SEAOSC Slender Wall Task Group
UBC 97 and ACI 318-02 Code Comparison
Summary Report

Executive Summary

Introduction

Recognizing there have been questions on the differences between the alternate slender wall design procedures in 1997 UBC and in ACI 318-02, the SEAOSC Board authorized a Task Group to provide a comprehensive review of the two design procedures. The ACI procedure was adopted by IBC 2000 and subsequent code editions. As quoted in ACI 318R-02 Commentary Section R14.8, Section 14.8 is based on the corresponding requirements in the UBC and experimental research of the Test Report by SCCACI-SEAOSC.

This summary report includes review of source documents, code comparison, and background of the design provisions under UBC and under ACI, respectively. A comprehensive review of the 1980 test data was made in addition to analytical comparison of sample wall panel design under each of the two procedures. Pursuant to the comparative design and validation of the original data, a list of findings is presented in the Report. Other design considerations though not part of the code comparison are discussed in order to encourage further studies by other groups. The report concludes with recommendations to SEAOSC Board and proposed changes to ACI.

Code Comparison

Under 97 UBC Section 1914.8, the cracked moment is based on $f_r = 5 \sqrt{f'_c}$; and in ACI 318-02 Section 14.8, the cracked moment is based on $f_r = 7.5 \sqrt{f'_c}$. This also means that the $M_{cr\,(UBC)} = 2/3\ M_{cr\,(ACI)}$ in the application of the two design procedures. In the 97 UBC, a linear interpolation between Δ_{cr} and Δ_n is permitted in obtaining Δ_s in order to simplify the slender wall panel design for $M_s > 5 \sqrt{f'_c}\, I_g/y_t$. The ACI procedure employs effective moment of inertia and a magnified moment for the combined moment due to lateral and eccentric vertical load, also know as the P-Δ effect. Table 1 gives section by section comparison between the alternate slender wall design procedures.

Review of 1980 Test Data

This Task Group was able to review and re-analyze the original test data. Verification of the 1980 data using adjusted lateral force and deflection data was performed. The analytical result follows closely with the bilinear load deflection characteristic. Lateral deflection increases rapidly when the moment exceeds two-third (2/3) of M_{cr} (as defined by ACI). The calculated moments for each of the twelve test panel correlate closely with the empirical test data. The load deflection curves and plots for the low axial loads versus moment interaction curve further validate the UBC design procedure. ACI needs to improve its methodology in computing M_u and I_e so that computed results would follow a bilinear load deflection characteristic.

Summary of Findings

Summary of comparative design examples is given on Table 5. Design based on ACI procedure is normally controlled by strength with service load deflection less than Δ_{cr}. ACI procedure significantly under-estimates service load deflection in comparison to the UBC procedure with increase lateral force and/ or axial load. Where wall panel design based on ACI procedures meets strength and deflection limit, the corresponding wall panel calculation based on UBC procedure may exceed the deflection limit.

Recommendations

- To calculate service load deflection, use E/1.4 for earthquake forces.
- Recommend to appropriate enforcement agencies that adoption of the 2003 IBC provisions on alternate design of slender wall procedure should incorporate proposed changes to ACI 318-05 Section 14.8.4.
- Modification to ACI 318-05 Section 14.8.4 - delete equations (14-8) and (14-9) and the last paragraph in total, and replace with the following after the first paragraph:

  " $\mathbf{\Delta_s = 0.67\Delta_{cr} + (M_s - 0.67M_{cr})(\Delta_n - 0.67\Delta_{cr}) \div (M_n - 0.67M_{cr})}$; for $M_s > 0.67M_{cr}$ (14-8)

  $\mathbf{\Delta_s = 5\, M_s\, l_c^2 \div (48 E_c\, I_g)}$; for $M_s < 0.67M_{cr}$ (14-9)

- Send a letter to ACI-318 addressing the concerns in using the ACI alternate design of slender wall procedure and requesting ACI 318 to correct statements under Commentary R14.8.

SEAOSC Slender Wall Task Group
UBC 97 and ACI 318-02 Code Comparison
Summary Report

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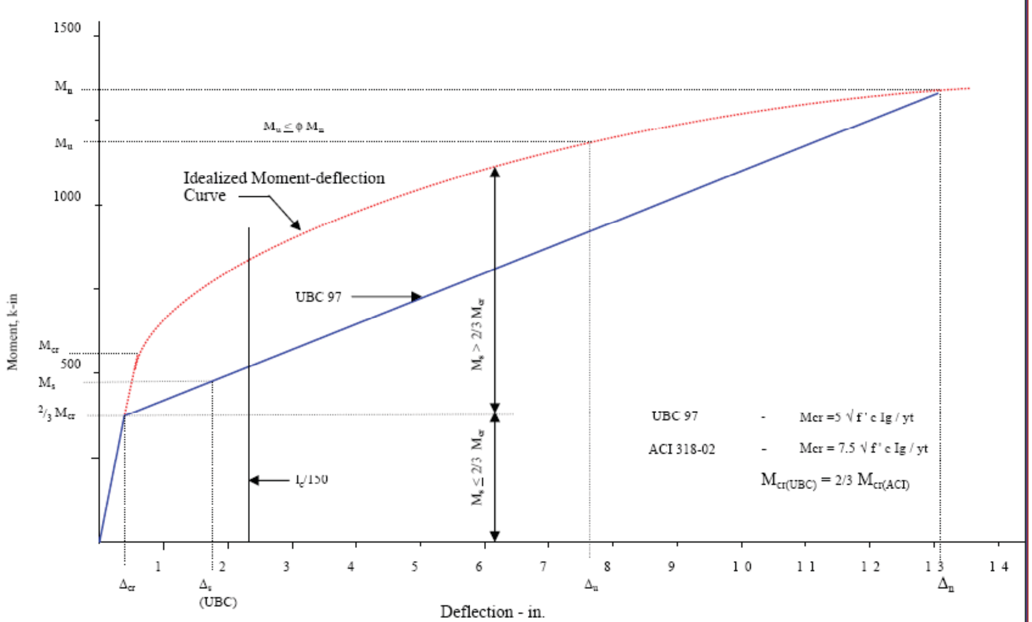
1. **Background**:

The original code development on alternate slender wall design was introduced into the 1987 UBC Supplement through efforts of SEAOC Building Code Committee. The provision was based on findings of Joint SCCACI- SEAOSC Task Committee on Slender Walls pursuant to full scale tests conducted in the early 1980's on twelve 4 feet wide by 24 feet high concrete wall panels of varying height to thickness ratios ranging from 30 to 60. [Refer to "Test Report on Slender Walls", aka "Green Book"]. The design procedure is predicated on control of out-of-plane deflection for serviceability under code prescribed forces in addition to required moment strength.

2. **Issue:**

In 1997 UBC Section 1914.8, the cracked moment is based on $f_r = 5 \sqrt{f'_c}$; and in ACI 318-02 Section 14.8, the cracked moment is based on $f_r = 7.5 \sqrt{f'_c}$. In the 97UBC, a linear interpolation between Δ_{cr} and Δ_n is permitted in obtaining Δ_s, the deflection at service load, in order to simplify the slender wall panel design for $M_s > 5 \sqrt{f'_c}\, I_g/y_t$. The conceptual moment-deflection curve shown in the figure below demonstrates the intent of the UBC provision. At the ordinate of $M_s > 2/3\, M_{cr}$, using the straight line linear interpolation between Δ_{cr} and Δ_n, UBC procedure gives a higher Δ_s, deflection under service load, than the corresponding value based on ACI 318 procedure. When the lower bound is raised from $f_r = 5 \sqrt{f'_c}$ to $f_r = 7.5 \sqrt{f'_c}$, the design of slender wall panels based on ACI procedure may significantly under-estimate service load deflection.



Moment Deflection Curve

3. Mission Statement

Recognizing there have been questions on the differences between the two design approaches, the SEAOSC Board authorized a Task Group to provide a comprehensive review of the two design procedures. In June, 2005, the Committee set forth to accomplish the following missions:

- Document review
- Review background of UBC provisions
- Review background of ACI provisions
- Perform sample calculations on an array of lateral force and axial load combinations
- Provide summary of findings
- Other design considerations
- Recommendations to SEAOSC Board
- Proposal for possible code change, if necessary

4. Document Review

Documents reviewed are listed in the reference section. The Green Book, "Test Report on Slender Walls" by SCCACI-SEAOSC Task Committee on Slender Walls, 1982 edition, was used as the primary data resource. Records of the 1980 test and data file were retrieved from archive. An abbreviated summary of the 1980 test panel properties and test data are given in Tables 6.1 to 6.8. Current draft of Design Guide for Tilt-up Concrete Structures, ACI Committee 551 was used as the source information on the development of the ACI design procedure.

5. Code Comparison

Table 1 gives section by section comparison between the alternate slender wall design procedure based on 97 UBC and that based on ACI 318-02. The ACI procedure was adopted by IBC 2000 and subsequent code editions. As quoted in ACI 318R-02 Commentary Section R14.8, Section 14.8 is based on the corresponding requirements in the UBC and experimental research of the Test Report by SCCACI-SEAOSC. The ACI Commentary further alleged that the procedure, as prescribed in UBC, has been converted from working stress to factored load design. This could also imply that the ACI procedure as written is a direct conversion of UBC procedure. In order to clarify and clearly understand the two procedures, several examples were used within a range of wall panel thickness, reinforcement ratio, axial load and lateral forces. Results of the analytical comparison are discussed in Section 9 of this Report.

6. Background of UBC Provisions on Alternate Slender Wall Procedure

Between late 1979 and 1982, a Joint Task Committee including members from the Southern California Chapter ACI and the Structural Engineers Association of Southern California was organized to study the design procedure of thin wall panels. Model building codes at that time limited the height to thickness ratio (h/t) to 25 for bearing walls and 30 for non-bearing walls. However tilt-up wall panels designed with variable moment of inertia accounting for the influence of axial loads and lateral instability such as $P\Delta$ moment were exempt from the h/t limitation. Non-bearing wall panels were designed with height to thickness ratio well in excess of 36.

While the 1980 Task Committee members agreed that elastic lateral instability (buckling) might be overly stated in building codes, the Committee concluded that full scale tests were needed in order to explore the inelastic behavior of tall slender wall. As a result of this non-profit research during the early 80's., results of the experimental work were presented in a "Test Report on Slender Walls." The test results gave better understanding in the performance of slender wall panels. There was no evidence of elastic and inelastic out-of-plane instability for the loading range tested. Subsequently, members of the SEAOC Building Code Committee authored and submitted proposed code change to ICBO offering an alternate design procedure for slender wall panels. The methodology emphasized deflection control in addition to strength to assure a wall of reasonable straightness after a service level loading. Required moment strength under UBC procedure is based on strength design. The slender wall provision was adopted and first included in 1987 UBC Supplement. During the ICBO code development hearing, the deflection limit of $l_c/100$, which was recommended by the 1980 Task Committee, was changed to $l_c/150$. While other minor changes were made in subsequent code development cycles on distribution of concentrated load, the alternate design procedure was not affected.

7. Review of 1980 Test Data

This current Task Group was able to review and re-analyze the original test data. All test panels were 24 feet in height and 4 feet in width reinforced with a single layer of 4 # 4 reinforcement bars. Analyses include adjusting the load based on the air bag contact area, the measured panel thickness and location of flexural reinforcement. Moment is calculated based on the following equation:

$$M_{(test)} = wl_c^2 \text{ x } 1.5 + P_1e + (P_1 + P_2)\,\Delta$$

Where

$M_{(test)}$ = equivalent moment based on test, in-kip
l_c = panel height, feet
w = applied lateral force on panel, kip
P_1 = applied axial load, kip
P_2 = panel weight at mid height, kip
e = eccentricity of applied axial load, inch
Δ = deflection at mid-height, inch

Results are shown in Figures 1.1 to 1.12. The upper curve shows load-deflection of the test panel, while the lower curve shows the moment-deflection relationship. On these plots, a ϕ –factor equal to one (1) was used. The ordinates for $^2/_3\ M_{cr}$ (cracked moment) and M_n (nominal moment strength) are shown. Lateral deflection increases rapidly when the moment exceeds $^2/_3\ M_{cr}$. A straight line joining $^2/_3\ M_{cr}$ (at $5\sqrt{f'_c}$) and M_n represents the permissible provision under UBC. The calculated moment-deflection for each test panel correlates closely with the empirical test data. The deflection limit $l_c/150$ is also shown on the plots.

An interaction envelop may be drawn for a range of axial load. The P-M values are calculated for a range of tensile strain up to 0.0020 based on the measured depths to reinforcement bars in each panel. Plots for the axial loads versus moment are shown in Figures 2.1 to 2.12. Nominal moment strength at an average load factor 1.5 times the axial load is shown for reference only. Except for wall panels 22 and 27, the calculated nominal moment strength is within the P-M envelop. These plots further validate the UBC design procedure.

An overlay of calculated moment-deflection based on ACI design procedure was studied. The plots for test panels 22 and 25 are shown in Figure 3.1; and for test panels 19 and 28 are shown in Figure 3.2. Below M_{cr} (at $7.5\sqrt{f'c}$), a straight line is drawn from zero to Δ_{cr} for moment within the uncracked segment. The ordinate for M_u and Δ_u are calculated based on ACI equations (14-5) and (14-6) for a range of lateral forces up to 50 lbs. per square foot and load combination based on ACI Appendix Equation (C-2.) In order to simulate an idealized bilinear relationship, a horizontal line is drawn from Δ_{cr} to intersect with the calculated value of Δ_u. It is important to note that the test results did not support the ACI $7.5\sqrt{f'_c}$ for modulus of rupture in any of the test panels. Also, the ACI procedure does not appear to correlate with the 1980 test results.

8. Background of ACI Provisions on Alternate Design of Slender Wall

Prior to the ACI 318-99, wall panels subject to combined axial and bending designed under ACI requirements must resort to second-order analysis in order to account for slenderness effects and lateral instability in accordance with Section 10.10. ACI Committee 318-D with input from Committee 551 introduced code change CD-121 in 1998. This code change was made in an effort to eliminate differences between ACI and UBC and in time for the adoption in the IBC 2000. For computing service load deflection the ACI procedure employs effective moment of inertia and a magnified moment for the combined moment due to lateral force and eccentric vertical load, also know as the P-Δ effect. Because the effective moment of inertia and magnified moment are dependent upon each other, some iteration is necessary.

The ACI procedure includes an additional restriction for walls based on the alternate design to be simply supported with constant cross section over height of panel, and revision of the axial stress limits from service load stress ≤ 0.04 f ´c to factor load stress ≤ 0.06 f ´c. The latter is the same as applying a load factor of 1.5 to service load. Within the normal range of load combinations for walls controlled by flexural tension as currently required by ACI 318-05, the axial load stress will never approach this stress limit.

In developing the equation for the bending stiffness $\Delta_{max} = M_{max}/ K_b$ where $K_b= 9.6\ E_cI_e/l_c^2$, Committee 551 drew on the similarity of the Euler critical buckling load of $P_{cr} = \pi^2\ E_cI_e/l_c^2 = 9.87\ E_cI_e/l_c^2$. ACI adopted the same equation as UBC for calculation of I_{cr} based on a rectangular stress block. However, the Branson equation for I_e is used for the calculation of service load deflection.

As an alternative to the second order analysis procedure, ACI Commentary R10.10 and R10.11 explain that the provisions under sections 10.11 and 10.12 present an approximate design method to account for the slenderness effect of slender columns based on a moment magnifier. One item lingers on is the 0.75 stiffness reduction factor in the denominators in ACI Equations (14 -5) and (14 -6) and its appropriateness for slender wall panels. The key question appears the lack of correlation to empirical data. In order to satisfy an idealized load deflection curve, an equation to express the portion of curve between Δ_{cr} and Δ_u under the ACI procedure would be prudent.

9. Analytical Comparison

Upon reviewing example A from draft document of ACI Committee 551, [Tilt-up Design Guide Examples – Draft No. 4], this Task Group formulated wall panels of similar geometry for comparative analyses using the UBC and ACI design procedures. Wall thicknesses of 6.25 and 7.25 inches were used for 29.5 feet high panels; and thicknesses of 5.75 and 6.25 were used for 24 feet high panels. Basic axial loads of 480 lbs. per foot dead load plus 500 lbs. per foot live load were applied with 3 inches eccentricity. The axial loads were increased to two times and three times the basic loads in order to explore high axial load parameters. Lateral forces of 20, 25, 30 and 35 lbs. per square foot were used in combination with each axial load condition. The reinforcement ratios generally varied between 0.0126 and 0.0162 which were within the maximum steel ratio of 0.0171 at $0.6\rho_b$. Loading increment for both lateral force and axial loadings were used in order to obtain the data points for moment-deflection curves.

In order to compare the two procedures similar load factors were used from the UBC and from ACI 318 Appendix C. Results of the comparative study are given on Tables 4.1 to 4.4 for single curtain reinforcement. Graphic representation of moment-deflection based on the range of calculations for seven wall panels are shown in Figures 4.1 to 4.4. A summary of the analytical comparative design is given on Table 5.

ACI 318 defines M_{cr} at a modulus of rupture of $7.5\sqrt{f'_c}$. For the purpose of this Report, the cracked moment as used in UBC procedure at $5\sqrt{f'_c}$ will be labeled as 2/3 M_{cr}. The load deflection characteristic for the UBC procedure is represented by a straight line from zero to 2/3 M_{cr} for the uncracked stage and another straight line from 2/3 M_{cr} to M_n for the cracked stage. For any given wall panel with reinforcement approaching the upper limit and with increase lateral force and/ or axial load, ACI procedure significantly under-estimates the service load deflection in comparison to the UBC procedure. In fact, in most cases, the service load deflection is less than Δ_{cr}.

For two curtains of reinforcement, the Task Group used a 29.5 feet high by 20 feet wide wall panel with a 10 feet wide by 15 feet off center opening. Effective pier width of 4 feet and 6 feet, with thickness of 6.25

inches and 7.25 inches and steel ratios ranging from 0.001 to 0.017, were used in the analytical comparison. This is similar to Example B under work in progress by Committee 551. Lateral forces of 17, 25 and 35 lbs. per square foot were used to provide a range of moments and deflections in this study. Results of the comparative study for double curtain reinforcement are given on Table 4.5. Contrary to the single curtain described above, the results for service load deflections are much closer between the UBC and ACI procedures. Nonetheless, the ACI procedure predicts service load deflection lower than UBC procedure.

A summary of all comparative design examples is given on Table 5. The table includes footnotes for $M_u/\phi < M_n$; $M_u/\phi > M_n$; $\Delta_s \le \Delta_{cr}$; $\Delta_{cr} \le \Delta_s \le l_c/150$ and $\Delta_s \ge l_c/150$. Of 28 comparative examples for single curtain reinforcement and 12 comparative examples for double curtain reinforcement, the ACI procedure shows 20 cases $\Delta_s \le \Delta_{cr}$ and only one case $\Delta_s \ge l_c/150$. Similarly, the UBC procedure shows 2 cases $\Delta_s \le \Delta_{cr}$ and 19 cases $\Delta_s \ge l_c/150$. The significance of this comparative study demonstrates that ACI procedure tends to under-predict serviceability.

10. Findings

Based on an array of analytical studies and comparison of code provisions, our findings are as follows:

1. Verification of the Green Book (1980 Slender Wall Task Committee Report) data using adjusted lateral force and deflection data was performed. The analytical result follows closely with the bilinear load deflection characteristic. Lateral deflection increases rapidly when the moment exceeds 2/3 of M_{cr} (as defined by ACI).

2. ACI needs to improve its methodology so that computed results would follow a bilinear load deflection characteristic observed during full scale testing. There are concerns from other sources researching appropriateness of I_e in the traditional Branson Equation for wall panel out-of-plane deflection calculation.

3. Both design procedures are applicable to walls controlled by flexural tension. The ACI code now defines tension control based on tensile strain, $\varepsilon_t \ge 0.0050$.

4. For wall panels with low percentage of reinforcement, panel design based on ACI procedure is normally controlled by strength with deflection less than Δ_{cr}. UBC procedure is more sensitive to out-of-plane deflection with increase in lateral force and/ or axial load.

5. For wall panels with reinforcement ratio approaching the upper limit, panel design based on ACI procedure significantly under-estimates service load deflection in comparison to the UBC procedure and empirical results with increase lateral force and/ or axial load.

6. Where wall panel design is based on ACI procedure meeting strength and deflection limits, the corresponding wall panel calculation based on UBC procedure may exceed $l_c/150$ deflection limit.

7. Designs using two curtains of reinforcement show closer correlation between the two procedures.

8. Control of maximum steel ratio based on tensile strain under ACI 318-05 procedure is appropriate.

9. The requirement for minimum reinforcement of $M_n \ge M_{cr} / \phi$ is appropriate.

10. ϕ – factor of 0.90 based on ACI 318-05 Section R9.3.2.2 is appropriate.

11. Load factors and load combinations should be based on generally accepted load factors from model code (ASCE 7-05.)

12. Change of $P_a/A_g < 0.04f'_c$ to $P_u/A_g < 0.06f'_c$ for maximum stress at mid-height does not impact design by either procedure since the normal range of axial load for slender wall does not approach the limit. In order to comply with tension controlled requirement, the normal range of axial loading will be substantially below the prescribed maximum stress level.

13. Approach for cracked moment of inertia (I_{cr}) is the same for both Codes.

14. Serviceability requirement of $\Delta_s \leq l_c/150$ (or 0.007 l_c) based on service load is the same for both Codes. The limit was apparently set by Building Officials. However, it does not appear the ACI procedure would exceed Δ_{cr} within the range of most loading and load combinations.

15. Seismic force prescribed on the strength basis will need to be divided by a load factor of 1.4 for equivalent service load calculations. Further code development for strength design force level should review the appropriate load factor for conversion to service load in serviceability check in addition to the appropriate inclusion of dead, floor and roof live loads.

16. In the ACI equations (14-5) and (14-6) for Δ_u and M_u, the 0.75 stiffness reduction factor tends to increase the required moment strength rapidly. The alternate slender wall design procedure includes the P-Δ effect; and it would appear further softening of the cracked moment of inertia is unnecessary.

17. In order to be consistent with ACI traditional modulus of rupture of $f_r = 7.5 \sqrt{f'_c}$ and $M_{cr} = f_r S$, the corresponding cracked moment in 97UBC should be limited to 2/3 $M_{cr.}$ For service load deflection, the UBC procedure should be revised to: $\Delta_s = 0.67\Delta_{cr} + (M_s - 0.67M_{cr}) (\Delta_n - 0.67\Delta_{cr}) \div (M_n - 0.67M_{cr})$

18. The following statements in ACI commentary R14.8 are found questionable and should be corrected:

    "Section 14.8 is based on the corresponding requirements in the Uniform Building Code (UBC) and experimental research" and

    "The procedure, as prescribed in the UBC, has been converted from working stress to factored load design."

11. Other Design Considerations

All engineering design includes considerable judgment in applying practical research and past experience. Building code provisions may not fully cover all design parameters. Some of those other design considerations that were discussed within this Task Group include the following:

- Effective Area of Steel –Traditionally, $A_{se} = A_s + P/f_y$. A unique problem in a double curtain wall is that the axial load modeled at the center of the wall is being used to increase the steel near the face of the wall, where its benefit is much greater than in reality. This tends to increase I_{cr} and thus help to reduce the calculated deflection and increase the nominal moment capacity. Further clarification is needed for double curtain wall reinforcement.

- Service level deflection – the model codes in other countries and practice in some parts of the United States are using deflection limitation of $l_c/100$ as was recommended in the "Green book." While the original research showed no lateral instability for thin wall panels under combined light axial load and large lateral forces, the enforcement agencies felt more comfortable with the more restrictive deflection limit of $l_c/150$ particularly in consideration of other brittle building materials. This Report does not address the validity or usefulness of service level deflection limit, except as an index in comparison of the design procedures. Parallel research is needed in service load deflection in order to justify different deflection limits.

- Location of rebar and tolerance – location of reinforcement sometimes is predicated on availability of commercial rebar chairs and the correct location of bars in orthogonal directions. ACI-318 permits 3/8 inch tolerance for $d \leq 8$ inches. Engineers should review if such tolerance would satisfy the design on thin panels. Construction observation should include the verification of reinforcement bar location.

- End condition versus simply support – ACI 318 puts emphasis under design limitations the importance of design based on simply supported wall panels regardless of end fixity. While some fixity may be realized either due to continuity of wall panels at the floor lines or fixity at a dock height wall panels, the inclusion of such end fixity to reduce service load deflection may be an academic exercise and should be based on further research.

