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UHRIG,R.E. Florida Power & Light Co.
RECIP.NAME RECIPIENT AFFILIATION
CLARK,R.A. Operating Reactors Branch 3

SUBJECT: Forwards response to IE Bulletin 80-11, "Masonry Wall Design," supplementing response to NRC 820805 request for addl info.((see repts"

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FPL
FLORIDA POWER & LIGHT COMPANY

October 4, 1982
L-82-428

Office of Nuclear Reactor Regulation
Attention: Mr. Robert A. Clark, Chief
Operating Reactors Branch #3
Division of Licensing
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Dear Mr. Clark:

Re: St. Lucie Unit 1
Docket No. 50-335
IE Bulletin 80-11
Additional Information

We have reviewed your letter dated August 5, 1982, which requested additional information concerning our response to IE Bulletin 80-11 (Masonry Wall Design). Our response is attached.

Should you or your staff have any further questions on this subject, please contact us.

Very truly yours,

Robert E. Uhrig

Robert E. Uhrig
Vice President
Advanced Systems & Technology

REU/PLP/mbd

Attachment

cc: Harold F. Reis, Esquire

A001

ST LUCIE PLANT - UNIT 1
IE BULLETIN 80-11: MASONRY WALLS
NRC REQUEST FOR ADDITIONAL INFORMATION

Item 1.

Indicate the number of walls of the stack bond type and provide a sample calculation to obtain moment and shear stresses of a typical stack bond wall.

Response:

The number of walls of the stack bond type is 65. A sample calculation of wall No. 34 that shows all analysis details is provided in Attachment A.

Item 2.

A sample calculation to indicate how the effect of higher modes of vibration is considered in the analysis.

Response:

The re-evaluation of all walls has been performed by ANSYS computer program using mode-frequency analysis. A minimum of 12 modes has been considered in the analysis.

Item 3.

With respect to Table 1 of Reference 2, justify by any existing test data the values for allowable shear and tension of collar joints.

Response:

Collar joints strength has not been relied upon in the re-evaluation of the walls.

Item 4.

With reference to Section V, Table 1 of Reference 2, justify the use of an increase factor of 1.7 for tension normal to bed joint. SEB criteria (5) allow only 1.3. If the Licensee intends to use any existing test data to justify this increase factor, the Licensee is requested to discuss the applicability of these tests to the masonry walls at the plant with particular emphasis on the following: boundary conditions, type of loads, sizes of walls, and type of masonry construction (block type, grouted, or ungrouted).

Response:

Based on the tests performed in the National Concrete Masonry Association (NCMA), the average modulus of rupture for walls built with Type M and S mortar is 88 psi (Table 1, Attach B) on net area of hollow units. The safety factor for factored loads is equal to $88/1.66 \times 0.5 \sqrt{mo} = 2.50$ (where mo = 1800 psi). For grouted units, the average modulus of rupture averaged 157 psi (Table 2, Attach B) from the tests by NCMA. The test values for 12" nominal thickness walls were not included in the average since the test model, composed of 4 inch concrete brick and 8 inch hollow block, does not correspond to St Lucie 1 construction. The safety factor for factored loads is equal to $157/67 = 2.34$ (since $1.67 \times 1.0 \sqrt{mo} > 67$ max value). Therefore, the increase of 1.67 for tension normal to bed joint appeared reasonable for factored loads based on the static tests.

The boundary conditions considered for a typical unreinforced wall are:

1) simple support at the top due to two clip angles restrained on both sides of the wall.

2). fixed end support at the bottom due to 3/8" mortar bedding provided on top of the slab.

3) On both sides of the wall, a free boundary is considered, even though in some cases the wall may have ties with other walls.

Since the wall is analyzed statically once its earthquake coefficients are obtained, the allowable tension normal to bed joint from the tests is applicable to the masonry walls at the plant.

Item 5.

a. In Reference 2, the Licensee indicated that "yield-line theory", "plastic design", and "arching analysis" have been used to qualify some of the masonry walls. The NRC, at present, does not accept the application of these methods to masonry walls in nuclear power plants in the absence of conclusive evidence to justify this application. Before any conclusion can be made about these methods, the Licensee is requested to provide any existing test data to justify the use of each technique metioned above. The applicability of the tests should be discussed for the following areas:

-Nature of the loads
-Boundary conditions
-Materials used
-Wall sizes
-Amount and distribution of reinforcement.

b. The Licensee is also requested to indicate the number of walls which were qualified by each method and provide the resulting stresses and displacements for these walls.

c. Provide a sample calculation illustrating how stresses and displacement were calculated by each method (yield-line theory, plastic design, and arching analysis).

Response:

a. The application of "yield-line theory" with "plastic design" techniques to masonry walls at the plant is merely a demonstration of finding the upper bound of the wall capability and its corresponding displacements. References on the use of "yield-line theory" to masonry walls are given in Appendix 3A (Attachment F) "A Literature Survey Transverse Strength Of Masonry Walls" pages 3A-3 and 3A-19 of "Recommended Guildlines for the Reassessment of Safety Related Concrete Masonry Walls" prepared by Owners and Engineering Firms Informal Group on Concrete Walls, October 6, 1980.

As indicated in the sample calculation (Attachment C) for wall No. 205, the boundary conditions are originally assumed fixed at the bottom and simply supported at the top. The resulting stresses are all within allowables according to the re-evaluation criteria except the splice or development length is slightly inadequate. Consequently, the boundary condition of the bottom of the wall was changed to a hinged support. As a result, the frequency was reduced, the earthquake coefficient increased and the maximum out-of-plane bending moment increased. However, the actual rebar tensile stress under this condition is still less than the yield strength which is 40 ksi although larger than the acceptance criteria of 0.9 fy. Then, a calculation was made to obtain the displacement of the wall assuming plastic hinges formed at the bottom and the middle of the wall. The plastic moment of the wall section was obtained by assuming a rectangular compression stress of 0.85 f́m and rebar yielding prior to crushing of masonry. The resulting stresses and displacements were then judged to be acceptable considering plant operability and conservative assumptions.

The application of "arching analysis" to masonry walls at St Lucie Unit 1 is based on the report "Response of Arching Walls and Debris from Interior Walls caused by Blast Loading" by Gabrielson G., Wilton C., and Kaplan K. URS Report 7030-23, URS Research Co., 1975 and its abstract written in the "Recommended Guidelines for the Reassessment of Safety Related Concrete Masonry Walls" prepared by Owners and Engineering Firms Informal Group on Concrete Masonry Walls, October 6, 1980.

b. The number of walls which were qualified by "yield-line theory with plastic design" is 3 (i.e, Wall No. 205, 201, 202). The number of walls which were qualified by "arching analysis" is 2 (i.e., wall No. 116A, 162A).

The resulting stresses and displacements for the group of walls No. 205, 201 and 202 are:

Rebar Tension	Flexual Compression	Displacement
39 Ksi	0.765 Ksi	0.98 in

The resulting stresses and displacements for walls qualified by arching analysis are:

Wall No.	Axial Compression	Shear	Displacement
116 A	0.021 Ksi	0.033 Ksi	0.045 in
162 A	0.113	0.026	0.083

c. Since only two special analysis methods were utilized, a sample calculation for each method is provided in Attachment C.

Item 6.

Provide sample drawings of wall modifications, and clarify whether the modified walls were qualified under working stress conditions.

Response:

The sketches of wall modification (wall No. 114) are included in Attachment D. All modified walls are qualified under working stress conditions.

Item 7.

The Licensee reported that one of the walls missing top supporting angles was inaccessible during normal plant operation, and that it would be repaired during the 1981 refueling outage. Indicate the current status of this wall, as well as the status of modifications of the other walls.

Response:

Modification to the one wall which was inaccessible during normal operation was completed during the 1981 outage. Modifications to the other walls have been completed.

Item 8.

Provide a sample calculation illustrating how stresses were calculated for a multi-wythe wall.

Response:

A sample calculation illustrating how stresses were calculated for a multi-wythe wall is provided in Attachment E.

Item 9.

Provide a sample drawing of a finite element model to illustrate how openings and attachments were considered in the model.

Response:

A sample drawing of a finite element model to illustrate how openings and attachments were considered in the model is provided in Attachment A.

Item 10.

Indicate the critical damping value used for the operating basis earthquake (OBE). Justification should be given if it is higher than 4% as specified in Regulatory Guide 1.61.

Response:

The critical damping values specified for use in the analysis are as follows:

For OBE, 2% for uncracked walls and 4% for cracked reinforced walls.

For DBE, 2% for uncracked walls and 7% for cracked reinforced walls.

The largest damping value available from the original St Lucie 1 floor response spectra curves is 5%. These curves were conservatively used in lieu of 7% curves for DBE analysis.

In those cases where the integrity of the wall could not be demonstrated using the more conservative 5% curves, the applicable 7% curves were generated specifically for this analysis and used in accordance with the aforementioned criteria.

ATTACHMENT A (10 SHEETS)

BY J. deMATOS DATE 11/11/80 SHEET 1 OF ____

CHKD. BY C. SHIH DATE 12/29/80 OFS NO. 2524902 DEPT. NO. ESC

CLIENT FLO

PROJECT ST. LUCIE #1

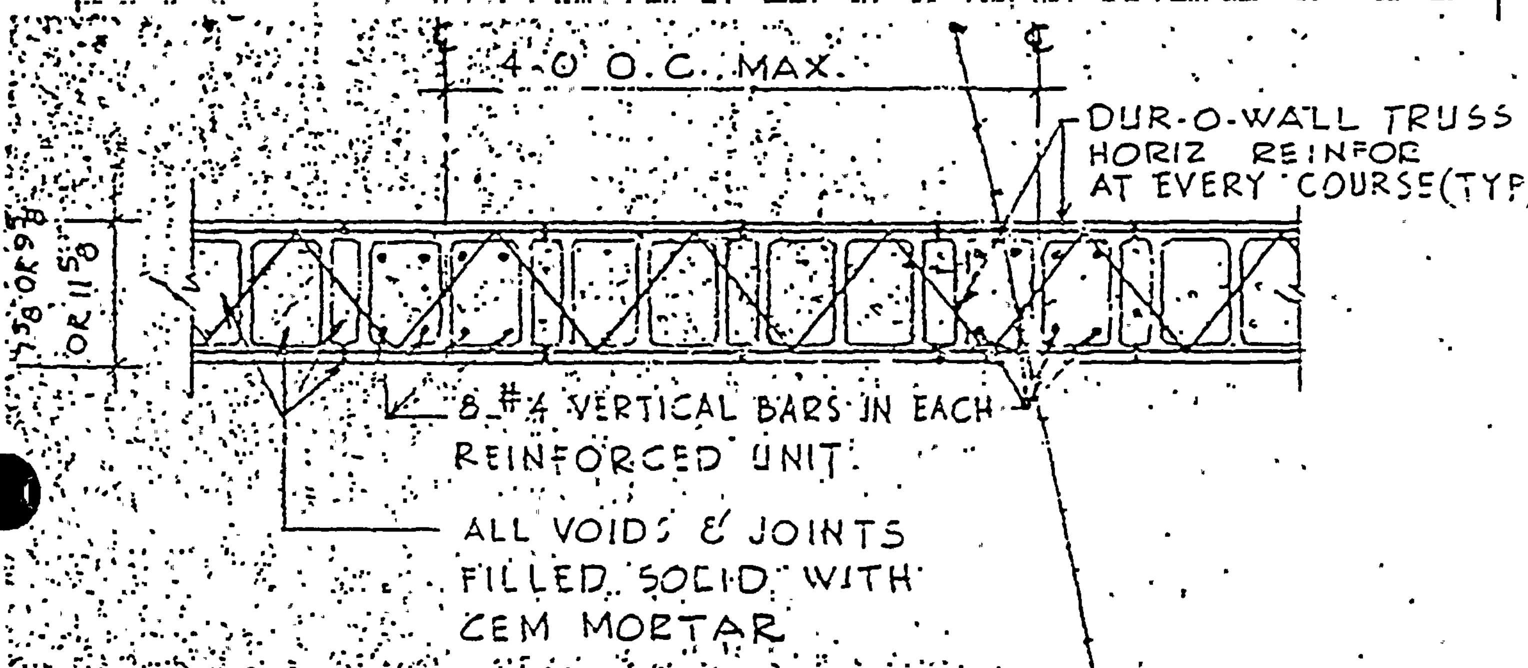
SUBJECT WALL # 34, 35 FREQUENCY ANALYSIS

WALL NO. 34 : FROM INSPECTION DATA SHEET

COMPOSITION : 12" × 8" × 16" BLOCK , REBAR , DUR-O-WAL

LOCATION : FL. EL. = 0.5 RAB , WALL SURFACE : N & S

DRAWING : BCS 128-4.300 ,



TYPICAL PLAN OF REINFORCED & STACK BOND CONC BLOCK WALL

MIN. COMPRESSIVE STRENGTH OF C-90 MASONRY UNIT = 600 PSI GROSS AREA

FOR NET AREA, COMPRESSIVE STRENGTH = 600/0.54 = 1111 PSI

* [ACTUAL TESTING DATA SHOWS COMPRESSIVE STRENGTH = 1302 PSI (GROSS AREA)
= 1302/0.54 = 2411 PSI (NET AREA) → f'_m = 1520 PSI

THE CORRESPONDING COMPRESSIVE STRENGTH OF MASONRY f'_m = 900 PSI FOR TYPE S MORTAR (ACI 531 TABLE 4.3)

$E_m = 0.9 \times 10^6$ PSI , $E_s = 29 \times 10^6$ PSI , m_0 = 1800 PSI

$n = E_s/E_m = 32$, REBAR f_y = 40,000 PSI (GRADE 40)

* SEE PAGE 10

581/8-81

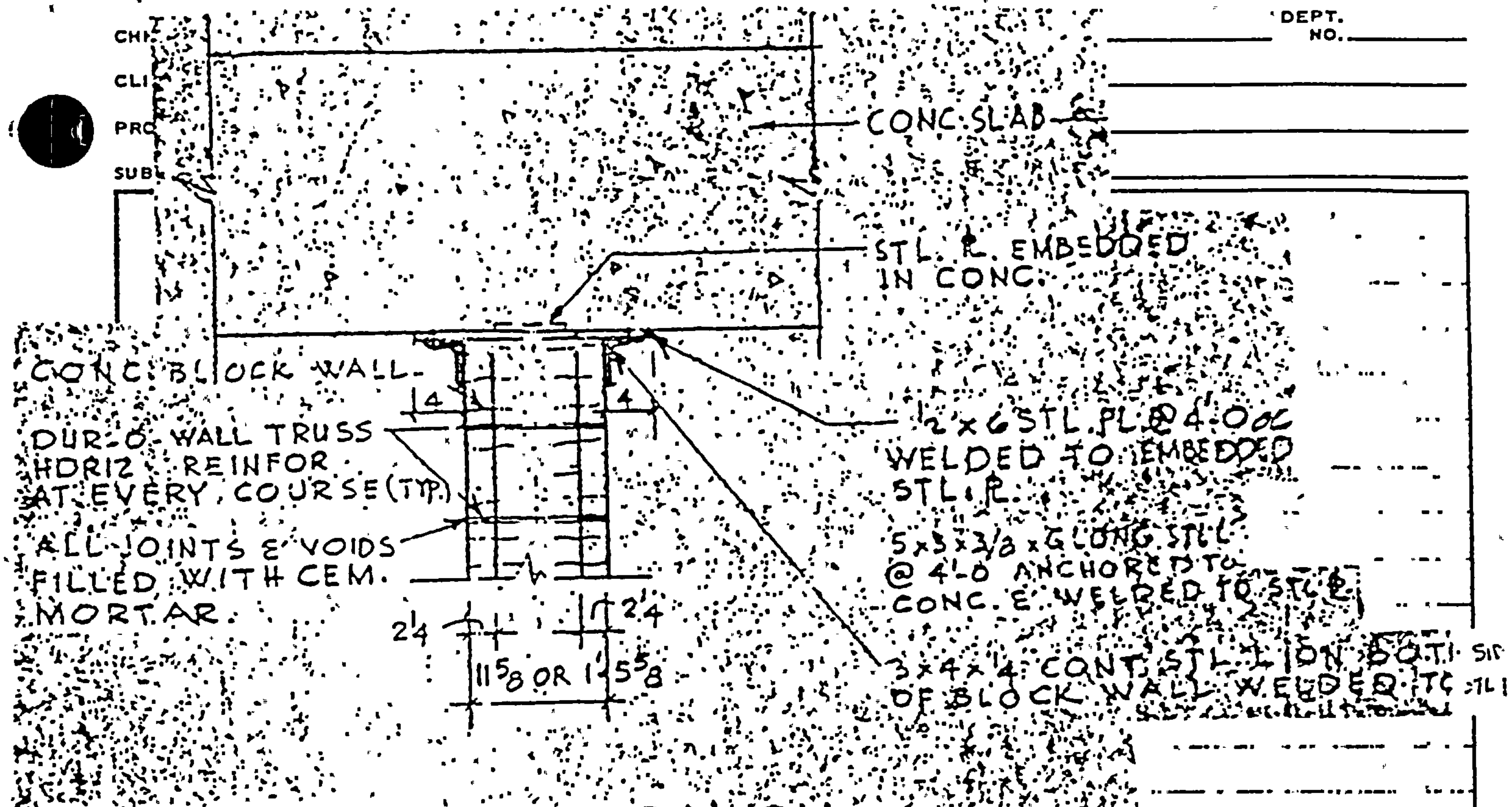
EBASCO SERVICES INCORPORATED

Att. A 2 of ?

