ATTACHMENT 3

ENERCON SERVICES, INC.	ENGINEERING CALCULATION COVER SHEET	Calc. No. PGE-009-CALC-007
		Rev. 0
		Sheet 1 of 160
Title: ISFSI Cask Storage Pad Steel Reinforcement		Client: PG&E
		Job No. PGE-009

Purpose Of Calculation:

The purpose of this calculation is to compute the size and spacing of the steel reinforcement necessary to accommodate the forces and moments within the storage pad for the temperatures resulting from the heat of hydration during the curing process, from the shrinkage of the concrete and from the seismic demand. The ISFSI Facility will contain (7) pads, which will support (20) HI-Storm Storage Casks per pad. Some results from this Calculation will be used in Calculation No. PGE-009-CALC-001 to evaluate the anchor plate embedment capacity.

NOTE: This Calculation is furnished as part of PG&E Contract No. 4600010841, Change Order No. 001

Scope Of Revision:

Initial issue.

Revision Impact On Results:

N/A

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☒ Safety Related ☐ Non-Safety Related

☐ Preliminary Calculation ☒ Final

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ENCERCON SERVICES, INC.

ENGINEERING CALCULATION
REVISION STATUS SHEET

CALCULATION NO. PGE-009-CALC-007

ENGINEERING CALCULATION REVISION SUMMARY

REVISION NO.	DATE	DESCRIPTION
0	3/11/03	Initial Issue

CALCULATION SHEET REVISION STATUS

SHEET NO.	REVISION NO.	SHEET NO.	REVISION NO.
All	0	N/A	N/A

APPENDIX AND ATTACHMENT REVISION STATUS

APPENDIX NUMBER	ISSUE DATE	REV. DATE	REISSUE DATE	APPENDIX NUMBER	ISSUE DATE	REV. DATE	REISSUE DATE

Method of Review:

The calculation has been independently reviewed in accordance with the requirements of ENERCON Corporate Standard Procedure 3.01. The independent verification of the calculation was performed by a detailed review and check of the entire calculation. This included verification of inputs, methodology, results and conclusions as well as a check of the mathematical accuracy of the computations.

Results:

The calculation has been independently verified to be mathematically correct and to be performed in accordance with license and design basis requirements and applicable codes. Inputs are appropriate and are obtained from verified source documents. The calculation is sufficiently documented and detailed to permit independent verification. No assumptions are made other than conservative simplifying assumptions, which are identified and do not require confirmation.

The methodology used to evaluate the Cask Storage Pad Steel Reinforcement has been independently verified to be appropriate and consistent with design and code requirements. The design requirements specified in the reference documents have been adequately addressed and the code requirements have been appropriately considered. All applicable provisions of the design codes have been addressed and all design requirements have been met. The results are clearly identified and the conclusions are supported by the results of the calculation.

The steel reinforcement is capable of resisting all applied loads including those due to curing stresses and seismic loads. Furthermore, the steel reinforcing has been shown to be adequate to limit cracks during curing to widths that are within acceptable limits. Forces have been developed for use in design of the embedment structure.

Thus, the analysis has been independently verified to be technically correct and to be consistent with license and design basis requirements. The results and conclusions accurately reflect the findings of the calculation.

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Introduction

The purpose of this calculation is to size the reinforcement for the ISFSI storage pads. This calculation will therefore refer to the governing document for the design, which is the PG&E Specification, 10012-N-NPG (Reference 1) including its subordinate specifications and referenced regulatory requirements. The PG&E Specification is written to procure an entire Dry Cask Storage System which includes the casks, cask transfer equipment etc. as well as related structures. This calculation will only draw upon those requirements of the Specification that relate specifically to the ISFSI Storage Pad. Those requirements are specified in Section 6.4, Proposed ISFSI Facility, of Reference 1. The various requirements of Section 6.4 will be addressed as well as the various subordinate specifications and regulatory documents.

The hierarchy of the design requirement documents begins with the governing document for the work, which is the PG&E specification, Reference 1. This requires the cask supplier to develop a design criteria document, which is the Holtec Design Criteria, Reference 2. The Holtec document, in turn, invokes some regulatory and industry documents, as does the PG&E specification. These subordinate documents are NUREG 1536, NUREG 1567, USNRC DG 1098, ANSI 57.9 and ACI 349 (References 7, 8, 9, 10 and 11 respectively.) In some areas, the requirements overlap and are slightly different from one another. Further, the documentation that supports the design and demonstrates that the design complies with all the various criteria does not all reside in one complete document. Therefore, as much as possible, this calculation will compile all the criteria and identify where specific requirements have been met.

The implementing documents that demonstrate compliance with the criteria are the Holtec Cask analysis report (Reference 3) and the ENERCON calculations for the Embedment, Seismic Analysis of the pad/cask configuration, Thermal/Shrinkage Analysis of the pad (References 4, 5 and 6) and this calculation. The Holtec Cask analysis provides input loads for the Embedment and Pad Seismic Analyses. Further, the Holtec Cask analysis provides some "filtering" for the controlling loads. In addition, PG&E has had some analyses performed by others to fulfill the various aspects of the design criteria. Where this occurred, it is noted.

This calculation is organized into three main sections. The first is an assessment of the requirements of the PG&E Specification and its subordinate and referenced documents. This assessment provides a reference to the location of the documentation that demonstrates compliance to each requirement where that is known. It also sorts and assesses the load structural requirements, particularly, the load combinations specified by the design requirement documentation. The second is a calculation for the size of the reinforcement for the thermal and shrinkage demand that will be placed on the pad during construction. This reinforcement is then evaluated for the seismic demand in the third section.

References

1 PG&E Specification, 10012-N-NPG.
2 HOLTEC Report HI-2002511, Rev.2, Design Criteria Document for the ISFSI Pad for Anchored HI-Storm 100 Deployment at the Diablo Canyon Power Plant, dated July12, 2001.
3 HOLTEC Report HI-2012618, Rev. 5, Analysis of Anchored HI-Storm Casks at the Diablo Canyon ISFSI, dated December 11, 2001.
4 ENERCON Calculation PGE-009-CALC-001, Embedment Support Structure, latest revision.
5 ENERCON Calculation PGE-009-CALC-003, ISFSI Cask Storage Pad Seismic Analysis, latest revision.
6 ENERCON Calculation PGE-009-CALC-006, ISFSI Cask Storage Pad Concrete Shrinkage and Thermal Stresses, latest revision.
7 USNRC, NUREG 1536,: Standard Review Plan for Dry Cask Storage Systems
8 USNRC, NUREG 1567, Standard Review Plan for Spent Dry Fuel Storage Facilities
9 USNRC Draft Regulatory Guide DG-1098, Safety-Related Concrete Structures for Nuclear Power Plants (Other Than Reactor Vessels and Containments), dated August 2000.
10 American Nuclear Society, ANSI/ANS-57.9-1992, design criteria for an independent spent fuel storage installation (dry type)
11 ACI 349-97, Code Requirements for Nuclear Safety Related Concrete Structures
12 ACI 207.1R-96, Mass Concrete
13 ACI 207.R2-95, Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete.

Section 1 – Assessments of the Design Requirements

Design Requirements per PG&E Specification 10012-N-NPG Section 6.4

The various design requirements from the PG&E specification are presented below. All the requirements are listed for completeness. Those that are addressed elsewhere or those with no structural significance are noted. Those that are carried forward, to be addressed within this calculation, will also be noted. The requirements are listed in bullet form with the specific paragraph numbers noted in parentheses:

- (6.4.1) The supplier shall provide the design criteria for the pad. The supplier, in this case, is the cask storage supplier, Holtec. The design criteria document is Holtec Report HI-2002511, Reference 2. The requirements of the Holtec Design Criteria are discussed below.
- (6.4.2) Site Conditions
- (6.4.2.1) Location is already addressed on the site drawings.
- (6.4.2.2) Topography is already addressed on the site drawings.
- (6.4.2.3) Air Temperature and Marine Environment. The requirements are addressed in the thermal data calculation provided by PG&E. The PG&E calculation is Reference 2 of the ENERCON thermal calculation, Reference 6 herein. The finished concrete pad will be coated as required to provide protection from environmental conditions.