- Effectiveness of Branson equation – there has been questions on the suitability of using the Branson equation, ACI Equation (9-8) for the computation of effective moment of inertia. One academia from Canada pointed out that the equation may not work well for concrete members with an Ig/Icr ratio greater than about 4. Using Branson's method (I_e) to calculate service load deflections in slender walls, particularly with single layer of reinforcement may significantly underestimate service level deflection. An improve methodology to replace the Branson's equation for slender wall deflection calculations is currently understudy and is not available at this time.

- Roof live load – under service load combination, model codes allows exclusion of roof live less than 30 lbs. per sq. ft. when combination with wind or seismic forces. ACI 318 does not address whether such exclusion is permitted under load combination.

12. Recommendations to SEAOSC Board

- To calculate service load deflection, use E/1.4 for earthquake forces.

- Recommend to appropriate enforcement agencies that adoption of the 2003 IBC provisions on alternate design of slender wall procedure should incorporate proposed changes to ACI 318-05 Section 14.8.4 listed under Section 13 below.

- Send a letter to ACI-318 addressing the concerns in using the ACI alternate design of slender wall procedure for service load deflection and requesting ACI 318 to correct the statements in Commentary R14.8.

13. Proposed Changes to ACI

The following are proposed revision to ACI 318-05

14.8.4 – Delete equations (14-8) and (14-9) and the last paragraph in total, and replace with the following after the first paragraph:

" $\mathbf{\Delta_s = 0.67\Delta_{cr} + (M_s - 0.67M_{cr})(\Delta_n - 0.67\Delta_{cr}) \div (M_n - 0.67M_{cr})}$; for $M_s > 0.67M_{cr}$ (14-8)

$\mathbf{\Delta_s = 5\,M_s\,l_c^2 \div (48E_c\,I_g)}$; for $M_s \leq 0.67M_{cr}$ (14-9)

Where

$\mathbf{\Delta_{cr} = 5(M_{cr})\,l_c^2 \div (48\,E_c I_g)}$

$\mathbf{\Delta_n = 5(M_n)\,l_c^2 \div (48\,E_c I_{cr})}$"

14. Acknowledgement

In preparing this report, the 2005 Slender Wall Task Group attempted to do a thorough search of available reference sources. Each Task Group member has performed and contributed to this analytical research and the summary report. The Task Group wishes to acknowledge several individuals who assisted in furnishing material for our analytical research efforts. Luis Garcia who is current chairperson of ACI-318 D was gracious to forward the original ACI code change CD121 and the analysis by Committee 551. Gerry Weiler who was chairperson of ACI 551 when ACI 318 was converting the 97 UBC slender wall section to ACI format furnished material showing the comparison of the earlier analysis as well as portions of the current Tilt-up Design Guide. Professor Peter Bischoff of the University of New Brunswick, Canada, shared some of his recent findings on the ACI deflection equations. Other individuals including Messer Neil Hawkins, Robert Mast, Basile Rabbat and Charles Salmon have also kept this Task Group informed.

The Task Group is indebted to the vigorous efforts of members of the 1980 Joint Task Committee and those volunteer workers who devoted two years of their professional lives on the test program and report assignments. We hope this Report serves as a closure to the earlier research efforts that continue to serve the design profession and construction industry in future years. To the memories of those Joint Task Committee members who have since deceased including Ralph Mclean, William Simpson and Ulrich Foth, we dedicate this summary report.

15. References

1. ACI Committee 551, “Tilt-Up Construction Guide- ACI 551.1R-05,” American Concrete Institute, Farmington Hills, MI, 2005.

2. ACI Committee 551, “Tilt-Up Design Guide Examples – Draft No. 4,” American Concrete Institute, Farmington Hills, MI, April, 2005.

3. ACI Committee 318-02, “Building Code Requirements for Structural concrete and Commentary,” American Concrete Institute, Farmington, MI, 2002.

4. ACI Committee 318-05, “Building Code Requirements for Structural concrete and Commentary,” American Concrete Institute, Farmington, MI, 2005.

5. SCCACI-SEAOSC Task Committee on Slender Walls, “Test Report on Slender Walls,” Los Angeles, CA, 1982

6. *Uniform Building Code*, Volume 2, “Structural Engineering Provisions,” International Conference of Building Officials, Whittier, CA 1997.

16. Appendix –

Table 1 – Comparison of Slender Wall Design Procedures UBC vs. ACI

Section Reference	Topic	1997 UBC	ACI 318-02
1914.8 ACI 14.8	Title	Alternate Design Slender Walls	Alternative design of slender walls
1914.8.1 1910.10 ACI 14.8.1 ACI 10.10	Applicable in lieu of consideration for slenderness effects as a compression member	Walls controlled by flexural tension	Walls controlled by flexural tension
1914.8.2 ACI 14.8.1 ACI 14.8.2	Limitations		Design as simply supported axial loaded member subjected to uniformed lateral force; Constant cross section over height of panel
1914.8.2 ACI14.8.2.6	Maximum axial stress at mid-height	Vertical service load stress ≤ 0.04 f ‘c	Vertical stress $P_u / A_g < 0.06$ f $‘_c$
1914.8.2 ACI14.8.2.3	Maximum Reinforcement ratio	$\rho \leq 0.06\, \rho_b$	$\rho \leq 0.06\, \rho_b$ ACI 318-02 $\varepsilon_t \geq 0.0050$ ACI 318-05
1914.8.2 ACI14.8.2.4	Minimum reinforcement	$\phi M_n \geq M_{cr}$	$\phi M_n \geq M_{cr}$
1914.8.2 ACI 14.8.2.5	Concentrated load	Bearing width plus width at slope of 2 V to 1 H	Bearing width plus width on each side at slope of 2 V to 1 H; Not to exceed spacing of conc. Load
1909.2.2 1612.2.1 ACI 9.2.1 *Or* ACI Appendix C.2	Basic load combinations	1.4D + 1.7L 0.75 (1.4D + 1.7L + 1.7W) 0.9D + 1.3W 1.2D + 1.0E + (f_1L + f_2S) 0.9D $\pm$ 1.0E	1.2D + 1.6(L_r or S) + (0.8W) 1.2D + 1.6W + 1.0L + 0.5(L_r or S) 1.2D + 1.0E + 1.0L + 0.2S 0.9D $\pm$ (1.0E or 1.6W) Note: without Directional Effect use 1.3W in place of 1.6W
1909.3.2.2 ACI R9.3.2.2	ϕ - factor	$0.90 - 2.0P_u / f'_c A_g \geq 0.70$ OR $0.70 + (1-P_u/0.10f'_c A_g)(0.90 - 0.70)$	0.90 when $\varepsilon_t \geq 0.005$ $0.65 + (\varepsilon_t - 0.002)(250/3)$ when $\varepsilon_t < 0.005$ ACI318-05
1014.8.3 ACI 14.8.3	Design moment strength	$M_u \leq \phi\, M_n$	$\phi\, M_n \geq M_u$
1914.8.3 ACI 14.8.3	Required factored moment	$M_u = w_u l_c^2 x1.5 + P_{u1}\, e/2 + (P_{u1} + P_{u2})\, \Delta_n$	$M_u = M_{ua} + P_u\, \Delta_u$ *OR* $M_u = M_{ua} \div [1 - 5P_u l_c^2 \div (0.75)48E_c I_{cr}]$
1914.0 ACI 9.5.2.3	Cracking moment for normal weight concrete	$M_{cr} = 5\sqrt{f'_c}\; I_g / y_t$	$M_{cr} = 7.5\sqrt{f'_c}\; I_g / y_t$
1914.8.4 14.0 ACI 14.8.4	Service Load moment	$M_s = wl_c^2 x1.5 + P_1\, e/2 + (P_1 + P_2)\, \Delta_s$	$M_{sa} = wl_c^2 x1.5 + P_1\, e/2$ $M = M_{sa} + (P_1 + P_2)\, \Delta_s$ $= M_{sa} \div [1 - 5P_s l_c^2 / 48\, E_c I_e]$
1914.8.4 14.8.3	Effective tension reinforcement	$A_{se} = (P_u + A_s f_y) \div f_y$	$A_{se} = (P_u + A_s f_y) \div f_y$
1914.8.4 ACI 14.8.3	Moment of inertia of cracked transformed section	$I_{cr} = n\, A_{se}\, (d - c)^2 + bc^3 / 3$	$I_{cr} = n\, A_{se}\, (d - c)^2 + l_w c^3 / 3$ $I_{cr} = (E_s/E_c)(A_s + P_u/f_y)\, (d-c)^2 + l_w c^3 / 3$ ACI 318-05
14.8.4 ACI 9.5.2.3	Effective moment of inertia	NA	$I_e = (M_{cr}/M)^3\, I_g + [1 - (M_{cr}/M)^3]\, I_{cr}$
1914.8.4 ACI 14.8.3	Deflection at M_{cr}	$\Delta_{cr} = 5\, M_{cr}\, l_c^2 \div 48E_c I_g$	NA
ACI 14.8.3	Deflection due to factored load	NA	$\Delta_u = 5\, M_u\, l_c^2 \div [(0.75)48\, E_c I_{cr}]$
1914.8.4	Max. potential deflection	$\Delta_n = 5\, M_n\, l_c^2 \div 48E_c I_{cr}$	NA
1914.8.4	Deflection due to cracking moment	$\Delta_{cr} = 5\, M_{cr}\, l_c^2 \div 48E_c I_g$	NA
1914.8.4 ACI 14.8.4	Deflection at Service Load	$\Delta_s = \Delta_{cr} + (M_s - M_{cr})(\Delta_n - \Delta_{cr}) \div (M_n - M_{cr})$	$\Delta_s = (5M)\, l_c^2 \div 48E_c I_e$
1914.8.4 ACI 14.8.4	Permissible service load deflection	$\Delta_s = l_c / 150$	$\Delta_s = l_c / 150$

Note: Editorial changes in ACI 318-05 are highlight.

Appendix

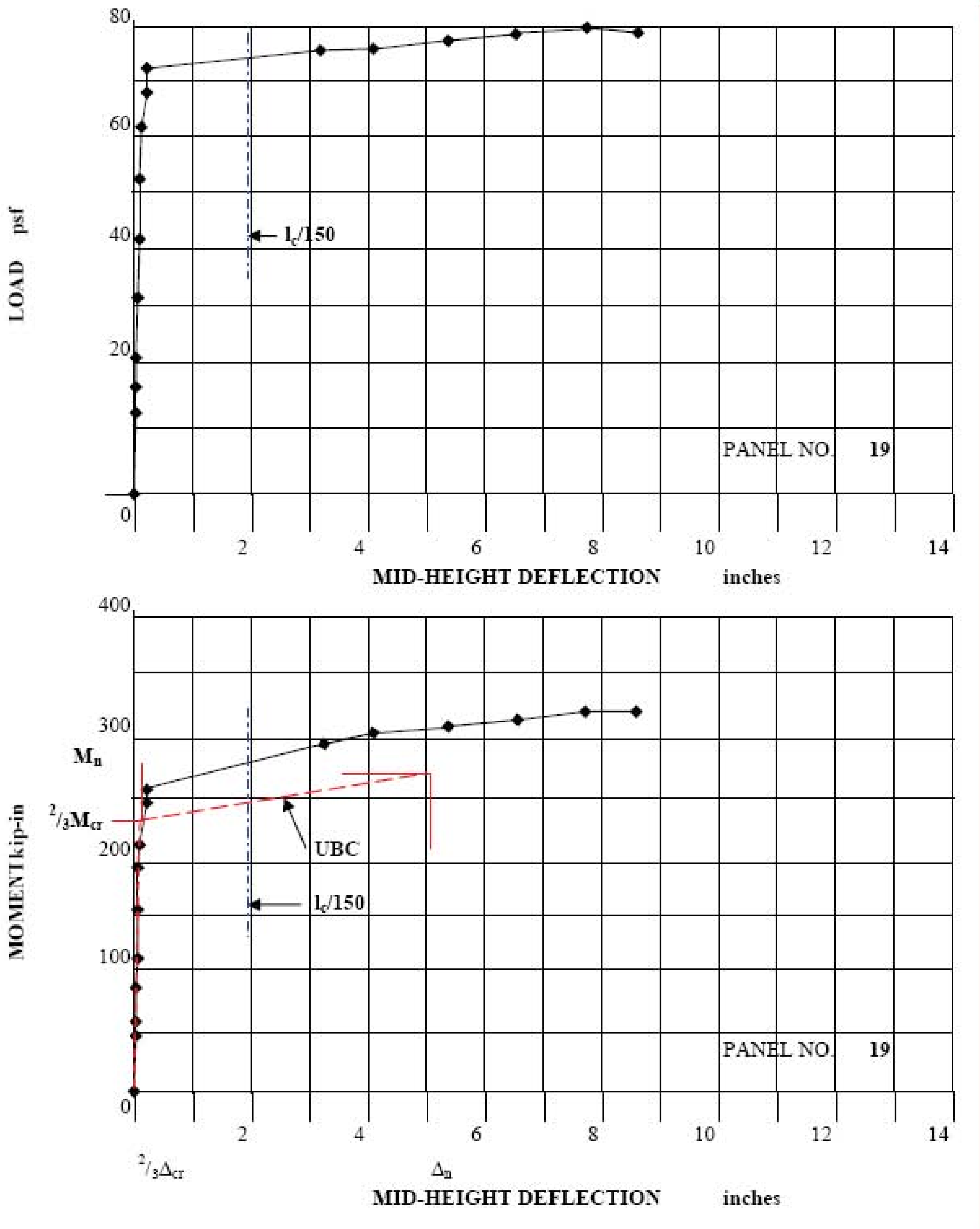


Figure 1.1 – Load-Deflection and Moment-Deflection Plots for Test Panel No. 19

Appendix

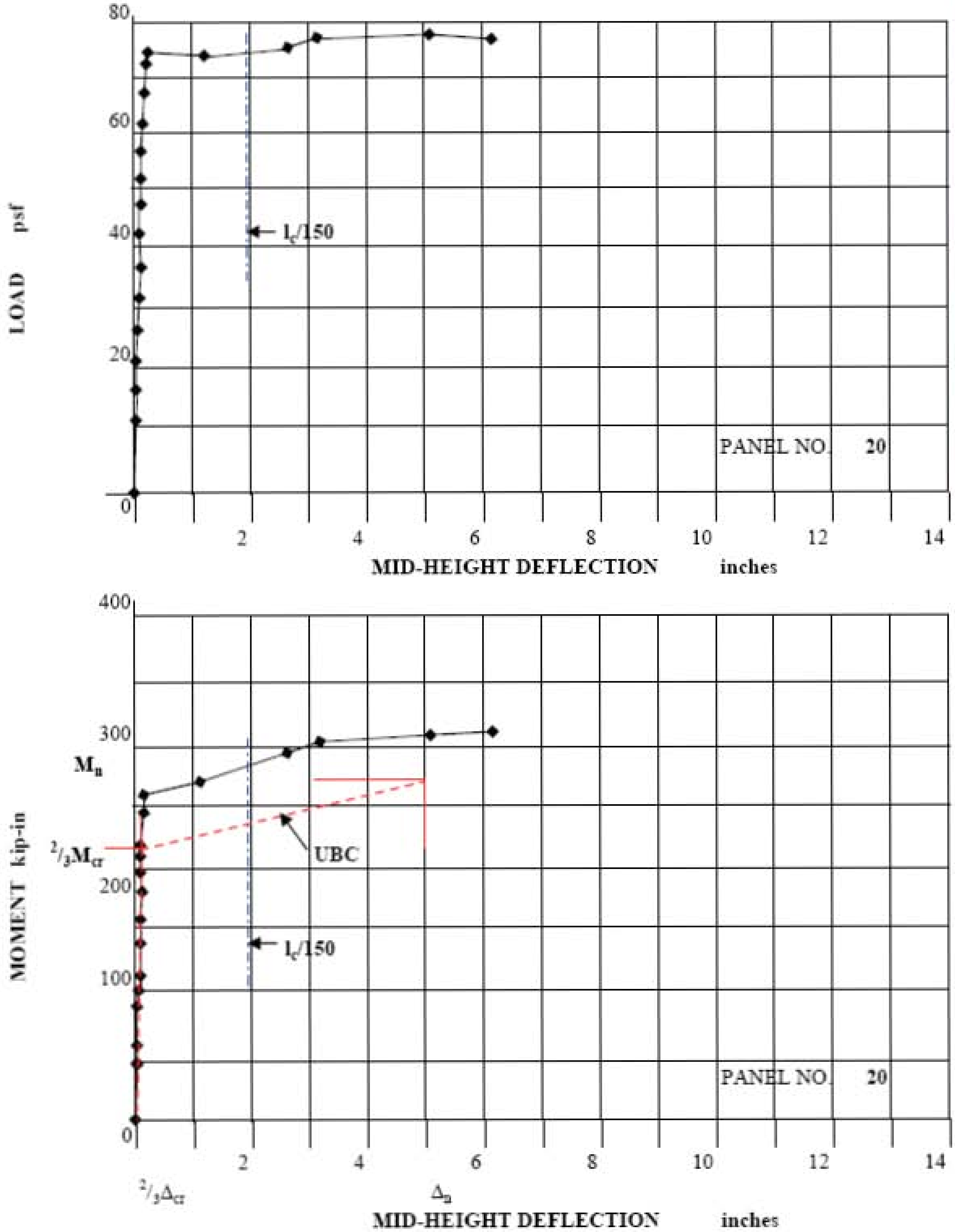
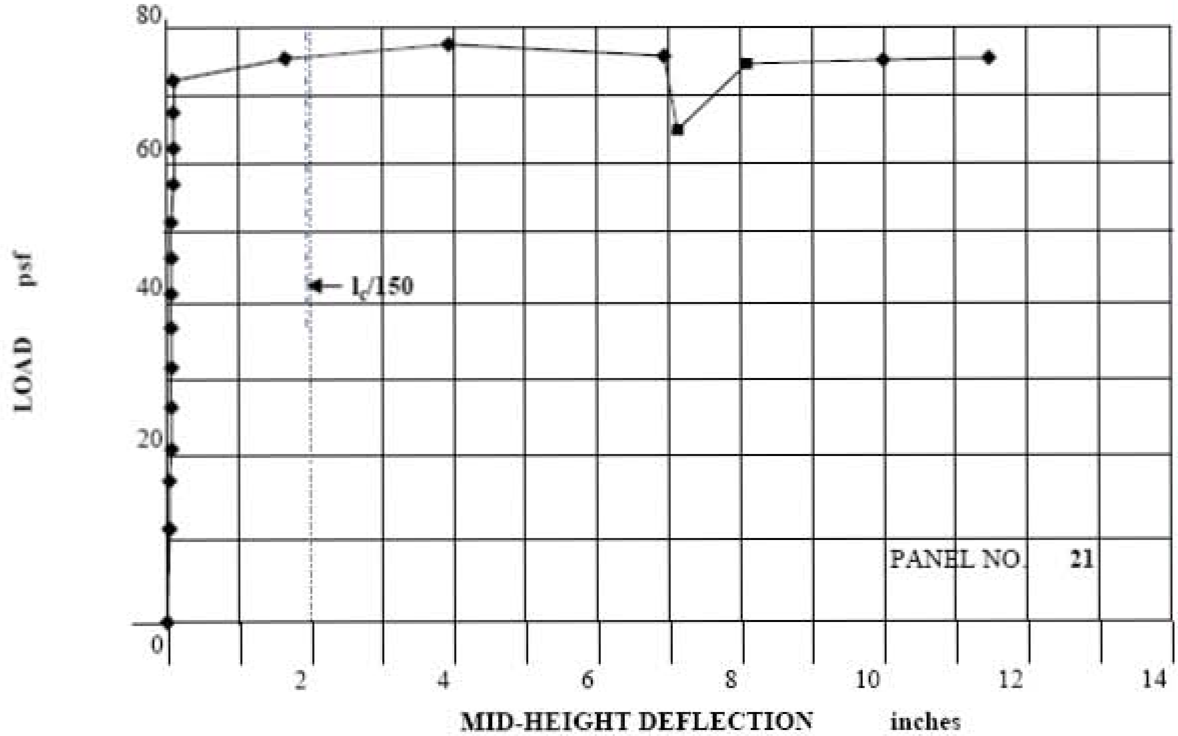