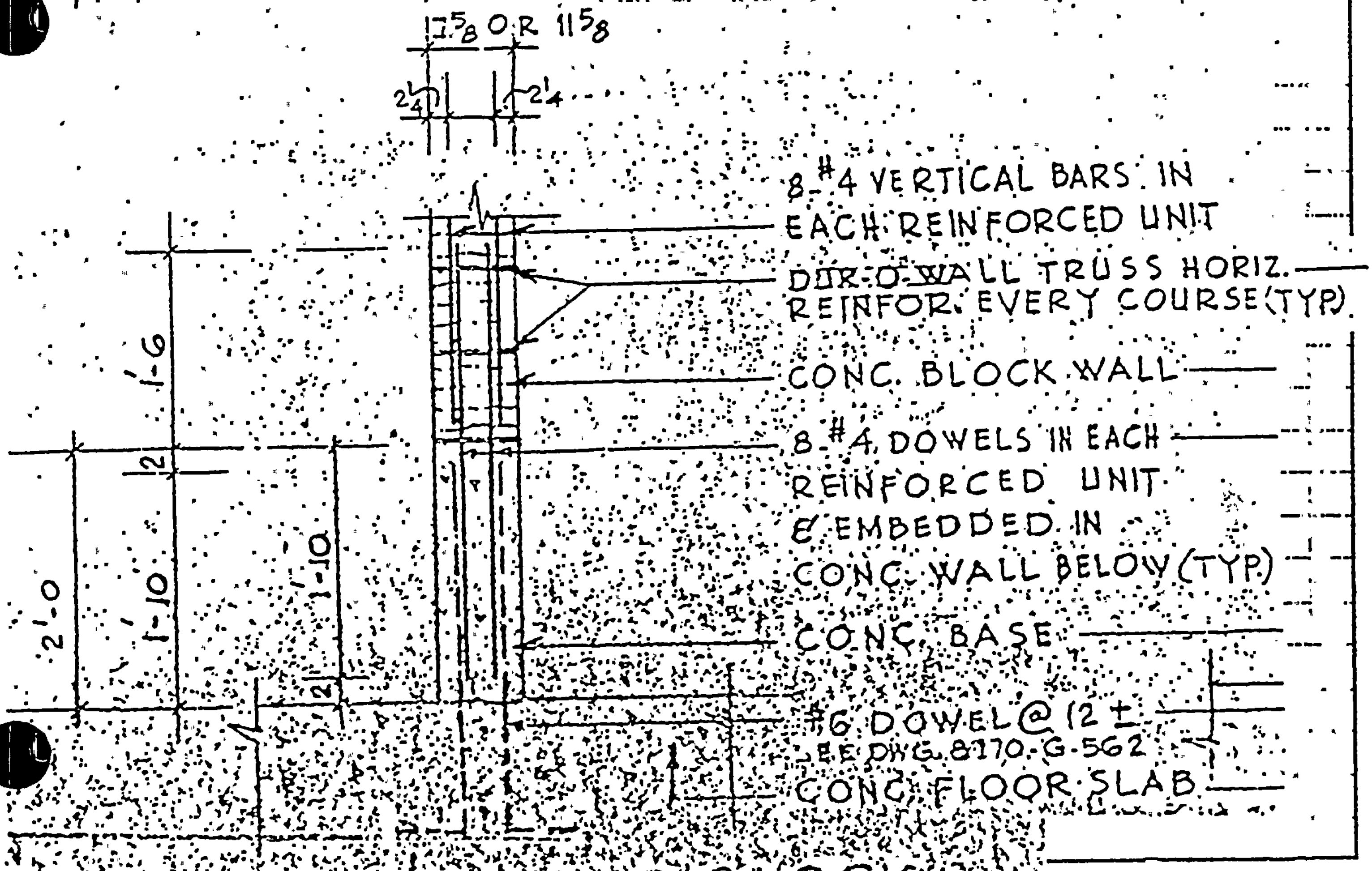
BY C. SHIH DATE 9/10/82

SHEET 2 OF

DEPT. NO.



WALL SECTION GH



8 & 10 CONC. BLOCK

A) UNCRACKED SECTION ANALYSIS:

FOR 1 BLOCK WITHOUT REBAR, MOMENT OF INERTIA: (UNCRACKED SECT)

$$I_m = \frac{1}{12}(12)(11.625)^3 = 2095 \text{ in}^4$$

BLOCK FILLED SOLID WITH CEMENT MORTAR.

FOR RINFORCED BLOCK, MOMENT OF INERTIA OF THE UNCRACKED SECT.

$$A_s = 8 \times 0.196 = 1.57 \square''$$

$$A_s(n-1) = 48.67 \square''$$ AREA OF REBARS TRANSFORMED TO MASONRY.

$$I_{m+s} = 2095 + \frac{1}{12}(48.67)(1)^3 + 48.67(8.81-5.81)^2$$

$$= 2537 \text{ in}^4$$

TOTAL MOMENT OF INERTIA OF THE TRANSFORMED, UNCRACKED SECTION: (THE WALL CONSISTS OF 4 REINFORCED UNITS, 8.75 UNREINF.)

$$I_t = 4(2537) + 8.75(2095) = 28,479 \text{ in}^4$$

EQUIVALENT PLATE THICKNESS FOR FINITE ELEMENT MODEL:

$$t_e = \sqrt[3]{\frac{12 I_t}{b}} = \sqrt[3]{\frac{12 \times 28479}{17 \times 12}} = 11.85'' = 0.99 \text{ FT.}$$

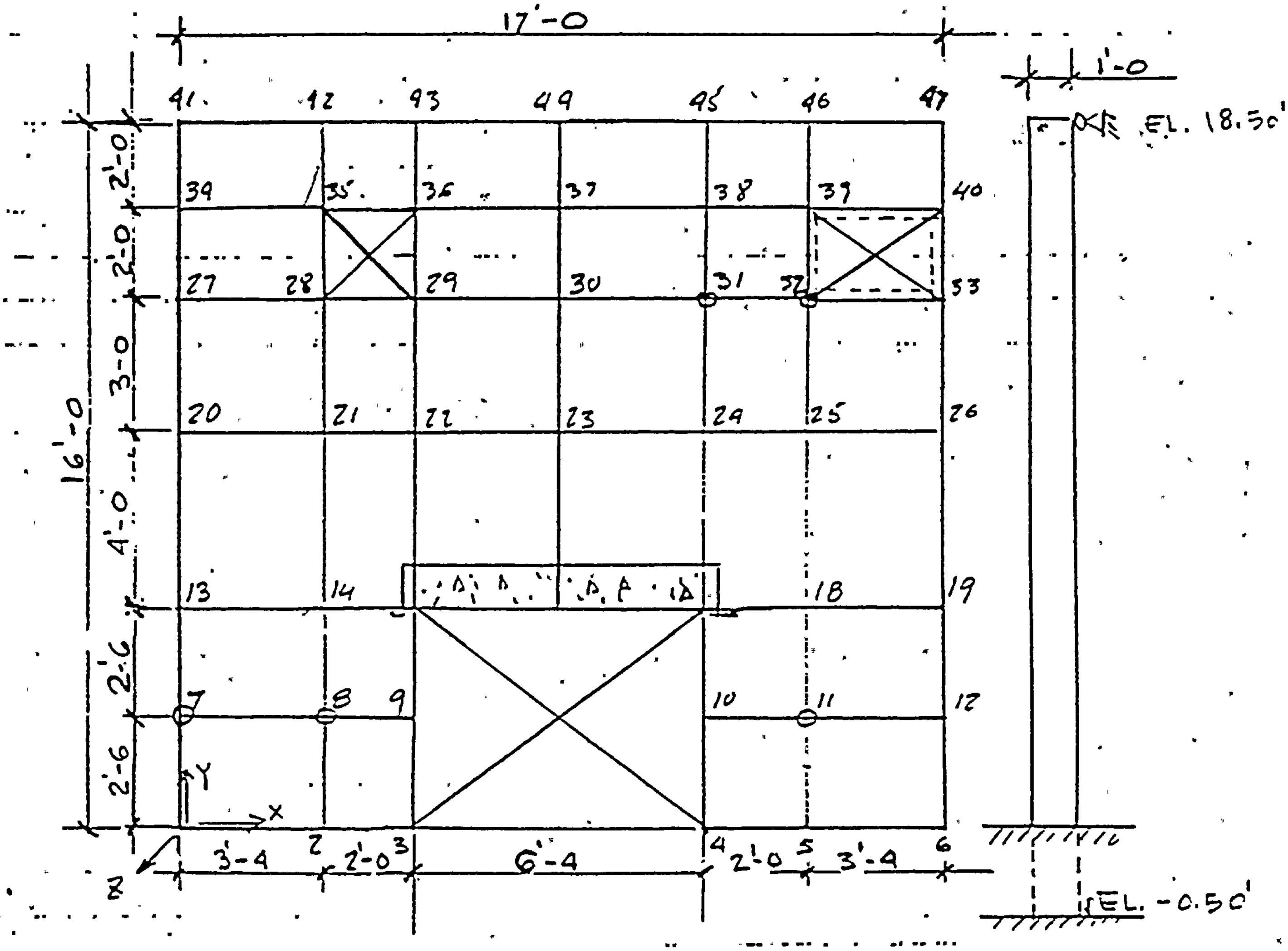
THE WALL SIMULATED BY THE FINITE ELEMENT MODEL AS SHOWN IN SKETCH.

ATTACHMENT WEIGHT INPUT AS GENERALIZD MASS AT NODES:

NODE	WT.	MASS
7	350#	350/1000×32.2 = 0.011 K-sec²/ft
8	75	0.002
11	400	0.012
31	175	0.005
32	175	0.005

EBASCO SERVICES INCORPORATED
NEW YORK

BY J. de MATUS DATE 11/11/80 SHEET 47 OF 204
CHKD. BY C. SHIH DATE 12/29/80 OFS NO. 3611.244 DEPT. NO. 550
CLIENT FLO.
PROJECT ST. LUCIE
SUBJECT WALL 34,35 FREQUENCY ANALYSIS



FINITE ELEMENT MODEL

ELEMENT — STIF 43

MASS — STIF 21.

$\mu = 0.2$

DENSITY $= 125/1000 \times 32.2 = 3.882 \times 10^{-3}$ K-sec^2/ft per ft^3

$E = 0.9 \times 10^3 \times 144 = 1.296 \times 10^5$ KSF.

FROM COMPUTER RUN. JOB NO. "JDMAGJF" 11/12/80 - FREQ. RUN.

MODE - FREQUENCY ANALYSIS BY "ANSYS" PROGRAM

LOWEST MODE FREQUENCY = 14.3 CYCLES/SEC.

CONSIDER MATERIAL VARIATION $f = 14.3 \pm 12.5\% = 12.5$ CPS

FROM FL. SPECTRA HORIZ N-S (OBE) DAMPING = 7% , EL.19.50

SSE = 2 x OBE

$SSE_{HOR.} = 0.125\,g \times 2 = 0.25\,g$

APPLY TO THE MODEL AS LATERAL PRESSURE :

$$p = 0.25 \times .125\ ^{K/ft^3} \times \frac{11.625}{12} = 0.030\ ^{K}/ft^2$$

APPLY ACCEL. IN VERTICAL Y DIRECTION. TO INCLUDE D.L.+SSE

$$Accel\ Y = 32.2 \times 1.24 = 39.9\ ft/sec^2$$

(0.24 g vertical to cover all RAB SSE effect.)

APPLY FORCES AT ATTACHMENT POINTS.

FROM COMPUTER RUN JOB NO. "JDMAGNA" 11/12/80 - STATIC RUN.

THE MAX. MOMENT AT FIXED BASE

$$M_a = 15^{FT\cdot K}$$

FIND THE MOMENT CAPACITY OF THE UNCRACKED SECTION AT BASE :

1) TENSION NORMAL TO BED JOINTS. SOLID OR GROUTED UNITS :

$$f_r = 1.67\sqrt{m_o} = 1.67\sqrt{1800} = 70.8 \text{ or } 67\ ^{psi}(max)$$

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SUBJECT WALL NO. 34 & 35

2) $M_{CR} = f_r \frac{I_t}{y}$ For FULL 17'-0 SECTION

$$M_{CR} = 0.067 \times \frac{28,479}{5.925} = 321.3^{"K} = 26.8^{'K}$$

3) FOR SECTION AT BASE: EFFECTIVE WIDTH = 10'-8"

$$M_{CR} = 26.8 \times \frac{10.67}{17} = 16.8^{'K} > M_a = 15$$

4) ∴ THE SECTION WILL NOT BE CRACKED ON AVERAGE

HOWEVER, IT MAY CRACK LOCALLY AT NODE 5

$$\frac{3.96}{2} \times 17 = 34^{'K} > 26.8$$

B) CRACKED SECTION ANALYSIS:

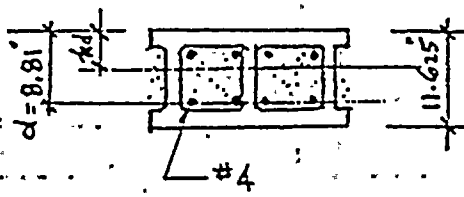
1) FIND NEUTRAL AXIS:

$$\tfrac{1}{2}(17\times12)(kd)^2 - 32(4\times4\times0.196)(8.81-kd) = 0$$

$$kd = 2.494^{IN}$$

$$k = 2.494/8.81 = 0.283$$

$$j = 1 - k/3 = 0.91$$



2) FIND I_{CR}:

$$I_{CR} = \tfrac{1}{3}\times(17\times12)\times(2.494)^3 + 32(3.14)(8.81-2.494)^2 = 5,063^{IN^4}$$

3) FIND I_e : EQUIVALENT MOMENT OF INERTIA OF CRACKED SECTION.

$$I_e = \left(\frac{M_{CR}}{M_a}\right)^3 I_t + \left[1 - \left(\frac{M_{CR}}{M_a}\right)^3\right] I_{CR}$$

$$I_e = \left(\frac{26.8}{34}\right)^3 (28479) + \left[1 - \left(\frac{26.8}{34}\right)^3\right] (5063)$$

$$I_e = 16{,}531 \text{ IN}^4$$

$$t_e = \sqrt[3]{\frac{12 I_e}{b}} = \sqrt[3]{\frac{12 \times 16531}{12 \times 17}} = 9.907^{\text{IN}} = 0.826 \text{ FT}$$

4) FREQUENCY OF THE CRACED SECTION:

FROM COMPUTER RUN "JDMAJZØ"

$f = 11.9$ CPS $\qquad 11.9 \times (1 - 12.5\%) = 10.4$ CPS

$SSE_{N-S} = 0.135 \times 2 = 0.27\ g$ TO ACT LIKE PRESSURE ON WALL

$$\text{PRESS.} = 0.27 \times \frac{125}{1000} \times \frac{11.625}{12} = 0.032 \text{ KSF.}$$

5) STATIC RUN RESULTS :

FROM COMPUTER RUN "JDMAJ1X"

$M_a = \frac{4.18}{2} \times 17 = 35.6^{\text{IK}}$ (FULL SECTION)

$M_{CAP} = f_s A_s j d$

$M_{CAP} = (0.9 \times 40) \times (4 \times 4 \times 0.196) \times (0.91)(8.81) = 906^{\text{"K}} = 75.5^{\text{IK}}$

$M_{CAP} > M_a$ (FULL SECTION)

BY: J. deMatos DATE 11/12/80 SHEET 8 OF

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CLIENT FPL

PROJECT ST. LUCIE No. 1

SUBJECT WALL NO. 34 & 35

c) STRESS CHECK :

CHECK : MOMENT AT BASE :

$$\Sigma M_a = 15.83^{\,\prime k}$$

$$f_s = \frac{M_a}{A_s\, j d} = \frac{15.83 \times 12}{3.14 \times \frac{10.67}{17} \times 0.91 \times 8.81} = 12.02^{\,ksi} < 36 \quad \text{O.K}$$

SHEAR : $\Sigma V = 5.2^{\,k}$ AT BASE.

$$v = \frac{V}{bd} = \frac{5.2}{(10.67 \times 12)(8.81)} = 0.0046^{\,ksi} < 0.051 \quad \text{O.K.}$$

ALLOW. $v = 1.7 \sqrt{f'_m} = 51^{\,psi}$

BOND :

$$u = \frac{V}{\Sigma o\, j d} = \frac{5.2}{16 \times 1.57 \times 0.91 \times 8.81 \times \left(\frac{10.67}{17}\right)} = 0.041^{\,ksi} < 0.186$$

O.K.

SPLICE :

$$u = \frac{A_s \cdot f_s}{\Sigma o \times l_{sp}} = \frac{3.14 \times \frac{10.67}{17} \times 12.02}{1.57 \times 16 \times \frac{10.67}{17} \times 18} = 0.083^{\,ksi} < 0.140$$

O.K.

$$u_{ALLOW} = 0.186 \times \tfrac{3}{4} = 0.140 \text{ ksi}$$

COMPRESSION :

$$f_a = \frac{29.4}{10.67 \times 12 \times 11.625} = 0.02^{\,ksi}$$

$$F_a = 0.44 f'_m = 0.44 \times .9 = 0.396 \text{ ksi}$$

$$f_m = \frac{M}{\frac{1}{2} \cdot bd^2 k j} = \frac{15.83 \times 12}{0.5 \times 10.67 \times 12 \times 8.81^2 \times 0.283 \times 0.91}$$

$$f_m = 0.148 \text{ ksi}$$

$$F_m = 0.85 f'_m = 0.85 \times 0.9 = 0.765 \text{ ksi}$$

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CLIENT FLO

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SUBJECT WALL NO. 34 & 35

$$\frac{f_a}{F_a} + \frac{f_m}{F_m} = \frac{-0.02}{-0.396} + \frac{0.148}{0.765} = 0.05 + 0.19 = 0.24 < 1$$

O.K.

DUR-O-WAL : $\frac{3}{16}$" WIRE ASTM A82. EVERY 8" SPACING

$f_y = 70$ KSI OBE $f_s = 0.5 f_y$ or 30 KSI max

SSE $f_s = 0.9 f_y = 63$ KSI

$M_{y\,cap} = A_s f_s jd = 0.0277 \times 30 \times 0.92 \times 10.72 = 8.2^{"K}$ (OBE)

$M_{y\,cap} = 8.2 \times \frac{63}{30} = \underline{17.22^{"K}}$ / COURSE OF BLOCK (SSE) ← USE

or $M_{y\,cap} = \frac{1}{2} \times f_c \times b \times kd \times jd$

$= \frac{1}{2} \times 0.85 \times 0.9 \times 8 \times 0.24 \times (10.72)^2 \times 0.92 = 77.64^{"K}$/course > 17.22

$\therefore M_{y\,cap} = 17.22 \times \frac{12}{8} = 25.83^{"K}$/FT. or $\underline{2^{'K}}$/FT

ALL STRESSES ARE LESS THAN THE ALLOWABLES.