- (6.4.2.4) Precipitation. This has no structural significance.
- (6.4.2.5) Isolation. These requirements are really for cask cooling. There are no effects on the pad.
- (6.4.2.6) Geology. No requirements for the pad design.
- (6.4.2.7) Ground Water. No requirements for the pad design.
- (6.4.2.8) Liquefaction. No requirements for the pad design.
- (6.4.2.9) Slope Stability. This requirement is being addressed elsewhere.
- (6.4.3) ISFSI Site-Specific Design Requirements
- (6.4.3.1) Soil-Rock Stiffness. The soil stiffness properties have been addressed in the seismic analysis calculation (Reference 5). The engineered backfill will be a concrete pad. Thus, there is no impact on the seismic analyses.
- (6.4.3.2) Seismic Design Criteria. Refers to (6.2.5) for criteria. Those portions of 6.2.5 that are applicable to the storage pad are:
  - (6.2.5.1 IV) 10CFR72 Criteria. Qualify the pad for the four (4) seismic events provided in Appendix A. These are 1) the Design Earthquake (DE), Figures A.1.1 and A.1.2; 2) Double Design Earthquake (DDE), Figure A.1.3; 3) Hosgri Earthquake (HE), Figures A.1.4, A.1.5 and A.1.6; and 4) the Long-Term Seismic Program earthquake (LTSP), Figures A.1.7 and A.1.8. The load combinations and allowables shall be per NUREG-1536, NUREG-1567 and other applicable requirements. These requirements are discussed below.
  - (6.2.5.3 I) Damping Ratios shall be per the included Table. This is not applicable for the pad design since the seismic forces from the casks will be supplied by Holtec and the inertia force applied to the pad itself is the ZPA of the response spectrum.
  - (6.2.5.4) Static Analysis. Applicable for structures with fundamental frequencies above 33 Hz. Use the ZPA, this is addressed in Reference 5.
  - (6.2.5.5 V) Methods of Directional Combinations of Loads.
    - (A) Use the SRSS or 100-40-40 rule for HE and LTSP. Addressed in Reference 5.
    - (B) Use the 2D-ABSUM for DE and DDE. Not required to be addressed, see the discussion in the Reduction of the Load Combinations section.
  - (6.2.5.6 VI) Use the maximum interface loads between the cask and pad to design the pad. Addressed in Reference 5. The implementing document, Reference 3 provides a Table of applied loads.
- (6.4.3.2) Requires an evaluation if the supplied spectra could be invalidated by the specifics of the pad design. Addressed in Reference 2, see below.
- (6.4.3.3) Lateral Soil Pressure (H). The lateral soil pressure is negligible since the pad is massive and a solid. Using the specified weight of 130 pcf and the at rest coefficient of pressure of 0.5, the maximum soil pressure at a height of 8 feet is (0.5)(130)(8) = 520 psf = 3.6 psi, neglect.
- (6.4.3.4) Interface Friction Coefficient. This is the value to use for the friction between the bottom of the pad and the top of the rock. This issue is addressed by others for the seismic analyses, and it is addressed in the calculations supporting the data in Table 3 herein.
- (6.4.3.5) Bearing Pressure. Evaluated in Reference 5.
- (6.4.3.6) Soil Elastic Settlement Loads (Sc). Not applicable since there is no soil beneath the pads.
- (6.4.3.7) Rock Displacement Loads. No expected displacement beneath the pads.

- (6.4.4) ISFSI Layout and Design Criteria
- (6.4.4.1) General Requirements. No structural requirements
- (6.4.4.2 I D) Sequencing addressed in Reference 5.
- (6.4.4.2 II) Structural Criteria for the Pad
- (6.4.4.2 II A) Required for thickness of pad and reinforcement. The pad thickness is determined in Reference 5 that demonstrates that the thickness is adequate to resist all the specified applied loads.
- (6.4.4.2 II B) Required fill beneath the pad. A concrete construction pad will be used, see Reference 6.
- (6.4.4.2 II C i) Loads and calculations for drop and tip-over. Addressed in Reference 2, see below.
- (6.4.4.2 II C ii) Loads and calculations for stability. Addressed in Reference 2, see below.
- (6.4.4.2.II C iii) Loads and calculations for rigging. Not applicable to the pad
- (6.4.4.2 II C iv) Loads and calculations on pad due to sequencing. Addressed in Reference 5
- (6.4.4.2.II D) Detailed design and material specifications for the anchor system. Design is provided in Reference 4.
- (6.4.4.2 II E) Utility requirements. No structural requirements
- (6.4.5) Instrumentation and Control Requirements. No structural requirements
- (6.4.6) Electrical Requirements. No structural requirements

Though not specifically referenced in Section 6.4, the pad is also included in the requirements of Section 6.1, DCPP Requirements:

- (6.1.4 (f)) The Storage Pad is specifically included as part of the Storage System.
- (6.1.9.1) Loads, as applicable, for the Storage System (including the pad) shall be evaluated in accordance with NUREG-1536, NUREG-1567, ANSI 57.9 and other applicable requirements.
- Loads are:
  - I. Dead Loads
  - II. Live Loads
  - III. Thermal Loads
    - Normal operational
    - Off-normal
    - Accidental or Abnormal (due to limitation or loss of cooling air for an extended period of time)
  - IV. Pressure loads
  - V. Earthquake Loads
  - VI. Wind Loads
  - VII. Tornado Loads
  - VIII. Cask/MPC Handling and Onsite Transport Loads
  - IX. External Man-induced events
  - X. Blast and Tip-Over Events
  - XI. Soil Loads at the ISFSI Site:
    - Lateral soil pressure

- Soil reaction
- Burial under debris

The applicable pressure loads and handling loads are addressed by the Holtec design documents. The other loads are addressed in the Holtec Design Criteria (Reference 2) and subsequently in the Holtec Cask Analysis (Reference 3), as well as the ENERCON analyses, References 4, 5 and 6. Sizing of reinforcement for the applicable design load is provided below.

- (6.1.9.2) The 10CFR72 load combinations shall be as specified in NUREG-1536, NUREG-1567 and other applicable requirements. These will be addressed below.
- (6.1.10) Requires detailed reports
- (6.2) DCPP Site-Specific Design Requirements. Specifies the magnitude of loads and specific methodology to be used for the Storage System. The loads are:

  - (6.2.1) Wind. The pad itself is not subject to wind loads since it is below grade. The applied wind load on the storage casks is insignificant and bounded by the seismic loads, see Reference 3, Section 9.3.
  - (6.2.2) Tornado and tornado missiles. The pad itself is not subject to tornado loads since it is below grade. The seismic loads bound the tornado loads (tornado missile plus tornado wind), see Reference 3, Section 9.3.
  - (6.2.3) Tsunami. The pad is not subject to tsunami.
  - (6.2.4) Flood. The pad is not subject to flood.
  - (6.2.5) Earthquake. See presentation above.

The net result of the requirements of the specification as they relate to the structural evaluations performed here are 1) a design criteria document from the cask supplier, 2) loads and load combinations shall be per NUREG-1536, NUREG-1567 and other documents as applicable, 3) seismic loads per Appendix A of the Specification.

In response to the requirement of 6.4.1 of 10012-N-NPG, Holtec, the cask supplier, provided HI-2002511, Rev. 2, Design Criteria Document for the ISFSI Pad for Anchored HI-Storm 100 Deployment at the Diablo Canyon Power Plant, dated July12, 2001, Reference 2.

Design Requirements per Holtec Design Criteria Document, Reference 2

This document is written in a narrative form. The requirements will be listed as they appear in the document. Further they will be related to PG&E specification 10012-N-NPG (Reference 1) as applicable.