Figure 1.2 – Load-Deflection and Moment-Deflection Plots for Test Panel No. 20

Appendix



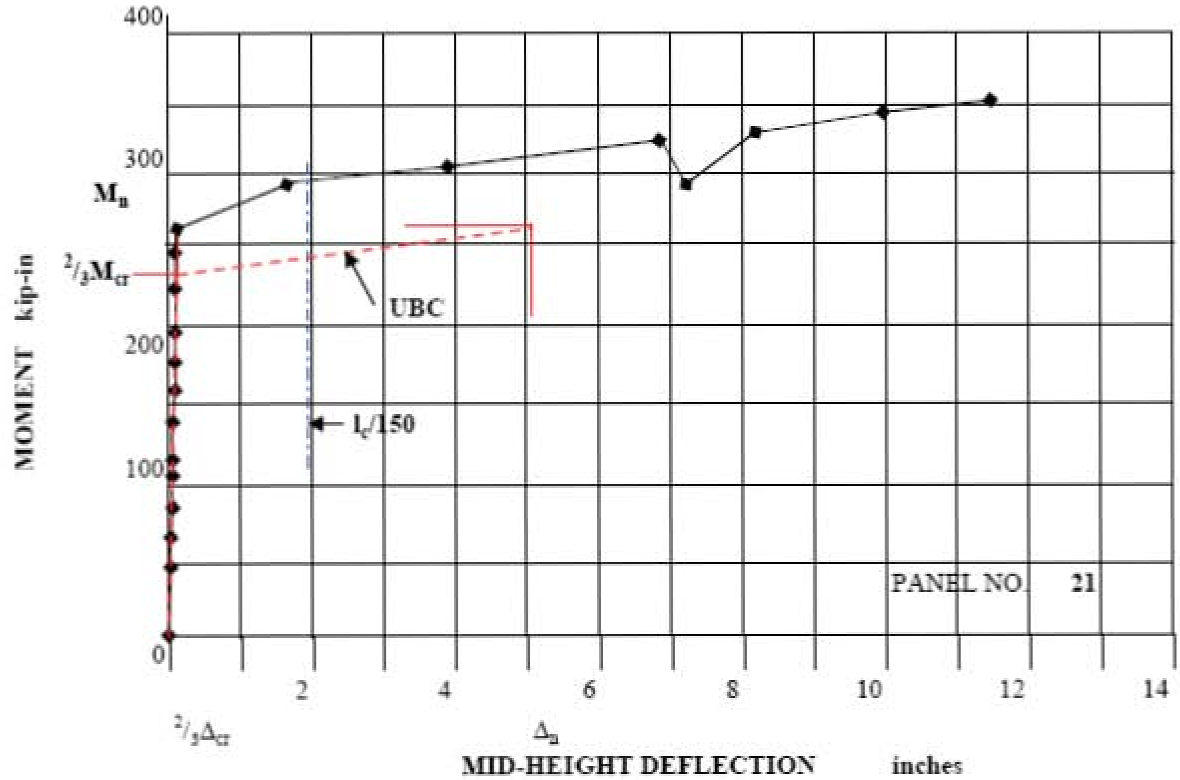


Figure 1.3 – Load-Deflection and Moment-Deflection Plots for Test Panel No. 21

Appendix

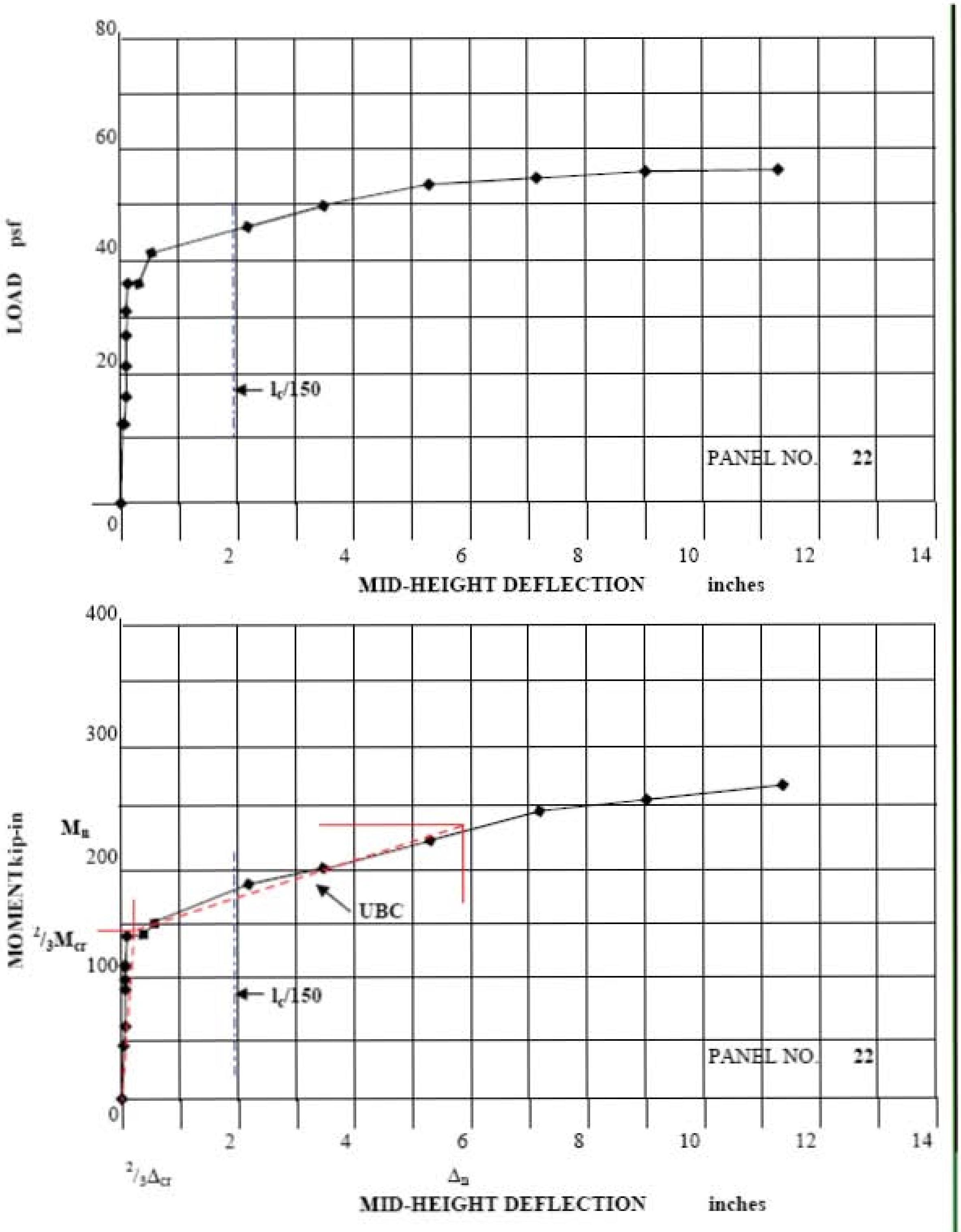


Figure 1.4 – Load-Deflection and Moment-Deflection Plots for Test Panel No. 22

Appendix

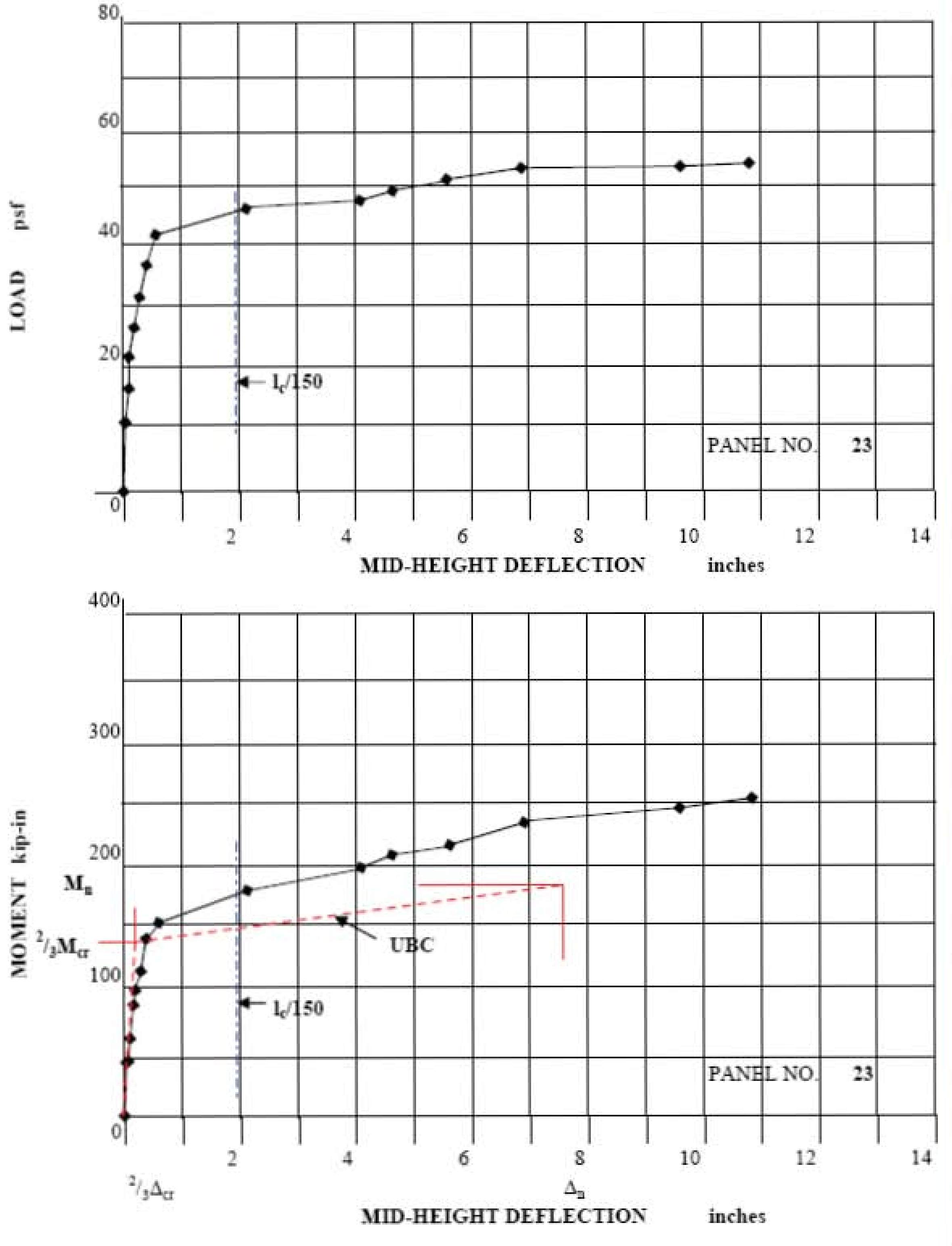


Figure 1.5 – Load-Deflection and Moment-Deflection Plots for Test Panel No. 23

Appendix

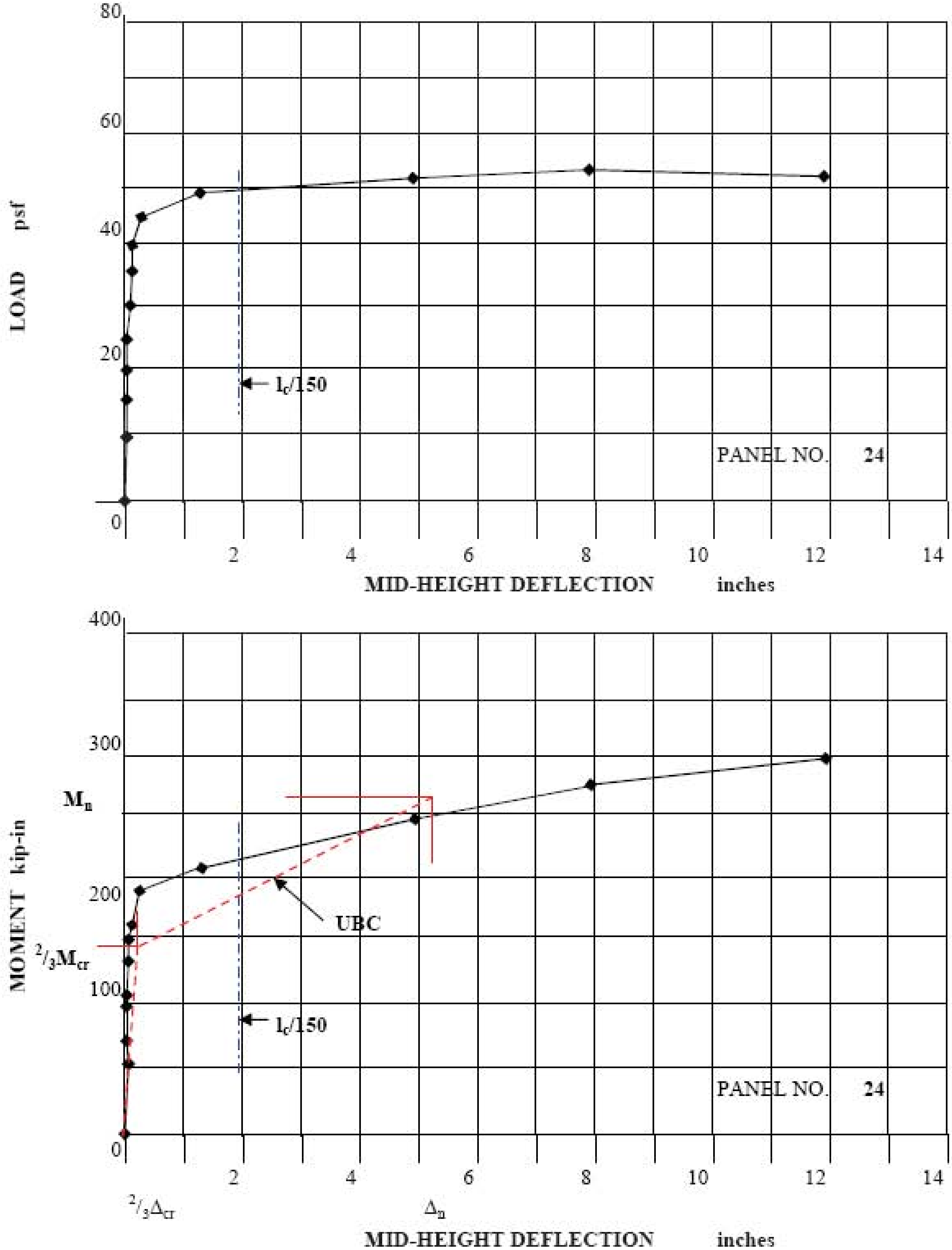


Figure 1.6 – Load-Deflection and Moment-Deflection Plots for Test Panel No. 24

Appendix

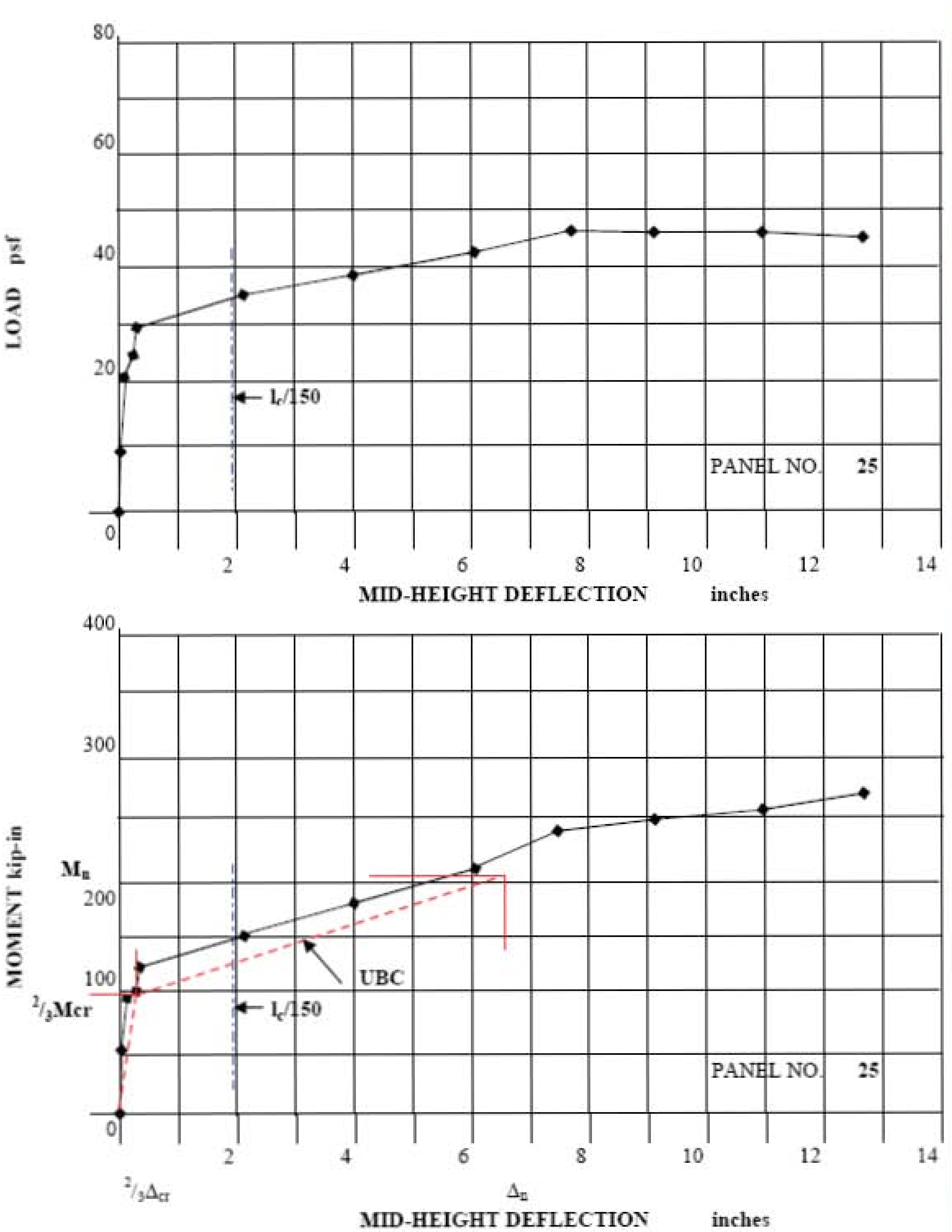
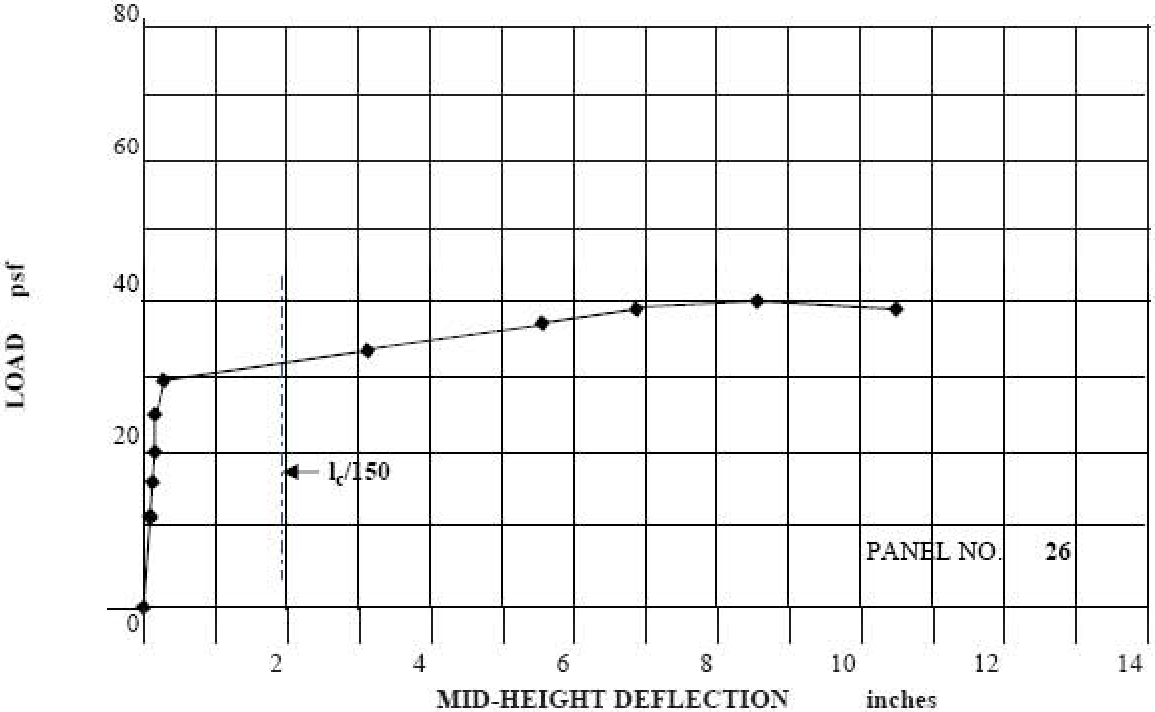