Att A 10 [illegible]

RINKER MATERIALS CORP.

QUALITY CONTROL LABORATORY

FIRST WITH THE BEST

BLOCKS — CONCRETE — STEEL — BUILDING SUPPLIES

P. O. BOX 231 WEST PALM BEACH, FLORIDA TELEPHONE TE 3-5555

STANDARD TESTS OF CONCRETE MASONRY UNITS

Specifications: ASTM C-140-65 and C-426-66T

Lab. No.: 47201
Date: JAN. 4, 1972
Mfg. At: FT. PIERCE

Reported To: ROUTINE TEST

Size of Block: MODULAR 8"X8"X16" Type of Block: 2 CELL 10 Wall Thickness: 1¼" Date of Mfg. DEC 6, 1971

COMPRESSION TESTS

Mark	Wt., Lbs.	Area Sq. In.	Total Load, Lbs.	Unit Load, Lbs./Sq.In.	Age, Days
1	35.5	119.14	152500	1280	28
2	35.8	119.14	148200	1245	28
3	35.7	119.14	159000	1335	28
4	35.1	119.14	154750	1299	28
5	35.9	119.14	161000	1352	28
Ave.	35.6			Ave. 1302	

MOISTURE TESTS

Mark	Wt. of Concrete lbs./Cu. Ft.	Absorption Lbs./Cu.Ft.	Absorption %	% Moisture as Compared to total Abs.	* Relative Humidity %
6	124.1	8.65	6.97	28.5	76.8
7	125.7	7.87	6.23	28.4	79.6
8	126.4	7.97	6.31	29.4	81.0
9	125.2	8.53	6.80	29.6	77.9
10	124.9	8.42	6.75	28.0	78.1
Ave.	125.26	8.292	6.622	28.98	78.68

* The Relative Humidity for , Florida - Month of is %.

LINEAR DRYING SHRINKAGE

ASTM C 426-66T

Mark:	11	12	13
% Shrinkage:	.0242	.0248	.0239
Average:	.0243		

Al Seay

Al Seay
Block Quality Coordinator

ATTACHMENT B (2 SHEETS)

Att. B 1 of 2

TABLE 1
FLEXURAL STRENGTH—SINGLE WYTHE WALLS OF HOLLOW UNITS—
UNIFORM LOAD--VERTICAL SPAN

Mortar Type Proportion ASTM C 270	Modulus of Rupture psi, Net Area	Reference
M	110	10
M	108	NCMA
M	102	10
M	97	10
M	95	NCMA
S	94 Ave = 93	NCMA
M	91	NCMA
M	89	NCMA
N	88	4
S	84	10
S	83	NCMA
S	81	10
S	75	NCMA
S	69	NCMA
N	67	4
N	62	4
S	60	10
N	58	4
N	45	4
O	60	10
O	41	4
O	36	4
O	36	4
O	33	4
O	32	4
O	30	10
O	27	4

AH B' Ed 2

TABLE 2 FLEXURAL STRENGTH, VERTICAL SPAN CONCRETE MASONRY WALLS FROM TESTS AT NCMA LABORATORY

	Wall				
				Modulus of Rupture	
ASTM Mortar Type*	Nominal Thickness in.	Max. Uniform Load psf.	Net Section Modulus in 3/ft	Gross Area, psi	Net Mortar Bedded Area, psi
		Monowythe Walls of Hollow Units			
M	8	85.15	80.97	61.74	88.73
M	8	87.10	80.97	63.15	90.76
M	8	91.00	80.97	65.97	94.82
M	8	103.35	80.97	74.93	107.69
S	8	62.40	80.97	45.24	69.47
S	8	72.15	80.97	52.31	75.18
S	12	183.3	164.64	57.11	93.94
S	12	161.2	164.64	50.22	82.62
		Composite Walls of Concrete Brick & Hollow CMU			
S	8	222.3	103.82	161.16	180.67
S	8	219.7	103.82	159.29 Av=	178.55
S	8	187.2	78.16	135.72 157	202.09
S	8	228.8	103.82	165.88	185.95
S	8	218.4	78.16	158.34	235.77
S	8	223.6	78.16	162.11	241.38
S	12	171.6	139.83	53.46	103.55
S	12	150.8	139.83	46.98	91.00
S	12	156.0	139.83	48.60	94.14
S	12	213.2	139.83	66.42	128.66
		Cavity Walls			
S	10	98.8	50.36	158.62	165.55
S	10	156.0	50.36	250.44	261.38
S	10	88.4	48.16	141.91	154.88
S	10	119.6	50.36	192.01	200.40
S	10	114.4	50.36	183.66	191.68
S	10	109.2	48.16	175.30	191.32
S	12(4-4-4)	145.6	50.36	233.73	243.94
S	12(4-4-4)	145.6	50.36	233.73	243.94
S	12(6-2-4)	135.2	77.80	127.38	146.63
S	12(6-2-4)	119.6	77.80	112.68	129.70

* Mortar type by proportial typical limit.

ATTACHMENT C (16 SHEETS)

EBASCO SERVICES INCORPORATED

NEW YORK

Att. C 1-f-1

BY C. GAVIDIA DATE 12/11/80

SHEET 168 OF 204

CHKD. BY [illegible] DATE [illegible]

OFS NO. 3611 249

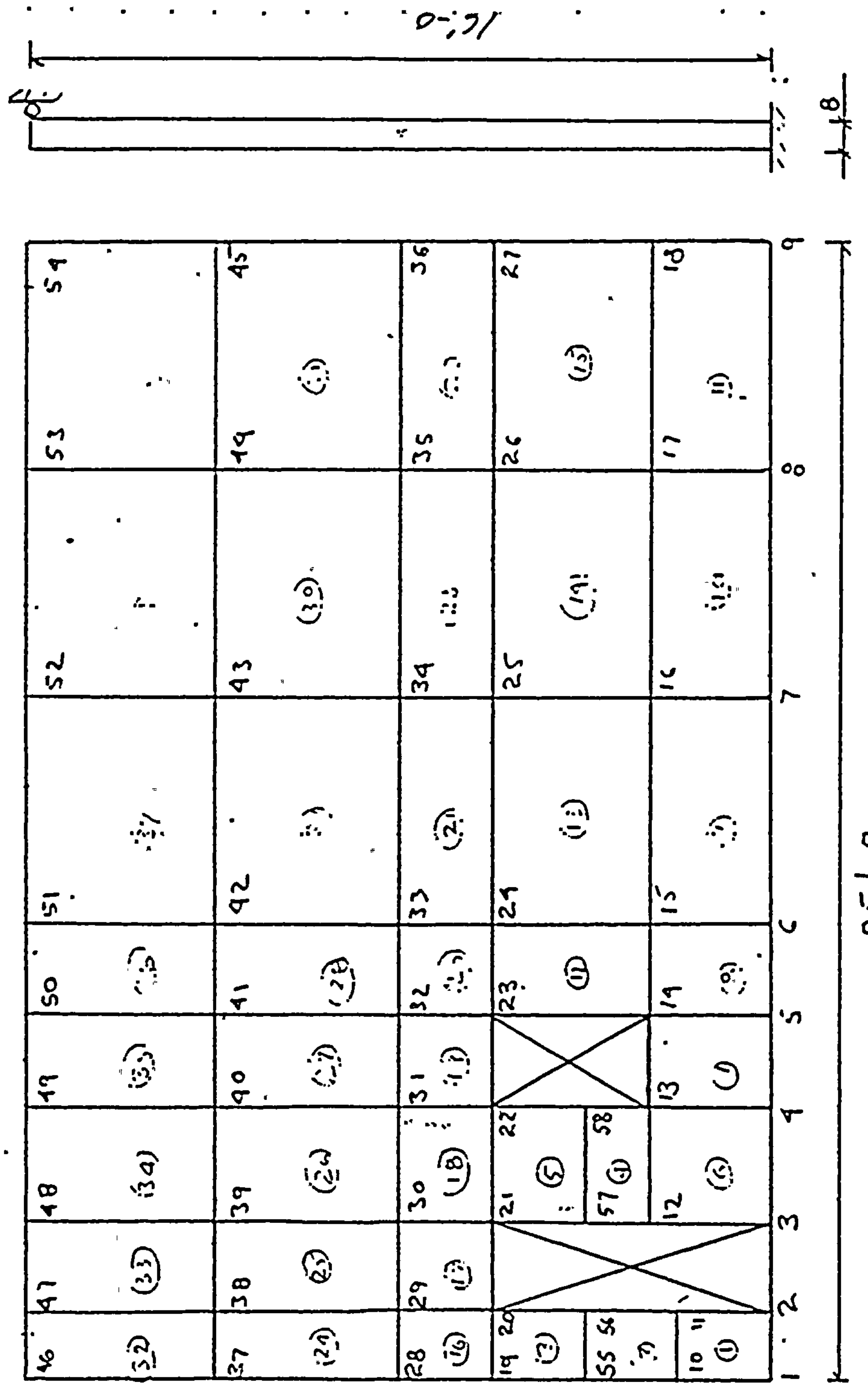
DEPT. NO. 550

CLIENT FLO

PROJECT ST LUCIE UNIT #1

SUBJECT MASONRY WALL RE-EVALUATION (201, 202, 205)

WALL # 205



FORM 881 REV 7-71

$AS = 1.57 \quad E_m = 0.9 \times 10^6 \, PSI \quad E_s = 29 \times 10^6 \, PSI$

$\bar{y} = 3.81$

BLOCK 16 x 7-5/8

W/O R. BARS → $I_m = 591 \, IN^4$

W/ R. BARS → $I_{m_s} = 704 \, IN^4$

7 UNITS W/R BARS & 11.75 UNITS W/O R. BARS

$$I_t = (7 \times 70.4) + (11.75 \times 591) = 11{,}872 \, IN^4$$

$$t_e = \sqrt[3]{\frac{\cancel{12} \times 11872}{25 \times \cancel{12}}} = 7.8 \text{ in}$$

$$= 0.65 \text{ FT}$$

$$M_{cr} = \frac{f_r I}{Y}$$

$$= \frac{0.067\,(11872)}{3.91} = 203 \text{ IN-K}$$

$$= 16.95 \text{ FT-K.}$$

SINCE SIZE OF WALL # 205 IS SIMILAR TO WALL # 170 THEN FREQUENCY OF WALL # 170 (SEE PAGE 156). IS SIMILAR TO FREQUENCY OF WALL # 205

$\therefore \quad f = 8.79 \text{ cps.}$

FROM FL. SPECTRA E-W & VERT @ ELE. 81.5

HORIZONTAL = 0.68 g

VERTICAL = 0.76 g. → ACC = 1.76 = 56.67 FT/SEC^2

SEISMIC LOADING:

$$SSE_H = (0.68)(.125)(.667) = 0.057 \ K/SQ.FT.$$

ESTIMATING MOMENT @ BOTT. OF WALL

$$M_a = \frac{(0.057 \times 25)(16)^2}{8} = 45.6 \ FT\text{-}K.$$

SINCE $M_a > M_{CR}$ ∴ SECTION IS CRACKED

FIND I_{CR} (28 #4 BARS IN TENSION)

$$(Kd)^2\left(\frac{b}{2}\right) - n A_s (d - Kd) = 0$$

$$\left(\frac{25 \times 12}{2}\right)(Kd)^2 - (32)(5.50)(5.31 - Kd) = 0$$

$$150 (Kd)^2 + 176 Kd - 934 = 0$$

$$Kd = \frac{-176 \pm \sqrt{(176)^2 - 4(150)(-934)}}{2(150)}$$

$$Kd = 1.98 \text{ in}$$

$$I_{cr} = \tfrac{1}{3}\, b\,(kd)^3 + n A_s (d - kd)^2$$

$$= \tfrac{1}{3}(25\times12)(1.98)^3 + (32)(5.5)(5.31-1.98)^2$$

$$= 2728 \text{ in}^4$$

$$I_e = \left(\frac{16.95}{45.6}\right)^3 (11872) + \left[1 - \left(\frac{16.32}{45.6}\right)^3\right](2728)$$

$$= 3198 \text{ in}^4$$

$$t_a = \sqrt[3]{\frac{\cancel{12}\times 3198}{(25\times\cancel{12})}} = 5.04 \text{ in}$$

$$= 0.42 \text{ FT.}$$

$f = 6.62$ cps. ← from Computer analysis

FROM FL. SPECTRA E-W & VERT @ ELEV. 81.5, D = 7%

HORIZONTAL = 1.09 g.

VERTICAL = 1.60 g → ACC = 2.60 g = 83.72 F/sec²

PRESSURE =

$$SSE_H = (1.09)(.650)(.125) = 0.087$$

$K_d = 1.98$

$K = 1.98/5.31 = 0.37$

$j = 1 - K/3 = 0.88$

$M_{cap} = f_s A_s j d$

$= (30)(5.5)(.88)(5.31)$

$= 925$ IN-K

$= 77.10$ FT-K.

$M_a = 67.16$ FT-K ← from Computer analysis

$M_a < M_{cap}$. OK

$F_a = 0.396$ Ksi

$F_m = 0.765$ Ksi

$A = (23 \times 12)(5.04) = 1391$ IN²

$S = 3198 / 5.04/2 = 1269$ IN³

$C = 51.99$ K.

$f_a = \dfrac{51.99}{1391} = 0.037$ Ksi

$f_m = \dfrac{67.16 \times 12}{1269} = 0.635$ Ksi

$$\frac{f_a}{F_a} + \frac{f_m}{F_m} = \frac{0.037}{0.396} + \frac{0.635}{0.765}$$

$$= 0.923 < 1.0 \quad \text{OK}$$

CHECK TENSILE STRESS:

$$f_s = \frac{67.16 \times 12}{(5.5)(.88)(5.31)}$$

$$= 31.36 \text{ Ksi} < 36 \text{ Ksi} \quad \text{OK}$$

SHEAR STRESS:

$V = 21.17$ K.

$$v = \frac{V}{b_w d} = \frac{21.17}{(23 \times 12)(5.31)}$$

$$= 0.014 \text{ Ksi} < 0.051 \text{ Ksi} \quad \text{OK}$$

BOND STRESS:

① FLEXURAL BOND

$$u = \frac{V}{(N \pi d_o)\, j\, d} = \frac{21.17}{(28)(1.57)(.88)(5.31)} =$$

$$= 0.103 \text{ Ksi} < 0.186 \text{ Ksi} \quad \text{OK}$$

EBASCO SERVICES INCORPORATED

NEW YORK

Att. C

BY C. GAVIDIA DATE 12/12/80 SHEET 174 OF 204

CHKD. BY [illegible] DATE 1/16/81 OFS NO. 3611.249 DEPT. NO. 550

CLIENT FLO

PROJECT ST LUCIE UNIT #1

SUBJECT MASONRY WALL RE-EVALUATION (201, 202, 205)

$u = \frac{A_s f_s}{\Sigma_0 \times [illegible]}$

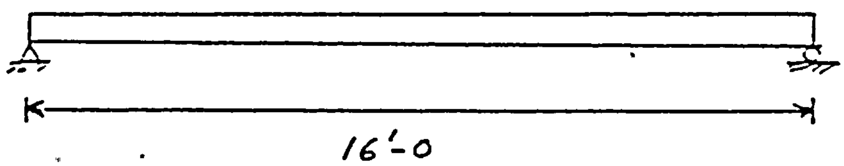
$= \frac{5.5 \times 31.36}{[illegible]}$

② ANCHORAGE

$$u_b = \frac{f_s d_b}{4L_1} = \frac{(31.36)(.5)}{4(18)}$$

$$= 0.218 \text{ ksi} > 0.186 \text{ ksi} \quad \text{N.G}$$

ASSUME WALL TO BE SIMPLE SUPPORTED



$f = 4.2$ cps. ← from Computer analysis

from FL. SPECTRA E-W & VERT., @ ELV. 81.5 ; D = 7%

HORIZONTAL = 1.3 g

VERTICAL = 2.4 g → ACC. = 3.4 g = 109.48 F/sec²

PRESSURE :

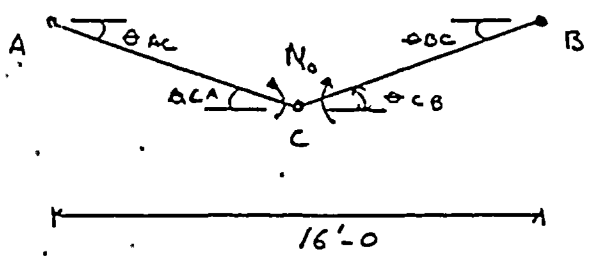
$SSE_H = (1.3)(.650)(.125) = 0.106$ K/SQ.FT.

