
1.1 Background	1
1.2 Objective and Scope	2
2. Limit States Design	3
2.1 Definitions	3
2.1.1 Limit States	3
2.1.2 Serviceability Limit States	3
2.1.3 Ultimate Limit States	3
2.2 Ultimate Limit States Expressions	3
2.3 Loads	3
2.4 Effects	4
2.5 Load Factors	4
2.6 Section Resistance	4
2.7 Resistance Factors	5
2.8 Short Section Resistance Interaction Diagram	5
3. Short Section Resistance Criteria	6
3.1 Strength Assumptions	6
3.1.1 Model Section	7
3.2 Uncracked Section Resistance	8
3.3 Cracked Section Resistance	8
3.3.1 Plain Masonry	8
3.3.2 Reinforced Masonry	8
3.3.2.1 Underreinforced Sections	9
3.3.2.2 Balanced Sections	9
3.3.2.3 Overreinforced Sections	11
4. Slenderness Effects	12
4.1 Introduction	12
4.2 Moment Magnifier Method, Background Theory	13
4.2.1 Concept and Expressions for Proposed Method	18
4.2.2 Proposed Design Procedure	20
4.2.3 Modified Design Procedure	21
4.3 P- δ Method	22
4.3.1 Concept and Expressions	22
4.3.2 Derivations and Sequence for Exact P- δ Method	23
4.4 Approximate P- δ Methods	26
4.4.1 Modified P- δ Method	26
4.4.2 Quadratic P- δ Method	26
4.4.2.1 Derivations and Expressions, Quadratic Method	26
4.4.2.2 Proposed Design Sequence	29

Contents (cont'd)	Page
5. Application to Test Data	31
5.1 Summary of Methods	31
5.2 Test Series Analysed	31
5.3 Graphic Tabulation of Results	32
5.4 Discussion of Results	32
6. Design Examples	40
6.1 Design Example 1	41
6.2 Design Example 2	50
6.3 Design Example 3	54
7. Comments on Methods	61
7.1 General	61
7.2 Exact P-6 Method	61
7.3 Modified P-6 Method	61
7.4 Quadratic P-6 Method	61
7.5 Moment Magnifier Method	62
8. Potential for Design Aids	62
8.1 Design Aids	63
9. Conclusions	63
10. Recommendations	64
11. Bibliography	65
Appendix A Test Series Output Tables	67
Appendix B Computer Listings	71
Appendix C Derivation of Effective Inertia	87

LIST OF FIGURES

Figure	Title	Page
1	Section resistance parameters	5
2	Uncracked resistance	6
3	Short section resistance diagram	6
4	Model section	7
5	Section forces	7
6	Underreinforced section forces	9
7	Balanced section forces	10
8	Cracked section geometry	10
9	Cracked section forces	10
10	Overreinforced section strains	11
11	P- δ Effect	12
12	Loads and effects	12
13	Beam column with transverse load	14
14	Beam column with end moment	15
15	Beam column	17
16	Primary loads and effects	19
17	Section resistance diagram and magnified e line	20
18	Initialization of P and M	20
19	Resistance diagram and magnified e line, modified procedure	21
20	Convergence of tertiary displacements	22
21	10-segment deflection analysis	23
22	Initialize P	23
23	Tertiary effects	24
24	P-E iteration	25
25	Primary moments at mid height	26
26	Derivation of δe_1 and δe_2	27
27	Interpolation of M_r for cracked transformed section	28

Test Series Graphic Tabulation of Results Ptest/Ptheory

28	S.C.P.I. Plain brick	34
29	Yokel Mathey and Dikkers Series 2	35
30	Hatzinikolas et al E1 to L4	36
31	Hatzinikolas et al L5 to K1	37
32	Hatzinikolas et al S1 to N1	38
33	Slender wall program	39
34 to 52	Design examples	41 to 60

NOTATION

A, A _n	Masonry net mortar bedded and grouted area
a	Depth of compression stress block
A _s , A _{s1} , A _{s2}	Reinforcing steel areas
A _{segx} , A ₁ , A ₂ , A ₃	Areas under M/EI diagram - For virtual work deflection analysis
C	Resultant compressive force
C ₁ , C ₂ , C ₃	Component compressive forces
C _m	Equivalent bending coefficient for moment magnifier method
d	Depth from compressive face to centroid of tension steel
δ, Δ _p , Δ _s , Δ	Flexural displacements
j _d	Moment arm of moment resisting couple T&C
k _d	Depth of compressive stress block (same as "a")
e, E	Virtual eccentricity at a section M/P
e ₁ , e ₂	Eccentricities of loading at wall ends
E _m , E _s	Masonry and steel moduli of elasticity
ε _m , ε _s	Masonry and steel strains
f' _m	Masonry prism strength
F _m	Factored masonry strength φ _m f' _m
F _y	Steel yield stress
F _s	Factored steel yield stress φ _s F _y
H, h	Wall effective height between lateral supports
I, I _o	Uncracked section moment of inertia
I ₁	Effective moment of inertia in Moment Magnifier Method
I ₁ , I ₂	Cracked or uncracked moments of inertia at wall ends 1 and 2 - the degree of cracking depending on e ₁ and e ₂
I _{eff}	Effective moment of inertia of a wall
I _{cr}	Cracked moment of inertia of a wall when P=0 and M ≥ M _{cracking} (transformed section)
L, L ₁ , L ₂	Lengths of model section
M _p	Total primary moment at a section
M, M ₁ , M ₂	Bending moments at section, end 1, end 2
M _i	Moment capacity selected for initial trial
M _w	Moment due to lateral loading (wind)
m	The ordinate of the unit load moment diagram at the centroid of the M/EI diagram - used for virtual work deflection analysis
P	Axial load at section
P ₁ , P ₂	Short section strength at ends 1 and 2 with eccentricities e ₁ and e ₂
P _o	Crushing strength of short section with e=0
P _i	Initial axial load selected for trial
P _{cr}	Euler buckling load
P _{max}	Theoretical maximum load capacity

NOTATION (con't.)

S	Axial load in moment magnifier theory derivations
T	Wall thickness
T _l	Wall face shell thickness
T _s	Steel tensile force
V	Transverse weak or strong axis horizontal shear

1. INTRODUCTION

1.1 BACKGROUND

Limit states design has, in the last two decades, come to be considered as the best modern design philosophy. It provides nearly uniform safety indices over a wide range of loading conditions and applications. The basic concepts are simple and logical and are easily understood and applied by designers and code authorities.

Masonry design, which began thousands of years ago, has accumulated a legacy of semi-empirical design "rules of thumb" that in many cases defy logic and are therefore difficult to extrapolate for design of the many special cases which are commonplace in modern construction.

Much research and testing of masonry walls and other elements has been carried out over a long period of time, but most of the work was carried out in an uncoordinated manner with much duplication of effort and poorly recorded data.

Masonry is, by nature, a complex arrangement of elements partially interconnected by a variety of cementitious materials, metal ties and anchors and involving many people with various skills, attitudes and shortcomings.

For these reasons, limit states design of masonry has lagged behind the other more easily controlled and manipulated materials structural steel and reinforced concrete which have enjoyed limit states design methods for many years.

C.S.A. Technical Committee S304 has established a task group headed by Professor Carl Turkstra of McGill University. This group has undertaken the following task leading to the creation of a Canadian Masonry Limit States Design Code.

- (a) Accumulation of available test data and extraction of useful information.
- (b) Organizing new test programs to fill gaps in the data
- (c) Carrying out safety index analyses of existing code provisions and requirements
- (d) Deriving new design philosophies, expressions and material performance factors reflecting workmanship, inspection and quality control parameters in plant and field
- (e) Carrying out safety index analyses of the proposed new methods and "tuning" them to achieve an acceptable balance between economy and safety

This work has been largely completed and at the present time the second draft of the new standard C.S.A. S304-1 1985 "Limit States Design of Masonry" has just been issued for comment by committee members.

1:2 OBJECTIVE AND SCOPE

As the writer has had experience in code writing and in design, it was generally agreed that a valuable contribution at this formative stage of the new masonry code writing process would be an in-depth designer's assessment of the proposed methods and requirements, with particular emphasis on comprehensibility and ease of applicability in the design office.

Accordingly this paper will be concerned with:

- (a) An assessment of the philosophies of the two proposed methods: the moment magnifier method and the load-deflection (P- δ) methods.
- (b) An assessment of the proposed design expressions, derivations and limited-case simplifications.
- (c) Application of the methods to specific tested cases and assessment of the relation between predicted and tested strengths.
- (d) Application of the methods to typical office design problems with exploration of potential for design aids and assessment of ease of comprehension and application.
- (e) Recommendations for the selection of methods and expressions based on the foregoing assessments.

2. LIMIT STATES DESIGN

2.1 DEFINITIONS

- 2.1.1 LIMIT STATES are the conditions or events that produce impairment of use or performance of a structure or part of a structure. They are usually classified under the following two headings:
- 2.1.2 SERVICEABILITY LIMIT STATES are those states that affect normal use and occupancy and include deflection, vibration, appearance, weathering, corrosion and similar long term effects.
- 2.1.3 ULTIMATE LIMIT STATES are those that affect safety, are related to strength and stability and include overturning, crushing, yielding, splitting, buckling and other effects of overloading and overstressing the members and materials in the structure. Since building codes and authorities are primarily concerned with structural safety, this paper will deal with the ULTIMATE LIMIT STATES.

2.2 ULTIMATE LIMIT STATE EXPRESSIONS

$$R \geq (P+M) \text{ or } V \quad (1)$$

This expression states the requirement of the Ultimate Limit State that the resistance R of a simple masonry wall element shall not be less than the applied loads and effects P , M and V defined as follows:

2.3 LOADS

2.3.1 VERTICAL IN-PLANE LOADS & EFFECTS

$$P = F_d D + F_c (F_L L + F_q Q) \quad \text{where:} \quad (2)$$

P is the total factored applied vertical in-plane (axial) compressive (or tensile) load on the element

D is gravity dead load

L is gravity live load

Q is a shearwall flange load effect caused by strong axis (in plane) shear and bending, usually due to lateral wind, earthquake or soil pressure loads transferred to the wall by floor or roof diaphragm

F_d , F_c , F_L and F_q are load and combination factors (see 2.5)

2.3.2 HORIZONTAL OUT-OF-PLANE LOADS

$$W = F_L W_L \quad \text{Where:}$$

W is the total applied horizontal out-of-plane uniform load on the element

W_L is the specified load

F_L is the specified load factor

2.4 MOMENT EFFECTS

$$M = P_e + M_w + P\delta \quad \text{where:} \quad (3)$$

M is the total weak axis bending at a section along the span of the element

P_e is the weak axis bending at the section induced by P_{e1} & P_{e2} , the end moments due to load eccentricities top & bottom of the wall element

M_w is the weak axis moment induced at the section by the out-of-plane load

$P\delta$ is the weak axis moment induced at the section by the curved path of the in-plane axial loads following the wall centroid which has been deflected out of plane by the applied weak axis moments

2.4.2 V is the in-plane (strong axis) or out-of-plane (weak axis) factored shear force applied at the section, usually caused by lateral loads.

2.5 LOAD FACTORS

The applied loads D, L, Q and W are specified by codes and are factored up (or down) to reduce to an acceptably low level the probability that a more severe load condition will occur during the life of the structure. Factors F_d, F_L, F_q and F_w are called load factors, F_c is called a combination factor and the design loads which are called Factored Loads are obtained by multiplying the specified loads by the load factors and combination factors as expressed in equation (2).

$$\text{Factored Loads} = \sum \text{Specified Loads} \times \text{Load} \times \text{Combination Factors (if any)}$$

2.6 SECTION RESISTANCE

R is the section resistance or strength which is a function of the following parameters (illustrated in Figure (1) and Figure (2))

(a) the factored material strengths $\phi m f'_m$ and $\phi s f_y$

- (b) the section geometry
- (c) the distribution of stress (eccentricity of loading)
- (d) and the degree of cracking (which alters the geometry)

Figure (1) SECTION RESISTANCE PARAMETERS

2.7 RESISTANCE FACTORS

The material strengths f'_m and F_y are factored down by factors ϕ_m and ϕ_s to account for variation in material strength, workmanship, tolerance, etc.

2.8 SHORT SECTION RESISTANCE INTERACTION DIAGRAM: (RESISTANCE DIAGRAM)

The strength or resistance of a given section for various eccentricities of loading is usually expressed as an "interaction diagram" (Figure (3)) which is obtained as follows:

- (a) The upper portion of the diagram represents the wall capacity as the load eccentricity travels from the centroid to the kern. When the load is at the centroid the compressive stress is uniform across the section, the moment is zero and $P = \phi f'_m A_n = P_o$ where A_n is the net area of the section. When the load is at the kern, the stress at the face on the other side of the centroid is zero and the stress at the compression face is $\phi f'_m$ therefore $P = P_o/2$. These stress relationships are shown in Figure (2).

Figure (2) UNCRACKED RESISTANCE

This portion of the curve can be drawn straight line from 0, P_o to $P_o E_k/2$, $P_o/2$ where E_k is the kern eccentricity.

- (b) The lower half of the curve can be obtained by stepping the neutral axis across the section from the tension face toward the compression face, calculating the values of P & M at each step. If the section is plain, the curve will pass through the origin. If the section is reinforced, the curve will pass through $M_o, 0$ where M_o is the pure moment capacity of the section. The moment capacity may be governed by compression in the masonry if over-reinforced or by tension in the steel if under-reinforced.

Figure (3) SHORT SECTION RESISTANCE DIAGRAM

3. SHORT SECTION RESISTANCE CRITERIA

Ojima and Turkstra (1979) studied the effects of linear and non-linear section stress-strain assumptions and concluded that a linear assumption with triangular stress block and ultimate masonry strain between .001 and .002 were conservative and appropriate until better information becomes available.

3.1 STRENGTH ASSUMPTIONS

(a) Modulus of Elasticity

$E_m = 1000f'_m \leq 20,000 \text{ MPa}$ for concrete block masonry

$E_m = 850f'_m \leq 20,000 \text{ MPa}$ for brick masonry

$E_{steel} = 200,000 \text{ MPa}$

(b) Distribution of Strain across Section: Linear

(c) Ultimate Strain: .002

(d) Tensile Strength in Masonry: Zero

(e) Ultimate Compressive Stress in Masonry: $\phi f'_m \leq .002 E_m$

(f) Steel Compressive Stress: Zero

(g) Ultimate Steel Tensile Stress: $\phi_s f_y$

(h) Reinforced Cracked Section Resistance: based on Transformed Section.

3.1.1. MODEL SECTION

Figure (4) shows the geometry and notation which has been adopted for describing a masonry wall section

Figure (4) MODEL SECTION

In this Figure $L1$ is the sum of the thicknesses of mortar bedded webs and grouted cores.

The section forces are illustrated in Figure (5)

Figure (5) SECTION FORCES

3.2 UNCRACKED SECTION RESISTANCE

As the eccentricity of loading moves from the section centroid toward the kern eccentricity the interaction line is straight between points O, P_o & $P_o E_k/2, P_o/2$. This relationship can be expressed as follows:

$$M_r = (P_o - P) E_k \quad (4)$$

where $E_k = S/A$ and $S = 2I/T$

3.3 CRACKED SECTION RESISTANCE

As the eccentricity of loading moves past the kern point, the section begins to crack but the section compressive capacity initially decreases more slowly than the eccentricity is increasing and the moment capacity, which is the product of these two, increases until $P \approx .25T$ in a plain solid section and at this point of maximum moment $P/P_o \approx .4$. However, for simplicity this point will be taken to occur at $P_o/2$.

3.3.1 PLAIN MASONRY

As the eccentricity of load moves from the kern toward the compression face, the moment capacity and load capacity both decrease and the curve finally passes through the origin. Reference (5) recommends the following expression for plain masonry with $P/P_o < .5$.

$$M_r = (1 - gP/P_o) P T / 2 \quad \text{in which } g = 2(1 - 4I/AT^2) \quad (5)$$

3.3.2 REINFORCED MASONRY

A reinforced section behaves like a plain section (because compression in the steel is ignored) until the flexural cracking extends past the first layer of reinforcement causing it to exert a new tensile force. This slows the cracking process, steepening the slope of the interaction curve which eventually passes through the point $M_o, 0$ in which M_o is the pure moment (no axial load) capacity of the section. This portion of the curve can conservatively be taken as a straight line between points $P_o E_k/2, P_o/2$ and $M_o, 0$.

M_o , the section pure moment capacity is governed by the tensile capacity of the steel in underreinforced sections and by the compressive capacity of the masonry in overreinforced sections.

3.3.2.1 UNDERREINFORCED SECTIONS

In these sections the tensile reinforcement has reached yield stress ($\phi_s F_y$) and elongates until the neutral axis has moved to a point where the stress in the masonry reaches the ultimate strength $\phi_m f'_m$ and a secondary compression failure occurs. At this point the moment arm $j d$ is measured from the centroid of the compression block to the centroid of the steel and the resisting moment $M_o = T_s \cdot j d$

For a rectangular section we can express this as follows: (Fig. (6))

$$F_s = \phi_s F_y A_s$$

$$F_m = \phi_m f'_m A_m$$

$$n = E_s / E_m$$

$$C = F_m a / 2$$

$$T_s = F_s A_s$$

$$C = T_s \text{ (if } P=0 \text{)}$$

Figure (6) UNDERREINFORCED SECTION FORCES

$$a = 2 F_s A_s / F_m L$$

$$j d = d - a / 3$$

$$M_o = F_s A_s j d$$

(6)

The depth of the compression zone "a" and the location of the resultant "C" for a hollow or flanged section can be found iteratively by increasing "a" in small increments until $C = T_s$.

3.3.2.2 BALANCED SECTIONS

A balanced section reaches yield stress in the steel and ultimate compressive stress at the compression face at the same time. Using transformed section conventions we can express this as follows:

(a) For a rectangular section: (Fig. (7))

$$F_m = \phi_m f'_m = \epsilon_m E_m \quad F_s = \phi_s F_y = \epsilon_s E_s$$

$$n = E_s / E_m \quad k = \epsilon_m / (\epsilon_s + \epsilon_m) = F_m / (F_s / n + F_m)$$

$$a = kd \quad jd = d - a/3$$

$$C = F_m a L / 2 \quad T_s = F_s A_s$$

$$M_o = M_{bal} = C jd$$

$$T_s = C \text{ therefore}$$

$$A_s'_{bal} = C / F_s \quad (7)$$

Figure (7) BALANCED SECTION FORCES

Similarly for a flanged section (using model section); Fig. (8) and Fig. (9)

Figure (8) CRACKED SECTION GEOMETRY

Note that if $kd \leq t_f$ the section capacity can be calculated as a rectangular section.

$$F_s = \phi_s F_y \quad n = E_s / E_m$$

$$F_m = \phi_m f'_m \quad \epsilon_m = 0.002 E_m$$

$$k = F_m / (F_s / n + F_m)$$

$$A_s'_{bal} = C / \phi_s F_y =$$

$$(C_1 + C_2 + C_3) / F_s \quad (8)$$

$$C = T_s$$

Figure (9) CRACKED SECTION FORCES

The primary function of the balanced section is to indicate the transition point between under-reinforced and overreinforced sections.