- (Section 2.0) Background. Tip over analysis not required since casks are anchored. Thus "soft pad" requirements are not applicable. Ref.1, (6.4.4.2 II C i).
- (Section 2.0) Background. Pad must be "...secured against excessive uplift (> 1/8" nominal) during extreme environmental events (viz., tornado missile, earthquakes, etc.". Ref. 1 (6.4.3.2). This is

interpreted to mean that that as long as the pad does not deflect more than 1/8 inch beneath the casks, the spectra used to analyze the storage system remain valid. The maximum pad displacements for any of the seismic events are as follows:

- Hosgri/LTSP seismic events, soft or hard rock, 0.101 inches anywhere, -0.045 inches at a cask. See Table 2, Reference 5.
- Hosgri/LTSP seismic events, soft rock, reduced concrete density, 0.112 inches anywhere, 0.0359 at a cask. See Table 13, Reference 5.
- Hosgri seismic events, soft rock, sequencing, all bounded by the basic load cases. See tables 2 and 14, Reference 5.
- Hosgri seismic events, soft rock, cask extraction, all bounded by the basic load cases. See Tables 2 and 15, Reference 5.
- Hosgri/LTSP seismic events, soft rock, reduced Poisson's ratio, 0.1023 inches anywhere, 0.0272 inches at a cask. See Table 16, Reference 5.

All displacements are computed to be less than the 1/8 inch; thus the spectra used are valid.

- The two key design objectives for the pad are (Reference 2):

  a. "The pad is sufficiently robust to withstand the overturning moments exerted on it by the cantilevered HI-STORMs due to destabilizing loadings such as the earthquake or a tornado missile. The overturning moments on the surface of the pad will act to cause local pad rotation that may need to be resisted by appropriately anchoring the pad to the subterranean half-space."
  b. "The anchored construction calls for a thick heavily reinforced pad, which means that the pad presents a stiff target to any object colliding with it. Because the maximum g-load that the MPC is permitted to sustain due to an impact event is limited by the HI-STROM FSAR, the handling device used to move and position the loaded cask on the pad shall be Single-Failure Proof. …"

- (Section 4.1) Design and Construction Criteria. The pad is categorized as a Category B Important-To-Safety Structure.
- (Section 4.1) Design and Construction Criteria. The pad does not need to be designed to accommodate cask tip-over since the casks are anchored. Ref. 1, (6.4.4.2 II C i) and (6.4.4.2 II C ii).
- (Section 4.1) Design and Construction Criteria. There is no need to perform a drop analysis and establish a lift height since the cask handling equipment meets NUREG-0612 criteria.
- (Section 4.2) Applicable Codes and Load Combinations. Factored load combinations for the pad design are to be from ACI-349-97, NRC DG 1098 and NUREG-1536.
- (Section 4.2 a) Holtec will supply earthquake loads to the pad from the casks. Pad designed to include pad self weight inertia terms. This issue is addressed in Reference 5.
- (Section 4.2.b) Load Combinations for the Concrete Pad. Notation and acceptance criteria of NUREG-1536 apply (see discussion of <u>ANSI/ANS 57.9</u> below for definitions):

Normal Events

Uc > 1.4D + 1.7L
Uc > 1.4D + 1.7 (L+H)

Off-Normal Events

Uc > 1.05D+1.275 (L+H+T)
Uc > 1.05D +1.275(L+H+T+W)

Accident-Level Events

Uc > D+L+H+T+(E or F)
Uc > D+L+H+Ta
Uc > D+L+H+T+Wt
Uc > D+L+H+T+A

These load combinations are addressed below.

The Design Criteria goes on to discuss overturning and sliding issues, which are being addressed by others.

- (Section 4.2.c) Load Combinations for the Anchor Studs. Addressed by Holtec
- (Section 4.2.d) Load Combinations for Embedment Structure. Addressed in Reference 4.
- (Section 4.3) Limiting Design Parameters. Embedment ductility. Addressed in Reference 4.
- (Section 4.3) Lift height limitation is not applicable since lifting devices are single-failure-proof.
- (Section 4.4) Additional Requirements.
- (Section 4.4 i) Interface. The details for the pad design and embedment structure are not in the HI-STORM FSAR
- (Section 4.4 ii) Applicable Code. The applicable code for the pad is ACI 349-97 or as specified by owner. Since 10012-N-NPG does not define a Code, the Code is ACI-349-97. Addressed below.
- (Section 4.4 iii) Grounding. No structural requirements
- (Section 4.4) There are no requirements in the CoC pertaining to cask load sequencing. Thus cask load sequence issues are left to the pad designer to evaluate. Sequencing is considered in Ref. 5.
- (Section 4.5) Provides general data for the design.
- (Section 4.6) Maximum Permissible Tornado Missile and Wind Load. Requires the cask designer to provide values but expects the seismic to control. Seismic bounds Wind and Tornado, including tornado missile, see Reference 3, Section 9.3.
- (Section 4.7) DCPP Cask Transporter Input Data. Provides dimensions and weights.
- (Section 5.1) General Comments. Requires the licensee to determine that the seismic and tornado loads are applicable for the site. These are specified in reference 1.

- (Section 5.2) General Requirements for the ISFSI Pad. These requirements are:

    1) Steel embedment to comply with ACI 349, and the 10/01/2000 Draft Appendix B. Addressed in Reference 4.
    2) Pad to comply with all structural requirements of NUREG-1567 and ACI 349. Addressed below.
    3) Pad/foundation qualified so that sliding will not have a significant impact on the cask qualification. This is being addressed elsewhere.
    4) Compression block/coupling in the embedment able to resist the maximum stud load provided by the cask designer. Addressed in Reference 4.

Design Requirements per ANSI/ANS-57.9 1992, Reference 10

ANSI/ANS-57.9 does not prescribe any detailed structural design requirements. Section 6.17 of ANSI 57.9 provides the design requirements. This section references ACI 349 and AISC N690 for load combinations but it does not specifically enforce them. The load definitions are:

- Normal Operating Loads
    - D - dead weight structure and attachments. Should be varied by +/- 5% to simulate the most severe response.
    - L - snow, rain, transient loads of equipment
    - T- thermal loads during operation, can be neglected if shown that the resultant stresses are secondary and self-limiting, and the response of the structure is ductile
    - H- lateral soil pressure

- Natural Phenomena Loads
    - E – Seismic, equivalent to the Safe Shutdown Earthquake
    - W - Design Basis Wind at a 100 year reoccurrence level
    - Wt - Design tornado – including wind pressure, differential pressure and missiles

- Off-Normal Operating and Accident Loads
    - Ta - Temperatures due to loss of cooling
    - A - Loads due to heavy load drop as described in ASCE Manual No. 58

Design Requirements per NUREG 1536, Reference 7

The structural design requirements per NUREG 1536 that are enforced by the PG&E design documents are the load combinations and allowable forces/stresses. These are provided in NUREG 1536 Table 3-1, Loads and load combinations:

Reinforced Concrete Structures

Normal Events and Conditions

Uc > 1.4D+1.7L
Uc > 1.4D+1.7(L+H)

Off-Normal Events and Conditions

Uc > 1.05D+1.275(L+H+T)
Uc > 1.05D+1.275(L+H+T+W)

Accidents and Conditions

Uc > D+L+H+T+(E or F)
Uc > D+L+H+T+A
Uc > D+L+H+Ta
Uc > D+L+H+T+Wt

Where Uc is the reinforced concrete available strength as computed from ACI 349 and the loads D, L etc are defined by ANSI 57.9 and F is the design basis flood. These load combinations are the same as specified in the Holtec Design Criteria Document and are addressed below.

<u>**Design Requirements per NUREG 1567, Reference 8**</u>

The structural design requirements per NUREG 1567 that are enforced by the PG&E design documents are the load combinations and allowable forces/stresses. These are provided in NUREG 1567 Table 7-1, Loads and load combinations

Steel and Reinforced Concrete Structures

Normal Events and Conditions

Uc > 1.4D+1.7L
Uc > 1.4D+1.7(L+H)

Off-normal Events and Conditions

Uc > 1.05D+1.275(L+H+T)
Uc > 1.05D+1.275(L+H+T+W)

Accident-Level Events and Conditions Missile concurrent with wind – same for 1536

Uc > D+L+H+T+E
Uc > D+L+H+T+A
Uc > D+L+H+Ta
Uc > D+L+H+T+Wt
Uc > D+L+H+T+F

The load combinations from NUREG 1536 and NUREG 1567 are the same. These load combinations are addressed below.