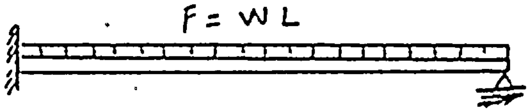
Max. M_a @ MIDSPAN

$M_a = 83.72$ FT-K ← from Computer analysis

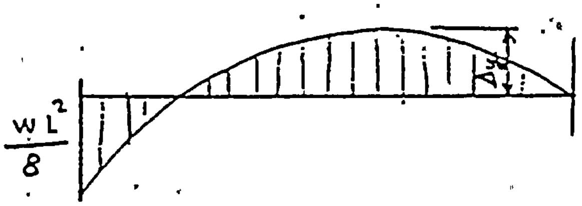
SINCE $M_a > M_{cap}$ → FURTHER analysis USING YIELD-LINE THEORY SHOULD BE PERFORMED

YIELD-LINE ANALYSIS





W = [illegible] = 2.[illegible] F



$M_u = F_y A_s (d - a/2) = 966$ in-K

$a = \frac{F_y A_s}{.85 f'_m \times b_w} = \frac{(36)(5.50)}{.85(.9)(25\times 2)} = 0.86$

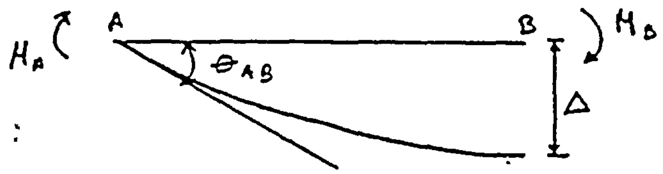
ELASTIC ANALYSIS

$$\Delta_y = WL^4/185EI = 8M_oL^2/185EI = 0.043\,D$$

$$M_o = WL^2/8 = 2.7\times(16)^2/8 = 86.4 \text{ FT-K.}$$

$$D = \frac{M_o L^2}{EI}$$

$$\Delta_y = 0.043\,(966)(16\times 12)^2\left(\frac{1}{900\times 3198}\right) = 0.53 \text{ IN}$$



FROM PLASTIC ANALYSIS OF STRUCTURES BY P.G. HODGE. PAGE 99 EQ. (4-42)

GENERAL SLOPE-DEFLECTION EQ. :

$$\boxed{\begin{array}{l}\theta_{AB} = \theta'_{AB} + \Delta/L + L/6EI\,(2M_A - M_B) \\ \theta'_{AB} = \text{ELASTIC ROTATION @ A}\end{array}}$$

$$\therefore\quad \theta_{AC} = \frac{(F/2)(L/2)^2}{24\,EI} + \frac{\Delta}{(L/2)} + \frac{(L/2)}{6\,EI}\,(2M_o - M_o)$$

$$\theta_{CA} = -\frac{(F/2)(L/2)^2}{24\,EI} + \frac{\Delta}{L/2} + \frac{(L/2)}{6EI}\,(-2M_o + M_o) = 0$$

AT THE MOMENT PRIOR TO COLLAPSE.

$$-\frac{FL^2}{192\,EI} + \frac{2\Delta}{L} - \frac{M_o L}{12EI} = 0 \quad \text{———} \quad (1)$$

COLLAPSE MECHANISM – HINGE ASSUMED @ MIDDLE

FROM BASIC YIELD LINE THEORY.

$$3\,M_o\,\cancel{\phi} = \frac{W_u L}{2} \times \frac{L}{2} \times \cancel{\phi} \times \frac{1}{\cancel{Z}} \times \cancel{Z}$$

$$M_o = \frac{W_u L^2}{12}$$

$$F_u = W_u L = \frac{12\,M_o}{L} \qquad \text{Sub. into eq. (1)}$$

$$2\Delta_u/L = M_o L/12EI + 12 M_o L/192\,EI$$

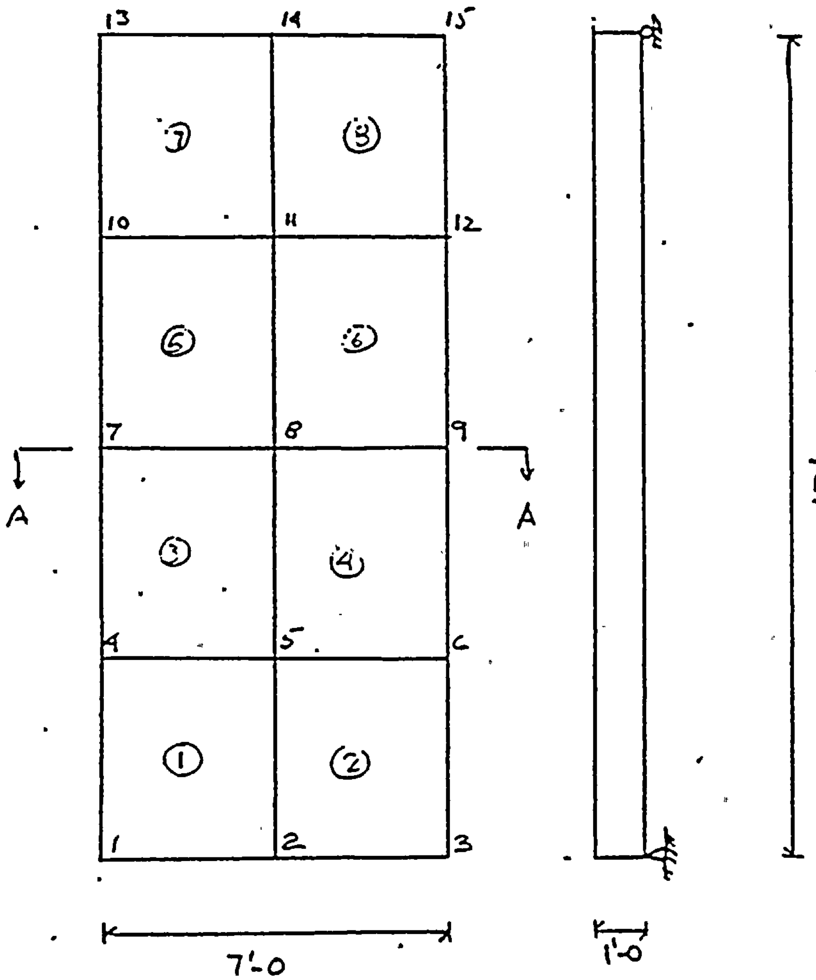
$$\Delta_u = \frac{7 \cdot M_o L^2}{96\,EI} = 0.073\,D$$

$$\Delta_v = 0.073\,(86.4 \times 12)(16 \times 12)^2 \left(\frac{1}{900 \times 3198}\right)$$

$$= 0.98 \text{ in}$$

SINCE $\Delta_v < 3\Delta_y$. OK//

WALL 162A (NO REINFORCEMENT)



$E_m = 0.9 \times 10^6$ psi $\quad t = 11.625$ in $\quad \bar{Y} = 5.81$ in

$$I = \frac{1}{12}(7\times12)(11.625)^3 = 10,997 \text{ in}^4 \qquad t = 0.969 \text{ ft}$$

$$M_{cr} = f_r \frac{I}{Y}$$

$$M_{cr} = \frac{(0.067)(10997)}{5.81} = 127 \text{ in-K}$$

$$= 10.57 \text{ FT-K}$$

$f = 8.56$ cps ← from Computer analysis.

From FL. SPECTRA N-S & VERT. @ ELV 61.5'

HORIZONTAL = 0.42 g.

VERTICAL = 0.34 g → acc. = 1.34 g = 43.15 FT/sec^2

PRESSURE :

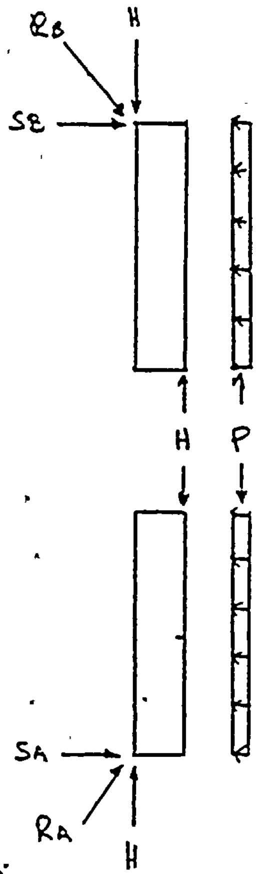
SSE $(0.42)(0.970)(.125) = 0.05$ K/SQ. FT.

@ SECTION A-A

$M_a = 14.54$ FT-K.

$M_a > M_{cr}$ ∴ SECTION IS CRACKED

RE-EVALUATION OF WALL USING ARCHING ANALYSIS.



$$H = P\ell^2/8t$$

$$S_B = S_A = P\ell/2$$

$$R_B = R_A = H/\ell \sqrt{\ell^2 + (4t)^2}$$

$$P = \text{SEISMIC LOAD} + 15\%\ \text{AREA}\ (0.025\ \#/FT^2)/\ell$$

$$P = (0.05)(7) + (.15 \times 7 \times 17)(.025)/17$$

$$P = 0.35 + 0.026$$

$$P = 0.38\ K/FT$$

$$H = (.38)(17)^2/8(0.969)$$

$$= 14.2\ \text{Kips}$$

$$S_A = S_B = (0.38)(17)/2$$

$$= 3.23\ \text{Kips}$$

$$R_A = R_B = 14.2/17 \sqrt{(17)^2 + (4 \times .969)^2}$$

$$= 14.56\ \text{Kips.}$$

COMPRESSIVE STRESS $= \dfrac{H}{b \cdot t_s}$

$= \dfrac{14.2}{(7\times12)(1.5)} = 0.113 \text{ Ksi} < 0.765 \text{ Ksi}$

OK

DEFLECTION : Δ (ALLOWABLE) = 0.3t

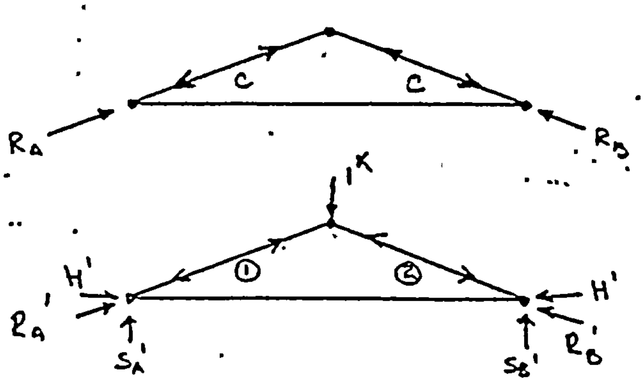
TOTAL $\Delta = \Delta_1$ (ELASTIC ANALYSIS) + Δ_2 (ARCHING ANALYSIS)

ELASTIC ANALYSIS : Δ_{max} @ ℄ , W = P

$$\Delta_1 = \Delta_{max} = \frac{5 w \ell^4}{384 E I} = \frac{5(0.38/12)(17\times12)^4}{384\,(0.9\times10^3)(10997)}$$

$= 0.072$ in

ARCHING ANALYSIS :



FORCES DUE TO EXTERNAL LOADS

C = R_A = 14.56 Kips.

FORCES DUE TO UNIT LOAD

$S_B' = S_A' = P\ell/2$

$= 1/2 = 0.5^K$

$$\frac{H'}{\ell/2} = \frac{S_A'}{t} \rightarrow H' = \frac{\ell S_A'}{2t} = \frac{(17)(.5)}{2(.969)}$$

$$H' = 4.39 \text{ K}$$

$$R_A' = R_B' = \sqrt{(H')^2 + (S_A')^2}$$

$$= \sqrt{(4.39)^2 + (.5)^2}$$

$$= 4.4^K$$

MEMBER	ℓ (IN)	A (IN²)	ℓ/A	S	μ	$S\mu\ell/AE$
①	102	977	0.104	14.56	4.4	6.68/E
②	102	977	0.104	14.56	4.4	6.68/E

$\Sigma = 13.37/E$

$$\Delta_2 = \frac{13.37}{0.9 \times 10^3} = 0.015 \text{ in}$$

TOTAL $\Delta = 0.072 + 0.015 = 0.087$ in

SINCE $\Delta = 0.083$ in $< 0.3t = 3.488$ in OK

CHECK FRICTION SHEAR:

Coeff. of friction (μ) = 0.8 $V_f = N\mu = 14.2 \times 0.8 = 11.36^K$

SINCE $V_f = 11.36^K > S_A = 3.23^K$

∴ ENDS of WALL WILL NOT SLIDE

EBASCO SERVICES INCORPORATED

NEW YORK

Att. C 16

BY C. GAVIDIA DATE 12/9/80 SHEET 150 OF 204

CHKD. BY [illegible] DATE 1/16/81 OFS NO. 3611.244 DEPT. NO. 550

CLIENT FLO

PROJECT ST LUCIE UNIT # 1

SUBJECT MASONRY WALL RE-EVALUATION (162A)

CHECK SHEAR STRESS:

$V = 3.23^{K}$

$$\tau = \frac{V}{b\,t_s} = \frac{3.23}{(7 \times 12)(1.5)}$$

$$= 0.026 \text{ ksi} < 0.051 \text{ ksi}$$

OK

FORM 881 REV 7-71

ATTACHMENT D (2 SHEETS)

EBASCO SERVICES INCORPORATED

NEW YORK

Att. D 1 of 3

BY C. SHIH DATE 1/16/81

SHEET 197 OF 204

CHKD. BY J. de MATOS DATE 1/21/81

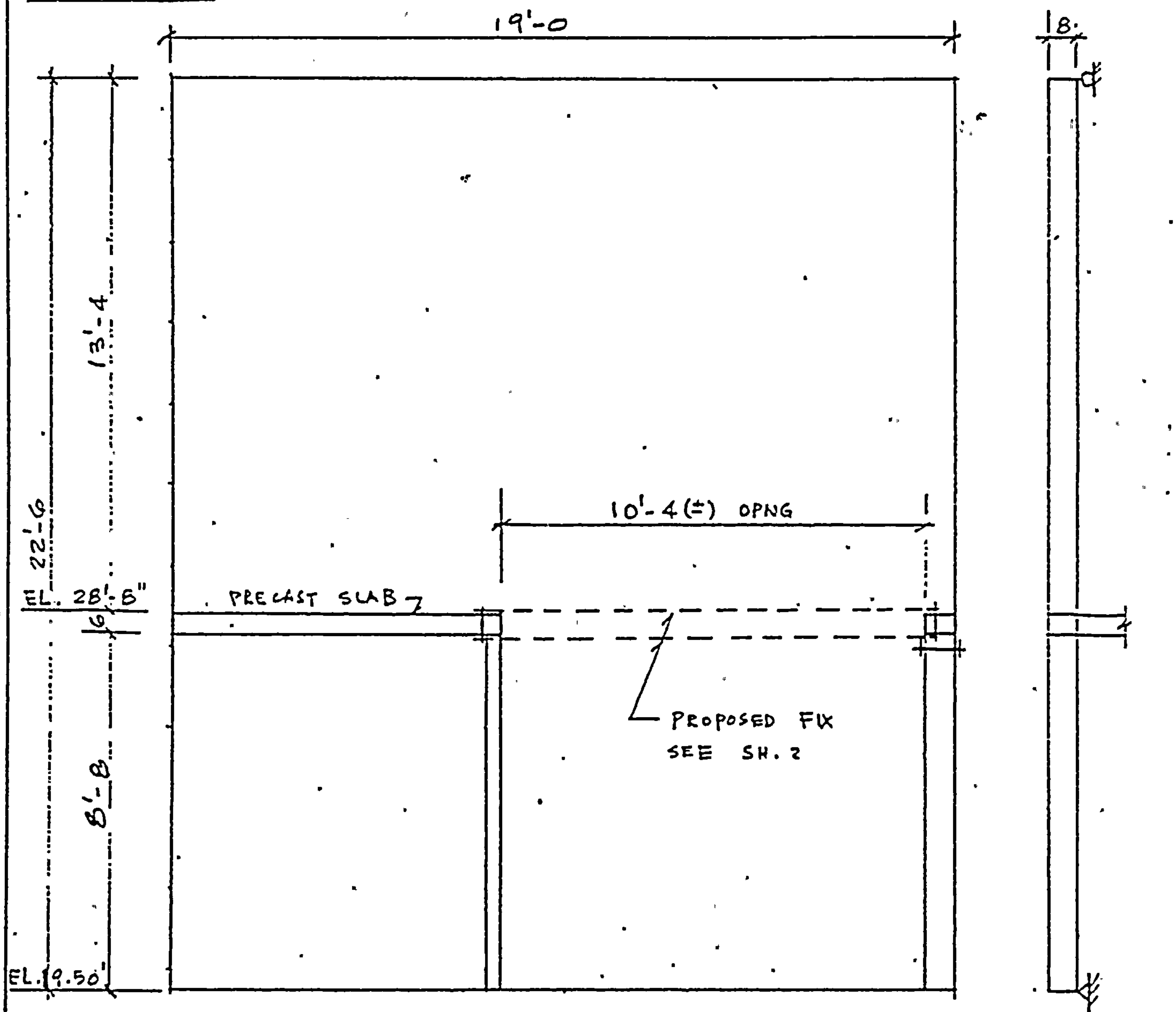
OFS NO. 3611-14 DEPT. NO. 550

CLIENT FLO

PROJECT ST. LUCIE #1 NRC IE BULLETIN 80-11

SUBJECT MASONRY WALL RE-EVALUATION, WALL NO 114, PROPOSED FIX

WALL NO. 114:



ELEVATION (WALL NO. 114)