3.3.2.3 OVERREINFORCED SECTIONS

In these sections the steel is not at yield and the section fails when the steel force T_s equals the maximum compressive force C that the masonry can develop. Since both T_s and C are dependent on the location of the neutral axis, the problem involves the solution of a quadratic equation or an iterative approach in which the neutral axis is stepped across the section until $T_s = C$.

For a solid overreinforced section the solution can be derived as follows: (Fig.(10))

$$C = T_s = \phi_m f'_m k d L / 2$$

$$T_s = A_s E_s \epsilon_s$$

$$F_m = \phi_m f'_m \neq .002 \epsilon_m$$

$$A_s E_s \epsilon_s = F_m L d \epsilon_m / 2 (\epsilon_m + \epsilon_s)$$

Multiply both sides -
by $(\epsilon_m + \epsilon_s)$

$$\epsilon_s^2 (A_s E_s) + \epsilon_s (A_s E_s \epsilon_m) - F_m L d \epsilon_m / 2 = 0$$

Which is a quadratic
equation the
solution to
which is:

$$\epsilon_s = .5 \epsilon_m \left(\sqrt{1 + \frac{2 F_m L d}{A_s E_s \epsilon_m}} - 1 \right)$$

$$M_p = A_s E_s \epsilon_s d \left(1 - \frac{\epsilon_m}{3(\epsilon_m + \epsilon_s)} \right) \quad (9)$$

For a hollow section the derivation of a rigorous solution does not seem practical and the iterative approach is more attractive.

Figure (10) OVERREINFORCED
SECTION STRAINS

4. SLENDERNESS EFFECTS

4.1 INTRODUCTION

If a slender wall becomes curved and is subjected to an axial load, a moment is induced at each section that is the product of the axial load P and the displacement of the centroid of the section from a centroidal plane through the wall ends. This moment is called the P - δ effect and is usually referred to as a secondary effect and is illustrated by Figure (11).

Figure (11) P - δ EFFECT

The area under this secondary moment curve produces an additional displacement which we will refer to as a tertiary effect and which if smaller than the secondary displacement will converge in subsequent cycles to a final value.

However, if P is sufficiently high, or the wall sufficiently slender, the flexural stiffness of the wall will be insufficient to limit the displacements which will increase until material failure occurs.

The lowest value of P that causes this type of failure is known as P_{cr} , the Euler critical buckling load.

4.1.1 GENERAL EXPRESSIONS

For a masonry wall subjected to the usual types of loads and effects (shown in Fig.(12)) the ultimate limit state expressions are:

Figure (12) LOADS AND EFFECTS

$$M1 \frac{(h-x)}{h} + M2 \frac{x}{h} + \frac{W}{2} (Hx - x^2) + P\delta x - Mr = 0 \text{ at any point } x \quad (10)$$

at mid height:

$$\frac{M1+M2}{2} + \frac{Wh^2}{8} + P\delta_{max} - Mr = 0 \quad (11)$$

$$\text{at end 1: } M1 - Mr = 0 \quad (12)$$

$$\text{at end 2: } M2 - Mr = 0 \quad (13)$$

where Mr is obtained from the resistance diagram at ordinate P .

The slenderness effect is contained in the term $P-\delta$ which occurs at points along the span that have been displaced from the centroidal plane through the wall ends.

Note that once the $P-\delta$ effect has been evaluated, the section strength governs capacity and if either P or δ are zero there is no slenderness effect. Walls that are not supporting axial loads have no tendency to buckle and therefore have no slenderness limitations other than those required to limit deflection under lateral loading. The displacement δ is zero at the wall ends (in most cases) so short section strength governs at these points. It has been proposed that slenderness effects can be ignored when:

$$h/r < 40 - 20e1/e2 \text{ for } 0 < e1/e2 \leq 1 \text{ or} \quad (14)$$

$$h/r < 40 - 40e1/e1 \text{ for } -1 \leq e1/e2 \leq 0 \quad (15)$$

in which r is the radius of gyration of the wall section.

There has been some controversy as to the best way of including the slenderness effect, to calculate it directly (the $P-\delta$ method), or to use the moment magnifier method. A main objection to the moment magnifier method was some uncertainty as to whether it applied to beam-columns with lateral loading. The following derivations confirm that it is applicable.

4.2 MOMENT MAGNIFIER METHOD, BACKGROUND THEORY

Timoshenko has derived the following expressions for the behaviour of beam-columns:

For a beam-column subjected to an axial load S and a transverse load P (Fig.(13))

Figure (13) BEAM-COLUMN WITH TRANSVERSE LOAD

the differential equations for the deflection curves for the two portions of the span are

$$EI \frac{d^2 y}{dx^2} = -Sy - \frac{Pc}{L} \quad \text{and} \quad (a)$$

$$EI \frac{d^2 y}{dx^2} = -Sy - \frac{P(L-c)}{L} (L-x) \quad (b)$$

letting $p^2 = \frac{S}{EI}$ and obtaining the constants of integration from the boundary conditions at the ends of the span and the slope continuity at 'P', we obtain for the left portion of the span

$$y = \frac{P \sin pc}{S p \sin pL} \sin px - \frac{Pc}{SL} \quad (c)$$

and by differentiating this we get

$$\frac{dy}{dx} = \frac{P \sin pc}{S \sin pL} \cos px - \frac{Pc}{SL} \quad \text{and} \quad (d)$$

$$\frac{d^2 y}{dx^2} = -\frac{Pp \sin pc}{S \sin pL} \sin px \quad (e)$$

Similarly for the right portion we can derive

$$y = \frac{P \sin p(L-c)}{S p \sin pL} \sin p(L-x) - \frac{P(L-c)}{SL} (L-x) \quad (f)$$

$$\frac{dy}{dx} = -\frac{P \sin p(L-c)}{S \sin pL} \cos p(L-x) + \frac{P(L-c)}{SL} \quad \text{and} \quad (g)$$

$$\frac{d^2 y}{dx^2} = -\frac{Pp \sin p(L-c)}{S \sin pL} \sin p(L-x) \quad (h)$$

If c is made very small, M_0 is substituted for Pc and pc substituted for $\sin pc$ in these expressions, we obtain solutions for a beam subjected to an end moment M_0 and axial load S as follows: (Fig. (14))

Figure (14) BEAM-COLUMN WITH END MOMENT

$$y = \frac{M_o}{S} \frac{(\sin px - x)}{\sin pL} \quad (i)$$

from which

$$\frac{dy}{dx} = \frac{M_o}{S} \frac{(P \cos px - 1)}{\sin pL} \quad (j)$$

An examination of expressions (c) to (j) reveals the following concepts:

- (a) The deflections, slopes and curvatures are directly proportional to the transverse loads P and applied end moments M_o .
- (b) The axial load S has a non-linear effect since it appears directly and indirectly (in p) in the expressions.

From this it follows that if S is held constant, superposition can be used to determine the effects of multiple transverse loads and end moments.

Using this principle we can derive for a beam-column with uniform transverse load q (many small concentrated loads) and axial load S :

$$M_{max} = -EI \left(\frac{d^2 y}{dx^2} \right) = EI q \frac{(1 - \cos pL/2)}{S \cos pL/2} = \frac{qL^2}{8} \frac{2(1 - \cos u)}{u^2 \cos u} \quad (k)$$

in which

$$x = L/2 \text{ and } u^2 = \frac{p^2 L^2}{4} = \frac{SL^2}{4EI}$$

The deflection curve of a beam can be represented by a trigonometric series as follows:

$$y = a_1 \sin \frac{\pi x}{L} + a_2 \sin \frac{2\pi x}{L} + a_3 \sin \frac{3\pi x}{L} \dots \quad (l)$$

The coefficients a_1, a_2 etc. are obtained by considering strain energy of the beam and are for a beam with a single transverse load P :

$$a_n = \frac{2PL^3}{EI\pi^4} \cdot \frac{1}{n^4} \cdot \frac{\sin n\pi c}{L}$$

from which

$$y = \frac{2PL^3}{EI\pi^4} \sum_{n=1}^{\infty} \frac{1}{n^4} \cdot \frac{\sin n\pi c}{L} \cdot \frac{\sin n\pi x}{L} \quad (m)$$

for a beam with a transverse load P and axial load S we can derive

$$a_n = \frac{2PL^3}{EI\pi^4} \cdot \frac{1}{n^2(n^2 - \frac{SL^2}{EI\pi^2})} \cdot \frac{\sin n\pi c}{L} \quad (n)$$

in which the term $\frac{SL^2}{EI\pi^2}$ is the ratio of the axial load S to the critical buckling load $S_{cr} = \frac{2EI}{L^2}$

letting $\alpha = S/S_{cr} = \frac{SL^2}{EI\pi^2}$ we obtain

$$a_n = \frac{2PL^3}{EI\pi^4} \cdot \frac{1}{n^2(n^2 - \alpha)} \cdot \frac{\sin n\pi c}{L} \quad (o)$$

Substituting this into equation (a) the deflection curve is:

$$y = \frac{2PL^3}{EI\pi^4} \sum_{n=1}^{\infty} \frac{1}{n^2(n^2 - \alpha)} \cdot \frac{\sin n\pi c}{L} \cdot \frac{\sin n\pi x}{L} \quad (p)$$

It can be shown that the first term of the series provides a good approximation of the deflection, letting $n=1$ we obtain

$$y = \frac{2PL^3}{EI\pi^4} \cdot \frac{1}{(1 - \alpha)} \cdot \frac{\sin \pi c}{L} \cdot \frac{\sin \pi x}{L} \quad (q)$$

If we compare this with equation (m), the deflection of a beam with transverse loading but no axial load, we can see that the effect of the axial load is contained in the expression $1/(1 - \alpha)$ which is a deflection "magnifier". Thus we have obtained the approximate relationship

$$\delta = \delta_0 / (1 - \alpha) \quad (r)$$

in which:

δ is the total deflection at a point due to axial and transverse loads, end moments and secondary effects.

δ_0 is the primary deflection at the point due to the transverse loads and end moments.

Note that the "magnifier" is applied to the deflection, not the moment, that is:

$$M_{\max} = S(e + \delta) = S\left(e + \frac{\delta_0}{1 - \alpha}\right) \text{ which is not the same as}$$

$$M_{\max} = Se\left(\frac{1}{1 - \alpha}\right) \quad (18)$$

as it is commonly used.

Using and expanding upon the examples given by Timoshenko, we can see the differences.

For a strut with axial load and equal end moments Se (Fig. (15))

$$p^2 = S/EI$$

$$pL = \sqrt{S/EI} \cdot L$$

$$L = H/2$$

$$S_{cr} = \pi^2 EI / 4L^2$$

$$\alpha = S/S_{cr} = \frac{4SL^3}{\pi^2 EI} = \frac{4(pL)^2}{\pi^2} = .405(pL)^2$$

Figure (15) BEAM-COLUMN

pl	.1	.5	1.0	1.5	$\pi/2$
α	.004	.101	.405	.911	1.0
$\frac{1}{1-\alpha}$	1.004	1.112	1.681	11.235	∞
Exact δ	.005e	.139e	.851e	13.136e	∞
Approx $\delta = \frac{SeL^2}{2EI} \cdot \frac{1}{1-\alpha}$	.005e	.139e	.841e	12.639e	∞
$M = S(e + \delta)$	1.005Se	1.139Se	1.851Se	14.13Se	∞
Approx $M = Se(\frac{1}{1-\alpha})$	1.004Se	1.112Se	1.681Se	11.235Se	∞

% Diff. M & Approx. M 1% 2.5% 10% 25.5%

Table (1) ACCURACY OF MOMENT MAGNIFIER APPROXIMATIONS

From all this we can conclude that the moment magnifier method is applicable to beam-columns with lateral loads.

4.2.1 CONCEPT AND EXPRESSIONS FOR PROPOSED METHOD

Hatzinikolas, Longworth and Warwaruk have proposed the following expressions and procedures for a moment magnifier method of design for masonry walls:

$$M = M_p \frac{CH}{1 - P/P_{cr}} \quad (19)$$

where M_p is the maximum primary moment along the span.

If $w=0$, $CH = .6 + .4M_1/M_2 \geq .4$ and $M_1 < M_2$ (20)

If $w > 0$ then $CH = 1$

For a plain or reinforced wall bent in single curvature

$$P_{cr} = \pi^2 EI / H^2 \quad (21)$$

where:

(a) For a plain wall with no tensile strength

$$I = 8(.5 - e/t)^3 I_o \quad (22)$$

$$\text{where } e = \frac{(e_1 + e_2)}{2}$$

(b) For a plain wall with tensile strength $f't$

$$II = 8I_o(.5 - e/t + D_t/2t)^3 \quad (23)$$

in which $D_t = EIf't/F_{max}$

$f't$ = Tensile strength (taken as .025Mpa)

$$F_{max} = 4 \left(\frac{P}{3(A(1-2e/t))} \right) \text{ and } El = 2t \frac{P}{AF_{max}}$$

(c) For a reinforced wall:

$$\text{If } e \leq e_k, \quad II = 8(.5 - e/t)^3 I_o$$

$$\text{If } e > e_k, \quad II = (.5 - e/t) I_o + .1 I_o$$

$$\text{in which } e = (e_1 + e_2)/2$$

For a plain or reinforced wall bent in double curvature:

$$P_{cr} = T_o E I_o / h^2 \quad (24)$$

in which T_o is a buckling coefficient for a stepped column. This is presented in the references as a table of values. The following algorithm was developed to provide approximate values for the computer program written to perform the calculations:

$$T_o = 10(B + A I^3(1-B)) + 2 A I^2 + B^3 \times 10 \quad (25)$$

$$\text{in which } B = I_1/I_o \text{ and } A I = e_1/(e_1 + e_2)$$

The ultimate limit state design requirements are illustrated by the following Loads and Effects diagram (Fig. (16)) and the Section Resistance diagram (Fig. (17)).

Figure (16) PRIMARY LOADS AND EFFECTS

Figure (17) SECTION RESISTANCE DIAGRAM & MAGNIFIED e LINE
For given e_1 , e_2 and w , Governing Load =
smallest of P_1 , P_2 and P_{max} .

4.2.2 PROPOSED DESIGN PROCEDURE:

The method proposed for finding the point of intersection of the magnified e line and the short section resistance interaction curve was the following:

- (a) Start with initial values of P_i and M_i obtained from the intersection of an unmagnified e line and the section resistance curve (Fig.(18))

Figure (18) INITIALIZATION OF P_i & M_i

- (b) Using this value of P , calculate P_{cr} using expressions (21) to (24)
- (c) Substituting these values of P_{cr} and M into the expression $M = PeCH / (1 - P/P_{cr})$, calculate a new value of P .
- (d) Determine the corresponding value of M from the resistance curve and without changing e repeat from (b) until $P_{new} \approx P_{old}$ with sufficient accuracy.

This method was studied and tried but finally abandoned for several reasons:

- (i) The rationale was obscure.
- (ii) There was no provision to include lateral loading w .
- (iii) Cases were tried that did not converge quickly and some that did not converge at all.
- (iv) The following procedure was developed that eliminated both disadvantages and could be

plotted by the computer so that the process could be directly observed.

4.2.3 MODIFIED DESIGN PROCEDURE

For given values of e_1 , e_2 and W to find P_{max} :

- (a) Obtain initial value of P at intersection of e_1 or e_2 line and resistance curve.
 - (b) Calculate P_{cr} and $M_{max} = M_p CM / (1 - P/P_{cr})$ where M_p is the maximum primary moment along span.
 - (c) If $M_{max} > M_r$ @ P , reduce P .
 - (d) Go to (b) and repeat until $M_{max} \leq M_r$ @ P .
- This is illustrated by Figure (19).

Figure (19) RESISTANCE DIAGRAM & MAGNIFIED e LINE MODIFIED PROCEDURE

This was particularly interesting where walls exhibited instability and no intersection occurred. Subroutine "H&WMM" in Appendix B was written to perform these calculations. Subroutine "H&WM1" also in Appendix B is a variation of the foregoing subroutine "H&WMM" that was modified to carry out the analysis of a wall with constant P , e_1 and e_2 and variable lateral load W . This analysis is accomplished in the following steps:

- (1) Calculate section moment resistance M_r at P
- (2) Calculate initial $W = 8M_r/H^2$
- (3) Find maximum primary Moment along span (M_p).
- (4) Calculate P_{cr} and $M_{max} = M_p CM / (1 - P/P_{cr})$.
- (5) If $M_{max} > M_r$ at P , reduce W .
- (6) Go to 3 and repeat until $M_{max} \leq M_r$ @ P

4.3 THE P- δ METHOD

4.3.1 CONCEPT AND EXPRESSIONS

Turkstra, Ojinaga and others have developed behavioural theory and expressions for a P- δ method for masonry design. This method has been proposed in various forms but the basic concepts are:

- The flexural stiffness of the wall is estimated, taking into account the effects of curvature, cracking and reinforcement.
- Displacements (δ_s) due to primary moments using a trial value for P (or W) are calculated.
- The secondary (P- δ) moments are added to the primary moments $M = M_p + P$
- If $M > M_r$ at P, reduce P (or W) and repeat from (b)

It has been proposed that tertiary effects - the displacements due to the P- δ moment itself - can be neglected within the range of practical slenderness ratios. However, in order to assess the importance of these effects, an iterative routine has been included in the program that cycles through the deflection analysis until the tertiary displacements have converged to a very small value. (Fig.(20))

Figure (20) CONVERGENCE OF TERTIARY DISPLACEMENTS

Ojinaga and Turkstra (1977) have recommended the following expressions for estimating the effective inertia of a wall. These are based on the assumptions shown by the diagram in Appendix C

- For plain masonry walls and columns

$$I_{eff} = (I_1 + I_2)/4 \text{ for } 0 \leq e_1/e_2 \leq 1 \quad (27)$$

$$I_{eff} = \text{lesser of } (I_1 + I_0)/4 \text{ or } (I_2 + I_0)/4$$

$$\text{for } -1 \leq e_1/e_2 \leq 0 \quad (28)$$

where I_1 and I_2 are cracked or uncracked moments of inertia (depending on e_1 and e_2) at ends 1 and 2 and I_0 is the uncracked moment of inertia of the wall section.

(b) For reinforced walls and columns:

$$I_{eff} = (I_1 + 2I_{cr} + I_2)/4 \text{ for } 0 < e_1/e_2 \leq 1 \quad (29)$$

$$I_{eff} = \text{lesser of } (I_1 + 2I_{cr} + I_0)/4 \text{ or } (I_2 + 2I_{cr} + I_0)/4 \text{ for } -1 \leq e_1/e_2 \leq 0 \quad (30)$$

in which I_{cr} is the cracked moment of inertia of the transformed section of the wall subjected to a pure bending moment equal to the pure moment resistance M_0 . I_1 and I_2 are also calculated using a transformed section if cracked.