Figure 1.7 – Load-Deflection and Moment-Deflection Plots for Test Panel No. 25

Appendix



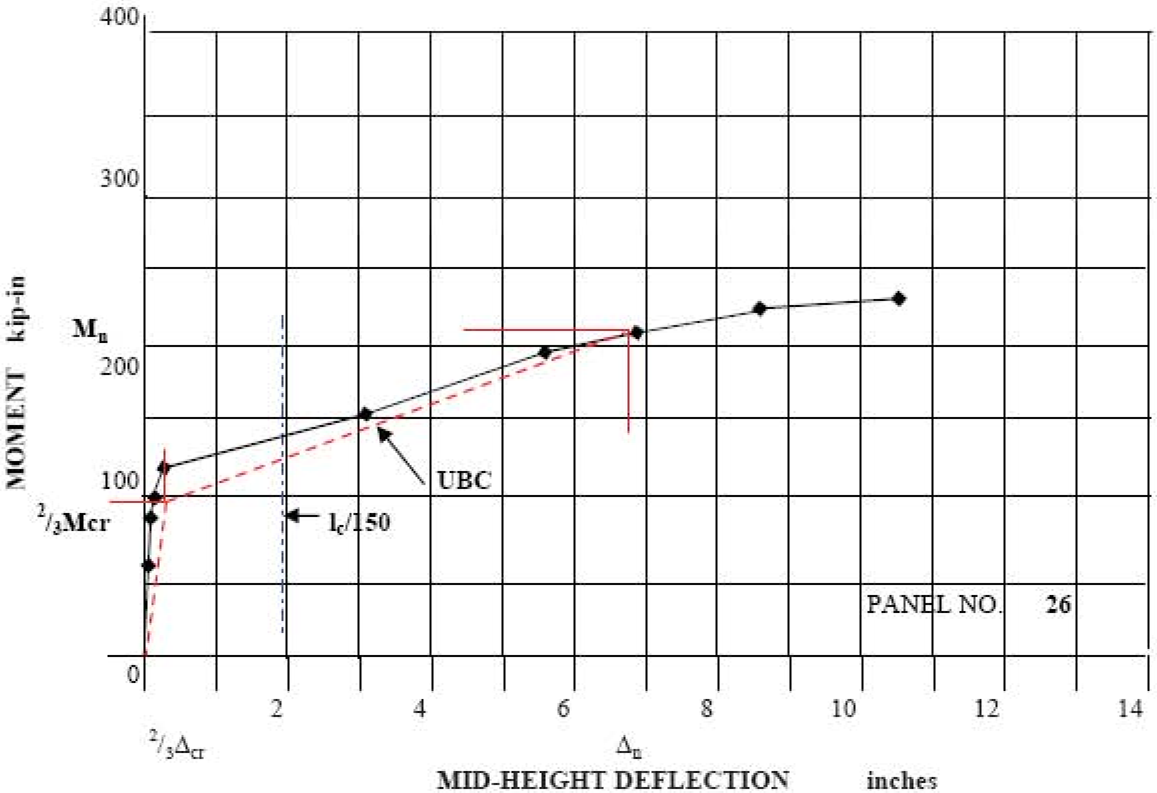


Figure 1.8 – Load-Deflection and Moment-Deflection Plots for Test Panel No. 26

Appendix

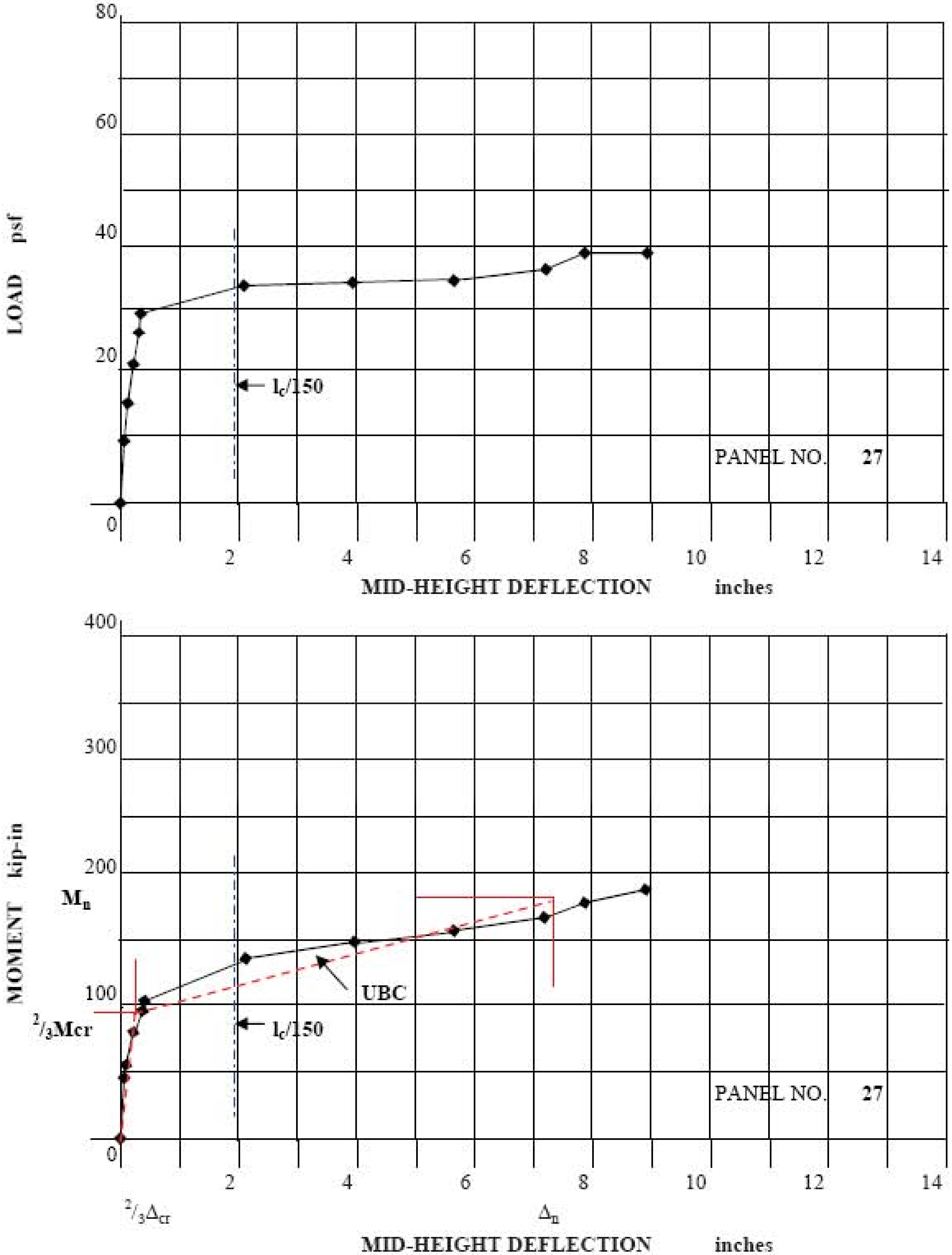


Figure 1.9 – Load-Deflection and Moment-Deflection Plots for Test Panel No. 27

Appendix

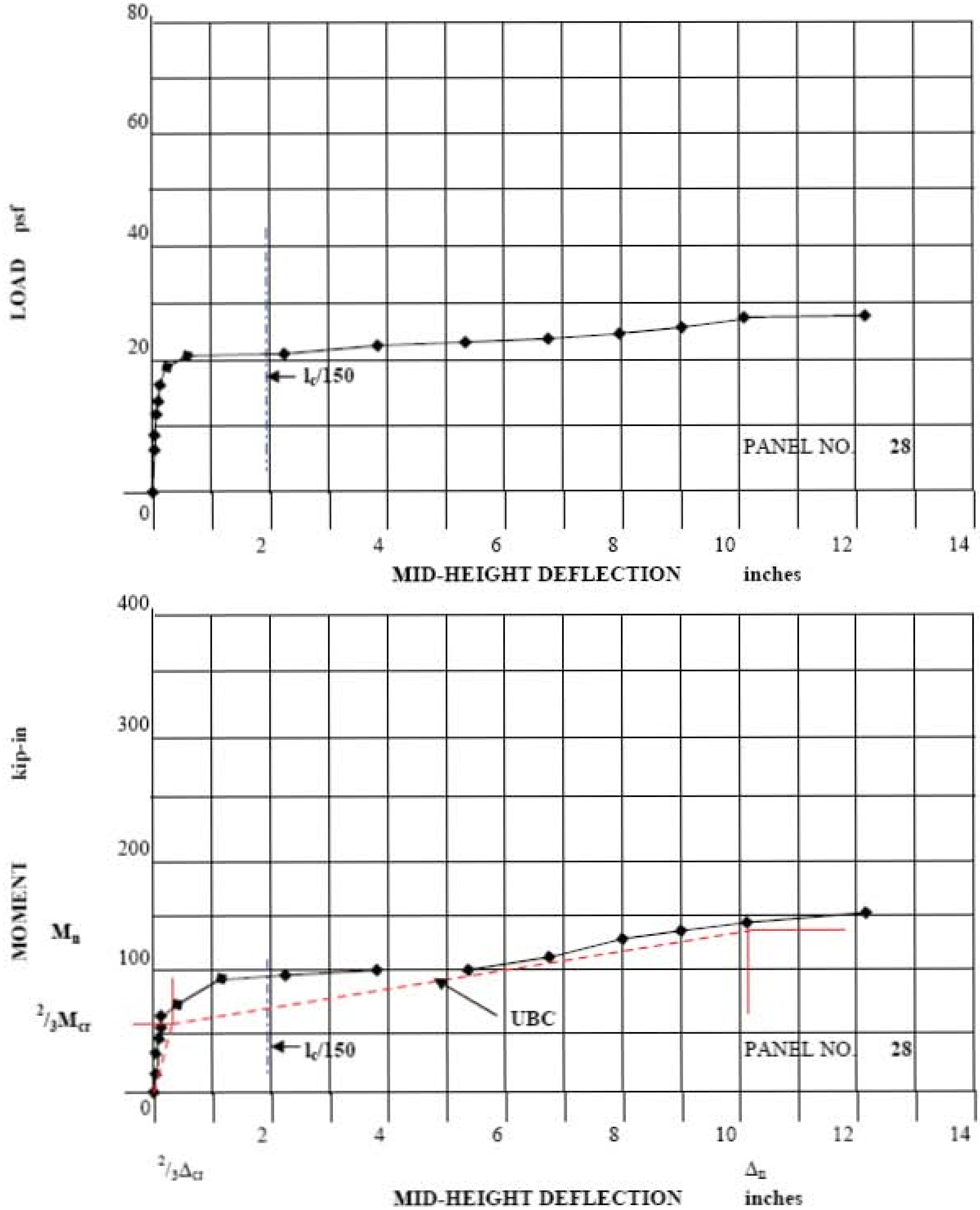


Figure 1.10 – Load-Deflection and Moment-Deflection Plots for Test Panel No. 28

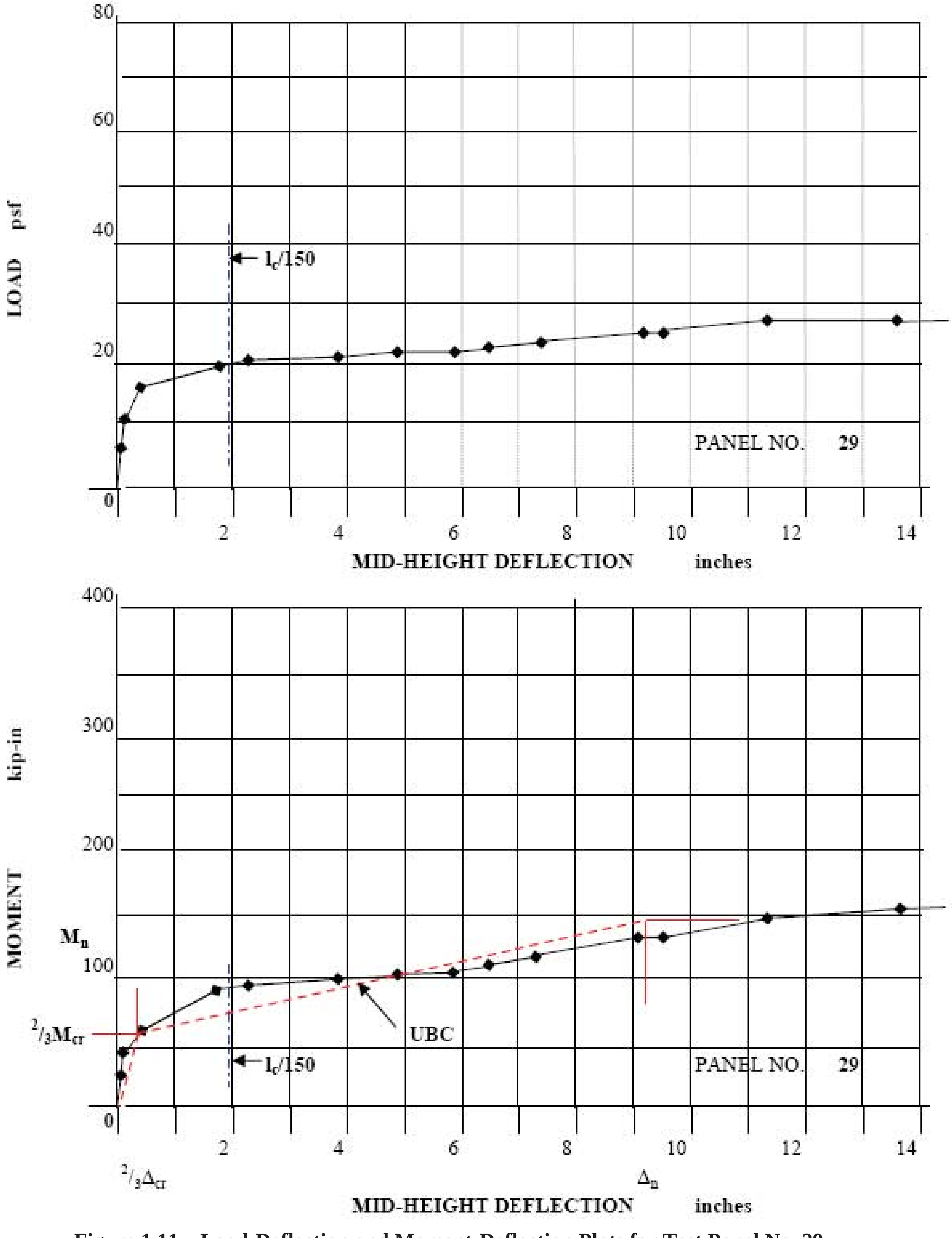


Figure 1.11 – Load-Deflection and Moment-Deflection Plots for Test Panel No. 29

Appendix

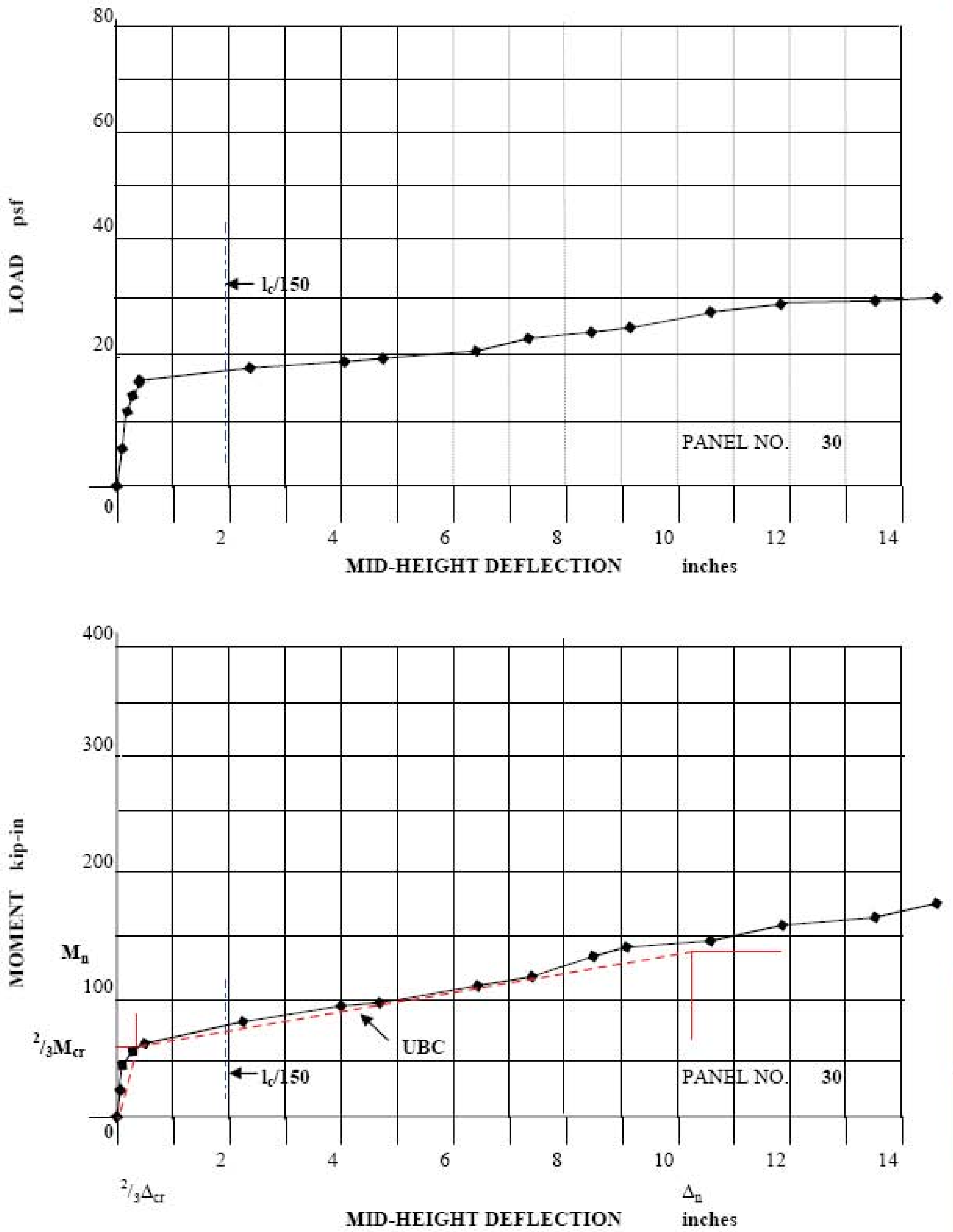
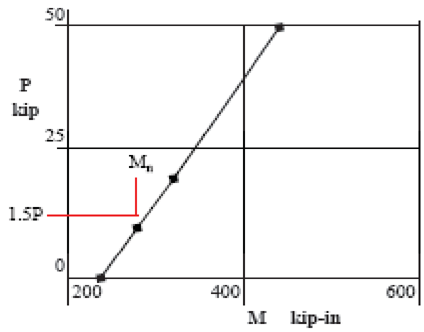


Figure 1.12– Load-Deflection and Moment-Deflection Plots for Test Panel No. 30

Appendix

SEAOSC -
Slender Wall Task Group
Develop interaction curve
Wall No. 19

f_y	=	67.5	ksi
f'_c	=	4.0	ksi
E_s	=	28,600	ksi
E_c	=	3540	ksi
A_s	=	0.196	sq in.
h	=	9.60	in
l_w	=	48.0	in
P	=	7.36	kip
P/ A_g	=	0.016	ksi
M_s(design)	=	276	kip-in



P/ f'_cA_g	P	M	M	s_t
0.0	0	238	19.9	0.0336
0.0005	1	242	20.2	0.0330
0.001	2	246	20.5	0.0324
0.0054	10	278	23.2	0.0278
0.0109	20	319	26.6	0.0236
0.0271	50	438	36.5	0.0159
0.05	92	594	49.5	0.0104
0.10	185	935	77.9	0.0052
0.103	190	952	79.3	**0.0050**
0.15	277	1204	100.4	0.0029
0.182	336	1350	112.5	**0.0020**
		kip-in	kip-ft	

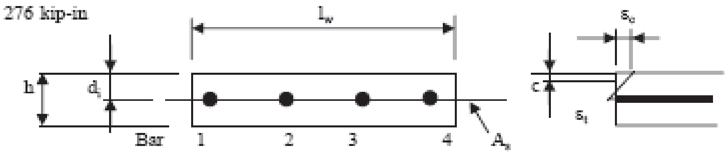
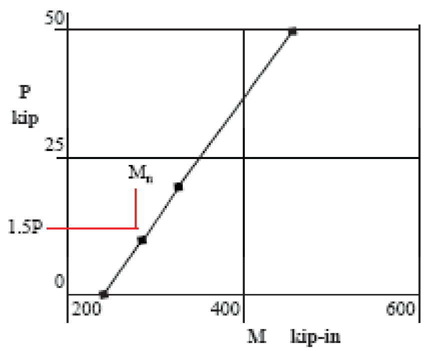


Figure 2.1 – Interaction Diagram for Test Panel No. 19

SEAOSC -
Slender Wall Task Group
Develop interaction curve
Wall No. 20

f_y	=	67.5	ksi
f'_c	=	4.0	ksi
E_s	=	28,600	ksi
E_c	=	3540	ksi
A_s	=	0.196	sq in.
h	=	9.40	in
l_w	=	48.0	in
P	=	7.36	kip
P/ A_g	=	0.016	ksi
M_s(design)	=	276	kip-in



P/ f'_cA_g	P	M	M	s_t
0.0	0	241	20.1	0.0340
0.0005	1	245	20.4	0.0334
0.001	2	249	20.7	0.0328
0.0055	10	284	23.7	0.0281
0.0111	20	327	27.3	0.0239
0.0276	50	452	37.7	0.0160
0.05	90	612	51.0	0.0107
0.10	181	933	77.7	0.0054
0.106	192	970	80.8	**0.0050**
0.15	271	1204	100.3	0.0030
0.186	336	1368	114.0	**0.0020**
		kip-in	kip-ft	

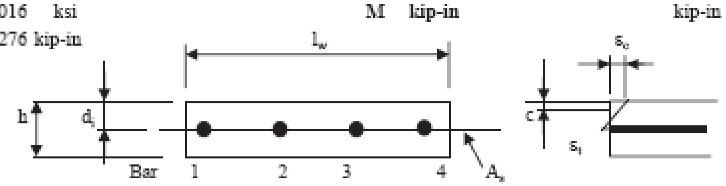
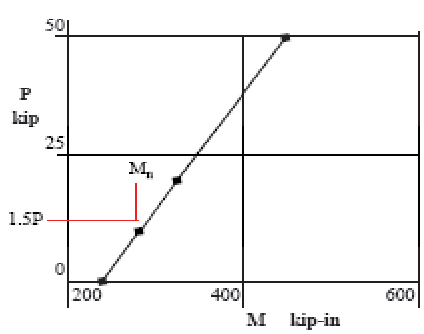


Figure 2.2 – Interaction Diagram for Test Panel No. 20

SEAOSC -
Slender Wall Task Group
Develop interaction curve
Wall No. 21

f_y	=	67.5	ksi
f'_c	=	4.0	ksi
E_s	=	28,600	ksi
E_c	=	3540	ksi
A_s	=	0.196	sq in.
h	=	9.50	in
l_w	=	48.0	in
P	=	5.97	kip
P/ A_g	=	0.013	ksi
M_s(design)	=	276	kip-in



P/ f'_cA_g	P	M	M	s_t
0.0	0	237	19.7	0.0334
0.0005	1	241	20.0	0.0328
0.001	2	244	20.4	0.0322
0.0055	10	279	23.3	0.0276
0.0109	20	321	26.8	0.0234
0.0274	50	444	37.0	0.0157
0.05	91	604	50.3	0.0104
0.10	183	921	76.7	0.0052
0.103	188	937	78.1	**0.0050**
0.15	274	1186	98.8	0.0029
0.180	329	1321	110.1	**0.0020**
		kip-in	kip-ft	

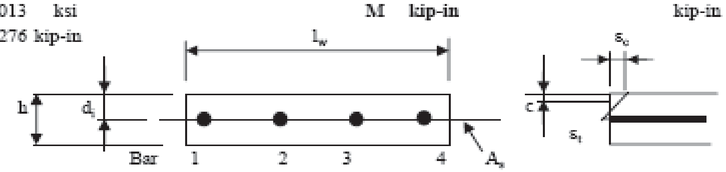
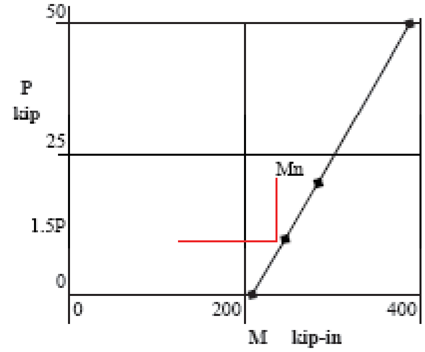


Figure 2.3 – Interaction Diagram for Test Panel No. 21

Appendix

SEAOSC -
Slender Wall Task Group
Develop interaction curve
Wall No. 22

f_y = 67.5 ksi
f'_c = 4.0 ksi
E_s = 28,600 ksi
E_c = 3540 ksi
A_s = 0.196 sq in.
h = 7.40 in
l_w = 48.0 in
P = 5.97 kip
P/ A_g = 0.017 ksi
M_u(design)= 235 kip-in



P/ f'_cA_g	P	M	M	ε_t
0.0	0	209	17.4	0.0292
0.0005	1	211	17.6	0.0288
0.001	1	214	17.8	0.0284
0.0070	10	246	20.5	0.0241
0.0140	20	283	23.6	0.0204
0.0351	50	390	32.5	0.0136
0.05	71	462	38.5	0.0108
0.10	142	684	57.0	0.0057
0.112	159	733	61.1	**0.0050**
0.20	285	1035	86.3	0.0021
0.205	292	1050	87.5	**0.0020**
		kip-in	kip-ft	

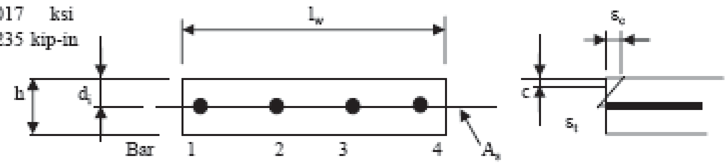
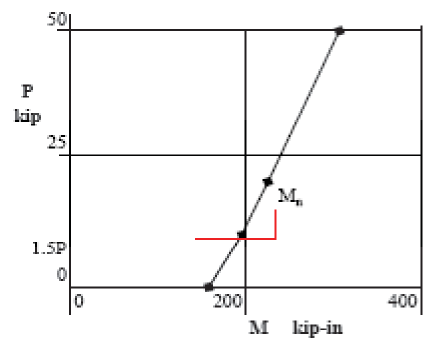


Figure 2.4 – Interaction Diagram for Test Panel No. 22

SEAOSC -
Slender Wall Task Group
Develop interaction curve
Wall No. 23

f_y = 67.5 ksi
f'_c = 4.0 ksi
E_s = 28,600 ksi
E_c = 3540 ksi
A_s = 0.196 sq in.
h = 7.34 in
l_w = 48.0 in
P = 5.97 kip
P/ A_g = 0.017 ksi
M_u(design)= 235 kip-in



P/ f'_cA_g	P	M	M	ε_t
0.0	0	166	13.8	0.0229
0.0005	1	168	14.0	0.0225
0.001	1	170	14.2	0.0222
0.0071	10	195	16.3	0.0188
0.0142	20	224	18.7	0.0158
0.0354	50	306	25.5	0.0103
0.05	71	360	30.0	0.0081
0.09	120	478	39.8	**0.0049**
0.10	141	524	43.6	0.0041
0.160	226	680	56.7	**0.0019**
0.20	282	760	63.3	0.0011
		kip-in	kip-ft	

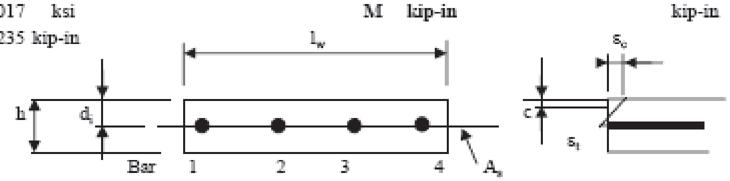
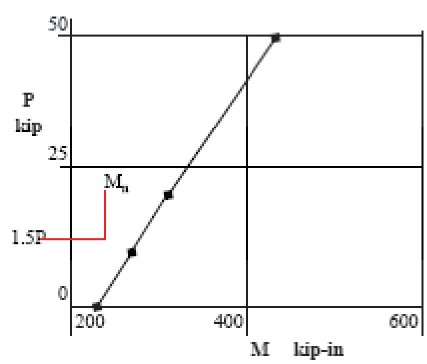


Figure 2.5 – Interaction Diagram for Test Panel No. 23

SEAOSC -
Slender Wall Task Group
Develop interaction curve
Wall No. 24

f_y = 67.5 ksi
f'_c = 4.0 ksi
E_s = 28,600 ksi
E_c = 3540 ksi
A_s = 0.196 sq in.
h = 7.38 in
l_w = 48 in
P = 8.12 kip
P/ A_g = 0.023 ksi
M_u(design)= 235 kip-in



P/ f'_cA_g	P	M	M	ε_t
0.0	0	231	19.3	0.0326
0.0005	1	234	19.5	0.0321
0.001	1	237	19.8	0.0317
0.0070	10	273	22.7	0.0270
0.0141	20	314	26.2	0.0229
0.0352	50	434	36.1	0.0153
0.05	71	514	42.8	0.0122
0.10	142	766	63.9	0.0067
0.128	182	894	74.5	**0.0050**
0.200	284	1178	98.2	0.0026
0.23	327	1277	106.4	**0.0020**
		kip-in	kip-ft	

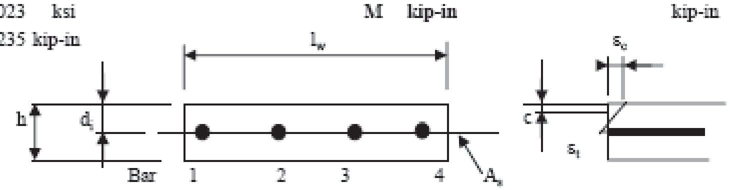
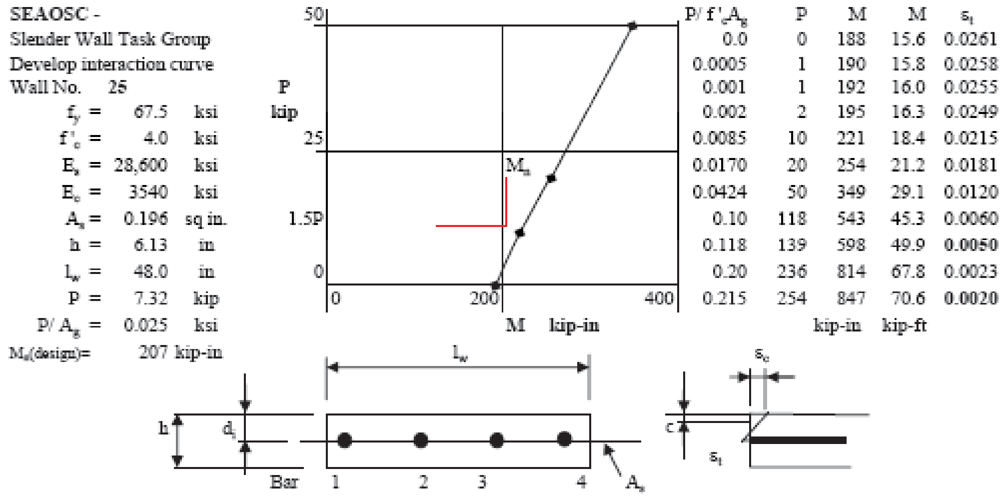