SCALE 1/4" = 1'-0

FORM 581 REV 7-71

EBASCO SERVICES INCORPORATED

NEW YORK

BY C. SHIH DATE 1/14/81

CHKD. BY J. de MATOS DATE 1/21/81

SHEET 198 OF 204

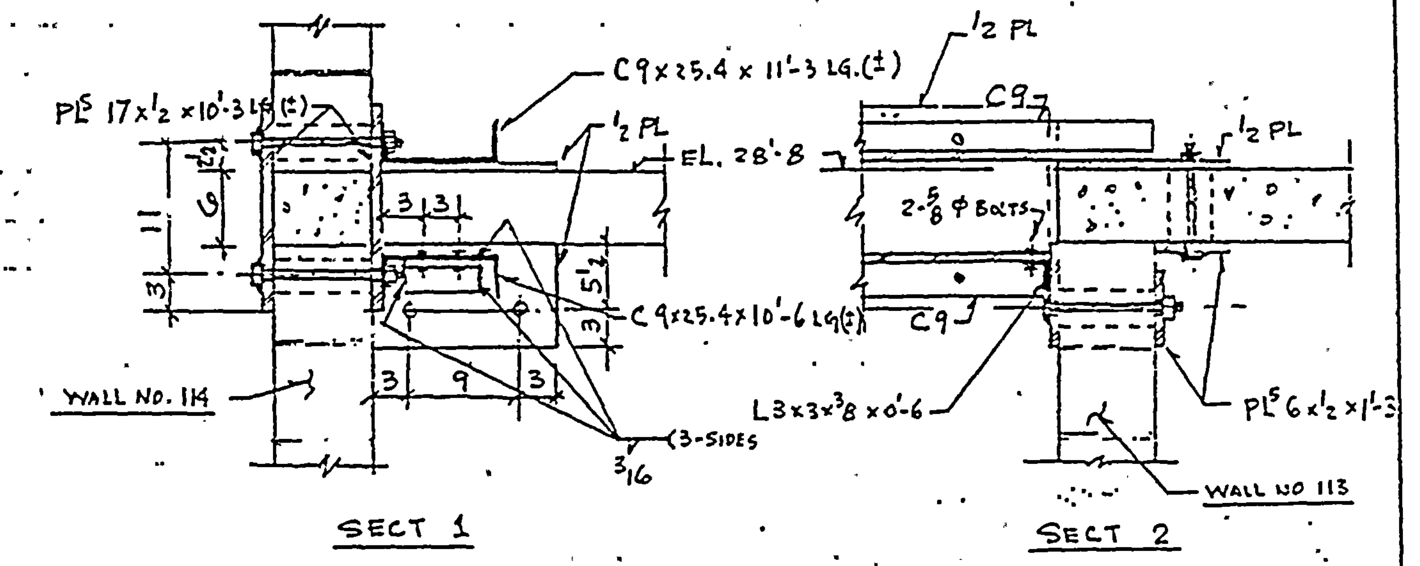
OFS NO. 3611.244 DEPT. NO. 530

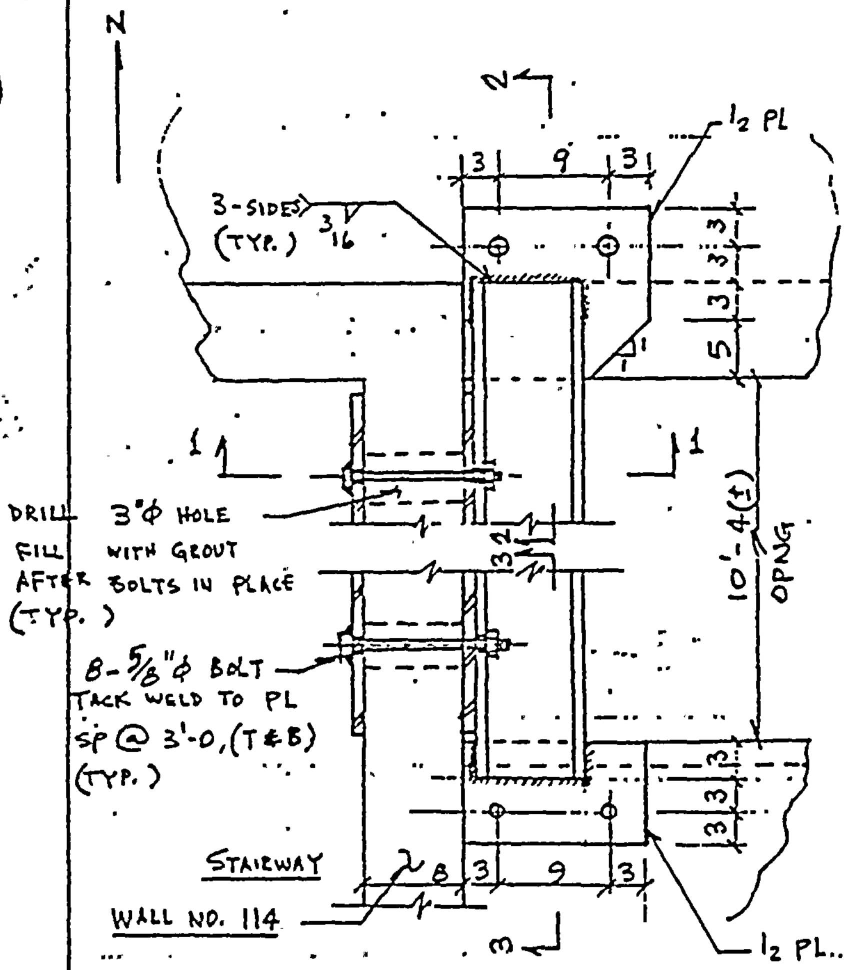
CLIENT FLO

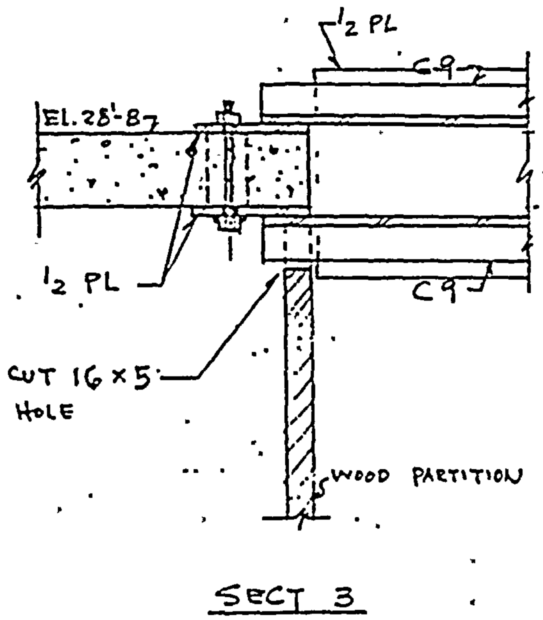
PROJECT ST LUCIE #1 NRC IE BULLETIN 80-11

SUBJECT MASONRY WALL RE-EVALUATION - WALL NO 114, PROPOSED FIX

WALL NO 114.







PLAN AT EL. 28'-8

SCALE : ¾" = 1'-0" (TYP.)

(ALL MATERIALS ARE AVAILABLE IN WAREHOUSE)

FORM 581 REV 7-71

ATTACHMENT E (10 SHEETS)

EBASCO SERVICES INCORPORATED

NEW YORK

AH. E 1 of 15

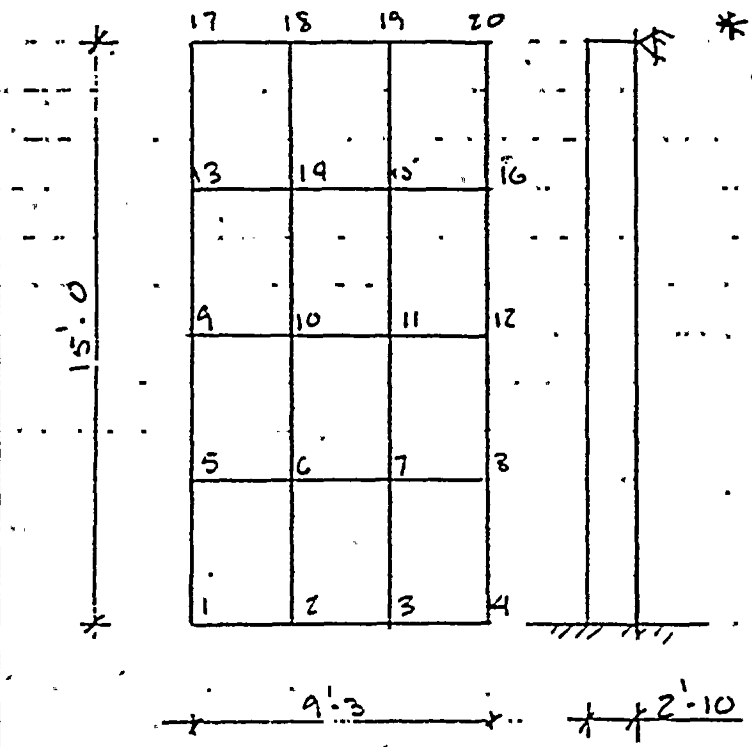
BY: JdeMatos DATE: 10/1/80 SHEET: ~~4A~~ OF 2?

CHKD. BY: C. SHIH DATE: 12/29/80 OFS NO.: 3611.24a DEPT. NO.: 550

CLIENT: FLO

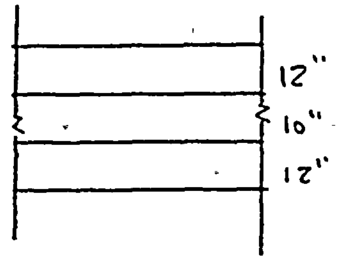
PROJECT: FLO ST. LUCIE

SUBJECT: WALL 1, 2, 3 FREQ. ANALYSIS



* Refer to speed letter by R. Russo dated 2/11/82, the following design pressure has been confirmed:

P = 3 PSI = 0.432 k/sf



Block 16 x 11 5/8 w/ R. BARS $I_{u+s} = 2{,}484\ in^4$

w/o R. BARS $I_u = 2{,}095\ in^4$

Block 16 x 9 5/8 w/ R. BARS $I_{u+s} = 1400\ in^4\ (1189 + 48.67 \times (7.06 - 4.81)^2)$

w/o R. BARS $I_u = 1189\ in^4\ (\frac{1}{12}\ 16 \times 9.625^3)$

16 x 11 5/8

$I_t\ (3)(2{,}484) + (4)(2{,}095) = 15{,}832\ in^4$ $t_e = 11.96\ in$

16 x 9 5/8

$I_t = (3)(1400) + 4(1189) = 8{,}956\ in^4$ $t_e = 9.89\ in$

FORM 561 REV 7-71

BY JdMatos DATE 10/6/80 SHEET 45 OF

CHKD. BY C. SHIH DATE 12/29/80 OFS NO. 3611.244 DEPT. NO. 550

CLIENT FLO

PROJECT FLO ST. LUCIE

SUBJECT WALL 1,2,3 FREQ. ANALYSIS

TOTAL

$I_t = \frac{1}{12}(9.25 \times 12)(9.89^3) + 2\left[\frac{1}{12}(9.25 \times 12)(11.96^3) + (9.25 \times 12)(11.96)(10.93^2)\right] = 357{,}790 \text{ in}^4 \Rightarrow t_e = \underline{33.819} \text{ in}$

$= 2.818 \text{ (ft)}$

$F = 47.5 \Rightarrow T = 0.03$

$accel_H = 0.34 g$ (E-W, D=2% ...)

$accel_V = 1.24 g : 39.9 : (ft/sec^2)$

$Wt_{WALL} = (2.818)(15)(9.25)(150^\#) = 58{,}650^\#$

$M_{cr} = f_r \left(I_t / y \right)$ $y = 33.819/2 = 16.910 \text{ in}$

$M_{cr} = (0.067)(357{,}790 / 16.910) = 1{,}418 \text{ in-k}$

$= \underline{118.2 \text{ Ft-k}}$

$SSE_H = \frac{(0.34)(58{,}650)}{(1000)(15)} = 1.32 \, ^K/_{Ft}$

Enter SSE_H like Prev...

$\frac{(1.32)(15)}{(9.25)(15)} = 0.143 \, K/SF$

TOTAL $P = 0.932 + 0.143 = 0.575 \, ^K/_{SF}$

MODEL FOR STATIC HAS 32 ELEMENTS

$M_a = 140.49$

$M_a > M_{cr}$ SECTIONS ARE CRACKED

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_T + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

Blk 16 x 11 58 $\begin{cases} A_s = 2.36 \text{ in}^2 \\ n = 32 \end{cases}$ $n A_s = 75.52 \text{ in}^2$

$$\frac{(9.25)(12)}{2}(kd)^2 - 75.52(8.81 - kd) = 0$$

$$55.5\, kd^2 + 75.52\, kd - 665.3 = 0$$

$kd = 2.85$ in

$$I_{cr} = \tfrac{1}{12}(9.25 \times 12)(2.85)^3 + (9.25 \times 12)(2.85)(1.425^2) +$$

$$\tfrac{1}{12}(75.52)(1^3) + (75.52)(8.81 - 2.85)^2$$

$$= \underline{3,545 \text{ in}^4}$$

Blk 16 x 9 58 $A_s = 2.36 \text{ in}^2$

$$\frac{(9.25)(12)}{2}(kd)^2 - 75.52(7.06 - kd) = 0$$

$kd = 2.49$ in

$I_{ck} = \frac{1}{12}(9.25 \times 12)(2.49)^3 + (9.25 \times 12)(2.49)(1.245^2) + \ldots$

$\frac{1}{12}(75.52)(1^3) + (75.52)(7.06 - 2.49)^2$

$= 2{,}154 \text{ in}^4$

TOTAL $I_{ck} = 9{,}244 \text{ in}^4$

$I_e = \left(\frac{1418}{1686}\right)^3 (357{,}790) + \left[1 - \left(\frac{1418}{1686}\right)^3\right] 9{,}244$

$= 216{,}644 \text{ in}^4$ $t_e = 28.61 \text{ in} = 2.384 \text{ (ft)}$

$F = 40.2$ (CPS) $\Rightarrow T = 0.025$ (sec)

$SSE_H = 0.39\,g$ $D = 5\%$ E-W From FL. SPECTRA

$SSE_V = 0.24\,g \Rightarrow acc = 39.9 \text{ ft/sec}^2$

$wt_{(WALL)} = (15)(9.25)(2.384)(150) = 49{,}617^{\#}$

$SSE = \frac{(0.39)(49{,}617)}{(1000)(15)} = 1.12 \text{ k/ft}$

Enter SSE_H like Pressure:

$\frac{(1.12)(15)}{(9.25)(15)} = 0.121 \text{ k/sf}$

TOTAL Pressure $= 0.432 + 0.121 = 0.553 \text{ k/sf}$

$M_a = 145.46$ ft-k

$f_s = 0.9\ F_Y = (0.9)(40) = 36$ ksi

$A_s = 2.36\ in^2$ $\qquad k_1 d_1 = k_3 d_3 = 2.85 \Rightarrow k_1 = k_3 = 0.32$

$j_1 = j_3 = 0.89$

$k_2 d_2 = 2.49 \Rightarrow k_2 = 0.35$

$j_2 = 0.88$

$M_{CAP} = \Sigma\ f_s A_s j d$

$= \left[(36)(2.36)(0.89)(8.81)\right](2) + (36)(2.36)(0.88)(7.06)$

$= 1,860$ in-k $= 155$ Ft-k

$M_a < M_{CAP}$ ok
$(145.46 < 155)$

$F_a = 0.44\ f'_m = 0.396$ ksi $\qquad A = (9.25 \times 12)(28.61)$

$F_m = 0.85\ f'_m = 0.765$ ksi $\qquad = 3175.7\ in^2$

$C = 61.4^k$

$f_a = \dfrac{61.4}{3175.7} = 0.019$ ksi $\qquad S = \dfrac{216\,649}{\frac{28.61}{2}} = 15,145\ in^3$

$f_m = \dfrac{(145.46)(12)}{15,145} = 0.111$

$\dfrac{f_a}{F_a} + \dfrac{f_m}{F_m} = \dfrac{0.019}{0.396} + \dfrac{0.111}{0.765} = 0.2 < 1$ ok

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BY J. de MATOS DATE 10/17/80 SHEET 6 OF 205

CHKD. BY C. SHIH DATE 12/29/80 OFS NO. 3611.244 DEPT. NO. 550

CLIENT FLO.

PROJECT ST. LUCIE

SUBJECT WALL 1, 2, 3

$M_c = 145.46$ Ft-K

$I_{ek} = 9{,}244\ in^4$

Blk 16×11^{58}: $\frac{3{,}545}{9{,}244}(M_a) = M_1 = 55.78$ Ft-K

$55.78(12) = f_s(2.36)(0.89)(8.81)$

$f_s = 36.2$ ksi N.G. (REQUEST D=7% E-W FL. SPECTRA)

Blk 16×9^{58}: $\frac{2{,}154}{9{,}244}(M_a) = 33.9$ Ft-K

SHEAR STRESS: $\nu = \frac{48.3}{(111)(30.06)} = 0.014$ ksi < 0.051 ksi OK

$V = 48.3^k$

BOND STRESS

① Flexural Bond $\mu = \frac{48.3/3}{12 \times 1.57 \times 0.88 \times 8.81} = 0.110 < 0.186$ ksi OK

② Anchorage: $\mu_b = \frac{f_s \times A_s}{1.57 \times l} = \frac{(36.2)(0.2)}{1.57 \times 18} = 0.256 > 0.186$ ksi N.G.

Simply supported wall at bottom should then be assumed

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CLIENT FLO.

PROJECT ST. LUCIE

SUBJECT WALL 1,2,3 RE-EVALUATION (RE-ANALYSIS)

ATTACHMENT LOADS

NODAL POINT	LOAD (lb)	Σ LOADS (K)	MASS
1			
2			
3			
4	9+175	0.184	0.006
5	15+10	0.025	0.001
6	30+33+80+10	0.153	0.005
7	28+200+12+20+65+32	0.357	0.011
8	27+15+50+13+11	0.116	0.004
9	15	0.015	0.0005
10	55+25+45+21	0.146	0.005
11	50+25	0.075	0.002
12	11	0.011	0.0003
13	10	0.010	0.0003
14	15+10+10+15	0.050	0.002
15	25+35+10+7+10	0.087	0.003
16	10+10	0.020	0.001
17	20	0.020	0.001
18	10+20+10+10	0.050	0.002
19	10+22	0.032	0.001
20			

w_u $P = 3\,PSI = 0.432\ ^K/_{S.F.}$

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BY J. de MATOS DATE 11/10/80 SHEET 8 OF ...