4.3.2 DERIVATIONS AND SEQUENCE FOR EXACT P- δ METHOD

Figure (21) illustrates the method used to calculate the wall deflection curve produced by the externally induced moments.

$$\delta_{total} = \int \frac{M \bar{m} dx}{EI} = \sum_{i=1}^{10} \frac{A_{SEG} \bar{m}_{SEG}}{EI}$$

Figure (21) 10-SEGMENT DEFLECTION ANALYSIS

Subroutine ITP-6 in Appendix B was written to do this analysis. The sequence of the analysis was the following:

- (a) Initialize P at intersection of maximum e line and resistance curve (Fig. (22)).

Figure (22) INITIALIZE P

- (b) Calculate primary deflections $D_p(2-10)$ due to areas under primary moment curves at nine stations (10 segments).
- (c) Check mid-height stability of plain walls
 $(A_1 \& A_2 = 0)$ is $D_p(6) + (e_1 + e_2)/2 > t/2$?
 If so, reduce P and repeat from (b).
- (d) Calculate tertiary effects (secondary deflections) $D_s(2-10)$ due to area under $P\delta$ moment curve (Fig. (23)).

Figure (23) TERTIARY EFFECTS

- (e) If $\max D_s(2-10)$ greater than $0.5(\max D_p(2-10))$ then poor convergence, so reduce P and repeat from (b).
- (f) Sum displacements: First time through set total displacement $D(2-10) = D_p(2-10) + D_s(2-10)$. On additional iterations, set $D(2-10) = D(2-10) + D_s(2-10)$ and set $D_p(2-10) = D_s(2-10)$.
- (g) If $\max D_s(2-10)$ greater than limit (2mm) then repeat from (d) (if previous cycle error less than $0.1P_0$).
- (h) Calculate final $P\delta$ moments and add to primary moments for total moments $M(2-10)$.
- (i) Select absolute maximum M along section and calculate $E = M/P$.
- (j) If $E \leq K_1$ then $P = P_0 K_1 / (E + K_1)$ and $M = P \cdot E$ and go to (L).
- (k) If $E > K_1$ then iterate neutral axis across section to find P at E . (Fig. (24))
 $P_2 = C - T_s$ and
 $M = T_s(d - t/2) + C(t/2 - c)$
 $E_4 = M/P_2$

Iterate until $E_4 = E$

Figure (24) P-E ITERATION

(L) $P_3 = P - P_2$ cycle error

(m) If $ABS(P_3) > .1P_o$ set flag to prevent tertiary displacement calculation on next cycle.

(n) If $ABS(P_3) < .03P_o$ go to (p)

(o) If $P_3 < 0$, $P = P + P_o/50$ and go to (b)

If $P_3 > .15P_o$, $P = P - P_o/10$ and go to (b)

If $P_3 \leq .15P_o$, $P = P - P_o/50$ and go to (b)

(p) $P(4) = P/1000$

(q) Finished.

Subroutine "ITP-61" in Appendix B is a variation of the foregoing subroutine "ITP-6" which has been modified to accommodate the analysis of a wall with P , e_1 and e_2 constant and lateral load W variable. This is accomplished in the following steps:

(a) Calculate section moment resistance M_r at $P = P_{test}$.

(b) Initialize W using $W = 8M_r/H^2$

(c) Calculate deflections and $P-\delta$'s at stations along span.

(d) Sum primary moments and $P-\delta$'s at stations and select maximum moment M_4 .

(e) Cycle error $M_3 = M_r - M_4$.

(f) If $M_3 < .1M_r$ then finished.

(g) If $M_4 > M_r$ then reduce W and repeat from (c).

(h) If $M_4 < M_r$ then increase W and repeat from (c).

4.4 APPROXIMATE P- δ METHODS

As previously discussed, these methods are proposed for walls with sufficient stiffness that tertiary effects can be safely ignored. It has been suggested that this limitation could be $H/t \geq 25$. The following two methods have been proposed in the references and expanded into design methods for this project:

4.4.1 MODIFIED P- δ METHOD

This is similar to that described for the exact analysis (subroutine "ITP- δ ") but only the primary displacements are calculated. The subroutine "ITP- δ " can be inhibited from calculating tertiary effects by assigning flag 22=1 in the argument list. The station with maximum primary + secondary moment along the span governs the design.

4.4.2 QUADRATIC P- δ METHOD

This method solves a quadratic equation $P^2A+PB+C=0$ for P at mid height of the wall. A, B and C are derived to include for primary moments, secondary P- δ moment and section resistance. The wall is checked at each end and at mid height as follows:

4.4.2.1 DERIVATIONS AND EXPRESSIONS QUADRATIC METHOD

- (a) At the ends, using equations (12) and (13):

$$Pe_1 - Mr = 0$$

(12)

$$Pe_2 - Mr = 0$$

(13)

- (b) At mid height calculate the primary moment
Figure (25).

$$\frac{e_1 + e_2}{2} \pm 0.05$$

Figure (25) PRIMARY MOMENTS AT MID-HEIGHT

Substituting into equation (13) we can derive the general expression:

$$P\left\{\frac{e_1+e_2}{2} + \frac{P(e_1+e_2)H^2}{16E_{\text{eff}}} + \frac{5WH^4}{384E_{\text{eff}}}\right\} + \frac{WH^2}{8} - Mr = 0 \quad (31)$$

in which the displacements δe_1 and δe_2 can be evaluated using virtual work as follows:

$$EI\delta e_1 = \frac{Pe_1}{2} \cdot \frac{H}{4} \cdot \frac{H}{12} + \frac{Pe_1}{2} \cdot \frac{H}{2} \cdot \frac{H}{8} + \frac{Pe_1}{2} \cdot \frac{H}{4} \cdot \frac{H}{6} = \frac{Pe_1 H^2}{16} \quad (\text{From Fig. (26)})$$

$$A_1 = \frac{Pe_1 \cdot H}{2 \cdot 4}$$

$$A_2 = \frac{Pe_1 \cdot H}{2 \cdot 2}$$

$$\text{Similarly } EI\delta e_2 = \frac{Pe_2 H^2}{16}$$

Figure (26) DERIVATION OF δe_1 AND δe_2

Letting $I = I_{\text{eff}}$ from equations (27) or (28) and Substituting into equation (31):

$$P\left\{\frac{e_1+e_2}{2} + \frac{P(e_1+e_2)H^2}{16E_{\text{eff}}} + \frac{5WH^4}{384E_{\text{eff}}}\right\} + \frac{WH^2}{8} - Mr = 0$$

$$P^2 \frac{(e_1+e_2)H^2}{16E_{\text{eff}}} + P\left\{\frac{e_1+e_2}{2} + \frac{5WH^4}{384E_{\text{eff}}}\right\} + \frac{WH^2}{8} - Mr = 0 \quad (32)$$

In order to be able to solve this equation for P, Mr must be evaluated. Using previously derived expressions for:

(a) Plain and reinforced masonry, uncracked (equation (4))

$$P \gg P_o/2 \text{ and } E \leq E_k$$

$$Mr = (P_o - P)E_k \text{ where } E_k = 2I/A_t \text{ (Kern eccentricity)}$$

Substituting into (32):

$$P^2 \frac{(e_1+e_2)H^2}{16E_{\text{eff}}} + P\left\{\frac{e_1+e_2}{2} + \frac{5WH^4}{384E_{\text{eff}}} + E_k\right\} + \frac{WH^2}{8} - P_o E_k = 0 \quad (33)$$

(b) Plain masonry, cracked (Equation (5))

$$P < P_o/2 \text{ and } E > E_k$$

$$M_r = \frac{P_t - P^2 t + 4 P^2 I_o}{2 P_o P_o A t} = \frac{P_t - P^2 t + 2 P^2 E_k}{2 P_o P_o}$$

Substituting into equation (32)

$$P^2 \left\{ \frac{(e_1 + e_2) H^2 + t}{16 E I_{eff} P_o} - \frac{2 E_k}{P_o} \right\} + P \left\{ \frac{e_1 + e_2}{2} + \frac{5 W H^4}{384 E I_{eff}} - \frac{t}{2} \right\} + \frac{W H^2}{8} = 0 \quad (34)$$

(c) Reinforced masonry, cracked transformed section

$$P < P_o/2 \quad E > E_k$$

Figure (27) INTERPOLATION FOR M_r , CRACKED TRANSFORMED REINFORCED SECTION

In Fig. (27) interpolate M_r between M_o and $P_o E_k/2$ as P goes from $P_o/2$ to Zero.

$$M_r = M_o + P \frac{(.5 P_o E_k - M_o)}{.5 P_o}$$

$$= M_o + P \frac{(E_k - 2 M_o)}{P_o} \quad (35)$$

In this case, we must also evaluate M_o .

Referring to equations (7) and (8) we can evaluate A_{sbal} .

If $A_s \leq A_{sbal}$ the section is underreinforced.

If $A_s > A_{sbal}$ the section is overreinforced.

For an underreinforced section using equation (6)

$M_o = F_s A_s j d$ where $F_s = \phi_s F_y$ and $j d$ is evaluated as in 3.3.2.1

For an overreinforced solid section we can use equation (11)

$$M_o = A_s E_s E_{sd} \left(1 - \frac{\epsilon_m}{3(\epsilon_m + \epsilon_s)} \right) \quad (11)$$

where ϵ_s is obtained from equation (10)

As previously discussed, M_o can be evaluated for a hollow or flanged section by iteration (see Fig. (24)). The neutral axis can be stepped across the section, starting at the tension face (where $E=E_k$), evaluating T_s and C at each step until $C=T_s$ at which point $P=0$ and the section pure moment capacity M_o will have been reached. Subroutine "P-E" in Appendix B performs this procedure when flag Z (in the argument list) is set = 3.

Substituting M_r from equation (35) into equation (32) we obtain:

$$\frac{P^2(e_1+e_2)H^2}{16EmI_{eff}} + P\left(\frac{e_1+e_2}{2} + \frac{5WH^4}{384EmI_{eff}} + \frac{2M_o-E_k}{P_o}\right) + \frac{WH^2}{8} - M_o = 0 \quad (36)$$

Equations (33), (34) and (36) provide approximate mid-height capacities for plain and reinforced masonry walls with axial loads, end moments and applied lateral pressures.

4.4.2.2 PROPOSED DESIGN SEQUENCE

Subroutine "APP-6" in Appendix B calculates wall capacities by this method in the following steps:

- (a) Calculate short section capacities P_1 and P_2 at wall ends (at eccentricities e_1 and e_2).
- (b) Substitute e_1 , e_2 , W and H into equation (33) and solve for P .
- (c) If $P \geq .5P_o$, finished, go to (g).
- (d) If $P < .5P_o$, section cracked, go to (e) or (f).
- (e) If plain ($A_{s1}=0$ & $A_{s2}=0$) use equation (36) and solve for P .
- (f) If reinforced ($A_{s1}>0$ or $A_{s2}>0$) use equation (36) and solve for P .
- (g) Wall capacity $P(5)$ is the smallest of P_1 , P_2 or P .

Subroutine "APP-61" in Appendix B is a variation of the foregoing subroutine "APP-6" which has been modified to accommodate the analysis of a wall with P , e_1 and e_2 constant and W variable. This is accomplished in the following steps:

(a) Calculate section moment resistance M_r at $P = P_{test}$.

(b) Using the general load resistance expression equation (32), which has been rearranged as:

$$W = \frac{\left\{ M_r - P^2 \frac{(e_1 + e_2)H^2 + P(e_1 + e_2)}{2} \right\}}{\left(\frac{5PH^4}{384E_m I_{eff}} + \frac{H^2}{8} \right)} \quad (37.)$$

Calculate W

5. APPLICATION TO TEST DATA

As it is assumed that loads, material strengths and tolerance were accurately controlled in the tested walls, no load or resistance factors have been applied in the applications of the methods to test data. Tests with zero eccentricities were run with .05t single curvature eccentricity to assess the adequacy of the proposed minimum code eccentricity and to avoid problems with some of the expressions which cease to function at zero eccentricity.

5.1 SUMMARY OF METHODS

Variable	Subroutine	
	P	W
Exact P- δ method	ITP- δ	ITP- $\delta 1(Z2=0)$
Modified P- δ method	ITP- $\delta 2$	ITP- $\delta 3(Z2=1)$
Quadratic P- δ method	APP- δ	APP- $\delta 1$
Moment Magnifier method	H&WMM	H&WMM1

5.2 TEST SERIES ANALYSED

The wall test series analysed are as follows:

	No. walls
(a) S.C.P.I. Plain brick wall test series	14
(b) Yokel Mathey & Dikkers test series #2	16
(c) Hatzinikolas Longworth and Warwaruk test series	67
(d) Slender walls research program test series California Structural Engineers	14

The first three series (a), (b) and (c) were wall elements loaded axially and eccentrically. The analytical routines calculated the maximum axial load P that each wall is theoretically capable of carrying and compared this with the ultimate test load Ptest. The output tables give values for Ptest/P

The last series were slender wall elements with a small constant axial load applied eccentrically at one end and a variable transverse pressure applied uniformly to one face. The analysis routines were modified to calculate the theoretical maximum transverse pressure W that each wall is capable of resisting and compare this value with the ultimate test load Wtest.

5.3 GRAPHIC TABULATION OF RESULTS

The following six pages are the graphically tabulated results of the test series analysis. The values have been classified according to individual test parameters H/t , e_1/e_2 and e/t in an attempt to expose any limits of applicability for the methods.

The numerical output tables which contain the wall geometry and material parameters, P_{test} (or W_{test}), P_{test}/P_{theory} (or W_{test}/W_{theory}) and calculated values of H/t , e_1/e_2 and e/t are contained in Appendix B.

5.4 DISCUSSION OF RESULTS

All methods gave fairly consistent predictions for the strength of plain and reinforced concrete block, values of P_{test}/P_{theory} ranging from 0.8 to 4.1 for the P- δ methods and up to 5.5 for the moment magnifier method. However, if the cases for plain walls with e/t greater than $1/3$ are eliminated, the spread for the moment magnifier method reduces to .8 to 2.5 and for the P- δ methods .8 to 4.0.

The methods all gave safe prediction of strength for plain brick walls, but the scatter was large, ranging from:

For the P- δ methods

1.8 to 5.6 for $h/t \leq 23$ and $e/t \leq .33$

1.8 to 21.4 for h/t up to 46 and e/t up to .34

For the Moment Magnifier Method

3.8 to 7.6 for $h/t \leq 23$ and $e/t \leq .33$

3.8 to 19.8 and higher (no intersection) for h/t up to 46 and e/t up to .34

GRAPHIC TABULATION
OF RESULTS

METHOD SYMBOLS

- ▲ Exact P-S Method
- ◻ Modified P-S Method (Primary Deflections)
- ◉ Quadratic P-S Method
- ◊ Moment Magnifier Method

TEST SERIES: YOKEL MATHEY & DIKKERS SERIES 2

Figure (29)

- TEST SERIES: HATZINIKOLAS et al E1 to L4

36

METHOD SYMBOLS

- ▲ Exact P-S Method
- Modified P-S Method (Primary Deflections)
- Quadratic P-S Method
- × Moment Magnifier Method

TEST SERIES: HATZINIKOLAS et al L5 to K1

Figure (31)

METHOD SYMBOLS

- ▲ Exact P-S Method
- Quadratic P-S Method
- Modified P-S Method (Primary Deflections)
- Moment Magnifier Method

TEST SERIES: HATZINIKOLAS et al S1 to N1

Figure (32)

METHOD SYMBOLS

- ▲ Exact P-S Method
- Modified P-S Method (Primary Deflections)
- Quadratic P-S Method
- Moment Magnifier Method

TEST SERIES: SLENDER WALL PROGRAM

Figure (33)

DESIGN EXAMPLES

6 DESIGN EXAMPLES

6.1 DESIGN EXAMPLE 1

Warehouse bearing sidewall

Specified gravity loads:

Roof snow load $2.0 \text{ KPa} \times 6 \text{ m} = 12 \text{ kN/m}$

Roof dead load

tar & gravel .6

steel .3

dead load $.9 \times 6 = 5.4$

Total load $2.9 \text{ KPa} \quad 17.4 \text{ kN/m}$

Specified wind load .7 KPa

Eccentricities:

At top zero - use hinged bearing

At bottom - partially fixed

Assume .05t top & bottom
(single curvature)

Trial section:

240 concrete block $f'_m = 15 \text{ MPa}$

2-15M vert. at 1200 $AS1 = 200$

Fig. (36) MODEL SECTION

FIG. (35)

Material Strength Factors:

Masonry flexure and shear $\phi_m = .4$

Reinforcement $\phi_s = .5$

EVERY 7TH CORE REINFORCED

FIG. (38)

ACTUAL SECTION
FIG. (37)

Section Properties:

$$\text{Area } A_n = (1200)(70) + (189)(170) = 116130 \text{ mm}^2$$

$$F_m = \phi_m f'_m = (.4)(15) = 6 \text{ MPa} \quad \phi F_y = (.5)(400) = 200$$

$$E_m = 1000 f'_m = 15000 \text{ MPa}$$

$$I_g = \frac{(189)(240^3)}{12} + (2) \frac{(1011)(35^3)}{12} + (2)(1011)(35)(102.5^2)$$

$$= (968.5)(10^6 \text{ mm}^4)$$

INPUT L, L1, T, T1, e1, e2

1200, 189, 240, 35, 12, 12

INPUT f'_m , F_y , A_{s1} , A_{s2} , D_{s1}

6, 200, 200, 0, 120

INPUT E_m & Max. M & s. STRAIN

15000, .002

$P_o = 6.97E+005$

$A_n = 116130$

$I_1 = 9.68E+008$

$I_{eff} = 5.21E+008$

$I_{cr} = 7.35E+007$

$M_o = 6.92E+006$

$I_n = 9.68E+008$

$I_2 = 9.68E+008$

$E_{I_{eff}} = 7.81E+012$

$e_k = 69$

Figure (39) TRIAL SECTION RESISTANCE DATA

For wall detailed in Figures (36), (37) and (38) and 2-15M at 1200.

Factored Load Cases (from N.B.C. Part 4, Limit States Design)

(a) 1.25D+1.5L

(b) 1.25D+.7(1.5L+.5W)

(c) 1.25D+1.5W

(d) .85D+1.5W

Primary Moments:

Figure (40) PRIMARY MOMENTS

At mid-height $M_p = P \frac{(e_1 + e_2)}{2} + \frac{WH^2}{8}$

Unfactored loads (per 1.2m length)

(a) L.L. $(1.2)(12) = 14.4 \text{ kN/1.2m}$

(b) D.L. at top of wall = $(1.2)(5.4) = 6.48 \text{ kN/1.2m}$

(c) D.L. at mid ht. wall = $6.48 + (1.2)(3)(\frac{7.3}{2}) = 19.62 \text{ kN/1.2m}$

(d) D.L. at bottom wall = $6.48 + (1.2)(3)(7.3) = 32.76 \text{ kN/1.2m}$

(e) Wind $(.7)(1.2) = .84 \text{ kN/m} (.84 \text{ N/mm})$

(1) P-S METHOD:

Load Case (a) $1.25D + 1.5L$ governs at wall bottom (by inspection)

Axial load = $(1.25)(32.76) + (1.5)(14.4) = 62.55$

Moment ($e = .05t = 12 \text{ mm}$) = $(62.55)(.012) = .75 \text{ kN-m}$

From Fig.(39) M_r at $P = 62$ is 13.5 O.K.