The Holtec Design Criteria document invokes Load Combinations of ACI 349-97, as modified by NRC DG 1098 (Reference 2, Section 4.2).

Load Combinations per ACI 349-97, Reference 11

The load combinations are specified in Chapter 9 of ACI-349:

1. U > 1.4D+1.4F+1.7L+1.7H+1.7Ro
2. U > 1.4D+1.4F+1.7L+1.7H+1.7Eo+1.7Ro
3. U > 1.4D+1.4F+1.7L+1.7H+1.7W+1.7Ro
4. U > D+F+L+H+To+Ro+Ess
5. U > D+F+L+H+To+Ro+Wt
6. U > D+F+L+H+Ta+Ra+1.25Pa
7. U > D+F+L+H+Ta+Ra+1.15Pa+1.0(Yr+Yj+Ym)+1.15Eo
8. U > D+F+L+H+Ta+Ra+1.0Pa+1.0(Yr+Yj+Ym)+1.0Ess
9. U > 1.05D+1.05F+1.3L+1.3H+1.05To+1.3Ro
10. U > 1.05D+1.05F+1.3L+1.3H+1.3Eo+1.05To+1.3Ro
11. U > 1.05D+1.05F+1.3L+1.3H+1.3W+1.05To+1.3Ro

The load definitions are provided in ACI 349. They are consistent with ANSI 57.9 where they overlap, however, ACI includes more loads since it is a general code and not developed specifically for ISFSI applications. These applicable load combinations are addressed below.

ACI also specifies a few additional requirements:

In Section 9.2.2, when the effects of differential settlement, creep or shrinkage may be significant, include them with D in combinations 4-11. The code requires a realistic assessment of such effects occurring in service. It is, therefore, not intended that the designer consider a "bounding" assessment of these effects. Thermal stresses and shrinkage are addressed below.

In Section 9.2.3, where a load reduces effects of other loads in any combination the coefficient shall be 0.9 if it can be shown the load is always there, otherwise use 0. Thus, the dead load must be reduced by 10% if it assists, rather than the 5% specified by ANSI 57.9. This is addressed in Reference 5 where the mass of the pad is reduced by 10%. The reduction applied to both the horizontal seismic acceleration loads on the pad as well as the vertical accelerations holding the pad down. The cask loads were not reduced. The displacements of the pad and casks due to a reduced concrete density increased slightly, however, the applied stresses decreased as shown in Reference 5. The displacements are acceptable, see above, and the internal forces from the basic analyses using normal weight concrete bound those using the reduced density concrete.

The reduced mass of the pad also needs to be evaluated relative to the tornado wind plus missile loads. The applied tornado wind plus missile load is 29,376 in kips (Reference 3, Section 9.3) Vs the maximum seismic load of 61,000 in kips (Reference 3, Table 3). Thus, since the pad is acceptable for the seismic load with the 10% reduction in mass, it is also acceptable for the tornado wind plus missile load since the seismic applied load is more than twice the tornado wind plus missile load.

The Holtec Design Criteria document invokes NRC DG 1098 (Reference 2, Section 4.2).

Assessment of NRC DG 1098, Reference 9

The draft regulatory guide accepts 349-97 with 15 additional regulatory positions. The only position that could effect the design is position 6, which modifies some of the load factors in the ACI 349 load combinations presented above. These modifications are:

Load Combinations 9, 10 and 11, use 1.2To rather than 1.05To
Load Combination 6, use 1.5Pa rather than 1.25Pa
Load Combination 7, use 1.25Pa rather than 1.15Pa

The ACI load combinations are repeated below with the DG 1098 changes reflected in **BOLD.**

1. U > 1.4D+1.4F+1.7L+1.7H+1.7Ro
2. U > 1.4D+1.4F+1.7L+1.7H+1.7Eo+1.7Ro
3. U > 1.4D+1.4F+1.7L+1.7H+1.7W+1.7Ro
4. U > D+F+L+H+To+Ro+Ess
5. U > D+F+L+H+To+Ro+Wt
6. U > D+F+L+H+Ta+Ra+**1.5**Pa
7. U > D+F+L+H+Ta+Ra+**1.25**Pa+1.0(Yr+Yj+Ym)+1.15Eo
8. U > D+F+L+H+Ta+Ra+1.0Pa+1.0(Yr+Yj+Ym)+1.0Ess
9. U > 1.05D+1.05F+1.3L+1.3H+**1.2**To+1.3Ro
10. U > 1.05D+1.05F+1.3L+1.3H+1.3Eo+**1.2**To+1.3Ro
11. U > 1.05D+1.05F+1.3L+1.3H+1.3W+**1.2**To+1.3Ro

Reduction of the Load Combinations

The load combinations specified by the Holtec Design Criteria, NUREG 1536 and NUREG 1567 are all the same. The ACI Code load combinations are significantly more complex. The ACI load combinations will be reduced first, followed by the NUREG 1567 Code load combinations.

The load combinations are assessed in light of the specification of the loads:

- F = 0 Flood (and all other potential fluid loads) are zero. See Reference 1, (6.2.4).

- H = 0 Soil pressures are not applicable for these analyses, see discussion of PG&E specification section (6.4.3.3).
- Ro and Ra are normal operating and accident equipment reactions other than the casks. The transporter is the only equipment load, and since its load is always down, i.e., it can not apply any moment to the pad, it does not have any effect on the pad design. The casks are treated as live loads.
- W is insignificant, see Reference 3, Section 9.3.
- To and Ta are normal operating and accident thermal forces. The bounding transient concrete temperature is specified in the Holtec Design Criteria (Reference 2, Section 4.5) to be 150 degrees F. This is considered to be below the threshold where explicit analyses for structural response are required. Further, the temperature of 150 degrees is the long-term general area limit, and is well below the short-term limit of 350 degrees per ACI 349 (see Reference 11, A.4.1 and A.4.2). Thus no material degradation or reduction in strength will occur.
- Pa is an accident pressure load and is not applicable for this analysis
- Yr, Yj and Ym are all associated with pipe break and are not applicable for this analysis.

Given the above definitions and applicability of loads, the ACI load combinations reduce to:

1. U > 1.4D+1.7L
2. U > 1.4D+1.7L+1.7Eo
3. U > 1.4D+1.7L
4. U > D+L+Ess
5. U > D+L+Wt
6. U > D+L
7. U > D+L+1.15Eo
8. U > D+L+1.0Ess
9. U > 1.05D+1.3L
10. U > 1.05D+1.3L+1.3Eo
11. U > 1.05D+1.3L

The Eo is the Design Earthquake and Ess is the most severe of the Double Design Earthquake (DDE), Hosgri (HE) or the Long Term Seismic Program (LTSP) earthquake. The HE, 7% damped curve, has a ZPA of 0.75G and a peak of approximately 1.8G (Reference 1, Figures A.1.4 and A.1.5). The damping values used in this discussion are from the Table in Section 6.2.5.3.I, of the PG&E Specification, Reference 1. The envelope of the DE, 5% damped curves, has a ZPA of 0.20G and a peak of approximately 0.76G (Reference 1, Figure A.1.3). Thus the DDE, 5% damped curve, has a ZPA of 0.40G and a peak of 1.52G. Comparing both the numerical values and the shape of the curves, it is clear that the HE, 7% damped curve, bounds the DDE curve, and therefore bounds the DE by more than a factor of 2.0.

Considering the load factors for the Eo load combinations, and the results of the reduced concrete density analyses described above, the use of 1.4D will result in a less demanding pad response than using just the dead weight. The same is true for the 1.7 factor on cask weight, L.