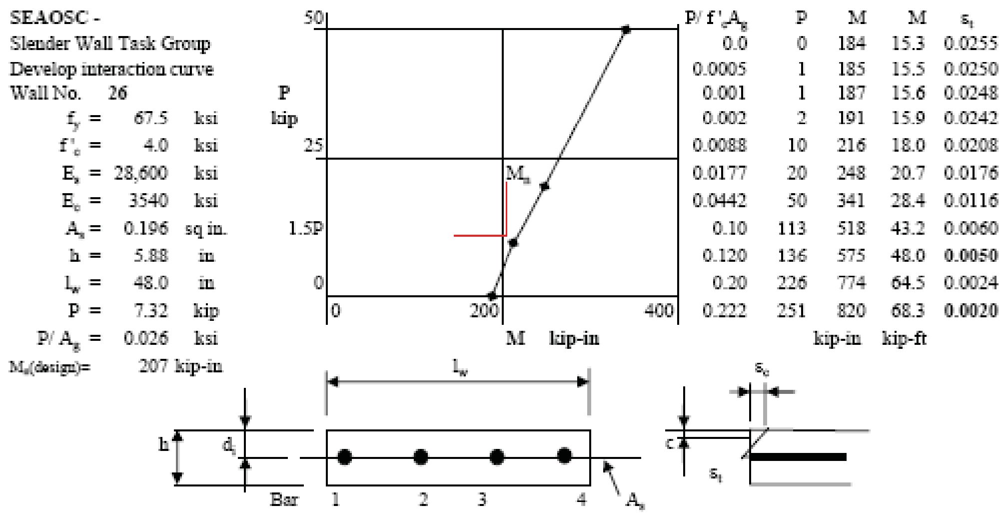
Figure 2.6 – Interaction Diagram for Test Panel No. 24

Appendix



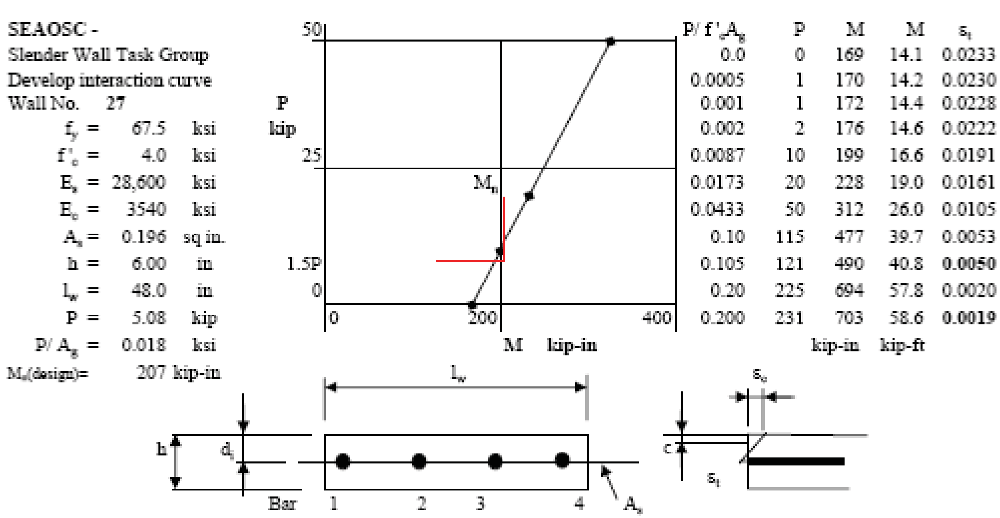
P/f'_cA_g	P	M (kip-in)	M (kip-ft)	ε_t
0.0	0	188	15.6	0.0261
0.0005	1	190	15.8	0.0258
0.001	1	192	16.0	0.0255
0.002	2	195	16.3	0.0249
0.0085	10	221	18.4	0.0215
0.0170	20	254	21.2	0.0181
0.0424	50	349	29.1	0.0120
0.10	118	543	45.3	0.0060
0.118	139	598	49.9	**0.0050**
0.20	236	814	67.8	0.0023
0.215	254	847	70.6	**0.0020**

Figure 2.7 – Interaction Diagram for Test Panel No. 25



P/f'_cA_g	P	M (kip-in)	M (kip-ft)	ε_t
0.0	0	184	15.3	0.0255
0.0005	1	185	15.5	0.0250
0.001	1	187	15.6	0.0248
0.002	2	191	15.9	0.0242
0.0088	10	216	18.0	0.0208
0.0177	20	248	20.7	0.0176
0.0442	50	341	28.4	0.0116
0.10	113	518	43.2	0.0060
0.120	136	575	48.0	**0.0050**
0.20	226	774	64.5	0.0024
0.222	251	820	68.3	**0.0020**

Figure 2.8 – Interaction Diagram for Test Panel No. 26



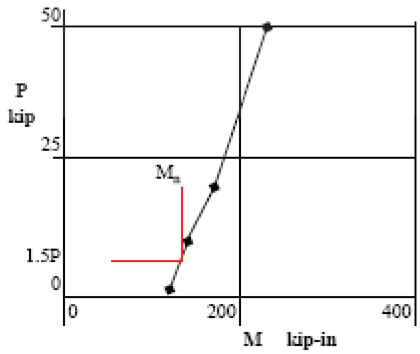
P/f'_cA_g	P	M (kip-in)	M (kip-ft)	ε_t
0.0	0	169	14.1	0.0233
0.0005	1	170	14.2	0.0230
0.001	1	172	14.4	0.0228
0.002	2	176	14.6	0.0222
0.0087	10	199	16.6	0.0191
0.0173	20	228	19.0	0.0161
0.0433	50	312	26.0	0.0105
0.10	115	477	39.7	0.0053
0.105	121	490	40.8	**0.0050**
0.20	225	694	57.8	0.0020
0.200	231	703	58.6	**0.0019**

Figure 2.9 – Interaction Diagram for Test Panel No. 27

Appendix

SEAOSC -
Slender Wall Task Group
Develop interaction curve
Wall No. 28

f_y =	67.5	ksi
f'_c =	4.0	ksi
E_s =	28,600	ksi
E_c =	3540	ksi
A_s =	0.196	sq in.
h =	4.82	in
l_w =	48.0	in
P =	4.33	kip
P/ A_g =	0.019	ksi
M_u(design)=	139	kip-in



P/ f'_cA_g	P	M	M	ε_t
0.0	0	127	10.6	0.0172
0.0005	0	128	10.7	0.0170
0.001	1	129	10.8	0.0168
0.002	2	132	11.0	0.0165
0.0108	10	149	12.5	0.0140
0.0216	20	171	14.2	0.0117
0.0539	50	232	19.3	0.0074
0.087	81	288	24.0	**0.0050**
0.10	93	309	25.7	0.0043
0.175	162	411	34.2	**0.0020**
0.200	186	438	36.5	0.0015
		kip-in	kip-ft	

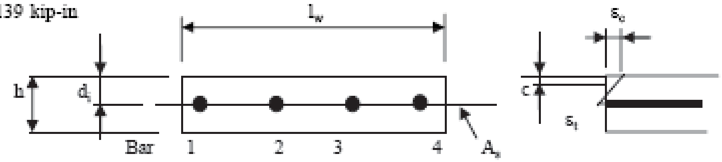
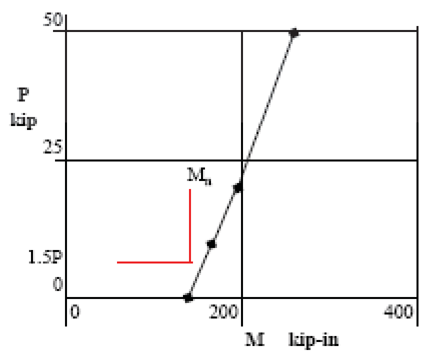


Figure 2.10 – Interaction Diagram for Test Panel No. 28

SEAOSC -
Slender Wall Task Group
Develop interaction curve
Wall No. 29

f_y =	67.5	ksi
f'_c =	4.0	ksi
E_s =	28,600	ksi
E_c =	3540	ksi
A_s =	0.196	sq in.
h =	4.78	in
l_w =	48.0	in
P =	4.31	kip
P/ A_g =	0.019	ksi
M_u(design)=	139	kip-in



P/ f'_cA_g	P	M	M	ε_t
0.0	0	139	11.5	0.0188
0.0005	0	140	11.6	0.0186
0.001	1	141	11.7	0.0185
0.002	2	143	11.9	0.0181
0.0109	10	163	13.6	0.0154
0.0217	20	186	15.5	0.0129
0.054	50	253	21.1	0.0082
0.100	92	338	28.2	**0.0050**
0.15	138	418	34.9	0.0031
0.195	179	480	40.0	**0.0020**
0.200	184	486	40.5	0.0019
		kip-in	kip-ft	

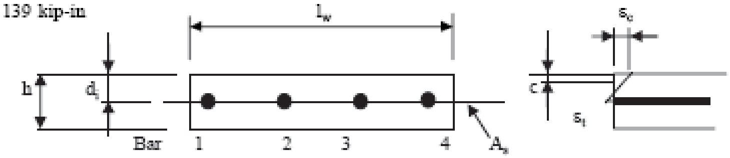
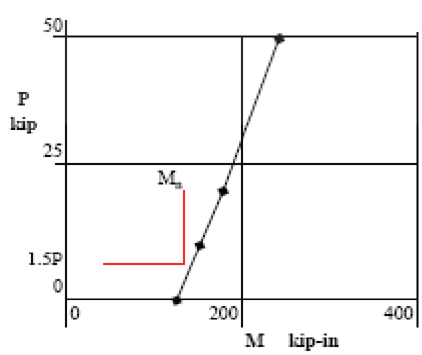


Figure 2.11 – Interaction Diagram for Test Panel No. 29

SEAOSC -
Slender Wall Task Group
Develop interaction curve
Wall No. 30

f_y =	67.5	ksi
f'_c =	4.0	ksi
E_s =	28,600	ksi
E_c =	3540	ksi
A_s =	0.196	sq in.
h =	4.89	in
l_w =	48.0	in
P =	4.38	kip
P/ A_g =	0.019	ksi
M_u(design)=	139	kip-in



P/ f'_cA_g	P	M	M	ε_t
0.0	0	126	10.5	0.0170
0.0005	0	128	10.6	0.0169
0.001	1	129	10.7	0.0167
0.002	2	131	10.9	0.0164
0.0106	10	148	12.4	0.0139
0.0213	20	170	14.1	0.0116
0.0531	50	230	19.2	0.0073
0.085	80	285	23.7	**0.0050**
0.10	94	309	25.7	0.0042
0.170	160	404	33.7	**0.0020**
0.200	188	437	36.4	0.0014
		kip-in	kip-ft	

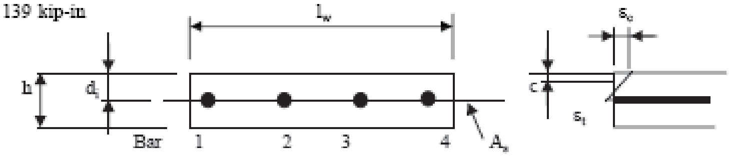
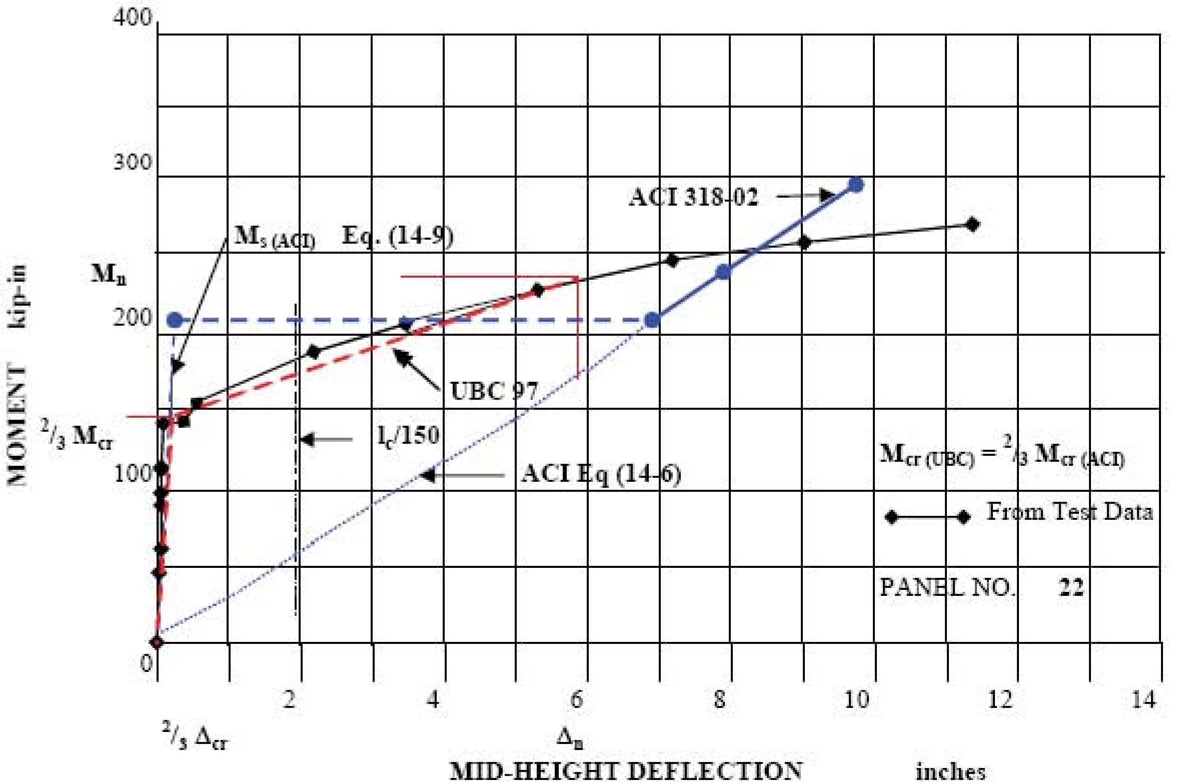


Figure 2.12 – Interaction Diagram for Test Panel No. 30

Appendix



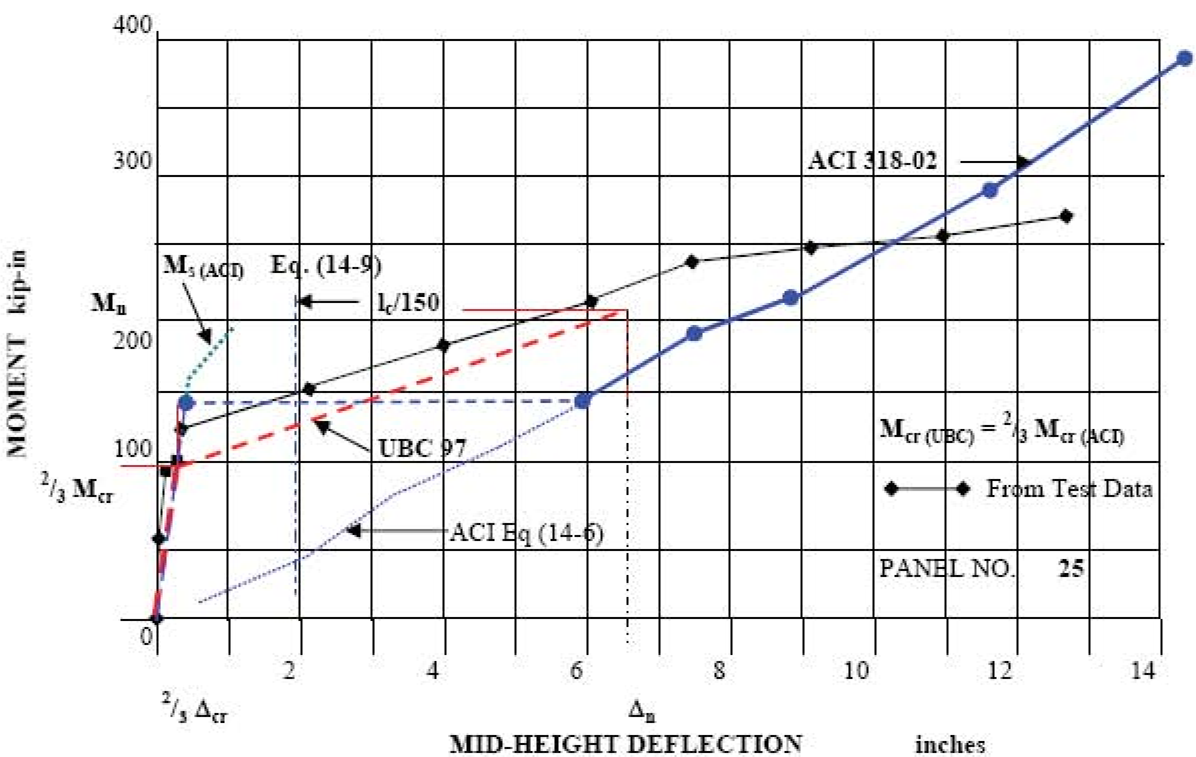
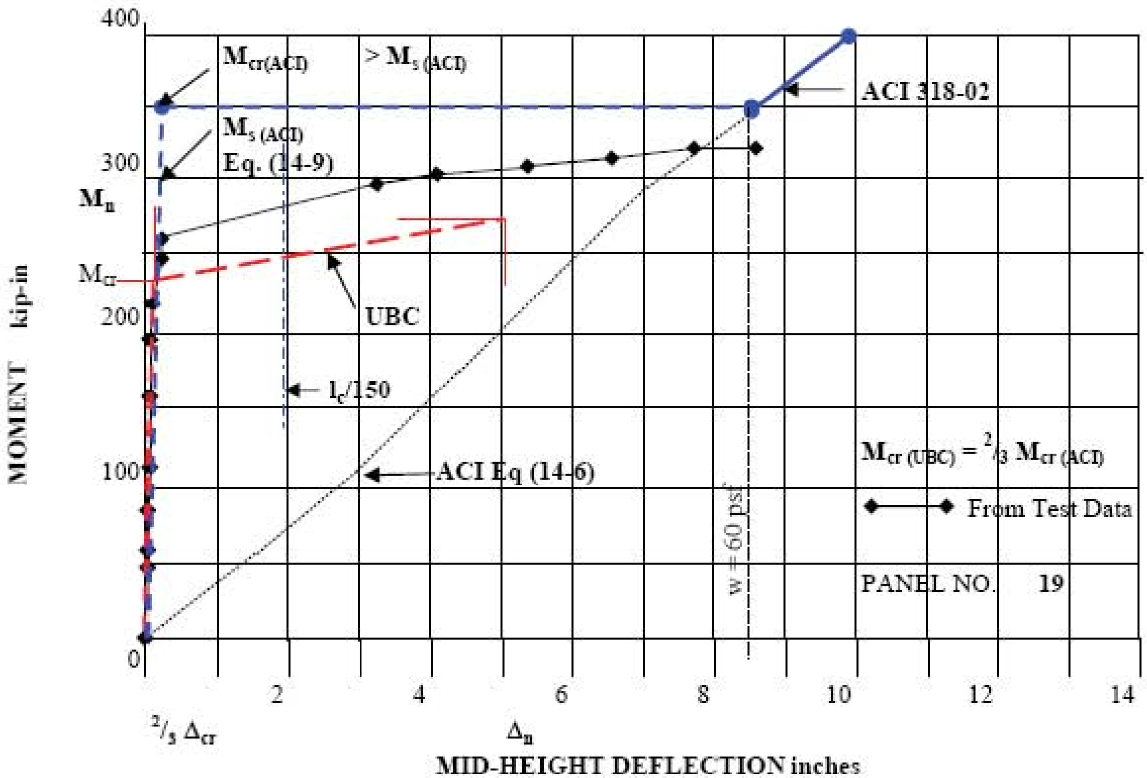


Figure 3.1 – Comparison ACI and UBC Procedure for Test Panel Nos. 22 and 25

Appendix



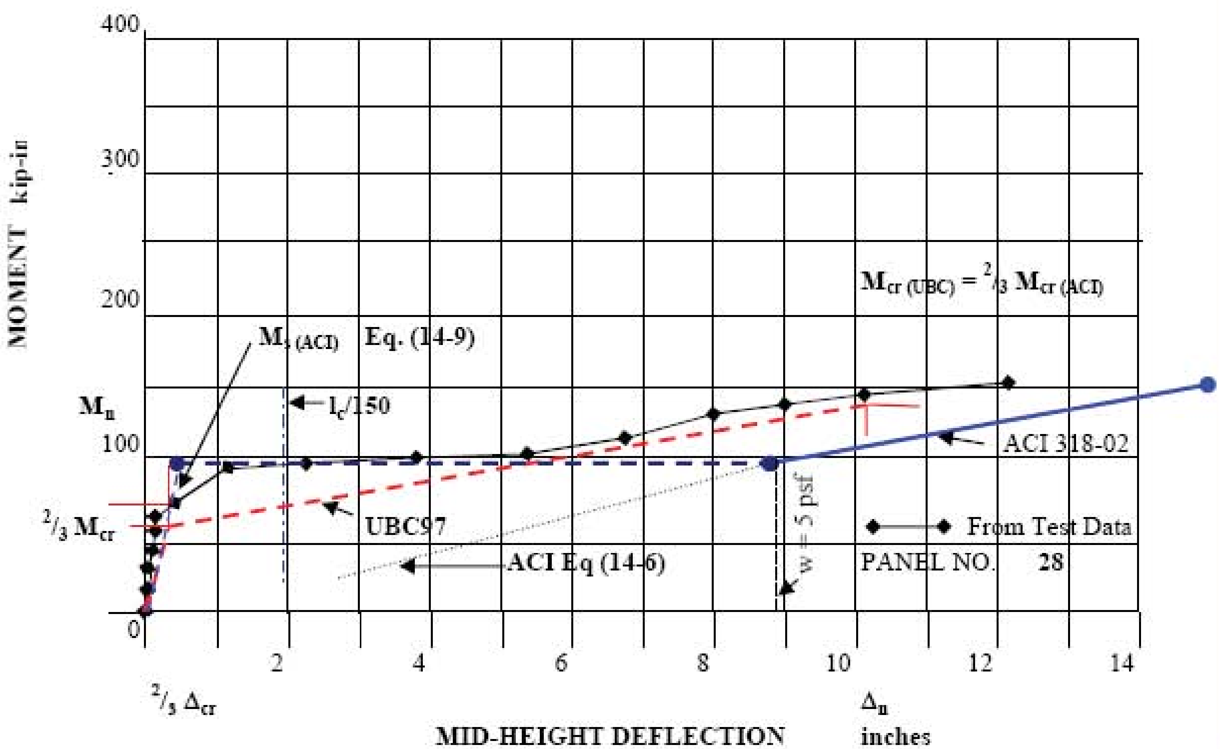
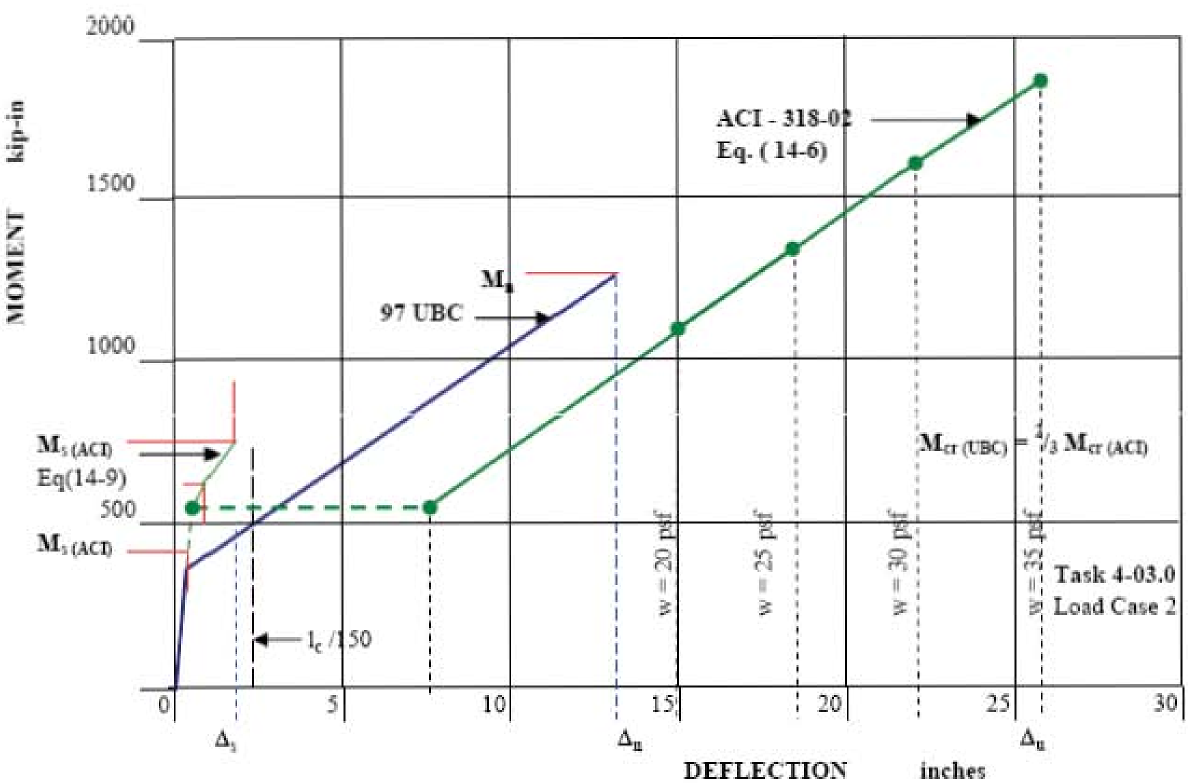