Load Case (b) $1.25D + 1.05L + 1.05W$ governs at mid height (by inspection)

Wind load = $(1.05)(.84) = .882 \text{ kN/m}$

Mid Ht axial load = $(1.25)(19.62) + (1.05)(14.4)$

= 39.65 kN/1.2m

$$\text{Primary Moment } M_p = \frac{(39.65)(.012 + .012)}{2} + \frac{(.882)(7.3)^2}{8}$$

$$= 6.35 \text{ kN-m}$$

Mid-Height deflection:

$$\delta_{\max} = \frac{P(e_1 + e_2)H^2}{16EI_{\text{eff}}} + \frac{5WH^4}{384EI_{\text{eff}}} \quad \text{at mid height}$$

$$I_{\text{eff}} = \frac{(11 + 2I_{cr} + 12)}{4} = \frac{((2)(968.5)(10^6) + (2)(73.5)(10^6))}{4}$$

$$= (521)(10^6) \text{ mm}^4$$

$$\delta(e_1 + e_2) = \frac{P(12 + 12)(7300)^2}{(16)(15000)(521)(10^6)} = .0000102 P(N) \quad \text{or}$$

$$= .0102 P(kN) \text{ mm}$$

$$\delta_W = \frac{5W(7300)^4}{(384)(15000)(521)(10^6)} = 4.73 W (N/\text{mm})$$

$$\delta_{\max} = (.0102)(39.65) + (4.73)(.882) = 4.57 \text{ mm}$$

$$P-\delta = (39.65)(.00457) = .181 \text{ kN-m}$$

$$M_p = 6.35$$

$$M_{\text{total}} = 6.53 \text{ kN-m}$$

From Fig.(39) M_r at $P=39.65$ is $11.7 > 6.53$ O.K.

Load Case (c) governs at mid height

$$\text{Factored wind load} = (1.5)(.84) = 1.26 \text{ kN/m}$$

$$\text{Factored gravity load} = (1.25)(19.62) = 24.5$$

$$\text{Mid Ht. deflections: } \delta_W = (1.26)(4.73) = 5.96 \text{ mm}$$

$$\delta(e_1 + e_2) = (.0102)(24.5) = .25 \text{ mm}$$

$$\delta_{\text{total}} = 6.2 \text{ mm}$$

$$P-\delta = (24.5)(.0026) = .15 \text{ kN-m}$$

Primary Moment

$$M_p = (24.5)(.012) + \frac{(1.26)(7.3)^2}{8} = 8.68 \text{ kN-m}$$

$$M_{\text{tot}} = M_p + P-\delta = 8.68 + .15 = 8.83$$

From Fig.(39) M_r at $P=24.5$ is $9.5 > 8.83$ O.K.

Load Case (d)

$$\text{Factored gravity load} = (.85)(19.62) = 16.67 \text{ /1.2m}$$

$$\text{Factored wind } W = (1.5)(.84) = 1.26 \text{ /m}$$

$$\text{Mid Ht deflections: } \delta = 5.96 \text{ mm}$$

$$\delta(e_1 + e_2) = (.0102)(16.7) = .17$$

$$P-\delta = (16.7)(.00613) = .1 \text{ mm}$$

$$M_p = (16.67)(.012) + \frac{(1.26)(7.3^2)}{8} = 8.59$$

$$M_{\text{tot.}} = M_p + P-\delta = 8.59 + .1 = 8.69 \text{ kN-m}$$

From Fig.(39) M_r at $P=16.7$ is $8.8 > 8.69$ o.k.

(2) MOMENT MAGNIFIER METHOD.

For reinforced masonry:

If mid ht. $e > e_k$ use $I_{\text{eff}} = (.5 - e/t) I_o < .1 I_o$

If mid ht. $e \leq e_k$ use $I_{\text{eff}} = 8 I_o (.5 - e/t)^3$

Load Case (a)

$$\text{Axial load} = 62.55 \text{ kN/1.2m}$$

$$\text{Mid Ht } M_p = .75 \text{ kN and } e = 12 \text{ mm}$$

$$e < e_k \text{ so use } I_{\text{eff}} = 8(.5 - 12/240)^3 (968.5)(10^6) = (706)(10^6)$$

$$P_{\text{cr}} = \frac{(\pi^2)(15000)(706)(10^6)}{7300^2} = 1961 \text{ kN}$$

$$M_{\text{tot.}} = (.75) \left(\frac{1}{\left(1 - \frac{62}{1961}\right)} \right) = .77$$

From Fig.(39) M_r at $P=62$ is $13.5 > .77$ O.K.

Load Case (b)

$$\text{Mid ht. axial load } 39.65$$

$$\text{Mid ht. } M_p = 6.35 \quad e = M/P = 160 \text{ mm}$$

$$e < e_k \text{ use } I_{\text{eff}} = \left\{ .5 - \frac{160}{240} \right\} (968.5)(10^6) \text{ Negative! Use } .1 I_o$$

$$I_{eff} = (.1)(968.5)(10^6) = 96.85(10^6)$$

$$P_{cr} = \frac{(\pi^2)(15000)(96.85)(10^6)}{7300^2} = 269$$

$$M_{tot.} = 6.35 \left(\frac{1}{\left(1 - \frac{39.6}{269}\right)} \right) = 7.44$$

From Fig. (39) M_r at $P=39.6$ is $11.7 > 7.44$ O.K.

Try Turkstra's I_{eff} :

$$P_{cr} = \frac{(\pi^2)(15000)(521)(10^6)}{7300^2} = 1447 \text{ kN}$$

$$M_{tot.} = 6.35 \left(\frac{1}{\left(1 - \frac{39.6}{1447}\right)} \right) = 6.52 \text{ close to } 6.53 \text{ from } P-\delta \text{ method}$$

Load Case (c)

Mid ht. axial load 24.5

Mid ht. $M_p = 8.68$ $e = 354 \text{ mm}$

$$I_{eff} = \left(\frac{.5 - \frac{354}{240}}{1} \right) I_o \text{ use } (.1)(968.5)(10^6)$$

$$P_{cr} = 269$$

$$M_{tot.} = 8.68 \left(\frac{1}{\left(1 - \frac{24.5}{269}\right)} \right) = 9.54$$

From Fig. (39) M_r at $P=24.5$ is $9.5 = 9.54$ Close but O.K.

Try Turkstra's I_{eff} :

$$M_{tot.} = 8.68 \left(\frac{1}{\left(1 - \frac{24.5}{1447}\right)} \right) = 8.82 \text{ Close to } 8.83 \text{ from } P-\delta \text{ method}$$

Load Case (d)

Mid ht. axial load 16.67

Mid ht $M_p = 8.59$

$P_{cr} = 269$ as before

$$M_{tot.} = 8.59 \left(\frac{1}{\left(1 - \frac{16.7}{269}\right)} \right) = 9.158$$

From Fig.(39) Mr at P=16.67 is $8.8 < 9.15$

Need more steel

Try Turkstra's Ieff:

$$M_{tot.} = 8.59 \left(\frac{1}{\left(\frac{1-16.67}{1447} \right)} \right) = 8.69 = 8.69 \text{ from the P-δ method}$$

(3) S304 S.C.P.I. METHOD

$$F_m = .33 f'_m = 5 \text{ MPa}$$

$$F_s = 165 \text{ MPa max}$$

Using $e_1/e_2 = 0$ and $H/t = 30$ $C_s = .4$

Load combinations from N.B.C. Part 4 Working Stress

Load Case (a) D+L

$$\text{Axial load at mid ht.} = 14.4 + 32.76 = 47.2 \text{ kN/1.2m}$$

INPUT L, L1, T, T1, e1, e2

1200, 189, 240, 35, 12, 12

INPUT f'f'm, fFy, As1, As2, D

165, 200, 0, 120

INPUT Em&Max.Mas. STRAIN

15000, .002

$$P_o = 5.81E+005$$

$$A_n = 116130$$

$$I_1 = 9.68E+008$$

$$I_{eff} = 5.21E+008$$

$$I_{cr} = 7.350E+007$$

$$M_o = 5.751E+006$$

$$I_n = 9.68E+008$$

$$I_2 = 9.68E+008$$

$$E_{eff} = 7.81E+012$$

$$e_k = 69$$

Figure (41) RESISTANCE DATA FOR SECTION Fig.(36), (37), (38), 2-15M at 1200 and $F_m = 5$

From Fig-(41) $P_o = 581$

$$e < t/3 \therefore P = C_e C_s P_o$$

$$e/t \leq .05 \therefore C_e = .1$$

$$P = (.4)(1)(581) = 232 > 47.2 \quad \text{O.K.}$$

Load Case (b) .75(D+L+W)

$$\text{Grav. load at mid ht.} = .75(14.4 + 19.62) = 25.5 \text{ kN}$$

$$\text{Wind moment } M_w = \frac{(.75)(.84)(7.3^2)}{8} = 4.2 \text{ kN-m}$$

$$e = 4.2/25.5 = .164 \text{ m}$$

$$\text{Plot } e \text{ line: } M = .164P \text{ (Figure (42))}$$

$$\text{At } M=0 \quad P=0$$

$$\text{At } M=20 \quad P=20/.164 = 122$$

From Figure (42) intersection at $M=15.5$, $P=100$

$$PCs = (.4)(100) = 40 > 25.5 \quad \text{O.K.}$$

$$MCs = (.4)(15.5) = 6.2 > 4.2 \quad \text{O.K.}$$

Figure (42)

Load Case (c) D+W

Gravity load at mid ht. = 19.62 kN

$$M_w = \frac{(.84)(7.3^2)}{8} = 5.6 \text{ kN-m}$$

$$e = 5.6 / 19.62 = .285 \text{ m}$$

Plot e line: $M = .285P$ (Figure (43))

At $M=0$, $P=0$

At $M=20$, $P=70.1$

From Fig. (43) intersection at $M=9$, $P=25$

Allow $P = (.4)(25) = 10 < 19.62$ N.G.

Corresponding $M = (.4)(9) = 3.6 < 5.6$ N.G.

Figure (43)

6.2 DESIGN EXAMPLE 2

Same wall as example 1 but increased specified loads.

Roof snow load 150kN/m

Roof D.L. 50kN/m

Wind 1kPa

Specified loads:

L.L. $(1.2)(150)=180\text{kN}/1.2\text{m}$

D.L. at top of wall $(1.2)(50)=60$

D.L. at mid ht wall $60 + \frac{(3)(7.3)(1.2)}{2} = 73.14$

D.L. at bottom $60 + (3)(7.3)(1.2) = 86$

Wind $(1.2)(1)=1.2\text{kN}/\text{m}$

(1) P-S METHOD

Load Case (a) 1.25 D+1.5L

At wall bottom axial load $= (1.25)(86) + (1.5)(180)$
 $= 377.5$

$M_p = (377.5)(.012) = 4.53\text{ kN-m}$

Try same section (1-15M ea. face at 1200)

From resistance diagram, Fig. (39), M_r at $P=377$
 is $22 > 4.53$ O.K.

Load Case (b) 1.25D+1.05L+1.05W

Factored load at mid ht.:

Wind $(1.05)(1.2)=1.26\text{kN}/\text{m}$

Axial Load $(1.25)(73.14) + (1.05)(180) = 280\text{ kN}/1.2\text{m}$

From previous example:

$\delta(e_1 + e_2) = .0102P = (.0102)(280) = 2.86\text{mm}$

$\delta_w = 4.73W = (4.73)(1.26) = 5.96\text{mm}$

$\delta_{\text{tot.}} = 8.82\text{mm}$

$$P-\delta = (280)(.00882) = 2.47 \text{ kN-m}$$

$$M_p = \frac{(1.26)(7.3^3)}{8} + (280)(.012) = 11.75$$

$$M_{tot.} = 2.47 + 11.75 = 14.22$$

From Fig. (39) M_r at $P = 280 = 25.8 > 14.22$ O.K.

MOMENT MAGNIFIER METHOD using Turkstra's I_{eff} :

$$M_{tot} = \frac{11.75}{1 - 280/1447} = 14.56 \text{ Close}$$

Load Case (c) 1.25D+1.5W

Factored loads at mid ht.

$$\text{Wind } (1.5)(1.2) = 1.8 \text{ kN/m}$$

$$\text{Axial load } (1.25)(73.14) = 91.4 \text{ kN/1.2m}$$

Mid Ht. deflections:

$$\delta(e_1 + e_2) = (.0102)(91.4) = .93 \text{ mm}$$

$$\delta_w = (4.73)(1.8) = 8.5 \text{ mm}$$

$$\delta_{tot.} = .93 + 8.5 = 9.4 \text{ mm}$$

$$P-\delta = (91.4)(.0094) = .86$$

$$M_p = (91.4)(.012) + \frac{(1.8)(7.3^3)}{8} = 13.08$$

$$M_{tot.} = .86 + 13.08 = 13.95 \text{ kN-m}$$

From Fig. (39), M_r at $P = 91.4$ is $16.4 > 13.95$ O.K.

MOMENT MAGNIFIER METHOD using Turkstra's I_{eff} :

$$M_{tot.} = \frac{13.08}{1 - 91.4/1447} = 13.96 \text{ Close.}$$

Load Case (d) .85D+1.5W

Factored loads at mid Ht.:

$$W = 1.8 \text{ kN/m}$$

$$\text{Axial load} = (.85)(73.14) = 62 \text{ kN/1.2m}$$

$$\text{Deflections } \delta(e_1 + e_2) = (.0102)(62) = .63$$

$$\delta_w = 8.5$$

$$\delta_{tot.} = .63 + 8.5 = 9.1 \text{ mm}$$

$$P-\delta = (62)(.0091) = .56 \text{ kN-m}$$

$$M_p = (62)(.012) + \frac{(1.8)(7.3^3)}{8} = 12.73$$

$$M_{tot.} = .56 + 12.73 = 13.29$$

From Fig. (39), M_r at $P=62$ is 13.5 Close but O.K.

MOMENT MAGNIFIER METHOD, using-Turkstra's I_{eff} :

$$M_{tot.} = \frac{12.7}{1 - 62/1447} = 13.26 \text{ Close}$$

(3) S304 METHOD

Load Case (a) D+L

$$\text{Axial load at mid ht.} = 73 + 180 = 232$$

From Fig. (41) $P_o = 581$ (as before)

$$e/t \leq .05 \therefore C_e = 1$$

$$P = C_e C_s P_o = (.4)(1)(581) = 232 = 232 \text{ O.K.}$$

Load Case (b) $= .75(D+L+W)$

$$\text{Axial load } (.75)(253) = 189$$

$$M_w = \frac{(.75)(1.2)(7.3^3)}{8} = 5.99 \text{ kN-m}$$

$$e = 5.99/189 = .0317 \text{ m}$$

$$e/t = .132 \therefore C_e = .77$$

$$P = (.77)(.4)(581) = 179 < 189 \text{ N.G.}$$

Load Case (c) D+W.

$$\text{Axial load } 73.14 \text{ kN}$$

$$M_w = \frac{(1.2)(7.3^3)}{8} = 7.99 \text{ kN-m}$$

$$e = 7.99/73.14 = .109 \quad e/t = .109/.24 = .455 > .33$$

Plot e line $M = .109P$ (Fig. (44))

From Fig.(44) Intersection at $M=20$ and $P=183$

$$PCs = (.4)(183) = 73.2 \approx 73.14 \quad \text{O.K.}$$

$$MCs = (.4)(20) = 8 \approx 7.99 \quad \text{O.K.}$$

Figure (44)

6.3 DESIGN EXAMPLE 3

Same wall as example 1 but increase wind W to 1.5 kPa
Unfactored load (per 1.2m length, see page 43)

L.L. 14.4

D.L. at top 6.48

D.L. at mid ht. 19.62

D.L. at bottom 32.76

Wind W (1.5)(1.2) = 1.8 kN/m/1.2m width

Load Case (a) No wind, therefore same as Example 1

Load Case (b) 1.25D + 1.05L + 1.05W

Factored loads at mid ht.

$$W = (1.05)(1.8) = 1.89 \text{ kN/m}$$

$$\text{Axial load } (1.25)(19.62) + (1.05)(14.4) = 39.64$$

$$\delta(e_1 + e_2) = (.0102)(39.64) = .4$$

$$\delta_w = (1.89)(4.73) = 8.93 \text{ mm}$$

$$\delta_{\text{tot.}} = .4 + 8.93 = 9.33 \text{ mm}$$

$$M_p = \frac{(1.89)(7.3^3)}{8} + (39.64)(.012) = 14.25$$

$$P-\delta = \frac{(39.64)(9.33)}{1000} = .37$$

$$M_{\text{tot.}} = .37 + 14.25 = 14.62 \text{ kN-m}$$

From Fig(39), M_r at $P = 39.64$ is 11.7 < 14.62 ~~N.G.~~
Increase steel.

Try 2-20M's $A_{s1} = 300$

INPUT L, L1, T, T1, e1, e2

1200, 139, 240, 35, 12, 12

INPUT F'm, F'y, As1, As2, Ds1

6, 200, 300, 0, 120

INPUT Em&Max, Mas, STRAIN

15000, 002

Po=6.97E+005 Mo=1.005E+007
 Mn=116130 In=9.68E+008
 I1=9.68E+008 I2=9.68E+008
 Ieff=5.39E+008 EIf=8.08E+012
 Icr=1.085E+008 ek=69

Figure (45)

From new resistance data: (Figure (45))

$$I_{eff} = \frac{(2)(968.5)(10^6) + (2)(108.5)(10^6)}{4} = (538.5)(10^6)$$

Small change won't affect P-δ much this case

Check from Fig.(45) new Mr at P=39.6 is 14.8 ≈ 14.624 Q.K.

MOMENT MAGNIFIER:

$$\text{New } P_{cr} = \frac{(\pi^2)(15000)(538.5)(10^6)}{7300^2} = 1496$$

$$M_{tot.} = \frac{14.75}{1 - \frac{39.6}{1496}} = 14.63 \text{ (Very close)}$$

Load Case (c) 1.25D+1.5W

$$\text{Wind } (1.5)(1.8) = 2.7 \text{ kNm}$$

$$\text{Mid ht. grav. load} = (1.25)(19.62) = 24.5 \text{ kN}$$

$$\delta(e1+e2) = \frac{P(e1+e2)H^2}{16E_m I_{eff}} = \frac{P(12+12)(7300^2)}{(16)(15000)(538.5)(10^6)} = .0099 P \text{ mm}$$

$$\delta_w = \frac{(5)(W)(7300^4)}{(384)(15000)(538.5)(10^6)} = 4.58 W \text{ mm}$$

$$\delta(e1+e2) = (.0099)(24.5) = .24 \text{ mm}$$

$$\delta W = (2.7)(4.58) = 12.3$$

$$M_p = (24.5)(.012) + \frac{2.7(7.3^2)}{8} = 18.27$$

$$M_{tot.} = .3 + 18.27 = 18.57$$

From Fig. (45), M_r at $P=24.5$ is $12.5 < 18.57$ N.G.