Further, Wt is bounded by the seismic responses, Reference 3, Section 9.3. Given the above the 11 equations further reduce to:

1. U > 1.4D+1.7L
4. U > D+L+Ess

Finally, given that equation 1 has no horizontal force component associated with it, it is acceptable by inspection, including the effect of the transporter. Hence the ACI 349 load combinations reduce to:

4. U > D+L+Ess

Where Ess is the most demanding of either the Hosgri or LTSP seismic events. An evaluation of the Hosgri curves and LTSP curves did not result in a clear governing seismic event. Therefore, both seismic events were explicitly evaluated, see References 3 and 5.

Now, using the above evaluation, the NUREG 1536 and 1567 load combinations also reduce to:

Steel and Reinforced Concrete Structures

Normal Events and Conditions
Uc > 1.4D+1.7L
Uc > 1.4D+1.7(L)

Off-normal Events and Conditions
Uc > 1.05D+1.275(L)
Uc > 1.05D+1.275(L)

Accident-Level Events and Conditions (Tornado missile concurrent with tornado wind)
Uc > D+L+E
Uc > D+L
Uc > D+L
Uc > D+L+Wt
Uc > D+L

And, applying the same arguments as above regarding D+L combinations and the Wt being bounded by seismic, the NUREG 1567 equations further reduce to ACI equation 4, where E is Ess.

Therefore, all the relevant structural criteria for the applied loads remaining to be satisfied are embodied in equation 4 above. The seismic demand is provided in Table 11, Reference 5. These are to be factored up by 1.15 to demonstrate compliance with the Design Codes and Specifications, see Reference 5, Summary and Conclusions.

Section 2 – Sizing of Reinforcement for Thermal and Shrinkage Demand

Thermal Demand due to Curing Temperatures and Shrinkage

In addition to the seismic evaluation required above, the only other significant structural demands on the pad are the thermal stresses/forces due to heat up during the curing process and shrinkage stresses/forces. The requirements are provided in ACI 349 Section 7.12. Since the pad is more than 72 inches thick, the reinforcement is proportioned using the provisions of ACI 207, (References 12 and 13), see ACI 349 Section 7.12.4. Also, the minimum reinforcement ratio shall be 0.0018 unless the area of reinforcement is 1/3 greater than that required by analysis, see ACI 349 Section 7.12.5. Rather than compute the thermal demand using the approach prescribed by ACI 207, the demands on the pad due to temperature and shrinkage were computed using a finite element model similar to that used to compute the seismic demand, Reference 6.

A review of the seismic analysis and the thermal/shrinkage analysis shows that the thermal/shrinkage demand is highly likely to control the design of the reinforcement. Thus, the design of the reinforcement is based on the thermal/shrinkage demand, using the results from Reference 6 and additional guidance as applicable from ACI 207. Once sized, the reinforcement is evaluated for the seismic demand in accordance with ACI 349.

Relevant requirements of ACI 349-97:

3.3.2 Maximum aggregate must be $\leq$ ¾ distance between bars

7.6.1 Minimum clear spacing between bars is db, but not less than 1 inch
7.6.2 Where parallel bars are used in two or more layers, the upper bars shall be placed directly above the lower bars with a clear distance not less than 1 inch.
7.6.4 The clear distance requirements apply to clear distance between contact splices and adjacent bars or splices.

7.7.1 Cover requirements: 3 inches for concrete cast against earth, 2 inches for concrete exposed to weather.

7.12.4 Concrete sections thicker than 72 inches should be designed using ACI 207
7.12.5 Where tension steel required, the minimum reinforcement ratio is 0.0018 unless the area of reinforcement provided is 1/3 greater than required by analysis.

21.2.2.3 Members below base shall also comply with Chapter 21
21.2.5.1 Reinforcement shall be ASTM 706 or ASTM 615 with caveats
21.2.6.1 Reinforcement may be welded. Alternate bars. May also be mechanically connected per 12.14.3.3 or 12.14.3.4

Initial Sizing of Reinforcement

The layout of the embedment structures will only permit a spacing of the top steel of 9 inches. Further, the geometry of the embedment structures and their proximity on the pad will not allow a reasonable lap splice arrangement for the reinforcement. Hence, the design will be developed using mechanical connectors. The connectors used in the design are Bar Lock "L-series" couplers; however, equivalent connectors can be used providing they meet the strength and geometric requirements. The connectors are 11.5 inches long; therefore at 9 inch spacing the traverse bars need to be far enough apart that the couplers can fit between them. Figures 1 and 2 show a layout for two layers of #10 bars with the couplers.

The layout provides for the specified cover to the surface of the coupler rather than the reinforcement. However, should this be relaxed and the coupler is allowed to be closer to the surface than the reinforcement cover, these calculations will still be valid.

Addressing some of the basic code requirements:

3.3.2 The maximum aggregate is 1½ inches. The minimum clear distance between parallel reinforcement is 2.9 inches. And, 2.9 > 4/3x1½. Therefore OK.
7.6.1 The 2.9 inches is > db (1.27) and > 1 inch. Therefore OK.
7.6.2 The top bars are placed directly above the lower bars. Therefore OK.
7.6.4 The location of the connectors will be staggered along the lengths of the bars. The clear distance between a connector and a bar is 1.27 + (2.90-1.27)/2 = 2.085 inches. And, 2.085 inches > 4/3x1½. Therefore OK.
7.12.6 Minimum reinforcement is 0.0018 x 9 x 96 = 1.555 square inches per 9" width. Conservative since full depth of pad is used, rather than d. Area steel of 2 #10 is 2x1.27 = 2.54 >1.55. Therefore OK without enforcing the 1/3 greater requirement.