Figure 3.2 – Comparison ACI and UBC Procedure for Test Panel Nos. 19 and 28

Appendix



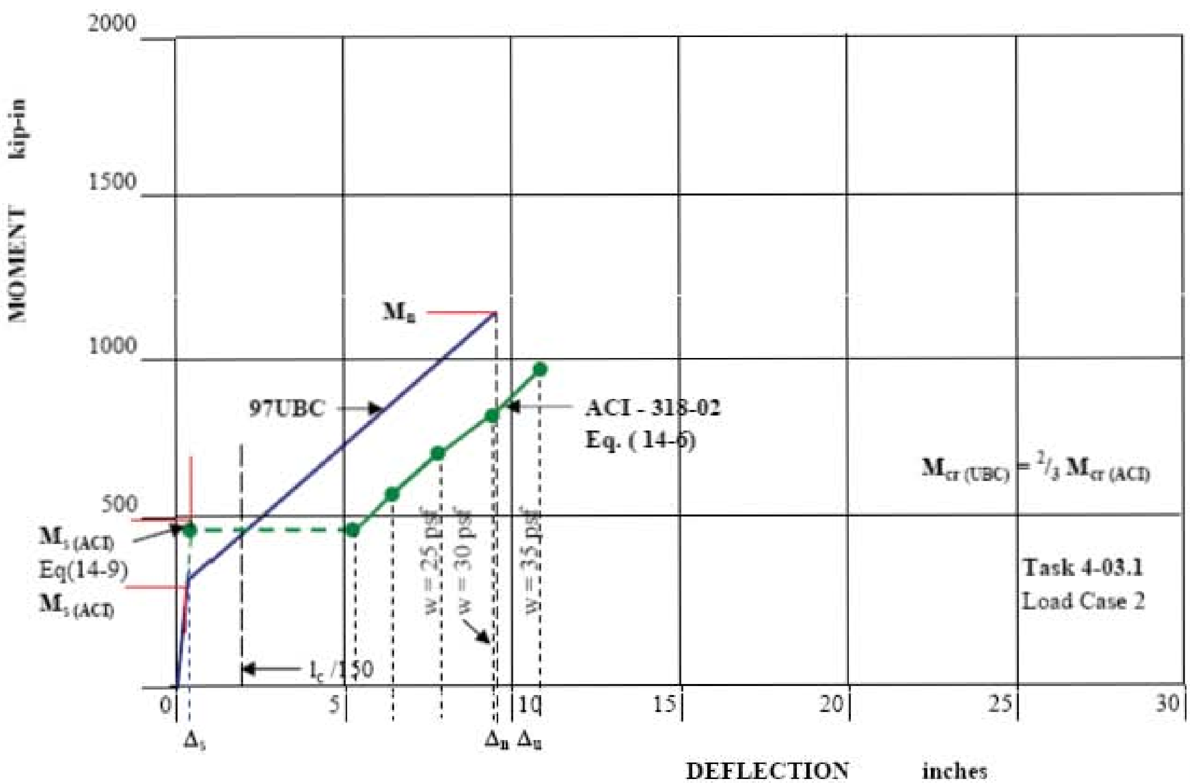


Figure 4.1 – Comparative Design Procedure Plot for Task 4 – 03.0 and Task 4 – 03.1

Appendix

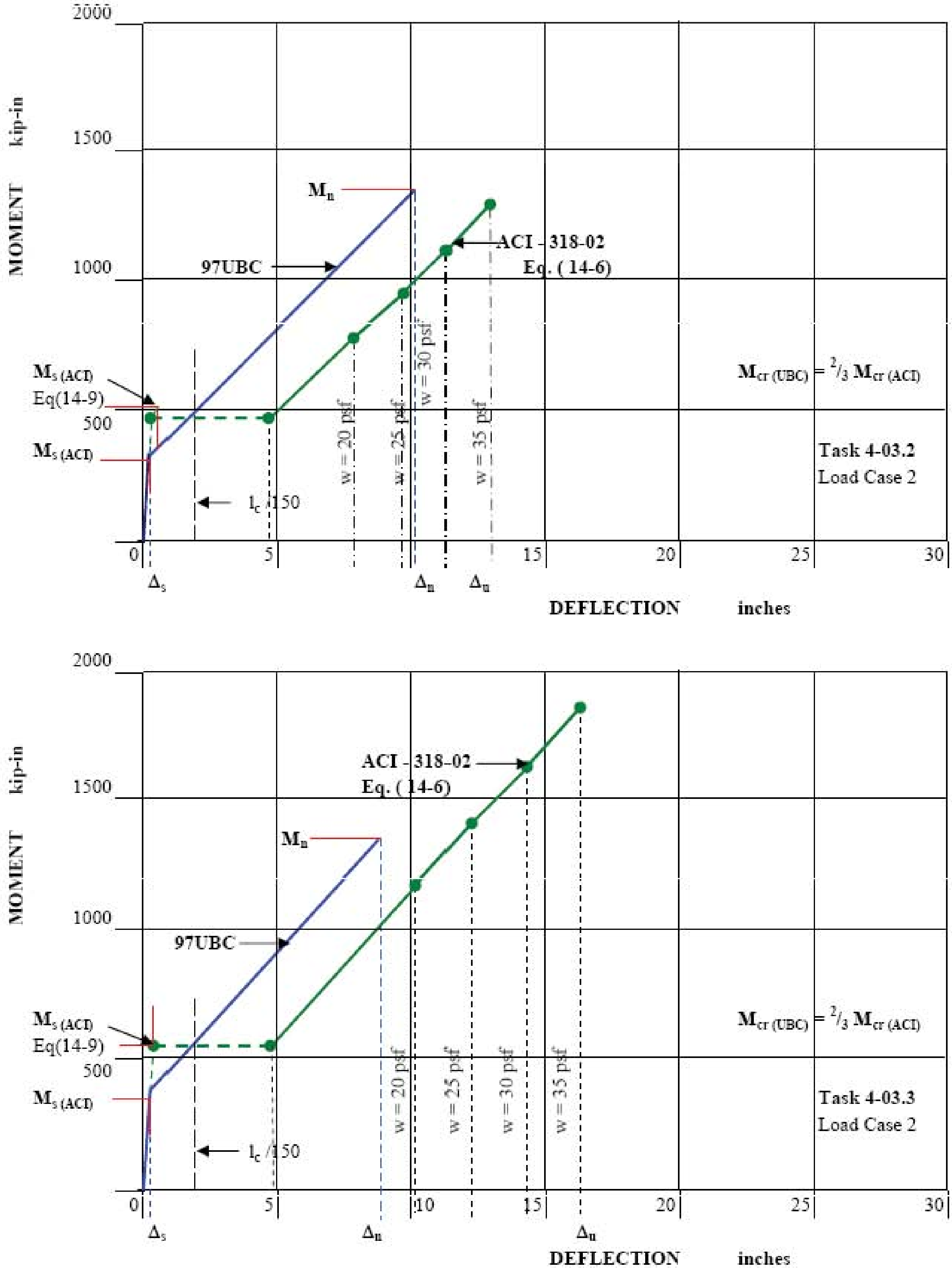
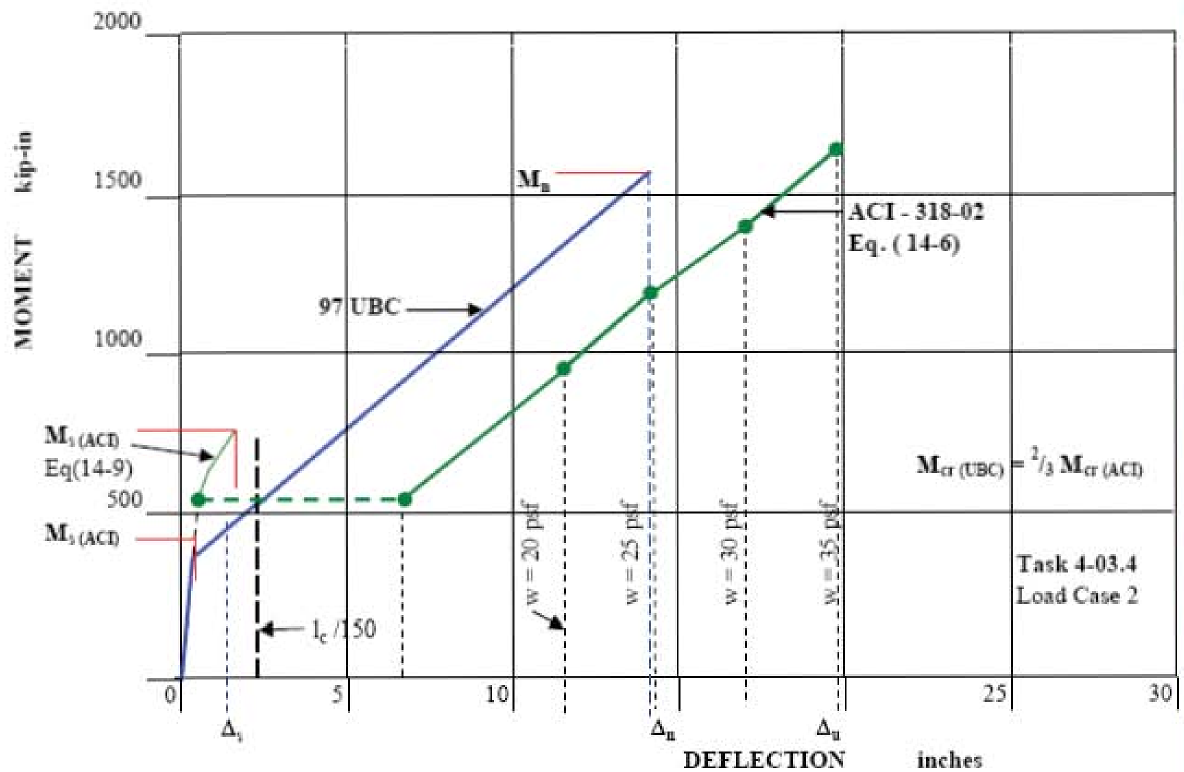


Figure 4.2 – Comparative Design Procedure Plot for Task 4 – 03.2 and Task 4 – 03.3

Appendix



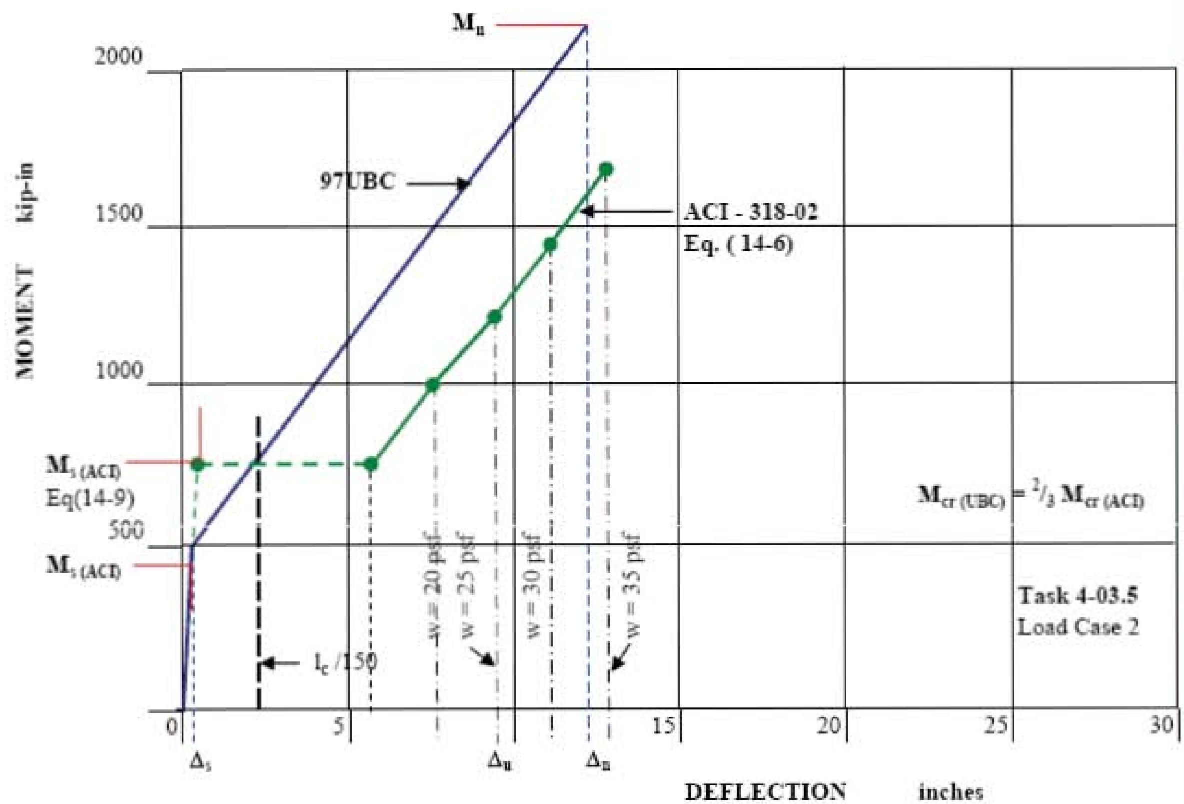


Figure 4.3 – Comparative Design Procedure Plot for Task 4 – 03.4 and Task 4 – 03.5

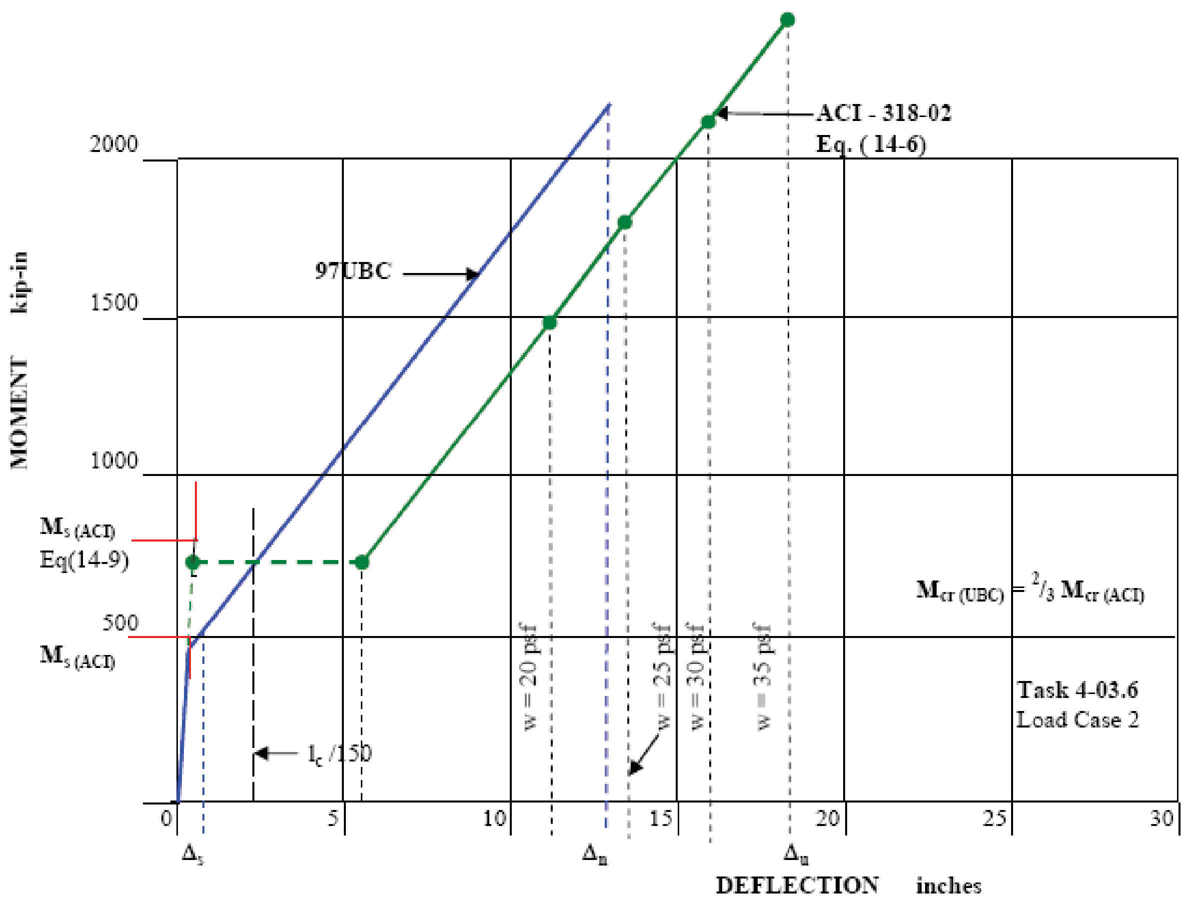


Figure 4.4 – Comparative Design Procedure Plot for Task 4 – 03.6

Appendix

Table 4.1 – Comparative Example Task 4 – 03.0 and Task 4 – 03.1

Slender Wall Task Group
Code Comparison
Summary of Calculation Data - Task 4 -03 **Load Case 2** U = 1.05 D+ 1.28 L + 1.3 W

Based on ACI 318-02 Procedure

Task			4-03.0	4-03.0	4-03.0	4-03.0	4-03.0	4-03.1	4-03.1	4-03.1	4-03.1	4-03.1
Lateral Force	w	psf	**9.7**	**20**	**25**	**30**	**35**	**16.2**	**20**	**25**	**30**	**35**
Panel Height	l_c	ft	29.5	29.5	29.5	29.5	29.5	24	24	24	24	24
Panel Thickness	h	in	6.25	6.25	6.25	6.25	6.25	5.75	5.75	5.75	5.75	5.75
Panel Length	l_w	ft	15	15	15	15	15	15	15	15	15	15
Eccentricity	e	in	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00
Rebar			16 # 6	16 # 6	16 # 6	16 # 6	16 # 6	16 # 6	16 # 6	16 # 6	16 # 6	16 # 6
Rebar d	d	in	3.13	3.13	3.13	3.13	3.13	2.88	2.88	2.88	2.88	2.88
Steel Ratio	ρ		0.0126	0.0126	0.0126	0.0126	0.0126	0.0137	0.0137	0.0137	0.0137	0.0137
Maximum Steel Ratio	$0.6\rho_b$		0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171
Factored Axial Load	P_u	kip	37.1	37.1	37.1	37.1	37.1	32.4	32.4	32.4	32.4	32.4
Average Axial Stress	P_u/A_g	ksi	0.033	0.033	0.033	0.033	0.033	0.0	0.031	0.031	0.031	0.031
Effective Steel Area	A_{se}	in^2	7.684	7.68	7.68	7.68	7.68	7.6	7.61	7.61	7.61	7.61
Moment Strength	M_n	k-in	1267	1267	1267	1267	1267	1142	1142	1142	1142	1142
$\phi = 0.90$	M_u/ϕ	k-in	618	1212	1500	1789	2077	521	633	780	928	1075
Required Strength	M_u	k-in	556	1091	1350	1610	1870	469	570	702	835	967
Cracked Moment	M_{cr}	k-in	556	556	556	556	556	470	470	470	470	470
Service Load Moment	M_s	k-in	219	428	529	641	765	238	288	355	421	489
	Δ_u	in	7.636	14.98	18.55	22.11	25.68	5.3	6.40	7.88	9.37	10.86
	Δ_{cr}	in	0.550	0.55	0.55	0.55	0.55	0.4	0.40	0.40	0.40	0.40
Service Load Deflection	Δ_s	in	0.217	0.42	0.52	0.92	1.71	0.2	0.24	0.30	0.35	0.46
Deflection Limit	$l_c/150$	in	2.360	2.36	2.36	2.36	2.36	1.9	1.92	1.92	1.92	1.92

Based on UBC 97 Procedure

Lateral Force	w	psf		**20**	**25**	**30**	**35**		**20**	**25**	**30**	**35**
Axial Load at mid-height	P_a	kip		33.7	33.7	33.7	33.7		29.3	29.3	29.3	29.3
Average axial stress	P_a/A_g	ksi		0.030	0.030	0.030	0.030		0.028	0.028	0.028	0.028
Effective Steel Area	A_{se}	in^2		7.68	7.68	7.68	7.68		7.61	7.61	7.61	7.61
Moment Strength	M_n	k-in		1267	1267	1267	1267		1142	1142	1142	1142
$\phi = 0.90$	M_u/ϕ	k-in		1142	1284	1425	1566		755	848	942	1035
Required Strength	M_u	k-in		1009	1134	1259	1384		668	750	833	916
Cracked Moment	$2/3\ M_{cr}$	k-in		371	371	371	371		314	314	314	314
Service Load Moment	M_s	k-in		476	664	851	1039		314	374	470	567
	Δ_n	in		13.05	13.05	13.05	13.05		9.61	9.61	9.61	9.61
	$2/3\ \Delta_{cr}$	in		0.37	0.37	0.37	0.37		0.26	0.26	0.26	0.26
Service Load Deflection	Δ_s	in		1.86	4.51	7.16	9.82		0.26	0.94	2.03	3.12
Deflection Limit	$l_c/150$	in		2.36	2.36	2.36	2.36		1.92	1.92	1.92	1.92

Appendix

Table 4.2– Comparative Example Task 4 – 03.2 and Task 4 – 03.3

SEAOSC
Slender Wall Task Group
Code Comparison
Summary of Calculation Data - Task 4 -03 **Load Case 2** U = 1.05 D + 1.28 L + 1.3 W

Based on ACI 318-02 Procedure												
Task			4-03.2	4-03.2	4-03.2	4-03.2	4-03.2	4-03.3	4-03.3	4-03.3	4-03.3	4-03.3
Lateral Force	w	psf	**10.8**	**20**	**25**	**30**	**35**	**6.9**	**20**	**25**	**30**	**35**
Panel Height	l_c	ft	24	24	24	24	24	24	24	24	24	24
Panel Thickness	h	in	5.75	5.75	5.75	5.75	5.75	6.25	6.25	6.25	6.25	6.25
Panel Length	l_w	ft	15	15	15	15	15	15	15	15	15	15
Eccentricity	e	in	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00
Rebar			19 # 6	19 # 6	19 # 6	19 # 6	19 # 6	16 # 6	16 # 6	16 # 6	16 # 6	16 # 6
Rebar d	d	in	2.88	2.88	2.88	2.88	2.88	3.13	3.13	3.13	3.13	3.13
Steel Ratio	ρ		0.0162	0.0162	0.0162	0.0162	0.0162	0.0126	0.0126	0.0126	0.0126	0.0126
Maximum Steel Ratio	$0.6\rho_b$		0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171
Factored Axial Load	P_u	kip	49.5	49.5	49.5	49.5	49.5	73.1	73.1	73.1	73.1	73.1
Average Axial Stress	P_u/A_g	ksi	0.0	0.048	0.048	0.048	0.048	0.065	0.065	0.065	0.065	0.065
Effective Steel Area	A_{se}	in^2	9.2	9.22	9.22	9.22	9.22	8.28	8.28	8.28	8.28	8.28
Moment Strength	M_n	k-in	1340	1340	1340	1340	1340	1351	1351	1351	1351	1351
$\phi = 0.90$	M_u/ϕ	k-in	520	866	1054	1242	1429	618	1296	1555	1814	2072
Required Strength	M_u	k-in	468	779	948	1117	1286	556	1166	1399	1632	1865
Cracked Moment	M_{cr}	k-in	470	470	470	470	470	556	556	556	556	556
Service Load Moment	M_s	k-in	191	315	382	450	523	169	347	414	482	549
	Δ_u	in	4.7	7.89	9.61	11.32	13.03	4.85	10.18	12.22	14.25	16.28
	Δ_{cr}	in	0.4	0.40	0.40	0.40	0.40	0.36	0.36	0.36	0.36	0.36
Service Load Deflection	Δ_s	in	0.2	0.26	0.32	0.38	0.58	0.11	0.23	0.27	0.32	0.36
Deflection Limit	$l_c/150$	in	1.9	1.92	1.92	1.92	1.92	1.92	1.92	1.92	1.92	1.92
Based on UBC 97 Procedure												
Lateral Force	w	psf		**20**	**25**	**30**	**35**		**20**	**25**	**30**	**35**
Axial Load at mid-height	P_a	kip		44.0	44.0	44.0	44.0		64.3	64.3	64.3	64.3
Average axial stress	P_a/A_g	ksi		0.042	0.042	0.042	0.042		0.057	0.057	0.057	0.057
Effective Steel Area	A_{se}	in^2		9.22	9.22	9.22	9.22		8.28	8.28	8.28	8.28
Moment Strength	M_n	k-in		1340	1340	1340	1340		1351	1351	1351	1351
$\phi = 0.90$	M_u/ϕ	k-in		1011	1106	1200	1294		1225	1320	1415	1510
Required Strength	M_u	k-in		886	969	1051	1134		1062	1145	1227	1310
Cracked Moment	2/3 M_{cr}	k-in		314	314	314	314		371	371	371	371
Service Load Moment	M_s	k-in		316	428	541	654		371	466	615	764
	Δ_n	in		10.18	10.18	10.18	10.18		8.85	8.85	8.85	8.85
	2/3 Δ_{cr}	in		0.26	0.26	0.26	0.26		0.24	0.24	0.24	0.24
Service Load Deflection	Δ_s	in		0.28	1.37	2.46	3.55		0.24	1.08	2.38	3.69
Deflection Limit	$l_c/150$	in		1.92	1.92	1.92	1.92		1.92	1.92	1.92	1.92