NEED MORE STEEL

Try 2-20M at 800

FIG. (47)

FIG. (48)

```
INPUT L, L1, T, T1, e1, e2
800, 189, 340, 35, 12, 13
INPUT f'c, f'y, As1, As2, Os1
20.0, 300, 0, 120
INPUT Em, Max. Mas, STRAIN
15000, 002
```

```
Po=5.29E+005 Mo=1.005E+007
An= 88130 In=6.71E+008
I1=6.71E+008 I2=6.71E+008
Ieff=3.90E+008 Eleff=5.86E+012
Icr=1.094E+008 ek= 63
```

Figure (46)

Resistance Data for section detailed in Figures (47) and (48), 2-20M at 800

Using new resistance data: (Fig. (46))

$$I_{eff} = (2)(671)(10^6) + \frac{(2)(109.4)(10^6)}{4} = (390.2)(10^6)$$

$$\delta(e_1, e_2) = \frac{P(12+12)(73002)}{(16)(15000)(390.2)(10^6)(1000)} = .0136P$$

$$\delta_w = \frac{5W(73004)}{(384)(15000)(390.2)(10^6)} = 6.317Wmm$$

Load Case (c) (cont'd) 1.25D+1.5W

$$\text{Grav. load} = 5.4 + \frac{(3.5)(7.3)(.8)(1.25)}{2} = 18.1 \text{ kN/.8m}$$

$$\text{Wind} = (1.5)(.8)(1.5) = 1.8 \text{ kN/m}$$

$$\delta(e1+e2) = (.0136)(18.1) = .247 \text{ mm}$$

$$\delta W = (6.317)(1.8) = 11.37 \text{ mm}$$

$$\delta_{\text{tot.}} = .247 + 11.37 = 11.61 \text{ mm}$$

$$P-\delta = \frac{(18.1)(11.61)}{1000} = .21 \text{ kN-m}$$

$$M_p = (18.1)(.012) + \frac{(1.8)(7.3^2)}{8} = 12.20$$

$$M_{\text{tot.}} = .21 + 12.2 = 12.41 \text{ kN-m}$$

From Fig.(46), M_r at $P=18.1$ is 11.6 < 12.41

NEED MORE STEEL

Try 2-20M's at 600

INPUT L,L1,T,T1,e1,e2

?
600,189,240,35,12,12

INPUT $f'_m, f_y, A_{s1}, A_{s2}, O_{s1}$

?
6,200,300,0,120

INPUT E_m & Max. Mas. STRAIN

?
15000, .002

$P_o = 4.45E+005$

$A_n = 74130$

$I_1 = 5.23E+008$

$I_{eff} = 3.16E+008$

$I_{cr} = 1.091E+008$

$M_o = 9.969E+006$

$I_n = 5.23E+008$

$I_2 = 5.23E+008$

$E_{I_{eff}} = 4.74E+012$

$e_k = 59$

Figure (49)

From new resistance data: (fig.(49))

$$I_{eff} = (2)(523)(10^6) + \frac{(2)(109)(10^6)}{4} = (316)(10^6)$$

$$\delta_w = \frac{5W(7300^4)}{(384)(15000)(316)(10^6)} = 7.8 \text{ mm}$$

Load Case (c) (cont'd) 1.25D+1.5W

$$\text{Grav. load} = (.6)(5.4) + \frac{(3.72)(7.3)(.6)(1.25)}{2} = 14.23 \text{ kN/.6m}$$

$$\text{Wind} = (.6)(1.5)(1/5) = 1.35 \text{ kN/m}$$

$$\delta(e_1 + e_2) = (.0168)(14.23) = .24 \text{ mm}$$

$$\delta_w = (1.35)(7.8) = 10.53 \text{ mm}$$

$$\delta_{\text{tot.}} = .24 + 10.53 = 10.77$$

$$P-\delta = \frac{(14.23)(10.77)}{1000} = .153$$

$$M_p = (14.23)(.012) + \frac{(1.35)(7.3^2)}{8} = 9.16$$

$$M_{\text{tot.}} = .153 + 9.16 = 9.31 \text{ kN-m}$$

From Fig. (49), M_r at $P=14.23$ is $11 > 9.31$ O.K.

MOMENT MAGNIFIER:

$$P_{cr} = \frac{(\pi^2)(15000)(316)(10^6)}{7300^4} = 877.9$$

$$M_{\text{tot.}} = \frac{9.16}{1 - \frac{14.23}{877.9}} = 9.31 \text{ (Right On!)}$$

Load Case (d) .85D+1.5W

$$\text{Grav. load} = (.85)(.6)(18.9) = 9.67 \text{ kN/.6m}$$

$$\text{Wind} = 1.35$$

$$\delta(e_1 + e_2) = (.0168)(9.67) = .162 \text{ mm}$$

$$\delta_w = 7.8 \text{ mm}$$

$$\delta_{\text{tot.}} = .162 + 7.8 = 7.96 \text{ mm}$$

$$P-\delta = \frac{(9.67)(7.96)}{1000} = .076$$

$$M_p = (9.67)(.012) + \frac{(1.35)(7.3^2)}{8} = 9.1$$

$$M_{tot.} = .076 + 9.1 = 9.17$$

From Fig. (49), M_r at $P = 9.67$ is $10.7 > 9.17$ O.K.

MOMENT MAGNIFIER

$$M_{tot.} = \frac{9.1}{1 - \frac{9.67}{877.9}} = 9.2 \quad (\text{Very close}).$$

S304 METHOD:

$$F_m = 5 \quad F_s = 165(\text{max}) \quad C_s = .4$$

INPUT L, L1, T, T1, e1, e2

? 600, 189, 240, 35, 12, 12

INPUT F'm, Fy, As1, As2, Ds1

? 5, 165, 300, 0, 120

INPUT Em & Max. Mas. STRAIN

? 15000, .002

$P_o = 3.71E+005$

$A_n = 74130$

$I_1 = 5.23E+008$

$I_{eff} = 3.16E+008$

$I_{cr} = 1.091E+008$

$M_o = 8.277E+006$

$I_n = 5.23E+008$

$I_2 = 5.23E+008$

$E_{eff} = 4.74E+012$

$e_k = 59$

Figure. (50)

Load Case (a) D+L

$$\text{Grav. Load} = (.6)(5.4) + \frac{(3.72)(7.3)}{2} + 12 = 18.6 \text{ kN/.6m}$$

$$P = C_e C_s P_o = (.4)(371) = 148.4 > 18.6 \quad \text{O.K.}$$

Load Case (b) .75(D+L+W)

$$\text{Mid ht. grav. load} = (.75)(18.6) = 13.95$$

$$M_w = \frac{(.75)(.6)(1.5)(7.3^2)}{8} = 4.49 \text{ kN-m}$$

$$e = \frac{4.49}{13.95} = .32$$

Plot a line $M = .32P$ (Fig. (51))

Figure (51)

From Fig.(51) intersection at $M=10.5$, $P=32.8$

$$PCs = (32.8)(.4) = 13.1 \approx 13.9 \quad \text{Close}$$

Load Case (c) D+W

$$\text{Mid Ht. grav. load} = (.6)(5.4 + (3.72)(\frac{7.3}{2})) = 11.38 \text{ kN}$$

$$M_w = \frac{(.6)(1.5)(7.3^2)}{8} = 5.99 \text{ kN-m}$$

$$e = \frac{5.99}{11.38} = .526 \text{ m}$$

Plot e line $M = .526P$ (Fig.(51))

From Fig.(51), intersection at $M=9.7$, $P=18.4$

$$PCs = (.4)(18.4) = 7.37 < 11.38 \quad \text{N.G.}$$

SUMMARY

7. COMMENTS ON METHODS

7.1 GENERAL

When load factors 1.5L and 1.25D and resistance factor .4 are applied, the predicted resistances will increase by approximately $1.3/4 = 3.25$ which should raise the least conservative results (.8) to around 2.5 which would seem adequately conservative to deal with most strength and loading variations.

7.2 THE EXACT P- δ METHOD

This method as would be expected gave slightly more conservative prediction of strength than the other methods, but it is questionable whether the extra computation time is justified.

7.3 THE MODIFIED P- δ METHOD

This method seems sufficiently accurate for most cases and has the capability to find the largest moment along the span for any type of curvature and loading and so should be able to deal with unusual or special cases more competently than the simpler methods.

7.4 THE QUADRATIC P- δ METHOD

This method uses considerably less computation time than either of the other P- δ methods, but has the disadvantage that it may not detect the maximum moment along a double curvature span with applied lateral loading since it checks only at ends and mid-height of span. It is also necessary to prevent the mid-height eccentricity from being zero in order to obtain reasonable results.

7.5 THE MOMENT MAGNIFIER METHOD

This method has the advantage that a simple mathematical expression provides a multiplier or "Magnifier" that is applied to the maximum moment along the span to obtain the critical design moment. However it is very sensitive to the value of P_{cr} used in the calculations.

It is suspected that the conservative scatter obtained for the plain brick test series is due to an underestimate of the tensile strength f'_t (taken as 0.025MPa) which affects the value of I_{eff} used in calculating P_{cr} .

In the design examples, this method gave identical answers to those given by the P- δ method when the same I_{eff} was used for both methods.

It is not clear, at this stage, how this method will cope with cases of double (or triple) curvature since the magnifier is applied to the maximum primary moment along the span, which may not be at the point where the maximum $M_p + P-\delta$ moment occurs. This problem deserves more study.

8. POTENTIAL FOR DESIGN AIDS

The design problem resolves into the following steps:

- (a) Define the factored loading and a trial geometry and material strengths.
e1, e2, w and P
f'm, F_y , E_m and E_s
H, L, L_1 , A_s , t and t_1
- (b) Calculate the section properties, the effective moment of inertia of the span I_{eff} and generate the resistance (interaction) curve using factored resistance parameters.
- (c) Calculate the primary moments.
- (d) For the P- δ method, calculate the displacements (δ 's) and add the P- δ 's to the primary moments, determining the maximum total moment M_{max} along the span.

- (e) For the moment magnifier method, multiply the maximum primary moment along the span by the "multiplier" $CM/(1-P/P_{cr})$ to determine M_{max} .
- (f) Using the resistance (interaction) curve, check if:
 - (i) $M_{max} \leq M_r$ at P then finished.
 - (ii) $M_{max} > M_r$ at P then increase strength and repeat from (b)

8.1 DESIGN AIDS

Step (b) can be easily tabulated in variations of the resistance data output diagrams produced by program "Resist" for the design examples.

Step (c) is best executed by a small computer or programmable calculator that steps along the span calculating and printing the primary moments as it goes.

Step (d) could be tabulated - station displacements for unit values of M_1 , M_2 and M_w which could be calculated by hand and the P - δ 's added at each station. This could also be easily done by a small computer or programmable calculator.

Step (e) values of P_{cr} could be tabulated - perhaps in the same table as the resistance data (see above).

9 CONCLUSIONS

- (a) The methods all seem applicable to the range of design problems that will be permitted by the proposed code.
- (b) The P - δ methods have the advantage that they encourage the designer to visualize the structural action and to approach the design in a simple direct manner.
- (c) The Moment Magnifier Method has the advantages of simple application and speed. However its use in special cases such as double curvature with lateral loading is not straight forward. This method gives identical results to those of the P - δ method for simple cases when the same I_{eff} is used for both methods.

- (d) The methods have not been adequately tested against walls with large axial and transverse loads.

10 RECOMMENDATIONS

- (a) That both the P- δ and Moment Magnifier Methods be permitted by the Code.
- (b) That the same I_{eff} be used for both methods.
- (c) That the Moment Magnifier Method be limited to single curvature where transverse loads are applied.
- (d) That the minimum design eccentricity $.05t$ be specified single curvature for all walls including those with equal and opposite end eccentricities.
- (e) That a test series be run for walls with large axial and transverse loads and single and double curvature end eccentricities.

- AMRHEIN, JAMES E. 1981. Slender walls research program by California Engineers. The Masonry Society Journal, Vol.1, No.2, pp G9-G15.
- BAKER, L.R. 1980. Design of masonry panels to resist lateral loads. Research Report No. CE2/80, Deakin University.
- BROWN, R.H., WATTAR, F.N. 1980. An evaluation of the moment magnifier technique for the design of unreinforced masonry. Proceedings, Second Canadian Masonry Symposium, Carleton University, Ottawa, pp 229-243.
- CHIK, S.W-K., LAI, F.W-K. 1981. The moment magnifier method for plain masonry walls. Project Report, Department of Civil Engineering, McGill University, Montreal.
- ESKINAZI, A., OJINAGA, J. and TURKSTRA, C.J. 1975. Some mechanical properties of brick and block masonry, interim report. Structural Masonry Series No.75-1, Department of Civil Engineering, McGill University, Montreal.
- HATZINIKOLAS, M., LONGWORTH, J., and WARWARUK, J. 1980. The analysis of eccentrically loaded masonry walls by the moment magnifier method. Proceedings, Second Canadian Masonry Symposium, Carleton University, Ottawa, pp 245-258.
- OJINAGA, J. 1976. The ultimate strength of loadbearing brick and block masonry walls. Master's Thesis, Department of Civil Engineering, McGill University, Montreal.
- OJINAGA, J., and TURKSTRA, C.J. 1977. The design of plain masonry walls. Internal Report, Department of Civil Engineering, McGill University, Montreal.
- OJINAGA, J. and TURKSTRA, C.J. 1978. The design of plain masonry: a study of alternatives.
- OJINAGA, J. and TURKSTRA, C.J. 1979. The design of masonry walls and columns, I-concentric loading, and minor axis bending. Internal Report, Department of Civil Engineering, McGill University, Montreal.
- SHRIVE, N.G. 1981. The failure mechanism of face shell bedded (ungrouted and unreinforced) masonry subjected to compressive loading. Research Report No. CE81-3, Department of Civil Engineering, University of Calgary.

SHYU, C.T. 1980. A safety index analysis for limit states design of plain masonry. Master's Thesis, Report No. ST80-5, Department of Civil Engineering, McGill University, Montreal.

TIMOSHENKO, S.P. 1976. Strengths of materials. Third Edition, Parts 1 and 2, Robert E. Krieger Publishing Co.

TURKSTRA, C.J. 1981. Limit states design in masonry, a status report. Printed Notes, A Short Course on Masonry for Structural Engineers, University of Calgary.

TURKSTRA, C.J. and OJINAGA, J. 1976. The moment magnifier method applied to brick walls. Proceedings, Fourth International Brick Masonry Conference, Belgium.

TURKSTRA, C.J. and REID, S.G. 1979. Serviceability limit states, basic criteria for design. Report No. ST79-5, Department of Civil Engineering, McGill University, Montreal.

TURKSTRA, C.J. and THOMAS, G.R. 1978. Strain gradient effects in masonry. Internal Report, Department of Civil Engineering, McGill University, Montreal.

WHITNEY, CHARLES S. 1942. Plastic theory of reinforced concrete design. A.S.C.E. Transaction 107.

YOKEL, F.Y., MATHEY, R.G. and DIKKERS, R.D. 1971. Strength of masonry walls under compressive and transverse loads. National Bureau of Standards, Building Science Series 34, Washington, D.C.

APPENDIX ^LA

Test series output tables

S.C.P.I. PLAIN BRICK

MK	TYPE	H	L	LI	T	TI	F'm	As	e1	e2	w	Ptest	ITP-6 ITP-62 APP-6 H&WHH				e/t	H/t	e1/e2
													Pt/P	Pt/P	Pt/P	Pt/P			
B2	BRK	2093	609	609	92	38	28.9	0	15	-15	0.0	1120	4.8	1.8	1.8	5.3	.16	23	-1.0
B3	BRK	4242	609	609	92	38	28.7	0	15	-15	0.0	510	1.7	1.1	1.5	13.9	.16	46	-1.0
B6	BRK	4166	609	609	203	38	23.4	0	61	-61	0.0	1334	2.0	2.0	2.0	5.5	.30	21	-1.0
B10	BRK	2093	609	609	92	38	28.8	0	31	-31	0.0	748	2.7	2.5	2.5	7.6	.34	23	-1.0
B11	BRK	4242	609	609	92	38	28.9	0	31	-31	0.0	330	2.7	2.2	1.4	19.8	.34	46	-1.0
B26	BRK	2222	609	609	92	38	28.6	0	0	15	0.0	919	4.3	2.0	2.0	3.8	.16	24	0.0
B27	BRK	4242	609	609	92	38	28.6	0	0	15	0.0	380	5.1	1.8	1.6	7.5	.16	46	0.0
B30	BRK	2093	609	609	92	38	29.1	0	0	31	0.0	483	4.8	3.2	2.4	3.7	.34	23	0.0
B31	BRK	4242	609	609	92	38	27.6	0	0	31	0.0	127	5.6	3.5	2.1	5.2	.34	46	0.0
B35	BRK	2751	609	609	197	38	33.1	0	33	33	0.0	2239	2.5	2.1	2.2	3.6	.17	14	1.0
B36	BRK	2093	609	609	92	38	28.5	0	15	15	0.0	615	2.0	2.2	2.1	5.3	.16	23	1.0
B38	BRK	4242	609	609	92	38	28.9	0	15	15	0.0	282	3.6	3.6	2.6	11.1	.16	46	1.0
B43	BRK	2093	609	609	92	38	29.0	0	31	31	0.0	192	20.4	20.4	13.7	0.0	.34	23	1.0
B46	BRK	4242	609	609	92	38	30.0	0	31	31	0.0	52	21.4	21.4	14.2	17.4	.34	46	1.0

YOKEL MATHEY & DIKKERS SERIES#2

MK	TYPE	H	L	LI	T	TI	F'm	As	e1	e2	w	Ptest	ITP-6 ITP-62 APP-6 H&WHH				e/t	H/t	e1/e2
													Pt/P	Pt/P	Pt/P	Pt/P			
Y1	BLK	2835	1219	50	194	33	11.7	0	0	0	0.0	961	.9	.9	1.1	.9	.00	15	0.0
Y1	BLK	2835	1219	50	194	33	11.7	0	10	10	0.0	961	1.1	1.3	1.1	1.2	.05	15	1.0
Y5	BLK	2835	1219	50	194	33	11.7	0	0	32	0.0	752	1.1	1.1	1.1	1.0	.16	15	0.0
Y7	BLK	2835	1219	50	194	33	11.7	0	0	48	0.0	934	1.6	1.6	1.6	1.5	.25	15	0.0
Y9	BLK	2835	1219	50	194	33	11.7	0	0	65	0.0	901	1.8	1.8	1.8	1.6	.34	15	0.0
Y13	BLK	2835	1219	50	194	33	11.7	0	0	32	0.0	787	1.2	1.2	1.2	1.1	.16	15	0.0
Y15	BLK	4865	1219	50	194	33	11.7	0	0	0	0.0	1192	1.2	1.2	1.5	1.2	.00	25	0.0
Y15	BLK	4867	1219	50	194	33	11.7	0	10	10	0.0	1192	1.7	1.5	1.5	1.6	.05	25	1.0
Y17	BLK	4867	1219	50	194	33	11.7	0	0	32	0.0	848	1.3	1.3	1.3	1.1	.16	25	0.0
Y19	BLK	4867	1219	50	194	33	11.7	0	0	48	0.0	770	1.4	1.3	1.3	1.2	.25	25	0.0
Y21	BLK	4867	1219	50	194	33	11.7	0	0	65	0.0	743	1.5	1.5	1.5	1.5	.34	25	0.0
Y23	BLK	5883	1219	50	194	33	11.7	0	0	0	0.0	986	1.4	1.0	1.3	1.0	.00	30	0.0
Y23	BLK	5883	1219	50	194	33	11.7	0	10	10	0.0	986	2.2	1.3	1.3	1.4	.05	30	1.0
Y27	BLK	5883	1219	50	194	33	11.7	0	0	32	0.0	818	1.8	1.2	1.2	1.2	.16	30	0.0
Y29	BLK	5883	1219	50	194	33	11.7	0	0	48	0.0	636	1.6	1.1	1.1	1.2	.25	30	0.0
Y31	BLK	5883	1219	50	194	33	11.7	0	0	65	0.0	663	2.1	1.3	1.3	2.3	.34	30	0.0