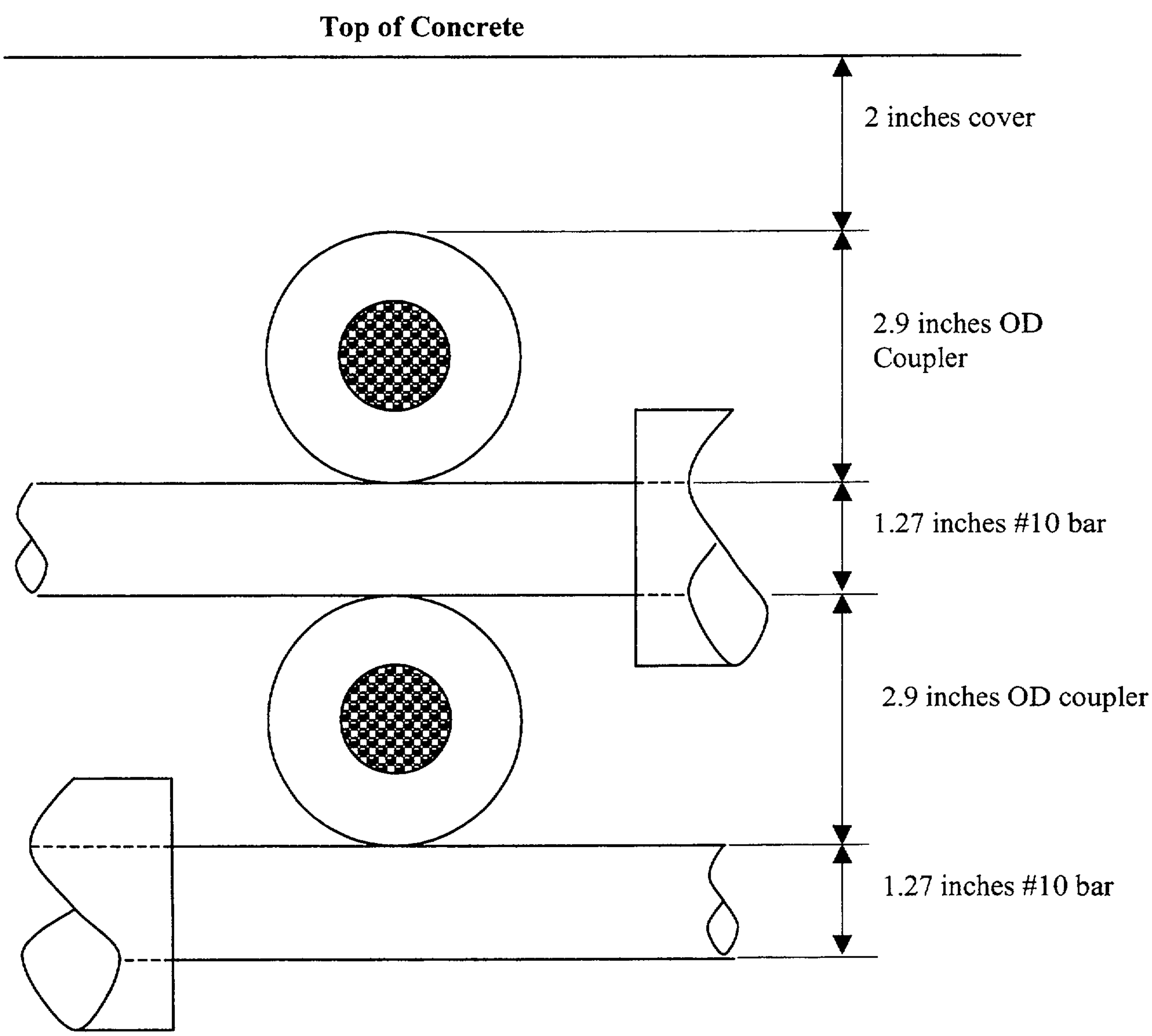


Figure 1 - Section of Top Steel – Looking N or S

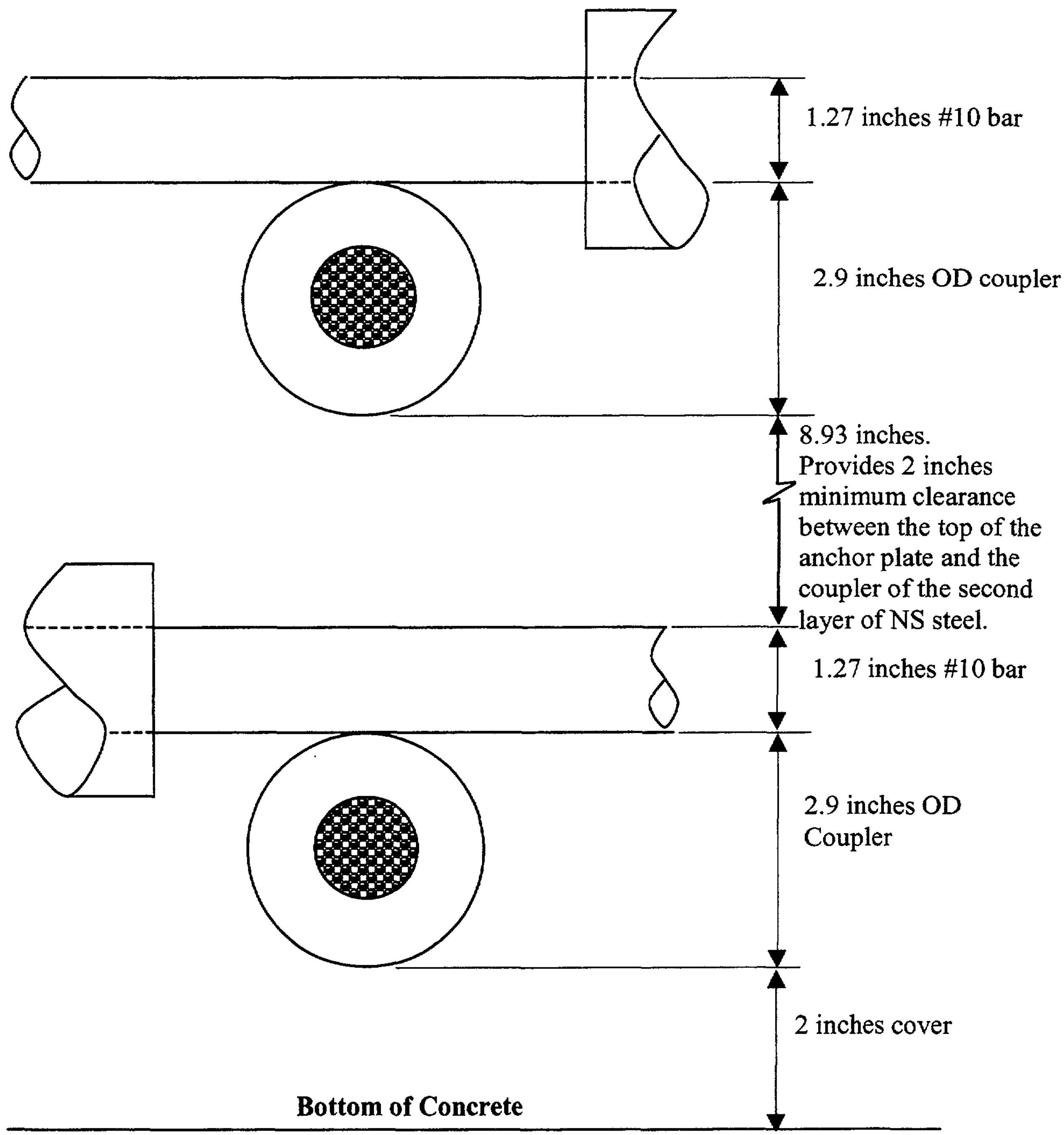
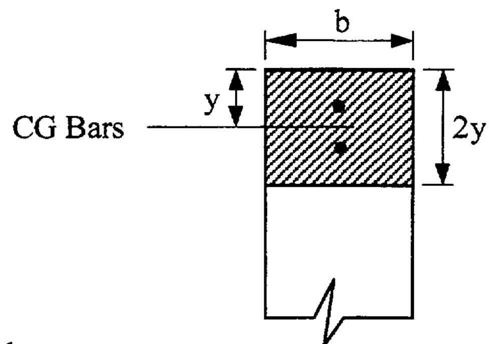


Figure 2 - Section of Bottom Steel – Looking N or S

Approximate Allowable Thermal and Shrinkage Stresses

The guidance from ACI 207.2, Reference 13, Chapter 5 discusses the issue of crack width vs. calculated steel stress. The following are approximate calculations to determine if the reinforcement arrangement and criteria are reasonable. The suggested acceptable crack width is 0.013 inches for exterior exposure. The relation between crack width and steel stress, is equation 5.1 from ACI 207:

$$w = 0.076\sqrt[3]{d_c A}\,\beta f_s 10^{-3}$$



Where:

- w is the crack width at surface
- d_c is the cover to the center of the bar
- A is the average effective concrete area around a bar = 2yb/N where N is the number of bars – see figure
- ß distance from neutral axis to the tensile face divided by the distance from neutral axis to the steel
- f_s is the calculated steel stress

For the NS top steel:

Now, allow w to equal 0.013 inches
d_c from the surface to the CG of the closest NS bar is 2+2.9/2 = 3.45 inches
A is 2 x (2+2.9+1.27/2) x 9/2 = 49.815 sq. in. per bar (see Figure 1 for dimensions)
Let ß equal approximately (96/2)/(96/2-(2+2.9+1.27/2)) = 1.13

Recasting the equation for with f_s as the unknown:

$$f_s = \frac{w}{0.076\sqrt[3]{d_c A}\,\beta 10^{-3}} = \frac{0.013}{0.076\sqrt[3]{3.45\text{x}49.815}\ \text{x}\ 1.13\text{x}10^{-3}} = 27.2 \text{ ksi for NS steel}$$

For the EW top steel:

d_c from the surface to the CG of the closest EW bar is 2+2.9+1.27/2 = 5.535 inches
A is 2 x (2+2.9+1.27+2.9/2) x 9/2 = 68.58 sq. in. per bar (see Figure 1 for dimensions)
Let ß equal approximately (96/2)/(96/2-(2+2.9+1.27+2.9/2)) = 1.19

$$f_s = \frac{w}{0.076\sqrt[3]{d_c A}\,\beta 10^{-3}} = \frac{0.013}{0.076\sqrt[3]{5.535 \times 68.58} \times 1.19 \times 10^{-3}} = 19.9 \text{ ksi for EW steel}$$

For the NS bottom steel:

Consider only the bottom bar, the second bar is too high into the concrete to consider.