Appendix

Table 4.3 – Comparative Example Task 4 – 03.4 and Task 4 – 03.5

SEAOSC
Slender Wall Task Group
Code Comparison
Summary of Calculation Data - Task 4 -03 **Load Case 2** U = 1.05 D + 1.28 L + 1.3 W

Based on ACI 318-02 Procedure												
Task			4-03.4	4-03.4	4-03.4	4-03.4	4-03.4	4-03.5	4-03.5	4-03.5	4-03.5	4-03.5
Lateral Force	w	psf	**11.1**	**20**	**25**	**30**	**35**	**14.4**	**20**	**25**	**30**	**35**
Panel Height	l_c	ft	29.5	29.5	29.5	29.5	29.5	29.5	29.5	29.5	29.5	29.5
Panel Thickness	h	in	6.25	6.25	6.25	6.25	6.25	7.25	7.25	7.25	7.25	7.25
Panel Length	l_w	ft	15	15	15	15	15	15	15	15	15	15
Eccentricity	e	in	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00
Rebar			21 # 6	21 # 6	21 # 6	21 # 6	21 # 6	24 #6	24 #6	24 #6	24 #6	24 #6
Rebar d	d	in	3.13	3.13	3.13	3.13	3.13	3.63	3.63	3.63	3.63	3.63
Steel Ratio	ρ		0.0165	0.0165	0.0165	0.0165	0.0165	0.0162	0.0162	0.0162	0.0162	0.0162
Maximum Steel Ratio	$0.6\rho_b$		0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171
Factored Axial Load	P_u	kip	37.1	37.1	37.1	37.1	37.1	57.4	57.4	57.4	57.4	57.4
Average Axial Stress	P_u/A_g	ksi	0.0330	0.033	0.033	0.033	0.033	0.044	0.044	0.044	0.044	0.044
Effective Steel Area	A_{se}	in^2	9.9	9.89	9.89	9.89	9.89	11.55	11.55	11.55	11.55	11.55
Moment Strength	M_n	k-in	1567	1567	1567	1567	1567	2120	2120	2120	2120	2120
$\phi = 0.90$	M_u/ϕ	k-in	618	1071	1326	1581	1836	829	1112	1364	1617	1869
Required Strength	M_u	k-in	556	964	1194	1423	1653	746	1001	1228	1455	1682
Cracked Moment	M_{cr}	k-in	556	556	556	556	556	748	748	748	748	748
Service Load Moment	M_s	k-in	248	428	529	640	763	337	450	552	653	755
	Δ_u	in	6.67	11.57	14.32	17.08	19.83	5.71	7.66	9.40	11.14	12.88
	Δ_{cr}	in	0.55	0.55	0.55	0.55	0.55	0.47	0.47	0.47	0.47	0.47
Service Load Deflection	Δ_s	in	0.24	0.42	0.52	0.91	1.66	0.21	0.29	0.35	0.41	0.49
Deflection Limit	$l_c/150$	in	2.36	2.36	2.36	2.36	2.36	2.36	2.36	2.36	2.36	2.36
Based on UBC 97 Procedure												
Lateral Force	w	psf		**20**	**25**	**30**	**35**		**20**	**25**	**30**	**35**
Axial Load at mid-height	P_a	kip		33.7	33.7	33.7	33.7		51.5	51.5	51.5	51.5
Average axial stress	P_a/A_g	ksi		0.030	0.030	0.030	0.030		0.039	0.039	0.039	0.039
Effective Steel Area	A_{se}	in^2		9.89	9.89	9.89	9.89		11.55	11.55	11.55	11.55
Moment Strength	M_n	k-in		1567	1567	1567	1567		2120	2120	2120	2120
$\phi = 0.90$	M_u/ϕ	k-in		1187	1328	1469	1611		1424	1566	1708	1850
Required Strength	M_u	k-in		1048	1173	1298	1423		1250	1375	1500	1625
Cracked Moment	$2/3\ M_{cr}$	k-in		371	371	371	371		499	499	499	499
Service Load Moment	M_s	k-in		461	621	781	941		499	580	738	895
	Δ_n	in		14.10	14.10	14.10	14.10		12.18	12.18	12.18	12.18
	$2/3\ \Delta_{cr}$	in		0.37	0.37	0.37	0.37		0.32	0.32	0.32	0.32
Service Load Deflection	Δ_s	in		1.40	3.24	5.07	6.91		0.32	0.91	2.06	3.21
Deflection Limit	$l_c/150$	in		2.36	2.36	2.36	2.36		2.36	2.36	2.36	2.36

Appendix

Table 4.4 – Comparative Example Task 4 – 03.6

SEAOSC
Slender Wall Task Group
Code Comparison
Summary of Calculation Data - Task 4 -03 **Load Case 2** U = **1.05D +1.28 L + 1.3W**

Based on ACI 318-02 Procedure

Task			4-03.6	4-03.6	4-03.6	4-03.6	4-03.6
Lateral Force	w	psf	**8.4**	**20**	**25**	**30**	**35**
Panel Height	l_c	ft	29.5	29.5	29.5	29.5	29.5
Panel Thickness	h	in	7.25	7.25	7.25	7.25	7.25
Panel Length	l_w	ft	15	15	15	15	15
Eccentricity	e	in	3.00	3.00	3.00	3.00	3.00
Rebar			24 # 6	24 # 6	24 # 6	24 # 6	24 # 6
Rebar d	d	in	3.63	3.63	3.63	3.63	3.63
Steel Ratio	ρ		0.0162	0.0162	0.0162	0.0162	0.0162
Maximum Steel Ratio	$0.6\rho_b$		0.0171	0.0171	0.0171	0.0171	0.0171
Factored Axial Load	P_u	kip	79.7	79.7	79.7	79.7	79.7
Average Axial Stress	P_u/A_g	ksi	0.061	0.061	0.061	0.061	0.061
Effective Steel Area	A_{se}	in^2	11.93	11.93	11.93	11.93	11.93
Moment Strength	M_n	k-in	2176	2176	2176	2176	2176
$\phi = 0.90$	M_u/ϕ	k-in	836	1663	2019	2376	2732
Required Strength	M_u	k-in	746	1484	1802	2120	2438
Cracked Moment	M_{cr}	k-in	748	748	748	748	748
Service Load Moment	M_s	k-in	248	486	589	691	801
	Δ_u	in	5.61	11.16	13.56	15.95	18.34
	Δ_{cr}	in	0.47	0.47	0.47	0.47	0.47
Service Load Deflection	Δ_s	in	0.16	0.31	0.37	0.44	0.61
Deflection Limit	$l_c/150$	in	2.36	2.36	2.36	2.36	2.36

Based on UBC 97 Procedure

Lateral Force	w	psf	**20**	**25**	**30**	**35**
Axial Load at mid-height	P_a	kip	70.6	70.6	70.6	70.6
Average axial stress	P_a/A_g	ksi	0.054	0.054	0.054	0.054
Effective Steel Area	A_{se}	in^2	11.93	11.93	11.93	11.93
Moment Strength	M_n	k-in	2176	2176	2176	2176
$\phi = 0.90$	M_u/ϕ	k-in	1797	1941	2084	2228
Required Strength	M_u	k-in	1563	1687	1812	1937
Cracked Moment	$2/3\ M_{cr}$	k-in	499	499	499	499
Service Load Moment	M_s	k-in	514	727	940	1154
	Δ_n	in	12.28	12.28	12.28	12.28
	$2/3\ \Delta_{cr}$	in	0.32	0.32	0.32	0.32
Service Load Deflection	Δ_s	in	0.42	1.94	3.47	4.99
Deflection Limit	$l_c/150$	in	2.36	2.36	2.36	2.36

Appendix

Table 4.5 – Comparative Example Task 4 – 04.0 and Task 4 – 04.1 for Double Curtain Reinforcement

SEAOSC
Slender Wall Task Group
Code Comparison
Summary of Calculation Data - Task 4 -03 **Load Case 2** U = 1.05 D + 1.28 L + 1.3 W

Based on ACI 318-02 Procedure			**Double curtain reinforcement**					
Task			4-04.0a	4-04.0b	4-04.1a	4-04.1b	4-04.1a	4-041b
Lateral Force	w	psf	**17**	**17**	**30**	**30**	**35**	**35**
Panel Height	l_c	ft	29.5	29.5	29.5	29.5	29.5	29.5
Panel Thickness	h	in	6.25	6.25	7.25	7.25	7.25	7.25
Panel Length	l_w	ft	4.00	6.00	4.00	6.00	4.00	6.00
Eccentricity	e	in	3.00	3.00	3.00	3.00	3.00	3.00
Rebar			(2) 8 #5	(2)10#5	(2) 9#6	(2) 9#6	(2)10#6	(2)10#6
Rebar d	d	in	4.44	4.44	5.38	5.38	5.38	5.38
Steel Ratio	ρ		0.0115	0.0096	0.0154	0.0103	0.0171	0.0114
Maximum Steel Ratio	$0.6\rho_b$		0.0171	0.0171	0.0171	0.0171	0.0171	0.0171
Factored Axial Load	P_u	kip	21.0	28.3	22.9	30.6	22.9	30.6
Average Axial Stress	P_u/A_g	ksi	0.070	0.063	0.066	0.059	0.066	0.059
Effective Steel Area	A_{se}	in^2	2.80	3.54	4.36	4.48	4.80	4.93
Moment Strength	M_n	k-in	660	850	1196	1298	1293	1410
$\phi = 0.90$	M_u/ϕ	k-in	495	607	661	834	775	953
Required Strength	M_u	k-in	445	546	595	750	682	858
Cracked Moment	M_{cr}	k-in	148	222	199	299	199	299
Service Load Moment	M_s	k-in	255	294	413	498	481	586
	Δ_u	in	8.17	7.37	5.38	5.55	5.85	6.02
	Δ_{cr}	in	0.55	0.55	0.47	0.47	0.47	0.47
Service Load Deflection	Δ_s	in	2.29	1.27	2.32	1.79	2.76	2.35
Deflection Limit	$l_c/150$	in	2.36	2.36	2.36	2.36	2.36	2.36
Based on UBC 97 Procedure								
Lateral Force	w	psf	**17**	**17**	**30**	**30**	**35**	**35**
Axial Load at mid-height	P_a	kip	19.2	25.6	21.0	27.8	21.0	27.8
Average axial stress	P_a/A_g	ksi	0.064	0.057	0.060	0.053	0.060	0.053
Effective Steel Area	A_{se}	in^2	2.80	3.54	4.36	4.48	4.8	4.93
Moment Strength	M_n	k-in	660	850	1196	1298	1293	1410
$\phi = 0.90$	M_u/ϕ	k-in	531	662	748	907	840	1020
Required Strength	M_u	k-in	459	575	649	790	729	888
Cracked Moment	$2/3\ M_{cr}$	k-in	99	148	133	199	133	199
Service Load Moment	M_s	k-in	269	324	414	512	480	593
	Δ_n	in	9.08	8.60	8.11	7.20	8.32	7.42
	$2/3\ \Delta_{cr}$	in	0.37	0.37	0.32	0.32	0.32	0.32
Service Load Deflection	Δ_s	in	3.01	2.43	2.38	2.27	2.71	2.63
Deflection Limit	$l_c/150$	in	2.36	2.36	2.36	2.36	2.36	2.36

Appendix

Table 5 – Summary of Comparative Examples

Summary of Findings - Task 4 - 03

Task	Panel Height	Panel Thickness	Panel Length	Rebar	Axial Load	Nominal Moment	w = 20 psf		w = 25 psf		w = 30 psf		w = 35 psf	
	l_c	h	l_w		P_a	M_n	M	Δ	M	Δ	M	Δ	M	Δ
	ft	in	ft		kip	in-kip								
Single Curtain Reinforcement				**- ACI Procedure**										
4-03.0	29.5	6.25	15.0	16 # 6	33.7	1267	1	3	2	3	2	4	2	4
4-03.1	24.0	5.75	15.0	16 # 6	29.3	1142	1	3	1	3	1	3	1	4
4-03.2	24.0	5.75	15.0	19 # 6	44.0	1340	1	3	1	3	1	3	2	4
4-03.3	24.0	6.25	15.0	16 # 6	64.3	1351	1	3	2	3	2	3	2	3
4-03.4	29.5	6.25	15.0	21 # 6	33.7	1567	1	3	1	3	2	4	2	4
4-03.5	29.5	7.25	15.0	24 # 6	51.5	2120	1	3	1	3	1	3	1	4
4-03.6	29.5	7.25	15.0	24 # 6	70.6	2176	1	3	1	3	2	3	2	4
Two Curtain Reinforcement				**- ACI Procedure**			w = 17 psf				w = 30 psf		w = 35 psf	
4-04.0a	29.5	6.25	4.0	(2) 8 # 5	19.2	660	2	4						
4-04.0b	29.5	6.25	6.0	(2)10 #5	25.6	850	2	4						
4-04.1a	29.5	7.25	4.0	(2) 9 # 6	21.0	1196					1	4		
4-04.1b	29.5	7.25	6.0	(2) 9 # 6	27.8	1298					1	4		
4-04.1a	29.5	7.25	4.0	(2) 10 # 6	21.0	1293							1	5
4-04.1b	29.5	7.25	6.0	(2) 10 # 6	27.8	1410							1	4

Notes:	1	$M_u/\phi < M_n$	3	$\Delta_s \leq \Delta_{cr}$
	2	$M_u/\phi > M_n$	4	$\Delta_{cr} < \Delta_s < l_c/150$
			5	$\Delta_s > l_c/150$

Task	Panel Height	Panel Thickness	Panel Length	Rebar	Axial Load	Nominal Moment	w = 20 psf		w = 25 psf		w = 30 psf		w = 35 psf	
Single Curtain Reinforcement				**- UBC Procedure**										
4-03.0	29.5	6.25	15.0	16 # 6	33.7	1267	1	4	2	5	2	5	2	5
4-03.1	24.0	5.75	15.0	16 # 6	29.3	1142	1	3	1	4	1	5	1	5
4-03.2	24.0	5.75	15.0	19 # 6	44.0	1340	1	4	1	4	1	5	1	5
4-03.3	24.0	6.25	15.0	16 # 6	64.3	1351	1	3	1	4	2	5	2	5
4-03.4	29.5	6.25	15.0	21 # 6	33.7	1567	1	4	1	5	1	5	2	5
4-03.5	29.5	7.25	15.0	24 # 6	51.5	2120	1	3	1	4	1	4	1	5
4-03.6	29.5	7.25	15.0	24 # 6	70.6	2176	1	4	1	4	2	5	2	5
Two Curtain Reinforcement				**-UBC Procedure**										
4-04.0a	29.5	6.25	4.0	(2) 8 # 5	19.2	660	1	5						
4-04.0b	29.5	6.25	6.0	(2)10 #5	25.6	850	1	5						
4-04.1a	29.5	7.25	4.0	(2) 9 # 6	21.0	1196					1	4		
4-04.1b	29.5	7.25	6.0	(2) 9 # 6	27.8	1298					1	4		
4-04.1a	29.5	7.25	4.0	(2) 10 # 6	21.0	1293							1	5
4-04.1b	29.5	7.25	6.0	(2) 10 # 6	27.8	1410							1	5

Notes:	1	$M_u/\phi < M_n$	3	$\Delta_s \leq 2/3\ \Delta_{cr}$
	2	$M_u/\phi > M_n$	4	$2/3\ \Delta_{cr} < \Delta_s < l_c/150$
			5	$\Delta_s > l_c/150$

Appendix

Table 6.1 – Summary of 1980 Test Panel Properties

SEAOSC - Slender Wall Task Group

Test Result from 1980 Task Committee on Slender Walls - Transcribed from Test Report

Reference:	Report of the Task committee on Slender Walls		
	ACI SCC - SEAOSC Task Committee		
Material:			
Concrete	Portland Cement	470	lbs (5sacks)
	Washed concrete sand	1,420	lbs (about 14 cu.ft)
	1 inch gravel	1,815	lbs (about 18 cu ft)
	Water	317	lbs (38 gal)
	water/ cement ratio	0.67	
Lab test results from supplier			
7 days		2,282	psi
28 days		3,181	psi
From Twining laboratories			
7 days	Compression	2,300	psi
	Splitting Tesile	270	psi
28 days	Compression	3,225	psi
	Modulus of elasticity	3,360	ksi
	Splitting Tensile	355	psi
	Modulus of rupture	695	psi
167 days	Compression	4,009	psi
	Modulus of elasticity	3,540	ksi
	Modulus of rupture	520	psi

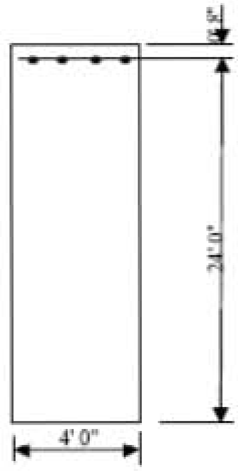
Panels cast on October 3, 1980 and lifted on October 15, 1980
Panels were stored on edge until testing

Reinforcement	ASTM 615-78 Grade 60 from Bethlehem		
Mill Report	Yield Strength	72,250	psi
	Ult Tensile Strength	102,750	psi
Lab Test	Yield Strength	67,500	psi
	Ult Tensile Strength	102,000	psi
	Modulus of elasticity	28,600	ksi
	Yield Strain	0.0025 to 0.0032	
	Elongation in 8 inches	17	%
	Average f_y	70,000	
	Average f_u	110,000	
Horiz #3			
Mill Report	Yield Strength	52,730	psi
	Ult Tensile Strength	75,910	psi
	Yield Strength	52,000	psi
Lab Test	Ult Tensile Strength	79,100	psi
	Modulus of elasticity	28,000	ksi
	Elongation in 8 inches	18	%

Appendix

Table 6.2 – Summary of 1980 Test Panel – Test Results

SEAOSC - Slender Wall Task Group
Typical Concrete Panel



Placement of Reinforcement

Panel No.	Thickness (h), inches		Distance (d) from outer face of wall					Variation in d	
	Nominal	Measured	Bar #1	Bar #2	Bar #3	Bar #4	Ave. d	%	inch
19	9.50	9.60	4.67	5.24	4.48	4.24	4.66	3	0.14
20	9.50	9.40	4.76	4.59	4.70	4.76	4.70	0	0.00
21	9.50	9.50	4.40	4.60	4.70	4.80	4.63	3	0.13
22	7.25	7.40	3.88	4.00	4.13	4.38	4.10	11	0.40
23	7.25	7.34	2.85	3.35	3.48	3.48	3.29	10	0.38
24	7.25	7.38	4.80	4.70	4.30	4.30	4.53	-23	-0.84
25	5.75	6.13	3.70	3.80	3.70	3.60	3.70	-21	-0.64
26	5.75	5.88	3.40	3.70	3.80	3.60	3.63	-23	-0.69
27	5.75	6.00	3.38	3.50	3.25	3.25	3.35	-12	-0.35
28	4.75	4.82	2.21	2.45	2.86	2.74	2.57	-6	-0.16
29	4.75	4.78	2.46	2.58	2.90	3.16	2.78	-16	-0.39
30	4.75	4.89	2.24	2.37	2.66	2.92	2.55	-4	-0.10

% variation in d = [1- d_{ave} / measured h/2] x 100

Variation in d = measured h/2 -d_{ave}

Slender Walls Test Results

Panel No.	Nominal Thickness t, inches	f'c psi	h/t Ratio	Vert Load plf	Load @ f_y psf	Defl at f_y inches	Max. Defl. inches	Date tested
19	9.50	4,000	30	320	87	7.3	9.9	5/14/81
20	9.50	4,000	30	320	83	5.3	7.0	5/12/81
21	9.50	4,000	30	320	83	7.5	12.3	4/27/81
22	7.25	4,000	40	320	57	5.4	12.2	4/28/81
23	7.25	4,000	40	320	52	7.4	11.8	4/29/81
24	7.25	4,000	40	860	57	7.6	11.8	4/14/81
25	5.75	4,000	50	860	51	8.1	13.2	3/14/81
26	5.75	4,000	50	860	42	7.2	11.1	3/18/81
27	5.75	4,000	50	320	42	8.5	12.4	3/23/81
28	4.75	4,000	61	320	32	11.6	13.0	5/5/81
29	4.75	4,000	61	320	34	12.6	19.2	5/15/81
30	4.75	4,000	61	320	34	13.1	15.2	5/14/81

Appendix

Table 6.3 – Summary of 1980 Test Panel Data (Panel Nos. 19 and 22)

SEAOSC - Slender Wall Task Group
Reference: Report of the Task committee on Slender Walls
1980 SCCACI - SEAOSC Task Committee

Panel No. 19
Date Tested 5/14/81
Material: 9.60 inch concrete

Reinf. Bars	4 No.4		As =	0.79	sq. in.	ρ =	0.0035
l_c =	288	in				ρ_{min} =	0.0030
f_y =	67.5	ksi					
f'_c =	4.0	ksi	n =	8.1			
E_s =	28,600	ksi					
E_c =	3540	ksi					
Eff d =	4.66	in					
A_{se} =	0.89	sq. in.	I_g =	3539	in^4		
$^2/_3 M_{cr}$ =	233	kip-in	$5\sqrt{f'c}$ =	316	psi		
M_n =	270	kip-in	a =	0.37	in		
I_{cr} =	130	in^4	c =	0.43	in		
Δ_n =	5.06	in	$^2/_3\Delta_{cr}$ =	0.16	in		

Adj. Load psf	Mid ht. Δ	Vert. Lc P_1	Wall Wt. P_2	wl_c^2x1.5	P_1e	$(P_1+P_2)\Delta$	M_{test}
11.2	0.02	1.28	6.08	38.6	10.0	0.2	49
15.6	0.01	1.28	6.08	53.9	10.0	0.1	64
20.8	0.02	1.28	6.08	71.9	10.0	0.2	82
31.2	0.05	1.28	6.08	107.8	10.0	0.4	118
41.6	0.06	1.28	6.08	143.8	10.0	0.5	154
52.0	0.07	1.28	6.08	179.7	10.0	0.5	190
62.4	0.09	1.28	6.08	215.7	10.0	0.6	226
67.4	0.23	1.28	6.08	233.0	10.0	1.7	245
72.6	0.23	1.28	6.08	251.0	10.0	1.7	263
75.1	3.22	1.28	6.08	259.5	10.0	23.7	293
76.6	4.10	1.28	6.08	264.6	10.0	30.2	305
77.5	5.31	1.28	6.08	267.8	10.0	39.1	317
78.8	6.54	1.28	6.08	272.4	10.0	48.1	331
79.6	7.74	1.28	6.08	275.0	10.0	57.0	342
78.3	8.64	1.28	6.08	270.6	10.0	63.6	344

SEAOSC - Slender Wall Task Group
Reference: Report of the Task committee on Slender Walls
1980 SCCACI - SEAOSC Task Committee

Panel No. 22
Date Tested 4/28/81
Material: 7.40 inch concrete

Reinf. Bars	4 No.4		As =	0.79	sq. in.	ρ =	0.0040
l_c =	288	in				ρ_{min} =	0.0030
f_y =	67.5	ksi					
f'_c =	4.0	ksi	n =	8.1			
E_s =	28,600	ksi					
E_c =	3540	ksi					
Eff d =	4.10	in					
A_{se} =	0.87	sq. in.	I_g =	1621	in^4		
$^2/_3 M_{cr}$ =	139	kip-in	$5\sqrt{f'c}$ =	316	psi		
M_n =	231	kip-in	a =	0.36	in		
I_{cr} =	96	in^4	c =	0.42	in		
Δ_n =	5.85	in	$^2/_3\Delta_{cr}$ =	0.21	in		