HATZINIKOLAS et al

MK	TYPE	H	L	LI	T	TI	F's	As	e1	e2	w	Ptest	ITP-6 ITP-62 APP-6 H&WHM				e/t	H/t	e1/e2
													Pt/P	Pt/P	Pt/P	Pt/P			
E1	BLK	3486	1000	76	194	38	13.4	0	89	-89	0.0	378	2.4	2.4	2.4	2.2	.46	18	-1.0
E2	BLK	3079	1000	76	194	38	13.4	0	89	-89	0.0	511	3.3	3.3	3.3	3.0	.46	16	-1.0
E3	BLK	2671	1000	76	194	38	13.4	0	89	-89	0.0	600	3.8	3.8	3.8	3.5	.46	14	-1.0
E4	BLK	2671	1000	76	194	38	13.4	0	64	-64	0.0	694	1.3	1.3	1.3	1.2	.33	14	-1.0
E5	BLK	2671	1000	76	194	38	13.4	0	32	-32	0.0	978	1.3	1.3	1.3	1.2	.16	14	-1.0
N21	BLK	3486	1000	76	194	38	13.4	0	32	-32	0.0	812	1.1	1.1	1.1	1.0	.16	18	-1.0
N31	BLK	3486	1000	76	194	38	13.4	0	64	-64	0.0	673	1.2	1.2	1.2	1.1	.33	18	-1.0
694	BLK	3486	1000	76	194	38	13.4	0	76	-76	0.0	574	1.2	1.2	1.2	1.1	.39	18	-1.0
N51	BLK	3486	1000	76	194	38	13.4	0	89	-89	0.0	342	2.2	2.2	2.2	2.0	.46	18	-1.0
65	BLK	3486	1000	76	194	38	13.4	0	64	76	0.0	670	1.5	1.5	1.5	1.5	.39	18	-0.8
66	BLK	3486	1000	76	194	38	13.4	0	64	89	0.0	642	4.1	4.1	4.1	3.7	.46	18	-0.7
68	BLK	3486	1000	76	194	38	13.4	0	76	-89	0.0	520	3.3	3.3	3.3	3.0	.46	18	-0.9
R2	BLK	2670	1000	622	194	38	13.4	1935	32	32	0.0	1423	1.5	1.4	1.5	1.6	.16	14	1.0
R3	BLK	2670	1000	622	194	38	13.4	1935	64	64	0.0	623	2.0	1.8	1.9	1.6	.33	14	-1.0
R4	BLK	2761	1000	622	194	38	13.4	1935	76	76	0.0	689	3.8	2.8	2.9	2.4	.39	14	1.0
R5	BLK	2671	1000	622	194	38	13.4	1935	89	89	0.0	511	3.1	2.2	2.6	2.1	.46	14	1.0
C3	BLK	3079	1000	622	194	38	13.4	1935	32	32	0.0	1112	1.2	1.1	1.3	1.4	.16	16	1.0
C4	BLK	3079	1000	622	194	38	13.4	1935	64	64	0.0	556	2.2	1.6	1.8	1.6	.33	16	1.0
C5	BLK	3079	1000	622	194	38	13.4	1935	76	76	0.0	545	4.0	2.3	2.5	2.1	.39	16	1.0
C6	BLK	3079	1000	622	194	38	13.4	1935	89	89	0.0	400	3.7	2.0	2.2	1.8	.46	16	1.0
D3	BLK	3486	1000	622	194	38	13.4	1935	32	32	0.0	889	1.0	1.0	1.1	1.3	.16	18	1.0
D4	BLK	3486	1000	622	194	38	13.4	1935	64	64	0.0	484	2.0	1.5	1.7	1.5	.33	18	1.0
D5	BLK	3486	1000	622	194	38	13.4	1935	76	76	0.0	420	3.1	1.7	2.1	1.8	.39	18	1.0
D6	BLK	3486	1000	622	194	38	13.4	1935	89	89	0.0	369	3.4	1.9	2.2	1.9	.46	18	1.0
L2	BLK	4706	1000	622	194	38	13.4	1935	32	32	0.0	667	1.0	.8	1.0	1.3	.16	24	1.0
L3	BLK	4706	1000	622	194	38	13.4	1935	64	64	0.0	400	1.8	1.7	1.7	1.7	.33	24	1.0
L4	BLK	4706	1000	622	194	38	13.4	1935	76	76	0.0	356	3.1	1.6	2.2	2.1	.39	24	1.0

HATZINIKOLAS CONT'D

MK	TYPE	H	L	LI	T	TI	F's	As	e1	e2	w	Ptest	ITP-6 ITP-62 APP-6 H&WHM				e/t	H/t	e1/e2
													Pt/P	Pt/P	Pt/P	Pt/P			
L5	BLK	4702	1000	622	194	38	13.4	1935	89	89	0.0	326	3.4	2.1	2.4	2.2	.46	24	1.0
H2	BLK	3486	1000	622	194	38	13.4	851	32	32	0.0	1154	1.4	1.2	1.5	1.6	.16	18	1.0
H3	BLK	3486	1000	622	194	38	13.4	851	64	64	0.0	384	2.0	1.4	1.6	1.3	.33	18	1.0
H4	BLK	3486	1000	622	194	38	13.4	851	76	76	0.0	289	1.8	1.5	1.7	1.3	.39	18	1.0
H5	BLK	3486	1000	622	194	38	13.4	851	89	89	0.0	249	1.9	1.8	1.9	1.3	.46	18	1.0
I2	BLK	3486	1000	622	194	38	13.4	193	32	32	0.0	965	1.1	1.0	1.5	1.4	.16	18	1.0
I3	BLK	3486	1000	622	194	38	13.4	193	64	64	0.0	240	1.3	1.0	2.0	.9	.33	18	1.0
I4	BLK	3486	1000	622	194	38	13.4	193	76	76	0.0	146	1.3	1.1	2.1	.8	.39	18	1.0
I5	BLK	3486	1000	622	194	38	13.4	193	89	89	0.0	108	2.9	1.5	2.5	.8	.46	18	1.0
J1	BLK	3486	1000	622	194	38	13.4	193	32	-32	0.0	1356	1.2	1.2	1.2	1.1	.16	18	-1.0
J2	BLK	3486	1000	622	194	38	13.4	193	64	-64	0.0	1170	1.8	1.8	1.8	1.8	.33	18	-1.0
J3	BLK	3486	1000	622	194	38	13.4	193	76	-76	0.0	1089	2.4	2.4	2.4	2.5	.39	18	-1.0
J4	BLK	3486	1000	622	194	38	13.4	193	89	-89	0.0	701	2.2	2.2	2.2	2.0	.46	18	-1.0
K1	BLK	3486	1000	622	194	38	13.4	193	-32	64	0.0	1023	1.6	1.6	1.6	2.0	.33	18	-0.5

HATZINIKOLAS CONT'D

MK	TYPE	H	L	LI	T	TI	F's	As	e1	e2	w	Ptest	ITP-G ITP-S2 APP-G H&MMH				e/t	H/t	e1/e2
													Pt/P	Pt/P	Pt/F	Pt/P			
S1	BLK	1000	1000	1000	194	38	13.4	0	10	10	0.0	1503	.8	.8	.8	.8	.05	5	1.0
S6	BLK	1000	1000	622	194	38	13.4	0	10	10	0.0	1348	.8	.8	.9	.9	.05	5	1.0
S8	BLK	1000	1000	622	194	38	13.4	1935	10	10	0.0	1690	1.1	1.1	1.1	1.1	.05	5	1.0
S10	BLK	1000	1000	622	194	38	13.4	851	10	10	0.0	1548	1.0	1.0	1.0	1.0	.05	5	1.0
S12	BLK	1000	1000	622	194	38	13.4	193	10	10	0.0	1388	.9	.9	.9	.9	.05	5	1.0
ES1	BLK	1000	1000	76	194	38	13.4	0	32	32	0.0	772	1.0	1.0	1.1	1.1	.16	5	1.0
ES4	BLK	1000	1000	76	194	38	13.4	0	65	65	0.0	618	1.2	1.2	1.4	1.8	.34	5	1.0
A1	BLK	2670	1000	76	194	38	13.4	0	10	10	0.0	978	1.0	1.0	1.1	1.1	.05	14	1.0
C1	BLK	3078	1000	76	194	38	13.4	0	10	10	0.0	845	.9	.9	.9	.9	.05	16	1.0
D1	BLK	3486	1000	76	194	38	13.4	0	10	10	0.0	969	1.1	1.1	1.1	1.1	.05	18	1.0
M1	BLK	4702	1000	76	194	38	13.4	0	10	10	0.0	925	1.1	1.1	1.1	1.1	.05	24	1.0
F1	BLK	3486	1000	76	194	38	13.4	0	10	10	0.0	712	.8	.8	.8	.8	.05	18	1.0
I1	BLV	3486	1000	622	194	38	13.4	193	10	10	0.0	1356	1.0	1.0	1.0	1.1	.05	18	1.0
H1	BLK	3486	1000	622	194	38	13.4	851	10	10	0.0	1668	1.3	1.2	1.2	1.4	.05	18	1.0
C2	BLK	3078	1000	622	194	38	13.4	1935	10	10	0.0	1401	1.0	1.0	1.0	1.1	.05	16	1.0
D2	BLK	3486	1000	622	194	38	13.4	1935	10	10	0.0	1779	1.4	1.3	1.3	1.4	.05	18	1.0
L1	BLK	4702	1000	622	194	38	13.4	1935	10	10	0.0	1705	1.5	1.5	1.3	2.2	.05	24	1.0
A2	BLK	2671	1000	76	194	38	13.4	0	32	32	0.0	707	1.0	1.0	1.0	1.1	.16	14	1.0
A3	BLK	2671	1000	76	194	38	13.4	0	65	65	0.0	355	.9	.9	1.1	5.5	.34	14	1.0
A5	BLK	2671	1000	76	194	38	13.4	0	76	76	0.0	116	2.9	2.9	2.7	0.0	.39	14	1.0
B1	BLK	3486	1000	76	194	38	13.4	0	0	32	0.0	711	1.0	1.0	1.0	.9	.16	18	0.0
B2	BLK	3486	1000	76	194	38	13.4	0	0	65	0.0	555	1.0	1.0	1.0	.9	.34	18	0.0
B3	BLK	3486	1000	76	194	38	13.4	0	0	76	0.0	380	1.0	.9	.8	.8	.39	18	0.0
B4	BLK	3486	1000	76	194	38	13.4	0	0	89	0.0	218	1.4	1.4	1.4	1.3	.46	18	0.0
M2	BLV	4695	1000	76	194	38	13.4	0	32	32	0.0	534	1.0	.9	.9	1.3	.16	24	1.0
N1	BLK	3486	1000	76	194	38	13.4	0	0	32	0.0	618	1.1	1.1	1.1	1.0	.16	18	0.0

SLENDER WALL PGM

MK	TYPE	H	L	LI	T	TI	F's	As	e1	e2	Pt	Ntest	ITP-G ITP-S2 APP-S1 H&MMH				e/t	H/t	e1/e2
													Wt/W	Wt/W	Wt/W	Wt/W			
S1	BLK	7520	1219	1219	244	38	17.0	529	0	220	5.7	6	1.7	1.7	1.7	1.7	.90	31	0.0
S2	BLK	7520	1219	1219	244	38	17.0	529	0	220	15.3	6	1.6	1.6	1.6	1.8	.90	31	0.0
S4	BLK	7520	1219	1219	194	38	18.0	529	0	197	15.3	5	2.0	2.0	1.8	2.0	1.02	39	0.0
S6	BLK	7520	1219	1219	194	38	18.0	529	0	197	5.7	6	2.3	2.3	2.3	2.3	1.02	39	0.0
S7	BLK	7520	1219	1219	143	38	22.0	529	0	171	5.7	3	1.5	1.5	1.5	1.6	1.20	53	0.0
S10	BLK	7520	1219	1219	241	38	21.0	529	0	221	5.7	6	2.3	2.3	2.3	2.3	.92	31	0.0
S13	BLK	7520	1219	1219	191	38	23.7	529	0	195	5.7	4	1.4	1.4	1.5	1.6	1.02	39	0.0
S16	BLK	7520	1219	1219	140	38	43.0	529	0	170	5.7	5	2.2	2.2	2.3	2.5	1.21	54	0.0
S19	BLK	7520	1219	1219	245	38	27.6	423	0	220	5.7	5	1.9	1.9	1.9	1.9	.90	31	0.0
S22	BLK	7520	1219	1219	194	38	27.6	423	0	197	5.7	4	1.7	1.7	1.7	1.7	1.02	39	0.0
S24	BLK	7520	1219	1219	194	38	27.6	423	0	197	15.3	4	2.0	2.0	1.8	2.0	1.02	39	0.0
S25	BLK	7520	1219	1219	143	38	27.6	423	0	171	15.3	3	1.9	1.9	1.7	2.3	1.20	53	0.0
S27	BLK	7520	1219	1219	143	38	27.6	423	0	171	5.7	3	1.8	1.8	1.8	2.0	1.20	53	0.0
S28	BLK	7520	1219	1219	121	38	27.6	423	0	160	5.7	2	1.4	1.4	1.4	1.6	1.32	62	0.0

APPENDIX B

Computer Program Listings

```

10 : MAIN "P-62"
20 : INITIALIZE
30 OPTION BASE 1
40 DIM I(2), P(8), T1(200), T2(200), N(10), L(200)
50 IORUFFER 16 2 16""
60 J=1 2 GOSUB 330
70 R1="ITP-6 ITP-62 APP-6 H4WHM"
80 PRINT R1 2 PRINT 1,2 : R1
90 R1="MK TYPE H L L1 T T1 F'a As e1 e2 u Ptest Pt/P Pt/P Pt/P Pt/P"
100 T1="MK TYPE H L L1 T T1 F'a As e1 e2 u Ptest Pt/P Pt/P Pt/P Pt/P"
110 PRINT R1 2 PRINT 1,3 : R1
120 FOR J=4 TO W1+3
130 FOR J2=1 TO 8 2 P(J2)=0 2 NEXT J2
140 GOSUB 330
150 CALL "CONST" ( A,D4,E1,E2,E3,E5,E6,E7,F1,F2,F3,I1,I(),I6,K1,L,L1,L2,N,P(),P(),S1,S2,T,T1,T1 )
160 CALL "ITP-6" ( D4,E1,E2,E6,F2,F3,H,K1,L1,L2,N,P,P(),P(),S1,S2,T,T1,W,0 )
180 : RE-RUN WITHOUT TERTIARY DEFL'N CALC'NS
190 CALL "ITP-6" ( D4,E1,E2,E6,F2,F3,H,K1,L1,L2,N,P,P(),P(),S1,S2,T,T1,W,1 )
200 CALL "APP-6" ( A,D4,E1,E2,E6,F2,F3,H,I1,K1,L1,L2,N1,N,P(),P(),S1,S2,T,T1,W )
220 CALL "H4WHM" ( A,D4,E,E1,E2,E5,E7,F2,F3,F6,H,I1,K1,L1,L2,N,P(),P(),S1,S2,T,T1,W )
230 IF E2=.1 THEN E2=0
240 FOR J2=4 TO 8
250 IF P(J2)=0 THEN P(J2)=0 ELSE P(J2)=P9/P(J2)
260 NEXT J2
270 PRINT USING 300 ; M1,T1,H,L,L1,T,T1,F1,S2,E1,E2,W,P9,P(4),P(8),P(5),P(7)
280 OUTPUT 16 USING 300 ; M1,T1,H,L,L1,T,T1,F1,S2,E1,E2,W,P9,P(4),P(8),P(5),P(7)
290 PRINT 1,J ; 16 2 16""
300 IMAGE 2(AAA,1X),3(DDDD,1X),DDD,1X,DD,1X,DD,D,1X,DDDD,1X,2(DDD,1X),DD,D,1X,DDDD,2X,4(DDDD,D)
310 NEXT J
320 GOTO 590
330 : SRR.DAT
340 N1=.7 : EQUIV.E.FACT
350 S1=0 2 D4=0 2 F6=.025
360 IF J=1 THEN READ T1,N1,N12 PRINT T1,N1 2 ASSIGN 1 TO N1 2 PRINT 1,1 ; T1,N1 2 RETURN
370 DATA "S.C.F.I. PLAIN BRICK", "PDAT8", 14
380 READ M1,T1,H,L,L1,T,T1,P1,S2,E3,E4,W2,P9
390 L=609
400 DATA B2,BRK,2093,609,92,38,28.9,0,15,-15,0,1120
410 DATA B3,BRK,4242,609,92,38,28.7,0,15,-15,0,510
420 DATA B6,BRK,4166,609,203,38,23.4,0,61,-61,0,1334
430 DATA B10,BRK,2093,609,92,38,28.8,0,31,-31,0,748
440 DATA B11,BRK,4242,609,92,38,28.9,0,31,-31,0,530
450 DATA B26,BRK,2222,609,92,38,28.6,0,0,15,0,919
460 DATA B27,BRK,4242,609,92,38,28.6,0,0,15,0,380
470 DATA B30,BRK,2093,609,92,38,29.1,0,0,31,0,483
480 DATA B31,BRK,4242,609,92,38,27.6,0,0,31,0,127
490 DATA B35,BRK,2751,609,197,38,33.1,0,33,33,0,2239
500 DATA B36,BRK,2093,609,92,38,28.5,0,15,15,0,615
510 DATA B38,BRK,4242,609,92,38,28.9,0,15,15,0,282
520 DATA B43,BRK,2093,609,92,38,29.0,0,31,31,0,192
530 DATA B46,BRK,4242,609,92,38,30.0,0,31,31,0,52
540 IF ABS(E3)<ABS(E4) THEN E1=E3 2 E2=E4 2 GOTO 560
550 IF ABS(E3)>ABS(E4) THEN E1=E4 2 E2=E3
560 IF E1=0 AND E2=0 THEN E2=.1
570 W=W2L/1000
580 RETURN
590 ASSIGN 1 TO 1
600 END