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CLIENT FLO

PROJECT FLO. ST. LUCIE

SUBJECT WALL #1,2,3 RE-EVALUATION (RE-ANALYSIS)

Blk 16x11 5/8

$I_t = (3)(2537) + (4)(2,095) = 15,991\ in^4$ $t_e = 12.002\ in$

Blk 16x9 5/8

$I_t = (3)(1400) + 4(1189) = 8,956\ in^4$ $t_e = 9.893\ in$

TOTAL: $I_t = \frac{1}{12}(9.25 \times 12)(9.893)^3 + 2\left[\frac{1}{12}(9.25 \times 12)(12.002)^3 + (9.25 \times 12)(12.102)(10.948)^2\right.$

$= 360,297\ in^4 \Rightarrow t_e = 33.898\ in = 2.825\ (ft)$

Revised wt of WALL (125 #/CF)

$Wt\ (WALL) = (2.825)(15)(9.25)(125) = 48,996\ \#$

$M_{cr} = (0.067)(360,297 / 16.949) = 1,424.3\ in\text{-}k = 118.7\ ft\text{-}k$

FORM 361 REV 7-71

EBASCO SERVICES INCORPORATED
NEW YORK

BY J. de MATO DATE 11/10/80 SHEET 9 OF 2__
CHKD. BY C. S___ DATE 12/29/80 OFS NO. 3611.2__ DEPT. NO. ___
CLIENT FLO.
PROJECT ST. LUCIE
SUBJECT WALL 1,2,3 RE-EVALUATION (RE-ANALYSIS)

$F = 59.4$ (CPS) → $T = 0.017$ (sec.) ± 15%

FL. SPECTRA D = 5% E-W @ elev = 19.50

$SSE_H = 0.35g$

$$SSE_H = \frac{(0.35)(48,996)}{(1000)(14)} = 1.22\ ^k/Ft$$

Enter SSE_H like Press:

$$\frac{(1.22)(14)}{(14)(9.25)} = 0.132\ K/SF$$

TOTAL $P = 0.432 + 0.132 = 0.564\ K/SF$

$Ma = 130.6$ Ft-k

$Ma > Mcr$ SECTION IS CRACKED

From page 3 & 4

$I_c = 216,644\ in^4$ $t_c = 28.61$ in $= 2.384$ ft

$F = 50$ (CPS) ± 15% $T = 0.02$ (sec)
(From new SPECTRA D = 7% E-W @ -0.50)

$SSE_H = 0.30g$

$SSE_H = (0.30)(0.125^k)(2.83) = 0.106\ ^k/SF$

$P = 0.432 + 0.106 = 0.538$ K/SF.

FORM 561 REV 7-71

EBASCO SERVICES INCORPORATED

NEW YORK

BY J. deMATOS DATE 11/10/80 SHEET 10 OF 2?

CHKD. BY C. SHIH DATE 12/29/80 OFS NO. 3611.249 DEPT. NO. 530

CLIENT FLO.

PROJECT ST. LUCIE

SUBJECT WALL 1,2,3 RE-EVALUATION (RE-ANALYSIS)

$M_a = 124.7$ ft-K

$M_a < M_{CAP}$ ok

$M_{CAP} = 155$ ft-K (from page 5)

Moment in Blk 16x11 5/8 : Moment in 16x9 5/8 Blk.

$$M_{a_1} = \frac{3,545}{9,244}(M_a) = 47.8 \text{ ft-K}$$

$$M_{a_2} = 124.7 \times \frac{2154}{9244} = 29^{\prime K}$$

$$(12)\,47.8 = f_{s_1}(2.36)(0.89)(8.81)$$

$$f_{s_1} = 31 \text{ ksi} < 36 \text{ ksi} \quad \text{ok}$$

$$f_{s_2} = \frac{29 \times 12}{2.36 \times 0.88 \times 7.06} = 23.73 \text{ ksi} < 36$$

o.k

Attachment F

APPENDIX 3A

A LITERATURE SURVEY
TRANSVERSE STRENGTH OF MASONRY WALLS

by

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and

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College of Engineering
University of California
Berkeley, California

March 1977

ABSTRACT

The literature survey presented collates most of the available relevant information on the transverse or out-of-plane strength of masonry walls. The report discusses several of the test techniques used and summarizes the most significant available test results. Formulations for predicting the capacity of walls subjected to transverse loads are presented together with their correlation with experimental results. Also included is a section relating test results to present design practices and code requirements.

4. FORMULATIONS TO PREDICT THE TRANSVERSE STRENGTH OF MASONRY WALLS

4.1 Introduction

The objective of most experimental research projects is to validate or improve a theoretical model. Because of the complexities associated with the non-homogeneity of masonry structural members, accurate theoretical models are difficult to develop and in many cases empirical or simplified relationships have been developed in their place. With respect to the transverse strength of masonry walls, several different theoretical approaches have been used. The most extensive work has been performed by Yokel et al.[6,30,31] who evaluated the theoretical capacity of unreinforced walls in a manner similar to that for concrete columns. In a correlation of the experimental results with their theory, inclusion of the slenderness effect of the walls produced reasonable agreement.

Both Scrivener[27] and Dickey[25] worked with reinforced masonry walls; they used formulations similar to those used for reinforced concrete beams and obtained reasonable correlation with experiments. Cajdert and Losberg[41] and Haseltine and Hodgkinson[42] used an analogy with the yield line theory for reinforced concrete slabs and performed tests on both reinforced and unreinforced walls with several different boundary conditions. Baker[43] used another method commonly used for reinforced concrete slabs; that of assuming the strength of a wall is given by the strength of two independent strips spanning in either direction. Baker performed experiments with one-third scale model panels simply supported on all edges.

Each of the above formulations and its correlation with experiments are described in the following sections.

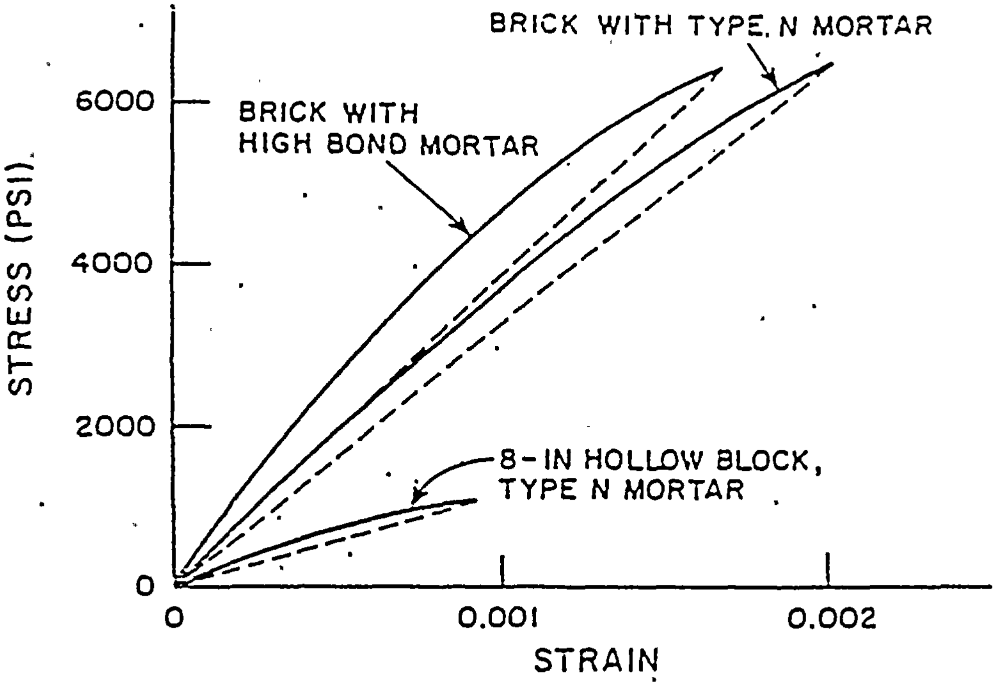
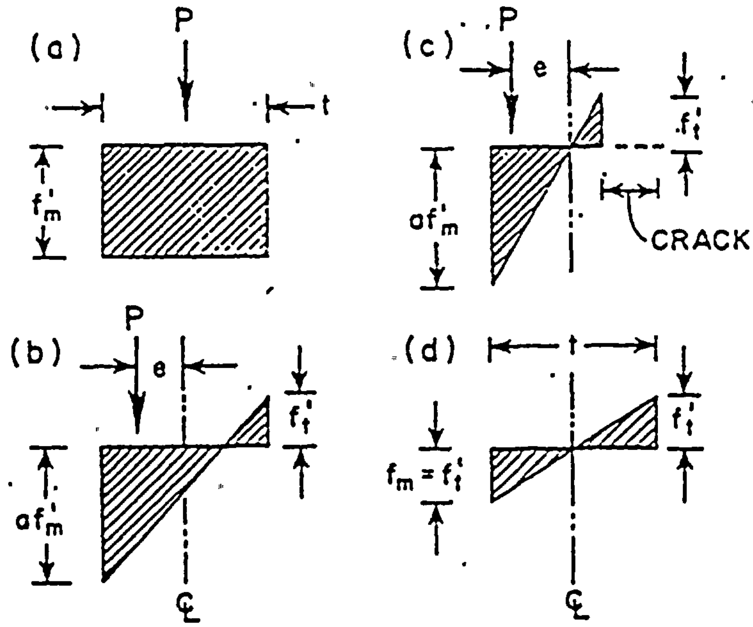


FIG. 4.1 STRESS-STRAIN PROPERTIES OF MASONRY



NOTE: t = wall thickness
f_t' = flexural tensile strength of masonry
f_m' = compressive strength of masonry
a = flexural compressive strength coefficient

FIG. 4.2 STRESS DISTRIBUTION AT FAILURE UNDER VARIOUS VERTICAL-LOAD AND MOMENT COMBINATIONS

From Reference (4)

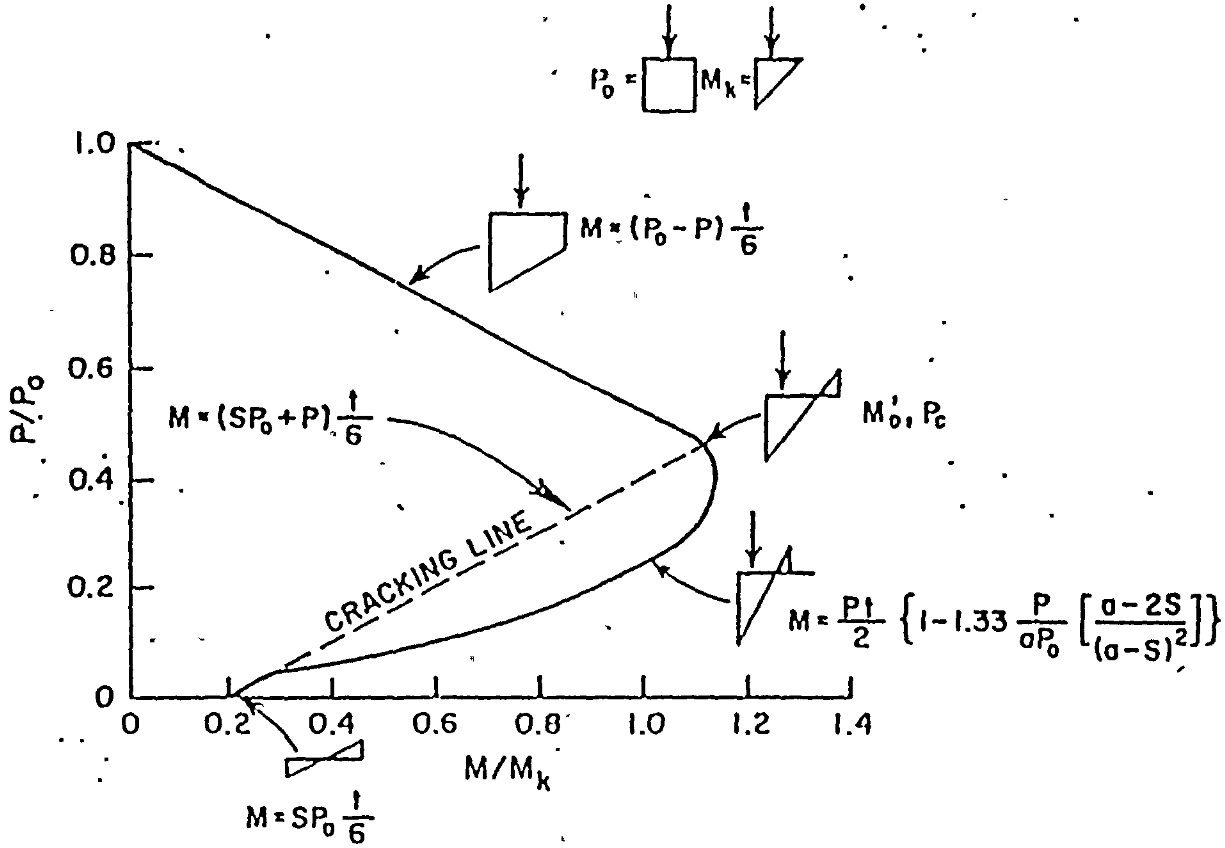
4.2 Cross-Sectional Capacity of Unreinforced Walls

The moment capacity of a cross section of a wall is not only a function of the tensile and compressive strengths of the masonry but also of the vertical load acting on the cross section. If the flexural, tensile and compressive strengths and the stress-strain properties of the masonry are known, an interaction curve between vertical load and moment can be drawn.

Yokel et al. show typical stress-strain curves for three different types of masonry, see Fig. 4.1. In order to simplify the analysis, a linear stress-strain relationship is assumed as shown by the dashed line in Fig. 4.1. Instead of this basic assumption, Meinheit[32] suggested that a stress-strain relationship more like that of concrete would give better agreement with experimental data.

If it is assumed that a plane section of the wall remains plane in flexure, and that a linear stress-strain relationship as shown in Fig. 4.1 is a valid approximation for masonry up to the point of failure, then the stress distribution at failure over a cross section under an eccentric vertical load can be determined as shown in Fig. 4.2. Figure 4.2(a) shows the stress distribution at failure under axial loading. In Fig. 4.2(b), the load eccentricity is increased to a point where, at failure, the section develops its flexural tensile strength at one wall face and its flexural compressive strength at the other wall face. If the load eccentricity is increased further, the stress distribution at failure will be associated with a cracked section as shown in Fig. 4.2(c). Finally, Fig. 4.2(d) shows the stress distribution at failure for pure flexure, when no resultant vertical load acts on the cross section. In this last case, the capacity depends entirely on the flexure tensile strength of the masonry.

3A-6



NOTE: P_o = axial load capacity $P_o = f'_m\ bt$

M_k = moment capacity $M_k = P_o t/12$ which corresponded to the stress distribution in Fig. 4.2(b)

t = thickness of wall

b = width of wall

s = ratio of tensile strength to axial compressive strength of masonry (f'_t/f'_m)

FIG. 4.3 CROSS SECTIONAL CAPACITY OF RECTANGULAR PRISMATIC SECTION WHEN $f'_t = 0.1f'_m$ AND $af'_m = f'_m$ (a=1)

From Reference (4)

Att F

Figure 4.3 shows an interaction curve for a solid rectangular section. The interaction curve is based on the assumption that flexural compressive strength equals the compressive strength under axial compression ($f'_m = a\ f'_m$, or $a = 1$). Typical stress distributions, associated with different portions of the curve, are shown in the figure and also the equations of these curves are shown. Further details of these interaction curves are discussed by Yokel and Dikkers[30].

4.3 Slenderness Effects Of Unreinforced Walls

The effects of slenderness on the moment capacity of walls are shown in Figs. 4.4 and 4.5. Figure 4.4 shows the free body of the upper half of a deflected wall under axial and transverse loads. The effective moment at any point along the height of this wall will be determined by the location of the line of action of the vertical force, relative to the location of the deflected centerline of the wall. Figure 4.5 shows a wall which is free to rotate at its upper and lower ends and is subjected to an eccentric vertical load which has a thrust line parallel to the axis of the wall. The moment acting on this wall is P e at the upper and lower ends of the wall. At midheight, the moment is equal to $P(e + \Delta)$. Thus the deflection of the slender wall causes a moment magnification equal to $P\Delta$. The moment magnification can be predicted approximately as

$$P(e + \Delta) = Pe \frac{1}{1 - \frac{P}{P_{cr}}} \qquad (4.1)$$

where $P_{cr} = \pi^2 EI/h^2$ (Euler load)

E = modulus of elasticity

I = moment of inertia of cross section

h = total height of wall.