Now, allow w to equal 0.013 inches
d_c from the surface to the CG of the closest NS bar is 2+2.9/2 = 3.45 inches
A is 2 x (2+2.9/2) x 9 = 62.1 sq. in. per bar (see Figure 2 for dimensions)
Let ß equal approximately (96/2)/(96/2-(2+2.9/2)) = 1.08

$$f_s = \frac{w}{0.076\sqrt[3]{d_c A}\,\beta 10^{-3}} = \frac{0.013}{0.076\sqrt[3]{3.45 \times 62.1} \times 1.08 \times 10^{-3}} = 26.5 \text{ ksi for NS steel}$$

For the EW bottom steel:

d_c from the surface to the CG of the closest EW bar is 2+2.9+1.27/2 = 5.535 inches
A is 2 x (2+2.9+1.27/2) x 9 = 99.63 sq. in. per bar (see Figure 2 for dimensions)
Let ß equal approximately (96/2)/(96/2-(2+2.9+1.27/2)) = 1.13

$$f_s = \frac{w}{0.076\sqrt[3]{d_c A}\,\beta 10^{-3}} = \frac{0.013}{0.076\sqrt[3]{5.535 \times 99.63} \times 1.13 \times 10^{-3}} = 18.5 \text{ ksi for EW steel}$$

Shrinkage forces from Reference 6, Table 14: (for the top 12 inches):

NS direction, sign convention: (-) indicates tension, see sign convention in Reference 5, forces adjusted from 17 feet (204 inches) to 9 inch bar spacing (where 2.54 is the area of 2 #10 bars):

$$f_{ns} = \frac{1510.694 \times 9/204}{2.54} = 26.2 \text{ ksi}$$

EW direction:

$$f_{ew} = \frac{1437.67 \times 9/204}{2.54} = 25.0 \text{ ksi}$$

The shrinkage forces are conservatively calculated because they do not include the stresses in the concrete at the bottom of the 12-inch layer. Thus it appears that the reinforcement arrangement and criteria are reasonable and warrant a more detailed assessment. The allowable stresses will be recomputed using more detailed information.

Detailed Evaluation for Thermal and Shrinkage Demands – Constrained Model Analyses

This bar arrangement will now be evaluated for moments and forces from the more detailed analysis results presented in Reference 6. First, the equations of equilibrium will be presented for the section. Since the reinforcement is not symmetric two sets of equilibrium equations will be developed: one where the sense of the moment produces tension on the bottom of the pad and one where the sense of the moment produces compression on the bottom of the pad. The equations are developed with the applied force and moment located at the mid-height of the pad since this is where the applied forces from the ANSYS analysis are computed, see Figures 3 and 4.

The thermal demand on the pad is computed using two bounds. The first analysis assumes that pad is constrained from sliding in any horizontal direction by the underlying rock. This is referred to as the "constrained model" in Reference 6. This is the most reasonable approximation to the actual expected behavior of the pad and it is the condition that is considered in the literature, see Reference 13. However, the potential that the pad may slide, locally, in some areas can not be denied. Thus, a second thermal model was analyzed in Reference 6 where the pad was allowed, analytically, to slide freely in any horizontal direction. This is referred to as the "unconstrained model" in Reference 6. The force results for the constrained model are provided in Table 9 of Reference 6, while the stress results for unconstrained model are provided in Table 11 and Figures 27 to 33 of Reference 6.

Since these assessments are focused on crack width estimation and to gain a sense for the actual pad stresses, the equations are developed using working stress methodology.

X or Z Strip Evaluation – moment produces tension on the bottom of the pad:

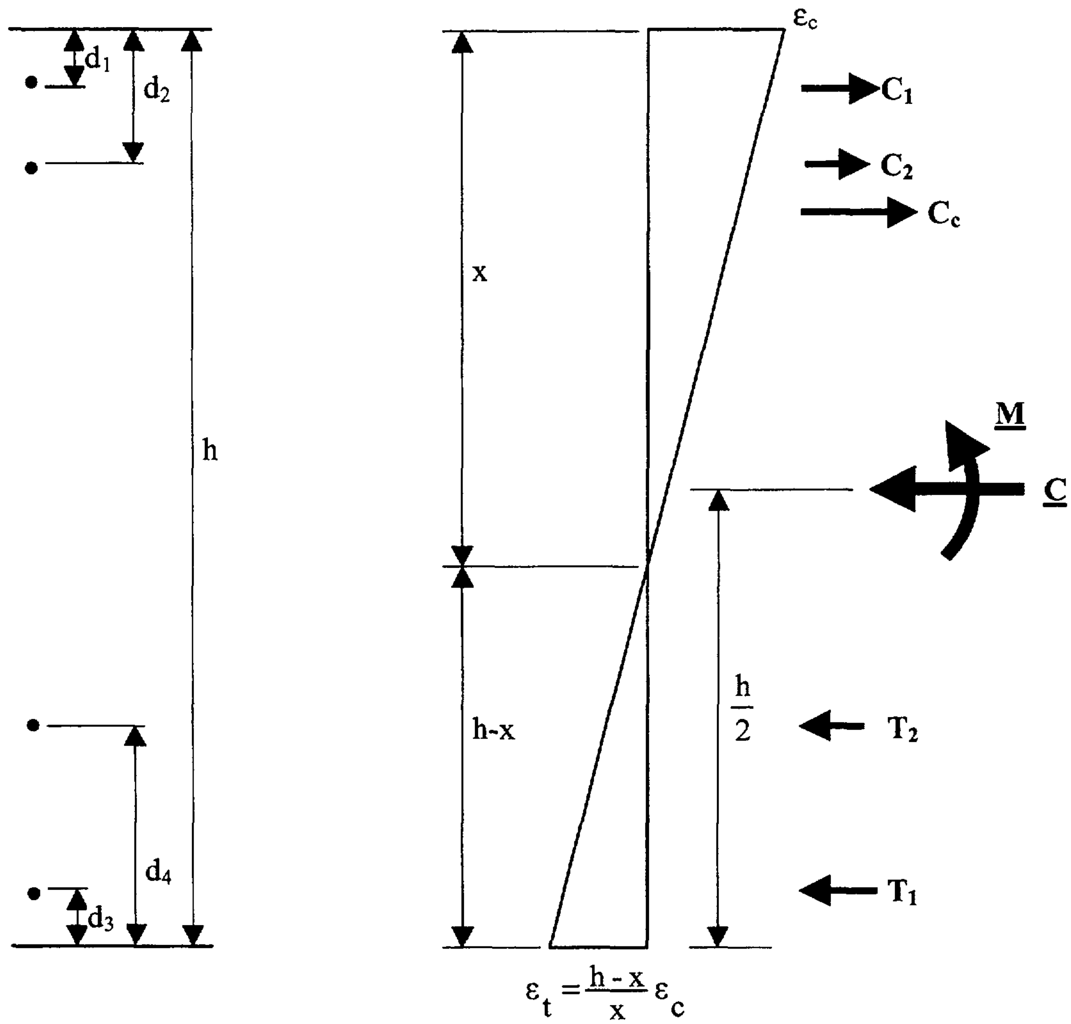


Figure 3 – Equilibrium for moment that produces tension on the bottom of the pad

Equilibrium equations:

The applied internal forces C and M are at h/2. Width of section is b.

$$\sum F = 0$$

$$\therefore\ C = C_c + C_1 + C_2 - T_1 - T_2$$

$$C_c = \frac{1}{2} xbE_c\varepsilon_c$$

$$C_1 = A_sE_s\left(\frac{x-d_1}{x}\right)\varepsilon_c$$

$$C_2 = A_sE_s\left(\frac{x-d_2}{x}\right)\varepsilon_c$$

$$T_1 = A_sE_s\left(\frac{h-x-d_3}{h-x}\right)\varepsilon_t = A_sE_s\left(\frac{h-x-d_3}{h-x}\right)\left(\frac{h-x}{x}\right)\varepsilon_c = A_sE_s\left(\frac{h-x-d_3}{x}\right)\varepsilon_c$$