```

```

10 SOR "CONST" (A,B4,E1,E2,E3,E5,E6,E7,F1,F2,F3,I1,I2,I3,I4,I5,I6,I7,I8,I9,I10,I11,I12,M,N,P,Q,S1,S2,T,T1,T2)
15 OPTION BASE 1
18 IF T4="BRK" THEN E5=220.54F1+1000 ELSE E5=MIN(4404F1+5041,20700)
19 E3=.003 & F6=.025 & F2=400
20 ! CALC.CONSTANTS
30 L2=L-L1 & F3=E3/E5 & IF F3=F1 THEN F3=F1
40 N=200000/E5
50 ! CALC UNCRACKED SECT PROP
60 A=T1L1+2I1I2L2
70 B1=L1I1*3/12+2I1I2*3I2L2/12+2I1I2L2(I1-T1)/2
80 S=2I1I1/T & P0=A/F3 & K1=S/A
90 ! PRINT USING 100 ; P0/1000
100 IMAGE /,"P0=",DDEDD
110 ! CALC Icr1
120 ! CALC Icr 148
130 FOR I=1 TO 2
140 IF I=1 THEN E=ARS(E1) ELSE E=ARS(E2)
150 IF E<K1 THEN I(1)=1 & R2=2I1I1/(M1T) & P(1)=P0+R2/(E+R2) & GOTO 200
160 Z=0 ! FIND X,P FOR GIVEN E
170 CALL "P-E" ( D1,D2,D3,D4,E,F2,F3,L1,L2,M,N,P,P1,S1,S2,T,T1,T5,T6,X,Z )
180 CALL "ICR" ( D1,D2,D3,I2,L1,L2,M,S1,S2,T,T1,T5,T6,X )
190 P(1)=P & I(1)=I2
200 NEXT I
210 IF ARS(E2)>ARS(E1) THEN E7=E1/E2 ELSE E7=E2/E1
220 IF S1>0 OR S2>0 THEN GOTO 260
230 ! PLAIN MASONRY
240 IF E7>0 AND E7<1 THEN I6=(I(1)+I(2))/4 & GOTO 340
250 I6=(I1+MIN(I(1),I(2)))/4 & GOTO 340
260 ! REINF'D MASONRY
270 ! CALC Icr (P=0)
280 Z=3 ! FIND X(N.A) FOR C=1s
290 CALL "P-E" ( D1,D2,D3,D4,E,F2,F3,L1,L2,M,N,P,P1,S1,S2,T,T1,T5,T6,X,Z )
300 ! CALC Icr (P=0)
310 CALL "ICR" ( D1,D2,D3,I2,L1,L2,M,S1,S2,T,T1,T5,T6,X )
320 IF E7>0 AND E7<1 THEN I6=(I(1)+2*I2+I(2))/4 & GOTO 340
330 I6=(MIN(I(1),I(2))+2*I2+I1)/4
340 E6=I6/E5
350 ! PRINT USING 360 ; A,I1,S,P0,K1
360 IMAGE "A=",DDEDDDD,"I1=",D.DDE,"S=",D.DDE,"P0=",D.DDE,"K1=",DDD
370 ! PRINT USING 380 ; E1,E2,E7
380 IMAGE "E1=",DDD,"E2=",DDD,"E7=",DDD
390 ! PRINT USING 400 ; I(1),I(2),I6,E5
400 IMAGE "I(1)=",D.DDE,"I(2)=",D.DDE,"I6=",D.DDE,"E5=",D.DDE
410 SUBEND

```

```

10 SUB "P-E" (D1,D2,D3,D4,E,F,L1,L2,M,N,P,S1,S2,T,T1,T5,T6,X,T)
20 Z=0 FINDS X FOR E; Z=1 FINDS X FOR P; Z=2 DRAWS INT CURVE; Z=3 FINDS X FOR C=15 W=0
30 IS=T/20 & 16=T/200 & I=T+15 & O1=T/2+D4/2 & D2=1/2 & D3=1/2-D4/2
40 FOR K=1 TO 100
45 IF X=1 THEN 420
47 IF X>1.2+15 THEN X=X-15 ELSE I=I-16
50 T2=T3,T4=0
52 IF X=0 THEN X=1
60 IF I=D1 THEN 180
70 IF S1=0 THEN 180
80 F=N*F31(D1-X)/I & IF F>F2 THEN F=F2
90 T2=F*I*S1
100 IF I=D2 THEN 180
110 IF S2=0 THEN 140
120 F=N*F31(D2-X)/I & IF F>F2 THEN F=F2
130 T3=F*I*S2
140 IF I=D3 THEN 180
150 IF S2=0 THEN 180
160 F=N*F31(D3-X)/I & IF F>F2 THEN F=F2
170 T4=F*I*S1
180 IF X>T1 THEN T6=T1 ELSE T6=X
190 IF X=T-T1 THEN T5=0 ELSE T5=X-T+T1
200 F4=F31(X-T6)/I
210 C1=(F3-F4)*T6*L2/2 & C2=F4*T6*L2
220 C3=F31*X*L1/2
230 C4=F31*T5-T1*L2/(2*X)
240 P=C1+C2+C3+C4-T2-T3-T4
250 IF Z<3 THEN 60TO 280
260 IF P<0 AND 15>16 THEN X=X+15 & 15=16 & 60TO 390
270 IF P>0 THEN 390
280 M=C1*(1/2-T6/3)+C2*(1/2-T6/2)+C3*(1/2-T1/3)+T2*D4/2-T4*D4/2-C4*(T2-T1+T5/3)
290 IF P>0 THEN 300
295 IF 15>16 THEN X=X+15 & 15=16 & 60TO 390
296 P=1 & 60TO 420
300 E4=M/P
310 PRINT USING 320 : X,F/1000,E4
320 IMAGE "X=",DDD,"P=",DDDD,"E4=",DDDD
330 IF Z=1 THEN 360
340 IF ABS(E1)<E4 AND 15>16 THEN X=X+15 & 15=16 & 60TO 390
350 IF ABS(E1)<E4 THEN 420 ELSE 390
360 IF Z=2 THEN 410
370 IF F<P1 AND 15>16 THEN X=X+15 & 15=16 & 60TO 390
380 IF F<P1 THEN 420
390 NEXT K
400 60TO 420
410 DRAW M/1000000,P/1000 & 60TO 390
415 60TO 390
420 SUREND

```

```

10 SUB "ICK" (D1,D2,D3,D4,L1,L2,M,S1,S2,T,T1,T5,T6,X)
20 A2,A3,A4=0
30 IF X<D1 THEN A2=N1S1
40 IF X<D2 THEN A3=N1S1
50 IF X<D3 THEN A4=N1S1
60 A5=T5/L2 & A6=T6/L2
70 A1=A2+A3+A4+A5+A6+T1/L1
80 C5=(A2/D1+A3/D2+A4/D3+A5/(T-T1+T5/2)+A6/(1/2+L1*X^2/2)/A1
90 I2=A5/(T-T1+T5/2-C5)^2+A2/(D1-C5)^2+A3/(D2-C5)^2+A4/(D3-C5)^2+A6/(C5-T1/2)^2
100 I2=I2+L1*X^3/12+L1*X/(X/2-C5)^2+L2*(T5^3+T6^3)/12
110 SUBEND

```

```

10 SUB "GRAPHCS" (A,D4,E,E1,E2,E5,E7,F2,F3,H,I1,K1,L1,L2,N,P0,R2,S1,S2,T,T1,W)
15 OPTION BASE 1
20 R2=2+11/(A+1) & U1=1000 & U2=1000000
30 Y1=1.2+P0/U1 & X2=R2+P0/U2 & X3=X2/4 & Y2=Y1/4
40 PEN 1 & GCLEAR
50 SCALE -X3,X2,-Y2,Y1
60 XAXIS 0,10,0,X2
70 YAXIS 0,100,0,Y1
80 MOVE -3*X3/4,Y1/2 & LDIR 90 & LABEL "P(W)"
90 MOVE X2/2,-Y2 & LDIR 0 & LABEL "M(KN-a)"
100 FOR J=1 TO Y1/500
110 K=500+J
120 MOVE -5*X3/6,K & LDIR 0 & LABEL VAL$(K) & DRAW 1,K
130 NEXT J
140 FOR J=1 TO X2/50
150 K=50+J
160 MOVE K,-3*Y2/4 & LDIR 90 & LABEL VAL$(K) & DRAW K,10
170 NEXT J
180 MOVE 0,P0/U1
190 P5=P0/2
200 N=R2/(P0-P5)
210 DRAW N/U2,P5/U1
220 Z=2 & P1=P5 & CALL "P-E" (D1,D2,D3,D4,E,F2,F3,L1,L2,M,N,P,P1,S1,S2,T,T1,T5,T6,X,Z)
230 SUBEND

```

```

10 SUB "TTP-6" (D4,E1,E2,E6,F2,F3,H,X1,U1,U2,M,F,P0,P1,S1,S2,I,T1,M,Z2)
15 : Z2=1 INHIBITS TERTIARY DEF 'N CALC'NS
20 OPTION BASE 1
30 DIM M5(2),C(2),D1(1),D1(1),R1(1),D2(1),M(1),P2(1)
35 : PRINT "OUT P-6 METHOD"
40 : PRIMARY DEFLECTIONS
50 IF P(1)≠P(2) THEN P4,P2=P+1 ELSE P4,P2=P(2)
60 : PRINT USING 70 : P2/1000
70 IMAGE "P2="DDDD
80 I4=M/10 : U1=1000 : U2=1000000 : I7,I9,U,K2=0 : I5=P0/10 : I6=I5/5 : MAT P2=ZER
90 DB=0
100 FOR I=1 TO 11
110 D1(I),D2(I),M(I),D(I)=0
120 NEXT I
130 M1=P2IE1 : M2=P2IE2
140 FOR I=2 TO 10
150 I=(I-1)*I4
160 FOR J=1 TO 10
170 C(I)=(J-1)*I4 : C(2)=J*I4
180 FOR K=1 TO 2
190 M5(K)=M4C(I)*(H-C(I))/2+M2IC(I)/H+M1*(H-C(I))/H
200 NEXT K
210 EOSUR 860 : M4
220 D1(I)=D1(I)+A7*M6
230 NEXT J
240 IF ABS(D1(I))>DB THEN DB=ABS(D1(I))
245 IF Z2=1 THEN D(I)=D1(I)
250 NEXT I
260 IF I7=0 AND ABS((E1+E2)/2)+DB>T/2 AND S1=0 AND S2=0 THEN P4,P2=.5*P2 : GO TO 830
265 I7=1
270 : PRINT "STA FROM S'S"
280 FOR I=1 TO 11
290 : PRINT USING "DD,3X,5DDD" : I,D1(I)
300 NEXT I
305 IF Z2=1 THEN 560
310 : SECONDARY DEFL'NS
320 K=0
330 D9=0 : K=K+1
340 FOR I=2 TO 10
350 I=(I-1)*I4
360 FOR J=1 TO 10
370 C(I)=(J-1)*I4
380 M5(1)=D1(J)*P2
390 M5(2)=D1(J)*P2
400 EOSUR 860 : M4
410 D2(I)=D2(I)+A7*M6
420 NEXT J
430 IF ABS(D2(I))>D9 THEN D9=ABS(D2(I))
440 NEXT I
450 : PRINT "STA S'S SEC'Y S'S"
460 FOR I=1 TO 11
470 IF K=1 THEN D(I)=D1(I)+D2(I)
480 IF K>1 THEN D(I)=D(I)+D2(I)
490 : PRINT USING "DD,3X,2(5DDD,3X)" : I,D1(I),D2(I)
500 D1(I)=D2(I)
510 D2(I)=0

```


530 NEXT I

535 IF D9=DE OR 1.36 THEN P4,12=.74P2 & GOTO 830 * UNSTABLE WALL REDUCE P2

535 IF 19=0 THEN 560 * BYPASS TERTIARY DEFL.CYCLES

540 D8=D9

550 IF D9/2 THEN 330

560 * CALC P-G'S

570 * PRINT *STM *PH *P-G *H *H/M*

580 FOR I=1 TO 11

590 X=(I-1)/14

600 M3=M1*(H1-X/21/2+M11*(H-X)/H+M211/H

605 IF M3=0 THEN M3=.001

610 P1=P21D(I)

620 M(I)=M3+P1 & R(I)=M(I)/M3

630 * PRINT USING "DD,2X,3(SDDDD,3X),D.DD" ; 1,M3/U2,P1/U2,M(I)/U2,R(I)

640 NEXT I

650 * SELECT MAX MOM

660 R4,M4=0

670 FOR I=1 TO 11

680 IF M4<ABS(M(I)) THEN M4=ABS(M(I))

690 IF R4<R(I) THEN R4=R(I)

700 NEXT I

710 E=M4/P2 & IF E<K1 THEN P=P0(K1)/(E+K1) & M=P1E & GOTO 740

720 I=0 : FIND I,F FOR GIVEN E

730 CALL "P-E" (D1,D2,D3,D4,E,F2,F3,L1,L2,M,N,P,P1,S1,S2,I,T1,T5,T6,X,Z)

740 * PRINT "E(00) M(N-0) P(KN) P/P0 *H/M*

750 * R=P/P0

760 * PRINT USING "DDD.D,2(DDDD.D,21),.DD,2X,D.DD" ; E,M/U2,P/U1,R,M4

765 IF P>1.21P4 THEN P2=P4 & GOTO 920

770 P3=P2-P

780 IF ABS(P3)<.18P0 THEN I9=1 ELSE I9=0

790 IF ABS(P3)<.031P0 THEN 910

795 IF U=1 AND P3<0 THEN 910

810 IF P3<0 THEN P2=P2+16 & GOTO 815

812 IF P3>.5115 THEN P2=P2-15 ELSE P2=P2-14

815 K2=K2+1 & FOR I=1,10 & P2(I)=P2(I+1) & NEXT I & P2(10)=P2

816 IF K2<10 THEN 830

817 FOR I=1 TO 9 & IF P2=P2(I) THEN 820 * OSCILLATION DETECTOR

818 NEXT I

819 GOTO 830

820 FOR I=1 TO 10 & IF P2=P2(I) THEN P2=P2(I)

821 NEXT I

822 GOTO 920

830 * PRINT USING 840 : P2/U1

840 IMAGE /,"TRY P=",DDDD

850 GOTO 90

860 * SUPR M6

870 A7=(M5(1)+M5(2))/14/(21E6)

880 C6=C(1)+M5(1)+21M5(2))/14/(31M5(1)+M5(2))

890 IF K<C6 THEN M6=(H-C6)/X/H ELSE M6=(H-X)/166/H

900 RETURN

910 IF S1<0 OR S2<0 THEN 920

915 IF E>1/2 THEN P2,P4=.98P2 & GOTO 830

920 IF I2=0 THEN P(4)=P2/U1 ELSE P(8)=P2/U1

930 SUBEND

```

10 SUB 'APP-S' (A,D4,E1,E2,E6,F2,F3,H,I1,I2,L1,L2,N,P0,P1,S1,S2,T,T1,W)
12 OPTION BASE 1
15 ! PRINT "TAS DIR. (APPROX) P-S METHOD"
20 U1=1000 & U2=1000000
25 ! CALC MID HI E
30 E=(E1+E2)/2 & IF ABS(E)<.051T THEN E=.051T
40 GOSUB 120 ! P>.51P0
50 IF P>=.51P0 THEN 300
60 IF S1>0 OR S2>0 THEN GOSUB 170 & GOTO 300
70 GOSUB 250 ! P<.51P0,PLAIN
75 GOTO 300
80 ! SUBR QUADR.
85 D=SQR(B^2-4AC)/2
85 ON ERROR GOTO 320
100 P=(-(B-D)/(2A))
102 IF P>MIN(P(1),P(2)) THEN P=MIN(P(1),P(2))
105 RETURN
120 ! SUBR P>.51P0
130 A1=H^2/E/(B+E6)
140 B=E+51W/H^4/(384E6)+K1
150 C=W/H^2/B-F0/K1
155 GOSUB 80
160 RETURN
170 ! SUBR P<.51P0,REINFORCED
180 Z=3 ! FINDS I,M ETC FOR P=0
190 CALL "P-E" (D1,D2,D3,D4,E,F2,F3,L1,L2,M,N,P,P1,S1,S2,T,T1,T5,T6,I,Z)
200 M0=M
210 A1=H^2/E/(B+E6)
220 B=E+51W/H^4/(384E6)+2IM0/P0-K1
230 C=W/H^2/B-M0
235 GOSUB 80
240 RETURN
250 ! SUBR FC.51P0,PLAIN
260 A1=H^2/E/(B+E6)+T/P0-2IK1/P0
270 B=E+51W/H^4/(384E6)-T/2
280 C=W/H^2/B
285 GOSUB 80
290 RETURN
300 ! PRINT USING 310 : P/U1,F/P0
310 IMAGE "P= ".DDDD."P/P0= ".D.DD
315 GOTO 335
320 OFF ERROR & PRINT "NO SOL'N PROBABLY P<M0/MF" & P=0
335 P(5)=P/1000
340 GOTO 440
350 CALL "BRPHCS" (A,D4,E,E1,E2,E5,E7,F2,F3,H,I1,I2,L1,L2,N,P0,R2,S1,S2,T,T1,W)
360 ! CALC. P(e+6Pe+6W)+HL^2/B
370 Q1=P0/20 & P3=P0/2
380 FOR J=1 TO 10
390 E3=E+P3/H^2/E/(B+E6)+51W/H^4/(384E6)
400 M3=P3/E3+W/H^2/B
410 IF J=1 THEN MOVE M3/U2,P3/U1 ELSE DRAW M3/U2,P3/U1
420 P3=P3-Q1
430 NEXT J
435 COPY
440 SUBEND