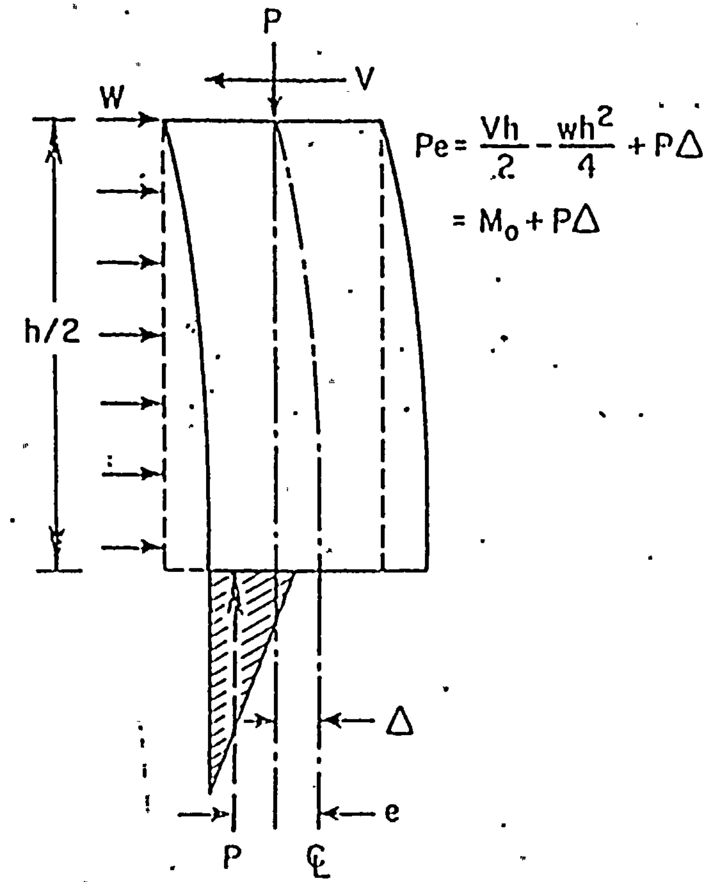


FIG. 4.4 SLENDERNESS EFFECTS ON EQUILIBRIUM
From Reference (30)

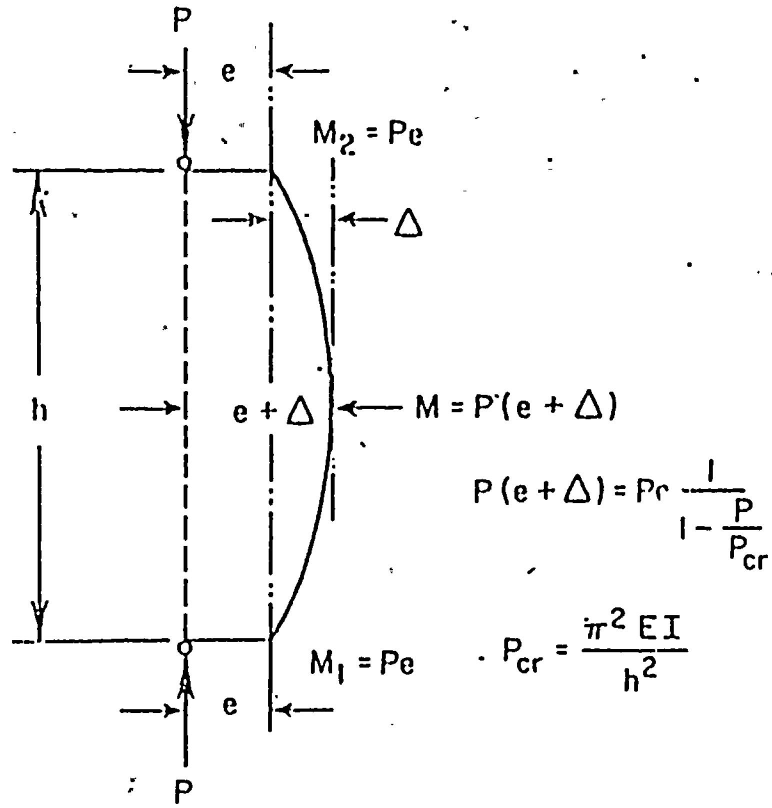


FIG. 4.5 SLENDERNESS EFFECT
From Reference (30)

The condition shown in Fig. 4.5 is not likely to occur in an actual building. A more realistic case is shown in Fig. 4.6 which shows an eccentrically loaded wall which is more or less fixed at its base and more or less free to rotate at the top. In this case the moment is not magnified as much as in Fig. 4.5, and if the wall is very stiff the moment may not be magnified at all.

An approximate prediction of moment magnification for any combination of end eccentricities and end fixities is given by[6,31,33]

$$M = M_o \frac{C_m}{1 - \frac{P}{P_{cr}}} \tag{4.2}$$

where M = maximum moment acting on the wall,

M_o = maximum moment imposed by external force.

(For an eccentric vertical load $M_o = P\,e$ and for a transverse load $M_o = \frac{wh^2}{8}$).

$C_m = 0.6 + 0.4\, M_1/M_2 \geq 0.4$,

where M_1 = the smaller end moment acting on the wall

M_2 = the greater end moment acting on the wall

$P_{cr} = \pi^2\, EI/(kh)^2$ critical load

k = length coefficient by which height is adjusted to equivalent height as shown in Fig. 4.7.

In Eq. (4.2), C_m is equal to zero for the case shown in Fig. 4.5 and for the case of transverse loading.

In order to estimate the value of the critical load P_{cr} in Eq. (4.2), the flexural wall stiffness EI is also important. Yokel et al.[31] in a study of vertically loaded unreinforced and reinforced

Att F. 10 of

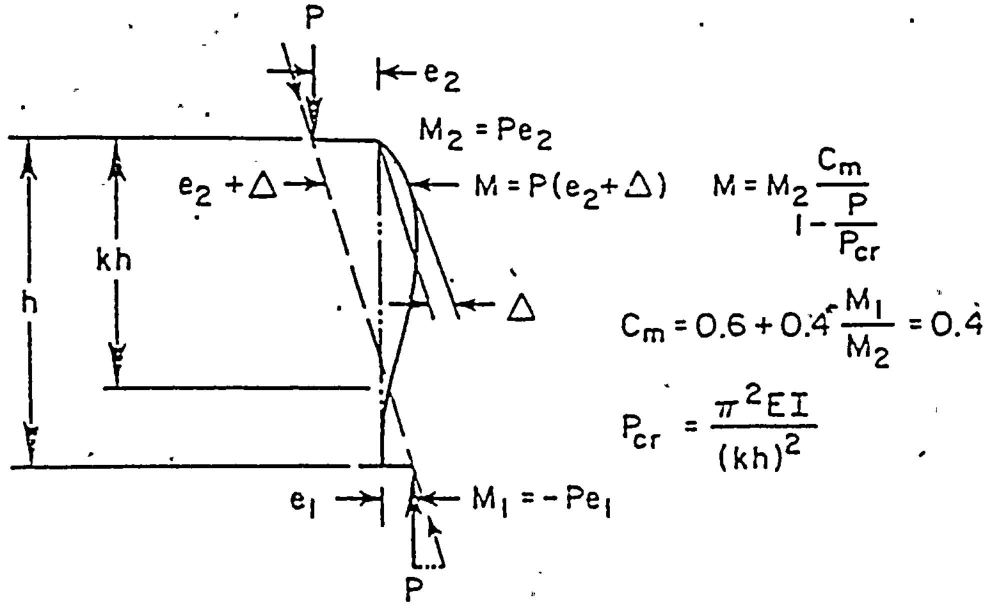


FIG. 4.6 EFFECT OF END CONDITIONS
From Reference (30)

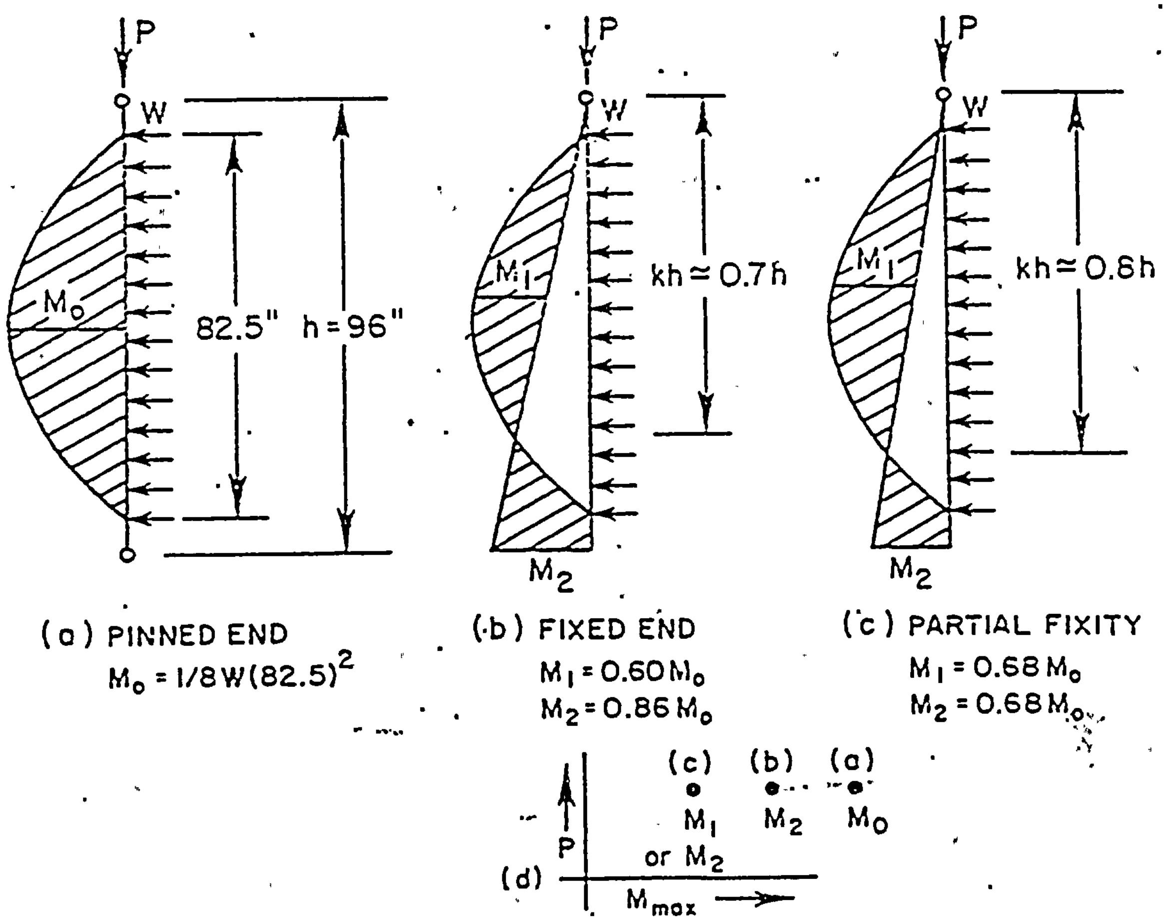


FIG. 4.7 INFLUENCE OF END CONDITIONS
From Reference (30)

concrete masonry walls suggested the following expressions to approximate to EI:

$$EI = E_i I_n/2.5 \text{ (reinforced masonry)} \qquad (4.3)$$

$$EI = E_i I_n/3.5 \text{ (unreinforced masonry)} \qquad (4.4)$$

where E_i = initial tangent modulus of elasticity

I_n = moment of inertia of uncracked net section.

For transverse loading combined with a vertical load for brick walls, Yokel[6] proposed that

$$EI = E_i I_n \cdot (0.2 + \frac{P}{P_o}) \leq 0.7\ E_i I_n,$$

where P_o = short wall axial load capacity determined on the basis of prism strength.

4.4 Correlation Between Theory And Experiments For Unreinforced Walls

Figure 4.8 shows an example of correlation of theory developed from Sections 4.2 and 4.3 with the combined vertical and transverse load tests on 4 inch brick walls with type N mortar conducted by Yokel et al.[6]. The test results are shown by solid circles and heavy horizontal lines. The left ends of these heavy lines represent the maximum moment caused by transverse load. The length of the horizontal line itself represents the added moment, equal to the product of the vertical load and the wall deflection at the point of maximum moment (mid-height). The magnitude of this added moment was computed using the horizontal deflections, measured at the time of wall failure.

The solid curve in Fig. 4.8 is the calculated cross-sectional capacity which is shown in Fig. 4.3 and should be compared with the

Att, F 12 of 1

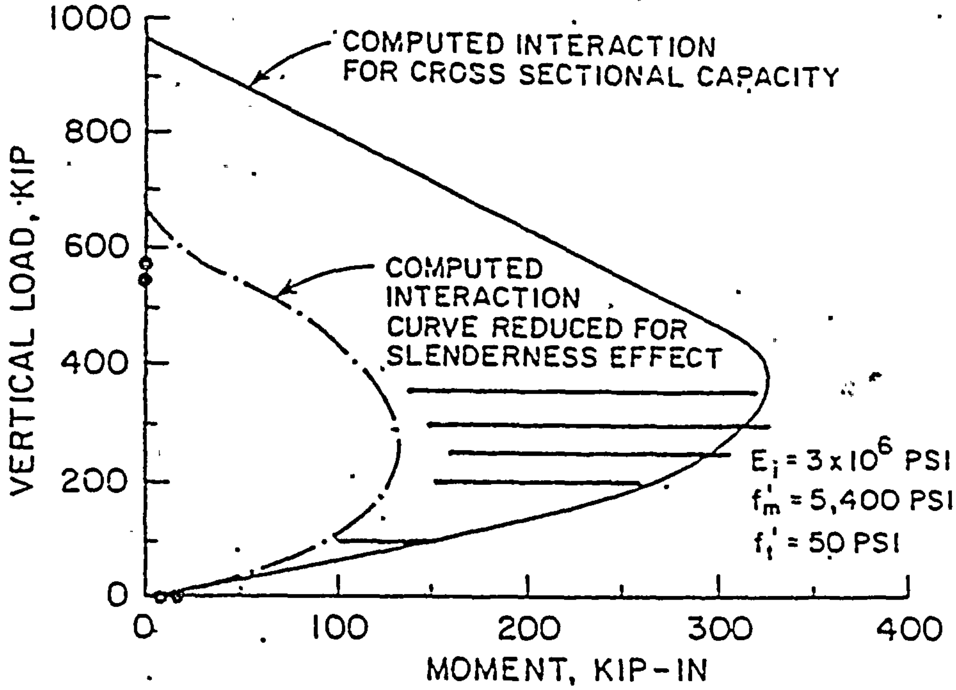


FIG. 4.8 4-IN BRICK WALLS WITH TYPE N MORTAR UNDER VERTICAL AND TRANSVERSE LOAD

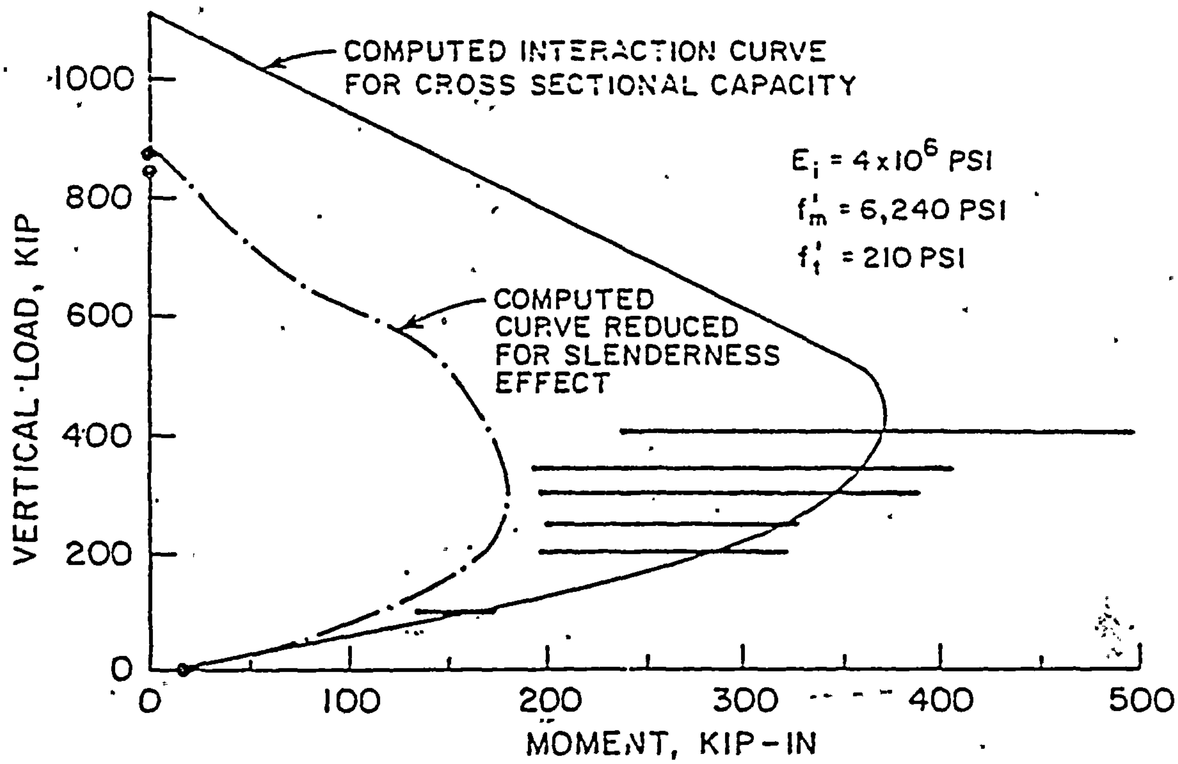


FIG. 4.9 4-IN BRICK WALLS WITH HIGH BOND MORTAR UNDER VERTICAL AND TRANSVERSE LOAD

From Reference (26)

right end of the horizontal line. The broken curve represents the wall capacity, computed by reducing the cross sectional capacity for slenderness effect in accordance with the theory discussed in Section 4.2. This reduced curve corresponds to the left ends of the horizontal solid lines. The intersection of the broken curve with the vertical load axis corresponds to the two solid circles on the load axis, which show the test results under vertical load without transverse load. Note that the theoretical curves closely predict the actual magnitude, as well as the trend of the test results. Slenderness effects are considerable in this case and their magnitude is well predicted by theory.

Similar comparisons are shown in Fig. 4.9 for 4 inch brick walls with high-bond mortar, and in Figs. 4.10 and 4.11 for 8 inch hollow block walls with type N mortar and high-bond mortar, respectively. The 4 inch brick walls with high-bond mortar show fair agreement between theoretical curves and test results, whereas the 8 inch hollow concrete walls show that the theoretical short-wall interaction curves (solid curves in Figs. 4.10 and 4.11) underestimate the wall strength for all panels. The reduced interaction curves (broken curves) predict moment capacities equal to or smaller than the observed reduced capacity

Figure 4.12 also compares the observed transverse strength of the walls with the theoretical interaction curves for 8 inch solid concrete block walls with type N mortar. All panels except one exceed the reduced moment capacity (dashed line) predicted on the basis of the axial prism test.