Adj. Load psf	Mid ht. Δ	Vert. Lc P_1	Wall Wt. P_2	wl_c^2x1.5	P_1e	$(P_1+P_2)\Delta$	M_{test}
10.4	0.03	1.28	4.69	35.9	8.6	0.2	45
15.6	0.09	1.28	4.69	53.8	8.6	0.6	63
20.8	0.09	1.28	4.69	71.8	8.6	0.5	81
26.0	0.09	1.28	4.69	89.7	8.6	0.5	99
31.2	0.10	1.28	4.69	107.7	8.6	0.6	117
36.4	0.14	1.28	4.69	125.6	8.6	0.8	135
36.3	0.35	1.28	4.69	125.3	8.6	2.1	136
41.3	0.51	1.28	4.69	142.9	8.6	3.1	154
45.6	2.16	1.28	4.69	157.7	8.6	12.9	179
49.7	3.46	1.28	4.69	171.8	8.6	20.7	201
53.6	5.27	1.28	4.69	185.3	8.6	31.4	225
54.6	7.17	1.28	4.69	188.6	8.6	42.8	240
55.4	9.04	1.28	4.69	191.4	8.6	54.0	254
56.1	11.33	1.28	4.69	193.8	8.6	67.6	270

Appendix

Table 6.4 – Summary of 1980 Test Panel Data (Panel Nos. 20 and 23)

SEAOSC - Slender Wall Task Group
Reference: Report of the Task committee on Slender Walls
1980 SCCACI - SEAOSC Task Committee

Panel No.	20						
Date Tested	5/12/81						
Material:	9.40	inch concrete					
Reinf. Bars	4 No.4		As =	0.79	sq. in.	ρ =	0.0035
l_c =	288	in				$\rho_{min.}$ =	0.0030
f_y =	67.5	ksi					
f'_c =	4.0	ksi	n =	8.1			
E_s =	28,600	ksi					
E_c =	3540	ksi					
Eff d =	4.70	in					
A_{se} =	0.89	sq. in.	I_g =	3322	in^4		
$^2/_3 M_{cr}$ =	224	kip-in	$5\sqrt{f'c}$ =	316	psi		
M_n =	272	kip-in	a =	0.37	in		
I_{cr} =	133	in^4	c =	0.43	in		
Δ_n =	5.01	in	$^2/_3\Delta_{cr}$ =	0.16	in		

Adj. Load	Mid ht.	Vert. Lc	Wall Wt.	wl_c^2x1.5	P_1e	$(P_1+P_2)\angle$	M_{tot}
psf	Δ	P_1	P_2				
10.4	0.02	1.28	5.95	35.9	9.9	0.1	46
15.6	0.04	1.28	5.95	53.9	9.9	0.3	64
20.8	0.06	1.28	5.95	71.9	9.9	0.4	82
26.0	0.07	1.28	5.95	89.7	9.9	0.5	100
31.2	0.08	1.28	5.95	107.7	9.9	0.6	118
36.4	0.09	1.28	5.95	125.6	9.9	0.6	136
41.5	0.05	1.28	5.95	143.6	9.9	0.3	154
46.7	0.13	1.28	5.95	161.5	9.9	0.9	172
51.9	0.15	1.28	5.95	179.5	9.9	1.1	190
57.1	0.17	1.28	5.95	197.2	9.9	1.2	208
62.2	0.17	1.28	5.95	215.1	9.9	1.2	226
67.4	0.19	1.28	5.95	233.0	9.9	1.4	244
72.6	0.22	1.28	5.95	251.0	9.9	1.6	262
74.2	0.22	1.28	5.95	256.3	9.9	1.6	268
73.9	1.15	1.28	5.95	255.6	9.9	8.3	274
75.6	2.69	1.28	5.95	261.1	9.9	19.5	290
77.6	3.16	1.28	5.95	268.1	9.9	22.8	301
78.0	5.06	1.28	5.95	269.6	9.9	36.6	316
76.7	6.14	1.28	5.95	265.1	9.9	44.4	319

SEAOSC - Slender Wall Task Group
Reference: Report of the Task committee on Slender Walls
1980 SCCACI - SEAOSC Task Committee

Panel No.	23						
Date Tested	4/29/81						
Material:	7.34	inch concrete					
Reinf. Bars	4 No.4		As =	0.79	sq. in.	ρ =	0.0050
l_c =	288	in				$\rho_{min.}$ =	0.0030
f_y =	67.5	ksi					
f'_c =	4.0	ksi	n =	8.1			
E_s =	28,600	ksi					
E_c =	3540	ksi					
Eff d =	3.29	in					
A_{se} =	0.87	sq. in.	I_g =	1582	in^4		
$^2/_3 M_{cr}$ =	136	kip-in	$5\sqrt{f'c}$ =	316	psi		
M_n =	183	kip-in	a =	0.36	in		
I_{cr} =	59	in^4	c =	0.42	in		
Δ_n =	7.56	in	$^2/_3\Delta_{cr}$ =	0.21	in		

Adj. Load	Mid ht.	Vert. Lc	Wall Wt.	wl_c^2x1.5	P_1e	$(P_1+P_2)\angle$	M_{tot}
psf	Δ	P_1	P_2				
10.4	0.06	1.28	4.65	35.9	8.5	0.3	45
15.6	0.07	1.28	4.65	53.9	8.5	0.4	63
20.8	0.12	1.28	4.65	71.8	8.5	0.7	81
25.9	0.20	1.28	4.65	89.6	8.5	1.2	99
31.1	0.31	1.28	4.65	107.4	8.5	1.8	118
36.2	0.42	1.28	4.65	125.2	8.5	2.5	136
41.3	0.53	1.28	4.65	142.9	8.5	3.1	155
45.6	2.13	1.28	4.65	157.7	8.5	12.7	179
46.9	4.09	1.28	4.65	162.2	8.5	24.3	195
49.1	4.65	1.28	4.65	169.6	8.5	27.6	206
50.8	5.63	1.28	4.65	175.7	8.5	33.4	218
52.6	6.88	1.28	4.65	181.6	8.5	40.8	231
52.7	9.63	1.28	4.65	182.1	8.5	57.1	248
52.9	10.81	1.28	4.65	182.7	8.5	64.1	255

Appendix

Table 6.5 – Summary of 1980 Test Panel Data (Panel Nos. 21 and 24)

SEAOSC - Slender Wall Task Group
Reference: Report of the Task committee on Slender Walls
1980 SCCACI - SEAOSC Task Committee

Panel No. 21
Date Tested 4/27/81
Material: 9.50 inch concrete

Reinf. Bars	4 No.4		As =	0.79	sq. in.	ρ =	0.0035
l_c =	288	in				$\rho_{min.}$ =	0.0030
f_y =	67.5	ksi					
f'_c =	4.0	ksi	n =	8.1			
E_s =	28,600	ksi					
E_c =	3540	ksi					
Eff d =	4.63	in					
A_{se} =	0.89	sq. in.	I_g =	3430	in^4		
$^2/_3\ M_{cr}$ =	228	kip-in	$5\sqrt{f'c}$ =	316	psi		
M_n =	268	kip-in	a =	0.37	in		
I_{cr} =	128	in^4	c =	0.43	in		
Δ_n =	5.10	in	$^2/_3\Delta_{cr}$ =	0.16	in		

Adj. Load psf	Mid ht. Δ	Vert. Lc P_1	Wall Wt. P_2	wl_c^2x1.5	P_1e	$(P_1+P_2)\Delta$	M_{tot}
10.4	0.01	1.28	6.02	35.9	9.9	0.1	46
15.6	0.01	1.28	6.02	53.9	9.9	0.1	64
20.8	0.04	1.28	6.02	71.9	9.9	0.3	82
26.0	0.05	1.28	6.02	89.9	9.9	0.3	100
31.2	0.06	1.28	6.02	107.8	9.9	0.4	118
36.4	0.07	1.28	6.02	125.6	9.9	0.5	136
41.5	0.08	1.28	6.02	143.6	9.9	0.6	154
46.7	0.09	1.28	6.02	161.5	9.9	0.6	172
51.9	0.10	1.28	6.02	179.5	9.9	0.7	190
57.1	0.11	1.28	6.02	197.4	9.9	0.8	208
62.3	0.12	1.28	6.02	215.4	9.9	0.9	226
67.5	0.13	1.28	6.02	233.3	9.9	1.0	244
72.7	0.15	1.28	6.02	251.3	9.9	1.1	262
76.5	1.66	1.28	6.02	264.5	9.9	12.1	287
77.8	3.91	1.28	6.02	268.9	9.9	28.5	307
75.9	6.94	1.28	6.02	262.4	9.9	50.6	323
65.0	7.16	1.28	6.02	224.5	9.9	52.2	287
74.9	8.09	1.28	6.02	258.8	9.9	59.0	328
75.1	9.98	1.28	6.02	259.5	9.9	72.8	342
75.7	11.43	1.28	6.02	261.6	9.9	83.4	355

SEAOSC - Slender Wall Task Group
Reference: Report of the Task committee on Slender Walls
1980 SCCACI - SEAOSC Task Committee

Panel No. 24
Date Tested 3/24/81
Material: 7.38 inch concrete

Reinf. Bars	4 No.4		As =	0.79	sq. in.	ρ =	0.0036
l_c =	288	in				$\rho_{min.}$ =	0.0030
f_y =	67.5	ksi					
f'_c =	4.0	ksi	n =	8.1			
E_s =	28,600	ksi					
E_c =	3540	ksi					
Eff d =	4.53	in					
A_{se} =	0.91	sq. in.	I_g =	1608	in^4		
$^2/_3\ M_{cr}$ =	138	kip-in	$5\sqrt{f'c}$ =	316	psi		
M_n =	265	kip-in	a =	0.37	in		
I_{cr} =	123	in^4	c =	0.44	in		
Δ_n =	5.24	in	$^2/_3\Delta_{cr}$ =	0.21	in		

Adj. Load psf	Mid ht. Δ	Vert. Lc P_1	Wall Wt. P_2	wl_c^2x1.5	P_1e	$(P_1+P_2)\Delta$	M_{tot}
9.9	0.02	3.44	4.68	34.1	23.0	0.2	57
14.7	-0.01	3.44	4.68	50.8	23.0	-0.1	74
19.4	0.00	3.44	4.68	66.9	23.0	0.0	90
24.6	0.03	3.44	4.68	84.9	23.0	0.3	108
29.6	0.06	3.44	4.68	102.4	23.0	0.5	126
35.2	0.16	3.44	4.68	121.6	23.0	1.3	146
39.9	0.18	3.44	4.68	137.8	23.0	1.5	162
44.9	0.25	3.44	4.68	155.1	23.0	2.0	180
49.0	1.26	3.44	4.68	169.3	23.0	10.2	202
51.7	4.95	3.44	4.68	178.6	23.0	40.2	242
53.4	7.95	3.44	4.68	184.4	23.0	64.5	272
51.8	11.93	3.44	4.68	179.1	23.0	96.8	299

Appendix

Table 6.6 – Summary of 1980 Test Panel Data (Panel Nos. 25 and 28)

SEAOSC - Slender Wall Task Group
Reference: Report of the Task committee on Slender Walls
1980 SCCACI - SEAOSC Task Committee

Panel No.	25						
Date Tested	4/27/81						
Material:	6.13 inch concrete						
Reinf. Bars	4 No.4		As =	0.79	sq. in.	$\rho =$	0.0044
$l_c =$	288	in				$\rho_{min.} =$	0.0030
$f_y =$	67.5	ksi					
$f'_c =$	4.0	ksi	$n =$	8.1			
$E_s =$	28,600	ksi					
$E_c =$	3540	ksi					
Eff d =	3.70	in					
$A_{se} =$	0.89	sq. in.	$I_g =$	921	in^4		
$^2/_3 M_{cr} =$	95	kip-in	$5\sqrt{f'c} =$	316	psi		
$M_n =$	212	kip-in	$a =$	0.37	in		
$I_{cr} =$	78	in^4	$c =$	0.43	in		
$\Delta_n =$	6.61	in	$^2/_3\Delta_{cr} =$	0.25	in		

Adj. Load psf	Mid ht. Δ	Vert. Ld P_1	Wall Wt. P_2	$wl_c^2x1.5$	P_1e	$(P_1+P_2)\Delta$	M_{test}
8.8	0.01	3.44	3.88	30.6	20.9	0.1	52
20.8	0.14	3.44	3.88	71.8	20.9	1.0	94
24.5	0.25	3.44	3.88	84.7	20.9	1.8	107
29.4	0.30	3.44	3.88	101.6	20.9	2.2	125
35.0	2.11	3.44	3.88	120.9	20.9	15.5	157
38.0	4.00	3.44	3.88	131.5	20.9	29.3	182
42.7	6.04	3.44	3.88	147.6	20.9	44.2	213
45.8	7.74	3.44	3.88	158.4	20.9	56.7	236
45.5	9.12	3.44	3.88	157.1	20.9	66.8	245
45.3	10.99	3.44	3.88	156.6	20.9	80.5	258
45.2	12.70	3.44	3.88	156.2	20.9	93.0	270

SEAOSC - Slender Wall Task Group
Reference: Report of the Task committee on Slender Walls
1980 SCCACI - SEAOSC Task Committee

Panel No.	28						
Date Tested	5/5/81						
Material:	4.82 inch concrete						
Reinf. Bars	4 No.4		As =	0.79	sq. in.	$\rho =$	0.0064
$l_c =$	288	in				$\rho_{min.} =$	0.0030
$f_y =$	67.5	ksi					
$f'_c =$	4.0	ksi	$n =$	8.1			
$E_s =$	28,600	ksi					
$E_c =$	3540	ksi					
Eff d =	2.57	in					
$A_{se} =$	0.85	sq. in.	$I_g =$	448	in^4		
$^2/_3 M_{cr} =$	59	kip-in	$5\sqrt{f'c} =$	316	psi		
$M_n =$	137	kip-in	$a =$	0.35	in		
$I_{cr} =$	33	in^4	$c =$	0.41	in		
$\Delta_n =$	10.16	in	$^2/_3\Delta_{cr} =$	0.32	in		

Adj. Load psf	Mid ht. Δ	Vert. Ld P_1	Wall Wt. P_2	$wl_c^2x1.5$	P_1e	$(P_1+P_2)\Delta$	M_{test}
2.6	0.00	1.28	3.05	9.0	6.9	0.0	16
5.2	0.01	1.28	3.05	18.0	6.9	0.0	25
7.8	0.02	1.28	3.05	27.0	6.9	0.1	34
10.4	0.05	1.28	3.05	35.9	6.9	0.2	43
13.0	0.08	1.28	3.05	44.9	6.9	0.3	52
15.6	0.16	1.28	3.05	53.8	6.9	0.7	61
18.2	0.23	1.28	3.05	62.7	6.9	1.0	71
20.6	0.61	1.28	3.05	71.3	6.9	2.6	81
20.8	2.27	1.28	3.05	71.8	6.9	9.8	89
22.4	3.86	1.28	3.05	77.3	6.9	16.7	101
22.9	5.34	1.28	3.05	79.2	6.9	23.1	109
23.9	6.76	1.28	3.05	82.6	6.9	29.3	119
24.8	7.97	1.28	3.05	85.7	6.9	34.5	127
26.3	9.01	1.28	3.05	90.9	6.9	39.0	137
27.3	10.19	1.28	3.05	94.3	6.9	44.2	145
27.4	12.20	1.28	3.05	94.7	6.9	52.9	154

Appendix

Table 6.7 – Summary of 1980 Test Panel Data (Panel Nos. 26 and 29)

SEAOSC - Slender Wall Task Group
Reference: Report of the Task committee on Slender Walls
1980 SCCACI - SEAOSC Task Committee

Panel No.	26						
Date Tested	3/18/81						
Material:	5.88	inch concrete					
Reinf. Bars	4 No.4		$As =$	0.79	sq. in.	$\rho =$	0.0045
$l_c =$	288	in				$\rho_{min.} =$	0.0030
$f_y =$	67.5	ksi					
$f'_c =$	4.0	ksi	$n =$	8.1			
$E_s =$	28,600	ksi					
$E_c =$	3540	ksi					
Eff d =	3.63	in					
$A_{se} =$	0.89	sq. in.	$I_g =$	813	in^4		
$^2/_3 M_{cr} =$	87	kip-in	$5\sqrt{f'c} =$	316	psi		
$M_n =$	207	kip-in	$a =$	0.37	in		
$I_{cr} =$	75	in^4	$c =$	0.43	in		
$\Delta_n =$	6.77	in	$^2/_3\Delta_{cr} =$	0.26	in		

Adj. Load psf	Mid ht. Δ	Vert. Ld P_1	Wall Wt. P_2	$wl_c^2x1.5$	P_1e	$(P_1+P_2)\Delta$	M_{test}
12.2	0.08	3.44	3.72	42.2	20.4	0.6	63
16.0	0.13	3.44	3.72	55.2	20.4	0.9	77
19.7	0.20	3.44	3.72	68.1	20.4	1.4	90
24.9	0.21	3.44	3.72	86.0	20.4	1.5	108
29.4	0.28	3.44	3.72	101.7	20.4	2.0	124
33.0	3.14	3.44	3.72	114.2	20.4	22.5	157
36.9	5.62	3.44	3.72	127.4	20.4	40.2	188
38.9	6.89	3.44	3.72	134.6	20.4	49.4	204
40.0	8.63	3.44	3.72	138.1	20.4	61.8	220
38.4	10.52	3.44	3.72	132.7	20.4	75.3	228

SEAOSC - Slender Wall Task Group
Reference: Report of the Task committee on Slender Walls
1980 SCCACI - SEAOSC Task Committee

Panel No.	29						
Date Tested	5/15/81						
Material:	4.78	inch concrete					
Reinf. Bars	4 No.4		$As =$	0.79	sq. in.	$\rho =$	0.0059
$l_c =$	288	in				$\rho_{min.} =$	0.0030
$f_y =$	67.5	ksi					
$f'_c =$	4.0	ksi	$n =$	8.1			
$E_s =$	28,600	ksi					
$E_c =$	3540	ksi					
Eff d =	2.78	in					
$A_{se} =$	0.85	sq. in.	$I_g =$	437	in^4		
$^2/_3 M_{cr} =$	58	kip-in	$5\sqrt{f'c} =$	316	psi		
$M_n =$	149	kip-in	$a =$	0.35	in		
$I_{cr} =$	39	in^4	$c =$	0.41	in		
$\Delta_n =$	9.23	in	$^2/_3\Delta_{cr} =$	0.32	in		

Adj. Load psf	Mid ht. Δ	Vert. Ld P_1	Wall Wt. P_2	$wl_c^2x1.5$	P_1e	$(P_1+P_2)\Delta$	M_{test}
5.2	0.11	1.28	3.03	17.9	6.9	0.5	25
10.4	0.22	1.28	3.03	35.9	6.9	0.9	44
15.5	0.45	1.28	3.03	53.6	6.9	2.0	62
19.4	1.75	1.28	3.03	67.0	6.9	7.5	81
20.3	2.25	1.28	3.03	70.1	6.9	9.7	87
21.1	3.84	1.28	3.03	73.0	6.9	16.6	96
22.1	4.85	1.28	3.03	76.3	6.9	20.9	104
21.8	5.88	1.28	3.03	75.3	6.9	25.3	108
22.7	6.46	1.28	3.03	78.4	6.9	27.8	113
23.7	7.37	1.28	3.03	82.0	6.9	31.7	121
25.4	9.14	1.28	3.03	87.7	6.9	39.4	134
25.2	9.52	1.28	3.03	87.1	6.9	41.0	135
26.9	11.33	1.28	3.03	93.0	6.9	48.8	149
26.8	13.69	1.28	3.03	92.6	6.9	59.0	159
26.8	15.83	1.28	3.03	92.7	6.9	68.2	168
26.6	18.13	1.28	3.03	91.9	6.9	78.1	177
31.7	19.30	1.28	3.03	109.6	6.9	83.2	200
35.7	19.60	1.28	3.03	123.3	6.9	84.4	215
39.8	19.82	1.28	3.03	137.5	6.9	85.4	230
42.9	20.37	1.28	3.03	148.3	6.9	87.8	243

Appendix

Table 6.8 – Summary of 1980 Test Panel Data (Panel Nos. 26 and 30)

SEAOSC - Slender Wall Task Group
Reference: Report of the Task committee on Slender Walls
1980 SCCACI - SEAOSC Task Committee

Panel No. 27
Date Tested 3/20/81
Material: 6.00 inch concrete

Reinf. Bars	4 No.4		As =	0.79	sq. in.	$\rho =$	0.0049
$l_c =$	288	in				$\rho_{min.} =$	0.0030
$f_y =$	67.5	ksi					
$f'_c =$	4.0	ksi	$n =$	8.1			
$E_s =$	28,600	ksi					
$E_c =$	3540	ksi					
Eff d =	3.35	in					
$A_{se} =$	0.86	sq. in.	$I_g =$	864	in^4		
$^2/_3 M_{cr} =$	91	kip-in	$5\sqrt{f'c}$	316	psi		
$M_n =$	184	kip-in	$a =$	0.36	in		
$I_{cr} =$	61	in^4	$c =$	0.42	in		
$\Delta_n =$	7.39	in	$^2/_3\Delta_{cr} =$	0.26	in		

Adj. Load psf	Mid ht. Δ	Vert. Ld P_1	Wall Wt. P_2	wl_c^2x1.5	P_1e	$(P_1+P_2)\Delta$	M_{test}
9.7	0.14	1.28	3.80	33.7	7.7	0.7	42
14.7	0.18	1.28	3.80	50.7	7.7	0.9	59
20.1	0.24	1.28	3.80	69.5	7.7	1.2	78
25.6	0.32	1.28	3.80	88.6	7.7	1.6	98
28.8	0.37	1.28	3.80	99.6	7.7	1.9	109
33.0	2.08	1.28	3.80	113.9	7.7	10.5	132
33.6	3.95	1.28	3.80	116.1	7.7	20.1	144
34.9	5.72	1.28	3.80	120.5	7.7	29.1	157
36.8	7.18	1.28	3.80	127.1	7.7	36.5	171
38.6	7.88	1.28	3.80	133.5	7.7	40.0	181
38.6	8.86	1.28	3.80	133.3	7.7	45.0	186

SEAOSC - Slender Wall Task Group
Reference: Report of the Task committee on Slender Walls
1980 SCCACI - SEAOSC Task Committee

Panel No. 30
Date Tested 5/14/81
Material: 4.89 inch concrete

Reinf. Bars	4 No.4		As =	0.79	sq. in.	$\rho =$	0.0064
$l_c =$	288	in				$\rho_{min.} =$	0.0030
$f_y =$	67.5	ksi					
$f'_c =$	4.0	ksi	$n =$	8.1			
$E_s =$	28,600	ksi					
$E_c =$	3540	ksi					
Eff d =	2.55	in					
$A_{se} =$	0.85	sq. in.	$I_g =$	468	in^4		
$^2/_3 M_{cr} =$	60	kip-in	$5\sqrt{f'c}$	316	psi		
$M_n =$	136	kip-in	$a =$	0.35	in		
$I_{cr} =$	32	in^4	$c =$	0.41	in		
$\Delta_n =$	10.25	in	$^2/_3\Delta_{cr} =$	0.32	in		

Adj. Load psf	Mid ht. Δ	Vert. Ld P_1	Wall Wt. P_2	wl_c^2x1.5	P_1e	$(P_1+P_2)\Delta$	M_{test}
5.2	0.10	1.28	3.10	17.9	7.0	0.4	25
10.4	0.20	1.28	3.10	35.9	7.0	0.9	44
13.0	0.30	1.28	3.10	44.8	7.0	1.3	53
15.5	0.41	1.28	3.10	53.6	7.0	1.8	62
17.7	2.36	1.28	3.10	61.3	7.0	10.3	79
18.8	4.04	1.28	3.10	64.9	7.0	17.7	90
19.6	4.70	1.28	3.10	67.8	7.0	20.6	95
21.0	6.41	1.28	3.10	72.6	7.0	28.0	108
22.9	7.44	1.28	3.10	79.0	7.0	32.6	119
24.2	8.47	1.28	3.10	83.5	7.0	37.1	128
25.4	9.15	1.28	3.10	87.7	7.0	40.0	135
27.1	10.58	1.28	3.10	93.7	7.0	46.3	147
28.5	11.83	1.28	3.10	98.5	7.0	51.8	157
29.0	13.54	1.28	3.10	100.1	7.0	59.3	166
29.8	14.64	1.28	3.10	103.0	7.0	64.1	174