```

```

10 SUB 'M4444' (A,D4,E,E1,E2,E3,E4,F2,F3,F4,H,I1,K1,L1,L2,N,P0,P1),S1,S2,T,T1,W)
15 OPTION BASE 1
20 ! PRINT "MODIFIED M44 MON. MAGNIFIER METHOD"
25 ! PRINT USING 26 ; B6
26 IMAGE 1, "F1", .000
30 ! PLOT SHORT SEC'M INT. CURVE
40 ! CALL 'GRAPHCS' (A,D4,E,E1,E2,E3,E4,F2,F3,H,I1,K1,L1,L2,N,P0,R2,S1,S2,T,T1,W)
50 ! INITIALIZE P-USE P'S FROM SUB'CONST'
60 P3=MIN(P1,P2) ? Q1=P3/10 ? Q2=Q1/4
70 J=0 ? U1=1000 ? U2=1000000
80 IF ABS(W)>0 THEN GOSUB 340 ? GOTO 100 ! SURR Max
90 IF E7>=0 THEN E=(E1+E2)/2 ELSE E=MAX(ABS(E1),ABS(E2))
100 IF E<0 THEN E=-E
110 IF P3>P0/2 THEN M=F1*(P0-P3) ? GOTO 150
120 Z=1 ? P1=P3
140 CALL "P-E" (D1,D2,D3,D4,E,F2,F3,L1,L2,M,N,P,P1,S1,S2,T,T1,T5,T6,I,Z)
150 M3=M
160 ! CALC Pcr (P5)
170 GOSUB 440 ! Pcr
180 IF P3>P5 THEN P3=.5*P5 ? GOTO 60
190 C5=.6+.4IE7
200 IF ABS(W)>0 THEN C5=1
210 GOSUB 440 ! Pcr
220 D5=C5/(1-P3/P5)
230 M5=P3*D5IE
240 ! IF J=0 THEN MOVE M5/U2,P3/U1 ? J=1
250 ! DRAW M5/U2,P3/U1
260 ! PRINT " P S P E M Pcr"
270 ! PRINT USING "DDDD,3(DDD,D,1),DDDDDDDD" ; P3/U1,D5,M5/U2,M3/U2,P5/U1
280 IF M5>M3 AND P3>0 THEN P3=P3-Q1 ? GOTO 60
285 IF P3<=0 THEN R(7)=0 ? SUREXIT
290 IF M5<M3 AND D1>D2 THEN P3=P3+Q1 ? Q1=Q2 ? GOTO 60
295 ! DRAW M3/U2,0 ? MOVE M3/U2,P3/U1 ? DRAW 6,P3/U1
296 ! MOVE 10,100+P3/U1 ? LDIR 0 ? LABEL VAL$(1P(P3/U1))
300 P(7)=P3/U1
310 ! PRINT "E(=) M(=) P(=) P/P0"
320 ! PRINT USING "SDDD,2(SDDDD,3),.DD,2X,D.DD" ; E,M3/U2,P3/U1,P3/P0,D5
330 GOTO 600
340 ! SURR Max
350 I4=H/10 ? M1=P3IE1 ? M2=P3IE2
360 M=0
370 FOR I=1 TO 10
380 I=I+1 ? I1=H-I
390 M6=M1I1/H+M2I1/H+M3I1(H-I)/2
400 IF M<ABS(M6) THEN M=ABS(M6)
410 NEXT I
420 E=M/P3
430 RETURN
440 ! SUBR Pcr

```

```

450 IF E>K1 AND S1>0 OR S2>0 THEN 6010 560
460 F9=4IP3/(3+H1(1-2IE/T))
470 E9=2ITIP3/(AIF9)
480 T9=E9IF6/F9
490 B9=8*(.5-E/T+T9/(2IT))3
500 ! E7=e1/e2
510 IF E7>0 THEN P5=9.87IE5189111/H2 2 6010 590
520 ! E7 NEG.
530 A9=ABS(E1)/(ABS(E1)+ABS(E2))
540 T6=10*(B9+A93(1-B9))+21A92+B93 2 IF T6>10 THEN T6=10
550 P5=T6IE5111/H2 2 6010 590
560 ! REINF'D WALL e>T/3
570 R3=.5-E/T 2 IF R3<.1 THEN R3=.1
580 P5=9.87IR3IE5111/H2
590 RETURN

```

```

10 SUB "APP-S1" (A,D4,E1,E2,E6,F2,F3,H,I1,K1,L1,L2,N,P0,P(),S1,S2,T,T1,M,M())
20 ! CALCULATES VARIABLE W (P=Ptest)
30 OPTION BASE 1
40 P=P(1)
50 ! CALC MID HT E
60 E=(E1+E2)/2 2 IF ABS(E)<.054T THEN E=.054T
70 ! CALC MrGP
80 IF P<.51P0 THEN 100
90 N=(P0-P)IK1 2 6010 150
100 IF S1>0 OR S2>0 THEN 140
110 ! PLAIN MASONRY, P<P0/2
120 M=PIT/2-P2IT/PG+24P2IK1/F0 2 6010 150
130 ! REINF'D, P<P0/2
140 P1=P 2 Z=1 2 CALL "P-E" (D1,D2,D3,D4,E,F2,F3,L1,L2,M,N,F2,P1,S1,S2,T,T1,T5,T6,X,Z)
150 W(5)=(N-P2IEH2/(6IE6)+P1E/2)/(54P1H4/(364IE6)+H2/6)
160 SURETIT

```

```

10 SUB "ITP-S1" (D4,E1,E2,E6,F2,F3,H,K1,L1,L2,N,P,P0,P1),S1,S2,T,T1,W,Z2,W1)
12 ! CALCULATES VARIABLE W (P CONSTANT)
15 ! Z2=1 INHIBITS TERTIARY DEFL'N CALC'NS
20 OPTION BASE 1
30 DIM M5(2),C(2),D(11),D1(11),R(11),D2(11),M(11)
35 ! PRINT "DLT P-C METHOD"
40 ! PRIMARY DEFLECTIONS
50 IF P(1)=P(2) THEN P4,P2=P(1) ELSE P4,P2=P(2)
60 ! PRINT USING 70 ; P2/1000
70 IMAGE "P2="DDDD
80 J4=H/10 ? U1=1000 ? U2=1000000 ? I7,I9,U,K2=0 ? I5=P0/10 ? I6=I5/5
81 ! CALC M2P=P1est+W2init.
84 IF P2>P0/2 THEN M=K1*(P0-P2) ? GOTO 86
85 CALL "P-E" ( D1,D2,D3,D4,E,F2,F3,L1,L2,N,N,P,P2,S1,S2,T,T1,T5,T6,X,I )
86 M2=81M/H^2
90 D8=0 ! PRIMARY DEFL'NS
100 FOR I=1 TO 11
110 D1(I),Q2(I),M(1),D(I)=0
120 NEXT I
130 M1=P2IE1 ? M2=P2IE2
140 FOR J=2 TO 10
150 I=(I-1)*14
160 FOR J=1 TO 10
170 C(I)=(J-1)*14 ? C(2)=J*14
180 FOR K=1 TO 2
190 M5(K)=M2/C(K)*(H-C(K))/2+M2/C(K)/H*M1*(H-C(K))/H
200 NEXT K
210 GOSUB 860 ! M6
220 D1(I)=D1(I)+A7*M6
230 NEXT J
240 IF ABS(D1(I))>D8 THEN D8=ABS(D1(I))
245 IF Z2=1 THEN D(I)=D1(I)
250 NEXT I
305 IF Z2=1 THEN 560
310 ! SECONDARY DEFL'NS
320 K=0
330 D9=0 ? K=K+1
340 FOR I=2 TO 10
350 I=(I-1)*14
360 FOR J=1 TO 10
370 C(I)=(J-1)*14
380 M5(1)=D1(J)*P2
390 M5(2)=D1(J+1)*P2
400 GOSUB 860 ! M6
410 D2(I)=D2(I)+A7*M6
420 NEXT J
430 IF ABS(D2(I))>D9 THEN D9=ABS(D2(I))
440 NEXT I

```

```

450 : PRINT 'STA S'S SEC'Y S'S'
460 FOR I=1 TO 11
470 IF K=1 THEN D(1)=D1(1)+D2(1)
480 IF K>1 THEN D(1)=D(1)+D2(1)
490 : PRINT USING 'DD,3X,2(SDD0,3X)'; 1,D(1),D2(1)
500 D1(1)=D2(1)
510 D2(1)=0
520 NEXT I
530 IF I9=0 THEN 560 : BYPASS TERTIARY DEFL.CYCLES
540 D8=D9
550 IF D9>2 THEN 330
560 : CALC P- 'S
570 : PRINT 'STN PM P-6 MT MT/MP'
580 FOR I=1 TO 11
590 X=(1-I)*14
600 M3=W2*(H*I-X^2)/2+M1*(H-X)/H+M2*I/H
605 IF M3=0 THEN M3=.001
610 P1=P2/D(1)
620 M(1)=M3+P1 : R(1)=M(1)/M3
630 : PRINT USING 'DD,2X,3(SDD0,3X),D.DD'; 1,M3/U2,P1/U2,M(1)/U2,R(1)
640 NEXT I
650 : SELECT MAX M3
660 R4,M4=0
670 FOR I=1 TO 11
680 IF M4<ABS(M(1)) THEN M4=ABS(M(1))
690 IF R4<R(1) THEN R4=R(1)
700 NEXT I
710 M3=M-M4
720 IF ABS(M3)<.1*M THEN 610
730 IF M4<M THEN W2=1.1*W2 : 605
740 IF M4>M THEN W2=.9*W2 : 605
750 IF Z2=0 THEN W(4)=W2 ELSE W(8)=W2
760 GOTO 930
770 : SUPR M
780 A7=(M5(1)+M5(2))*14/(2*E6)
790 C6=C(1)+(M5(1)+2*M5(2))*14/(3*(M5(1)+M5(2)))
800 IF X<C6 THEN M6=(H-C6)*I/H ELSE M6=(H-X)*C6/H
810 RETURN
820 SUBEND

```

```

10 SUB "HLMH1" (A,D4,E,E1,E2,E5,E7,F2,F3,F6,H,I1,K1,L1,L2,N,P0,P1,S1,S2,T,T1,W(Y)
15 OPTION BASE 1
30 : PLOT SHORT SEC'N INT. CURVE
40 : CALL "ERPHCS" ( A,D4,E,E1,E2,E5,E7,F2,F3,H,I1,K1,L1,L2,N,P0,R2,S1,S2,T,T1,W )
50 : INITIALIZE P-USE P'S FROM SUB"CONST"
60 P3=P(1)
70 J=0 : U1=1000 : U2=1000000
90 IF E7>=0 THEN E=(E1+E2)/2 ELSE E=MAX(ABS(E1),ABS(E2))
100 IF E<0 THEN E=-E
110 IF P3=P0/2 THEN M=K1*(P0-P3) : GOTO 150
120 Z=1 : P1=P3
140 CALL "P-E" ( D1,D2,D3,D4,E,F2,F3,L1,L2,M,N,P,P1,S1,S2,T,T1,T5,T6,X,Z )
150 M=81M/H^2
155 GOSUB 340 : SUBR Mmax
160 : CALC Pcr (P5)
170 GOSUB 440 : L Pcr
180 C5=.6+.41E7
200 IF ABS(M)>0 THEN C5=1
210 GOSUB 440 : Pcr
220 D5=C5/(1-P3/P5)
230 M5=M41D5
240 : IF J=0 THEN MOVE M5/U2,P3/U1 : J=1
250 : DRAW M5/U2,P3/U1
260 : PRINT " P S P E .M Pcr"
270 : PRINT USING "DDDD,3(DDD.D,11),DDDDDDDD" ; P3/U1,D5,M5/U2,M/U2,P5/U1
275 M3=M-M5
276 IF ABS(M3)<.11M THEN W(7)=M : SUBEXIT
280 IF M5>M THEN W=.91M : GOTO 155
285 IF M5<M THEN W=1.11M : GOTO 155
295 : DRAW M5/U2,0 : MOVE M5/U2,P3/U1 : DRAW 0,P3/U1
296 : MOVE 10,100+P3/U1 : LDIR 0 : LABEL VALS(1P(P3/U1))
300 P(7)=P3/U1
310 : PRINT "E(10) .M(EN-a) P(EN) . P/P0"
320 : PRINT USING "SDDD.,2(SDDDD,31),.DD,21,0.DD" ; E,M5/U2,P3/U1,P3/P0,D5
330 GOTO 600
340 : SUBR Mmax
350 I4=M/10 : M1=P31E1 : M2=P31E2
360 M4=0
370 FOR I=1 TO 10
380 I=114 : I1=M-I
390 M6=M1I1I1/M+M2I1I1/M+M3I1I1(M-I)/2
400 IF M4<ABS(M6) THEN M4=ABS(M6)

```

```

410 NEXT I
420 E=M4/P3
430 RETURN
440 ! SUBR Pcr
450 IF E>X1 AND S1>0 OR S2>0 THEN GOTO 560
460 F9=41P3/(31A1(1-21E/T))
470 E9=211P3/(A1F9)
480 T9=E9F6/F9
490 B9=81(.5-E/T+T9/(21T))^3
500 ! E7=e1/e2
510 IF E7>0 THEN P5=9.871E5189111/H^2 @ GOTO 590
520 ! E7 NEG.
530 A9=ABS(E1)/(ABS(E1)+ABS(E2))
540 T8=101(B9+A9^31(1-B9))+21A9^2+T9^3 @ IF T8>10 THEN T8=10
550 P5=T81E5111/H^2 @ GOTO 590
560 ! REINF'D WALL e>1/3
570 R3=.5-E/T @ IF R3<.1 THEN R3=.1
580 P5=9.871R31E5111/H^2
590 RETURN
600 SURFND

```



```

10 MAIN PGM RESIST
20 OPTION BASE 1
30 DIM I(3), P(2)
40 PRINT ALL
50 DISP "INPUT L, L1, T, T1, e1, e2"
60 INPUT L, L1, T, T1, E1, E2
70 DISP "INPUT F'u, Fy, As1, As2, Ds1"
80 INPUT F1, F2, S1, S2, B1
90 DISP "INPUT Ea&Max, Mas, STRAIN"
100 INPUT E5, E3
105 NORMAL
110 CALL "CONST2" ( A, D4, E1, E2, E3, E5, E6, E7, F1, F2, F3, I1, I(1), I6, K1, L, L1, L2, N, P(1), S1, S2, T, T1, T6 )
120 CALL "GRAPH2" ( A, D4, E, E1, E2, E5, E7, F2, F3, H, I1, K1, L1, L2, N, P(1), R2, S1, S2, T, T1, W )
130 COPY
140 END

```

```

10 SUB "CONST2" (A, D4, E1, E2, E3, E5, E6, E7, F1, F2, F3, I1, I(1), I6, K1, L, L1, L2, N, P(1), S1, S2, T, T1, T6)
15 OPTION BASE 1
20 ! CALC. CONSTANTS
30 L2=L-L1 ? F3=E3/E5 ? IF F3>F1 THEN F3=F1
40 N=200000/E5
50 ! CALC UNCRACKED SECT PROP
60 A=T*L1+2*T1*L2
70 I1=L*L3/12+2*T1*I1+3*L2/12+2*T1*L2*((T-T1)/2)2
80 S=2*I1/T ? P(1)=A/F3 ? K1=S/A
110 ! CALC Ieff
120 ! CALC Icr T&B
130 FOR I=1 TO 2
140 IF I=1 THEN E=ARS(E1) ELSE E=ARS(E2)
150 IF E<K1 THEN I(1)=T1 ? R2=2*I1/(A*I) ? P(1)=P(1)+R2/(E+R2) ? GOTO 200
160 I=0 ! FIND X, P FOR GIVEN E
170 CALL "P-E" ( D1, D2, D3, D4, E, F2, F3, L1, L2, N, P, P1, S1, S2, T, T1, T5, T6, X, Z )
180 CALL "ICR" ( D1, D2, D3, I2, L1, L2, N, S1, S2, T, T1, T5, T6, X )
190 P(1)=P ? I(1)=I2
200 NEXT I
210 IF ARS(E2)>ARS(E1) THEN E7=E1/E2 ELSE E7=E2/E1
220 IF S1>0 OR S2>0 THEN GOTO 260
230 ! PLAIN MASONRY
240 IF E7>0 AND E7<=1 THEN I6=(I(1)+I(2))/4 ? GOTO 340
250 I6=(I1+MIN(I(1), I(2)))/4 ? GOTO 340
260 ! REINF'D MASONRY
270 ! CALC Icr (P=0)
280 I=3 ! FIND X(N, A) FOR C=Ts
290 CALL "P-E" ( D1, D2, D3, D4, E, F2, F3, L1, L2, N, P, P1, S1, S2, T, T1, T5, T6, X, Z )
300 ! CALC Icr (P=0)
310 CALL "ICR" ( D1, D2, D3, I2, L1, L2, N, S1, S2, T, T1, T5, T6, X )
312 I(3)=I2
315 PRINT USING 316 : P(1), N
316 IMAGE 27, "Po=", D.DDE, 4X, "Mo=", D.DDE
320 IF E7>0 AND E7<=1 THEN I6=(I(1)+2*I2+I(2))/4 ? GOTO 340
330 I6=(MIN(I(1), I(2))+2*I2+I1)/4

```

```

340 E6=16+E5
350 PRINT USING 360 ; A,11
360 IMAGE "Am=",DDDDDD,6X,"In=",D.DDE
370 PRINT USING 380 ; 1(1),1(2)
380 IMAGE "I1=",D.DDE,4X,"I2=",D.DDE
390 PRINT USING 400 ; 16,E6
400 IMAGE "Ieff=",D.DDE,2X,"E1eff=",D.DDE
410 PRINT USING 420 ; 1(3),K1
420 IMAGE "Icr=",D.DDE,2X,"ek=",DD,2/
430 SUBEND

```

```

10 SUB "GRPHC2" (A,D4,E,E1,E2,E5,E7,F2,F3,H,11,K1,L1,L2,N,P0,R2,S1,S2,T,11,W)
15 OPTION BASE 1
20 R2=2+11/(A+1) : U1=1000 : U2=1000000
30 Y1=1-21P0/U1 : X2=.6+R21P0/U2 : X3=X2/4 : Y2=Y1/4
40 PEN 1 : GCLEAR
50 SCALE -X3,X2,-Y2,Y1
60 XAXIS 0,1,0,X2
70 YAXIS 0,20,0,Y1
80 MOVE -3+X3/4,Y1/2 : LDIR 90 : LABEL "P(KN)"
90 MOVE X2/2,-Y2 : LDIR 0 : LABEL "N(KN-a)"
100 FOR I=1 TO Y1/200
110 K=200+I
120 MOVE -5+X3/8,K : LDIR 0 : LABEL VAL$(K) : DRAW X2,K
130 NEXT I
140 FOR I=1 TO X2/10
150 K=10+I
160 MOVE X,-3+Y2/4 : LDIR 90 : LABEL VAL$(K) : DRAW K,Y1
170 NEXT I
175 MOVE 0,0 : DRAW P0:ABS(E1)/U2,P0/U1 : MOVE 0,0 : DRAW P0:ABS(E2)/U2,P0/U1
180 MOVE 0,P0/U1
190 P9=P0/2
200 N=R2+(P0-P9)
210 DRAW N/U2,P9/U1
220 CALL "P-E1" (D1,D2,D3,D4,E,F2,F3,L1,L2,H,K,F,P1,S1,S2,T,11,T5,T6,X)
250 SUBEND

```

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS
STANDARD REFERENCE MATERIAL 1010a
(ANSI and ISO TEST CHART No. 2)

APPENDIX C

a) single curvature

b) double curvature

FIG. 3 REASONING BEHIND THE EFFECTIVE INERTIA ASSUMPTION

LIBRARY

CARLETON UNIVERSITY

Manuscript Thesis

Unpublished theses submitted for degrees and deposited in the University Library are open for inspection, but are to be used only with due regard for the rights of the authors. Bibliographical references may be noted, but passages may be copied only with the permission of the authors, and proper credit must be given in subsequent written or published work. Extensive copying or publication of the thesis in whole or in part requires also the consent of the Chairman of the appropriate department.

Theses may be reproduced on microfilm for use in place of the manuscript itself provided the rules listed above are strictly adhered to and the rights of the author are in no way jeopardized.

This thesis by *Fenton V.C.* has been used by the following persons, whose signatures attest their acceptance of above restrictions.

A library which borrows this thesis for use by its patrons is expected to secure the signature of each user.

Name and address

Date

END

03-07-87

FIN