In the case of cavity walls or composite walls, theoretical interaction curves are somewhat different from those of single wythe

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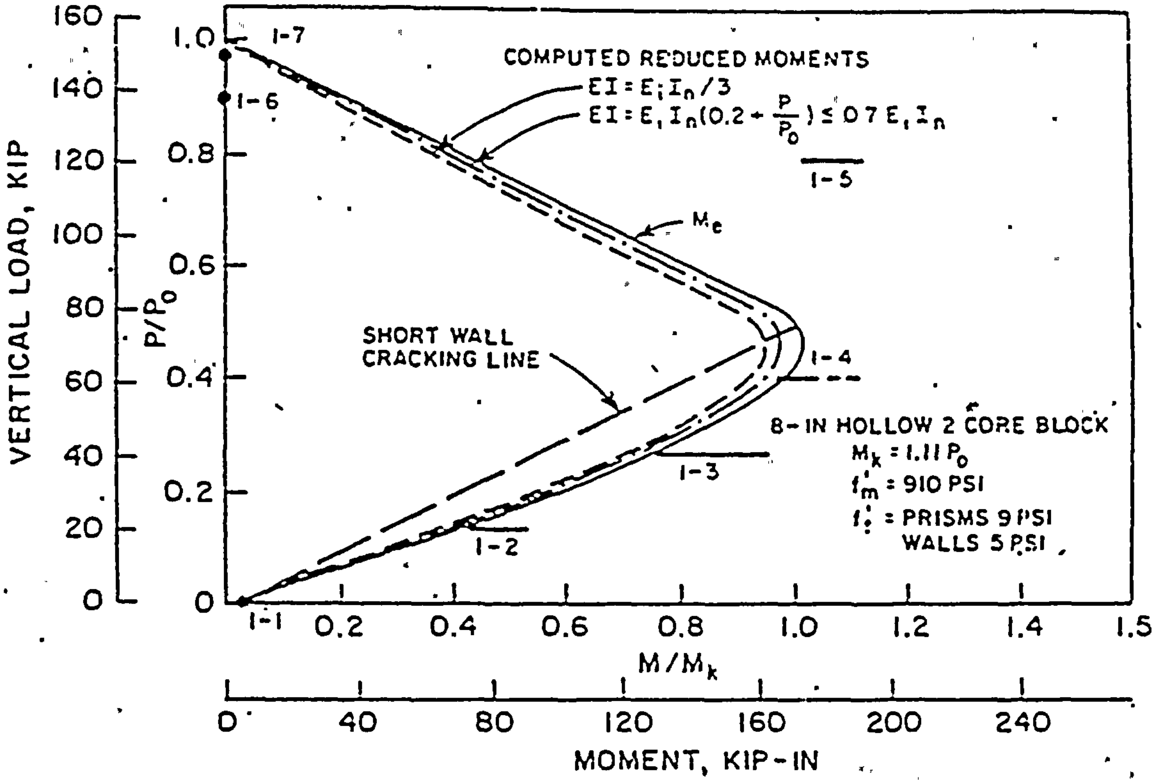

FIG. 4.10. 8-IN HOLLOW BLOCK WALLS WITH TYPE N MORTAR, CORRELATION WITH PRISM STRENGTH

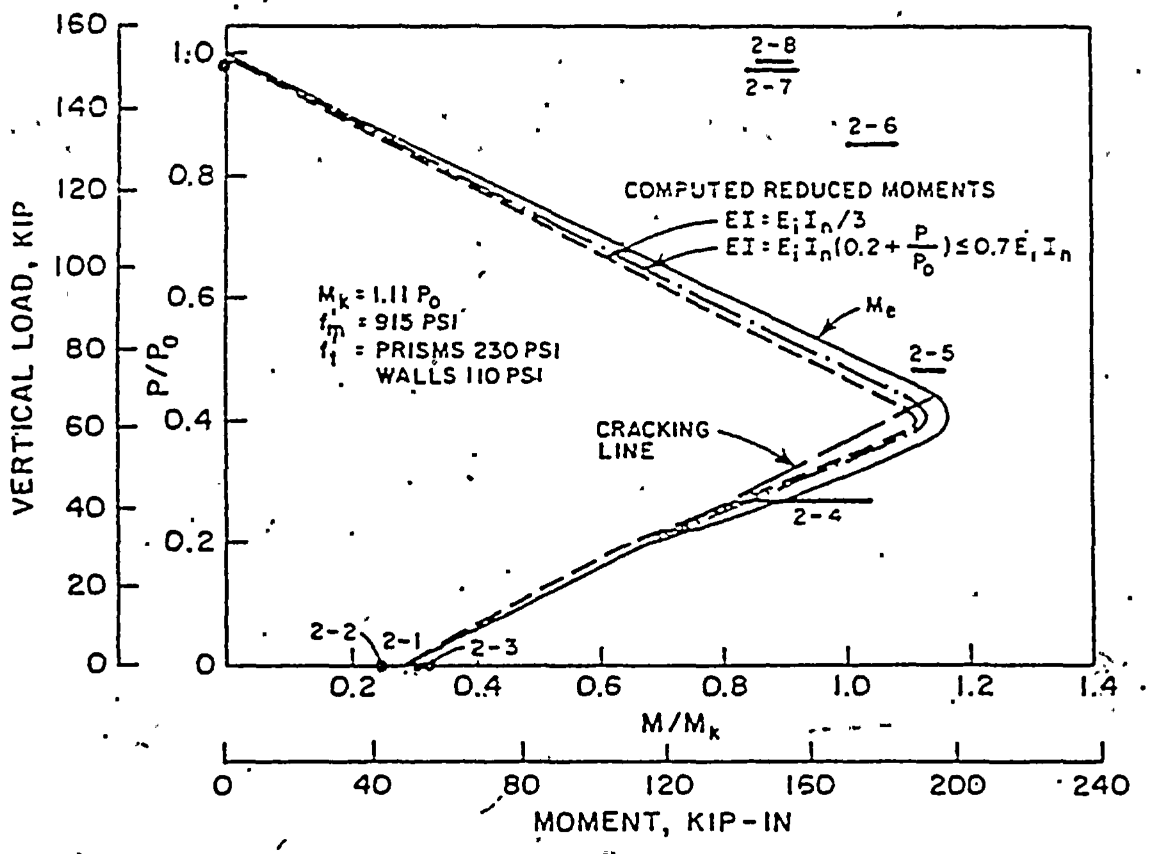

FIG. 4.11 8-IN HOLLOW CONCRETE BLOCK WALLS WITH HIGH BOND MORTAR, CORRELATION WITH PRISM STRENGTH

From Reference (4)

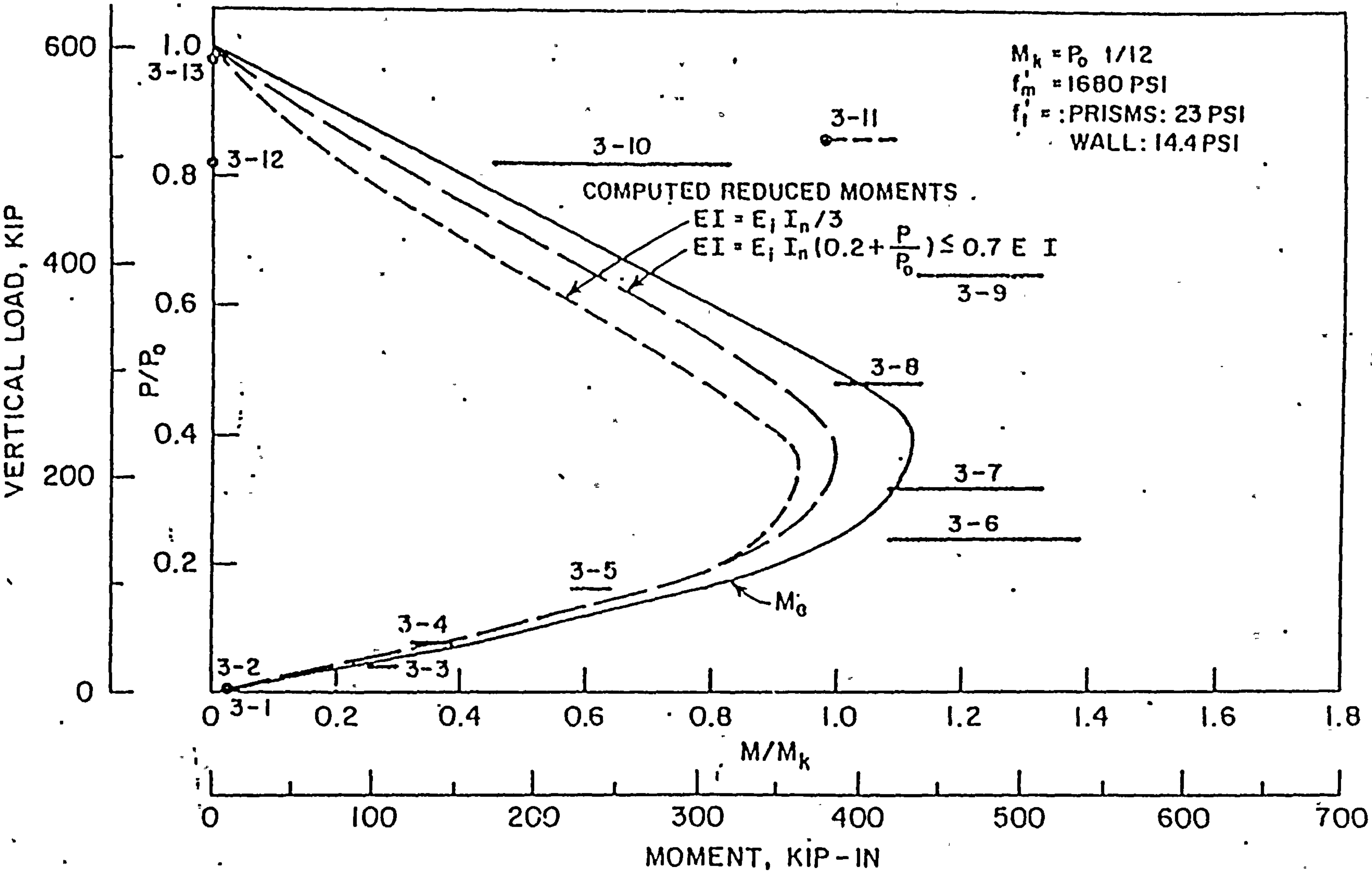

FIG. 4.12 8-IN SOLID CONCRETE BLOCK WALLS WITH TYPE N MORTAR, CORRELATION WITH PRISM STRENGTH
From Reference (4)

AH. F 16

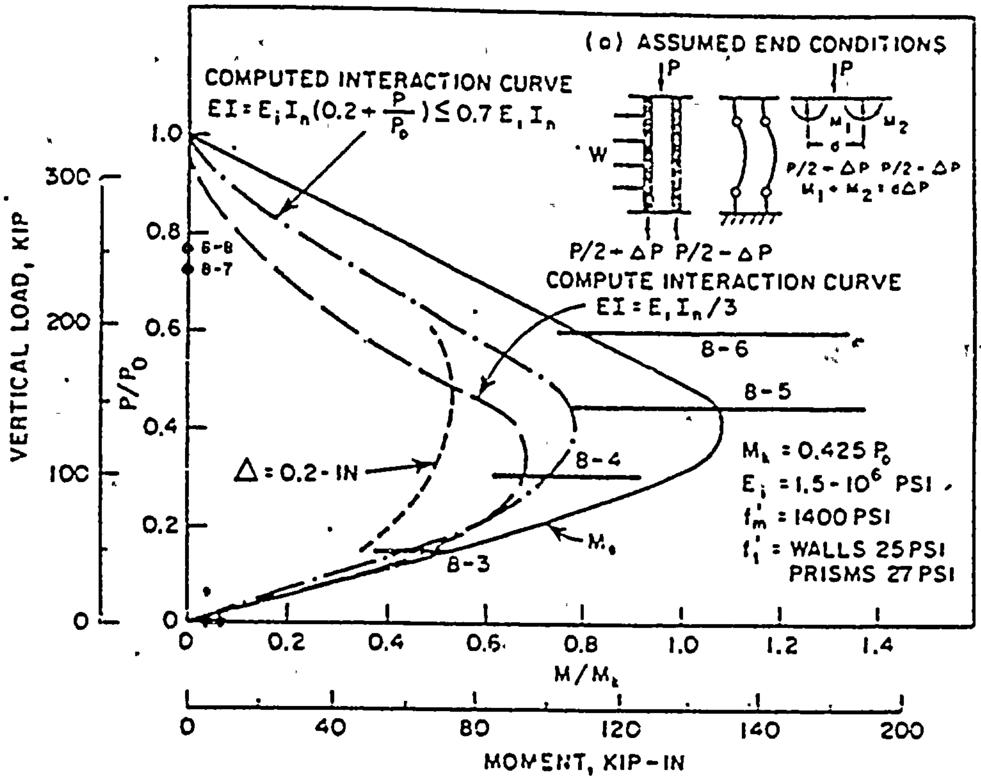

FIG. 4.13 4-2-4 IN CONCRETE BLOCK CAVITY WALLS, CORRELATION WITH PRISM STRENGTH

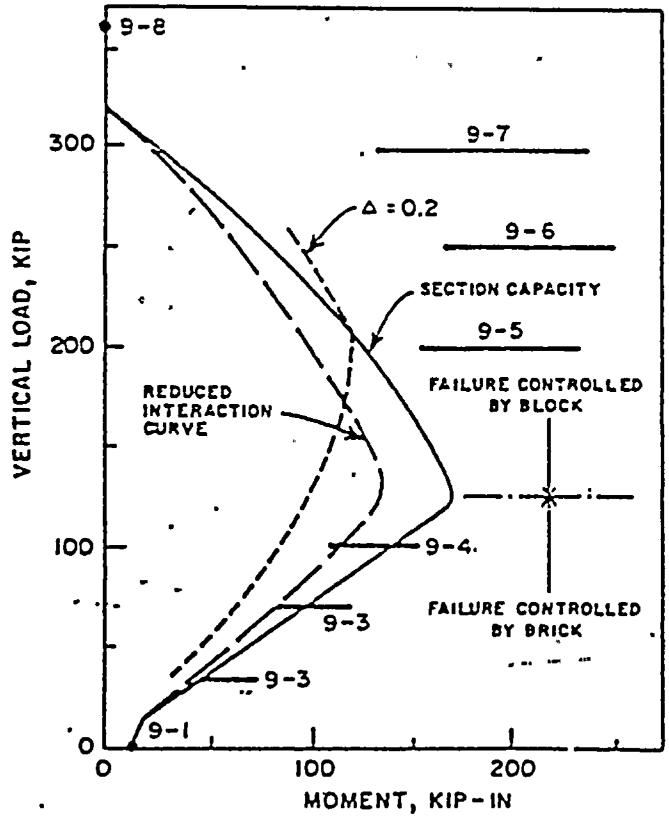

FIG. 4.14 4-2-4 IN BRICK AND CONCRETE BLOCK CAVITY WALLS, CORRELATION WITH PRISM STRENGTH

From Reference (4)

walls, but similar comparisons can be developed. The results of tests[6] of 4-2-4 in. concrete block cavity walls are plotted in Fig. 4.13 together with interaction curves computed on the basis of prism tests. The assumption was made that each wythe takes one half the vertical load and one half the moment. P_o was computed on the basis of the average strength obtained from prism tests on the 4 inch hollow block. Moments were computed conservatively, assuming that partial top-end fixity existed and this produced about one half the pin-ended moment, see Fig. 4.13(a). The analytical curve for section capacity reflects the tests reasonably well. It can be seen from the magnitude of the observed added moments which are due to deflection at failure (length of the horizontal solid line), that slenderness effects are an important factor in this wall system.

The prediction of wall capacity for brick-block cavity walls is more difficult and complicated because of the two different material properties and associated load transfer mechanism. Details of these prediction formulae are given by Yokel et al.[6], whose final results are shown in Fig. 4.14. Figure 4.14 shows that up to P = 100 kip, the moment capacity is controlled by the brick. In this range the computed reduced moment capacity (dashed line) agrees well with the test. The total moment capacity, which is shown by the solid line is somewhat less than observed capacity (right ends of the solid horizontal lines) and consequently the magnitude of the measured slenderness effect is larger than that of the computed effect. Above an axial load of 100 kips the computed strength underestimates observed wall strength considerably. In this range it is thought that strength is controlled by the concrete block which forms the back face with respect to the transverse load.

AH. F 12

Yokel et al. summarized their extensive investigations with the following conclusions:

(1) Transverse strength of masonry walls is reasonably predicted by evaluating the cross-sectional capacity and reducing that capacity to account for the added moment caused by wall deflection. The general trend of the test results is in good agreement with theory, and the magnitude of individual test results is conservatively predicted.

(2) Cross-sectional moment capacity of wall panels was conservatively predicted by a theoretical interaction curve which was based on compressive prism strength and linear strain gradients.

(3) Slenderness effects, computed by the moment magnifier method as modified to account for section cracking, predicted closely the slenderness effects observed in the 4 inch thick brick walls, and reasonably predicted these effects for concrete masonry walls, concrete block cavity walls, and brick and block cavity walls.

(4) The qualitative observation was made that with large eccentricities the flexural compressive strength of masonry exceeds the compressive strength developed in pure one-dimensional compression by a significant margin, and that flexural compressive strength increases with increasing strain gradients.

(5) The transverse strength of cavity walls was conservatively predicted by assuming that each wythe carries its proportional share of vertical loads and moments, and that transverse loads, but not shear forces parallel to the plane of the wall, are transmitted by the ties.

(6) The transverse strength of composite brick and block walls was approximately predicted by assuming that the walls act monolithically.

(7) Whenever walls did not fail by stability-induced compression failure, their axial compressive strengths were reasonably predicted by

prism tests. In the case of concrete masonry with high-bond mortar, compressive tests with prisms capped with high strength plaster over-estimated wall strength, while prisms set on fiberboard showed good correlation with wall strength.

(8) Flexural tensile strength of all the wall panels tested equaled or exceeded 1/2 of the flexural strength as determined by prism tests.

4.5 Flexural Capacity Of Reinforced Masonry Walls

Scrivener[27] suggested that a reinforced brick wall could be considered as a lightly reinforced wide beam, with the brick weak in tension similar to concrete. The yield load (ultimate load) can be predicted to within a few percent by considering the section in this way and applying ultimate moment theory (as for reinforced concrete). The stress strain curve for brick is assumed to be the same as that for concrete so that the concrete constant 0.59 in the Whitney equation can be used. The ultimate moment M_u is

$$M_u = A_s f_y (d - 0.59 A_s f_y/f'_c b) \qquad (4.5)$$

where A_s = cross-sectional area of steel

f_y = yield stress of steel

d = depth to center of gravity of steel

b = beam width

f'_c = brick crushing strength.

A comparison between the theoretical ultimate loads calculated by Eq. 4.5 and the transverse load tests performed by Scrivener are discussed in Section 3.4 and are shown in Table 3.12.