And similarly,

$$T_2 = A_sE_s\left(\frac{h-x-d_4}{x}\right)\varepsilon_c$$

Substituting:

$$C = \frac{b}{2}E_c\varepsilon_c x + A_sE_s\left(\frac{x-d_1}{x}\right)\varepsilon_c + A_sE_s\left(\frac{x-d_2}{x}\right)\varepsilon_c - A_sE_s\left(\frac{h-x-d_3}{x}\right)\varepsilon_c - A_sE_s\left(\frac{h-x-d_4}{x}\right)\varepsilon_c$$

Gathering terms:

$$C = \frac{\varepsilon_c}{x}\left\{\frac{b}{2}E_cx^2 + A_sE_s\left[x-d_1+x-d_2-(h-x-d_3)-(h-x-d_4)\right]\right\}$$

Canceling terms:

$$C = \frac{\varepsilon_c}{x}\left\{\frac{b}{2}E_c x^2 + A_s E_s\left[4x - 2h - d_1 + d_3 - d_2 + d_4\right]\right\}$$

Therefore:

$$0 = \frac{\varepsilon_c}{x}\left\{\frac{b}{2}E_c x^2 + A_s E_s\left[4x - 2h + d_{net}\right]\right\} - C$$

Where:

$$d_{net} = -d_1 + d_3 - d_2 + d_4$$

Multiply by $x \neq 0$, and expanding the terms:

$$0 = \varepsilon_c\left\{\frac{b}{2}E_c x^2 + 4A_s E_s x - 2A_s E_s h + A_s E_s d_{net}\right\} - Cx$$

Now, bring the Cx term into the coefficient for x, and recognize that $\varepsilon_c \neq 0$,

$$0 = \varepsilon_c\left\{\frac{b}{2}E_c x^2 + \left(4A_s E_s - \frac{C}{\varepsilon_c}\right)x - 2A_s E_s h + A_s E_s d_{net}\right\}$$

Therefore the { } must equal 0, since $\varepsilon_c \neq 0$,

$$0 = \left\{\frac{b}{2}E_c x^2 + \left(4A_s E_s - \frac{C}{\varepsilon_c}\right)x - 2A_s E_s h + A_s E_s d_{net}\right\}$$

Now divide by $E_c \neq 0$,

$$0 = \left\{\frac{b}{2}x^2 + \left(4A_s\frac{E_s}{E_c} - \frac{C}{E_c\varepsilon_c}\right)x - 2A_s\frac{E_s}{E_c}h + A_s\frac{E_s}{E_c}d_{net}\right\}$$

Let $\alpha = \frac{E_s}{E_c}$, and recognize that $E_c\varepsilon_c = \sigma_c$, the concrete stress:

$$0 = \left\{\frac{b}{2}x^2 + \left(4A_s\alpha - \frac{C}{\sigma_c}\right)x - 2A_s\alpha h + A_s\alpha d_{net}\right\}$$

This is a quadratic for x, therefore:

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) \pm \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 - 4\frac{b}{2}(-2A_s\alpha h + A_s\alpha d_{net})}}{b}$$

Only the + sign for the radical makes physical sense, since the – sign will always lead to a negative value for x, since the radical will always be larger than the first term in the numerator. This is true because the sum of the terms $2h+d_{net}$ is always positive. ($d_{net} \geq -2h$). Therefore:

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{net})}}{b} \qquad \text{Equation (1)}$$

Also,

$$\sum M = 0$$

Therefore:

$$M = C_c\left(\frac{h}{2} - \frac{x}{3}\right) + C_1\left(\frac{h}{2} - d_1\right) + C_2\left(\frac{h}{2} - d_2\right) + T_1\left(\frac{h}{2} - d_3\right) + T_2\left(\frac{h}{2} - d_4\right) \qquad \text{Equation (2)}$$

X or Z Strip Evaluation – moment produces compression on the bottom of the pad

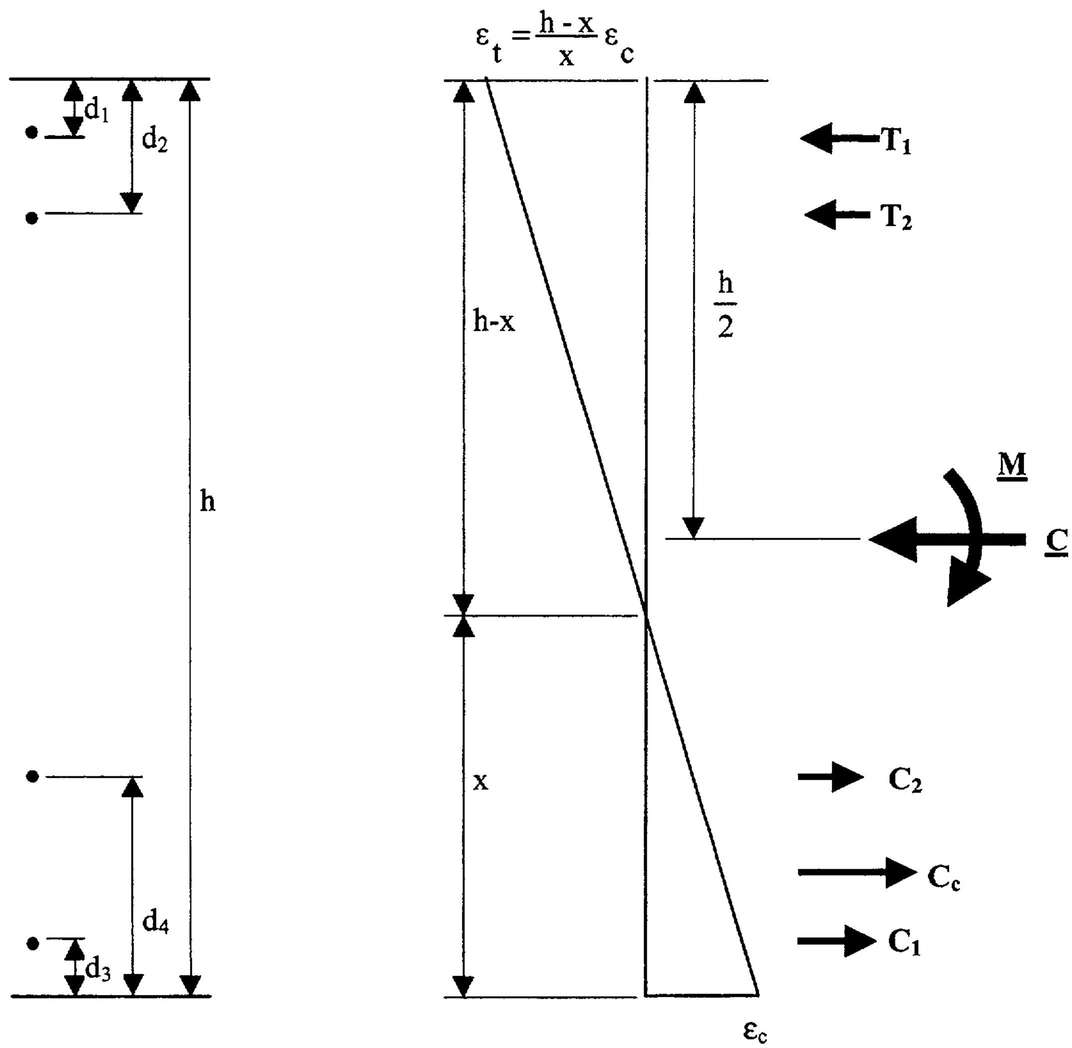


Figure 4 – Equilibrium for moment that produces compression on the bottom of the pad

Equilibrium equations:

The applied internal forces C and M are at h/2. Width of section is b.

$$\sum F = 0$$

$$\therefore\ C = C_c + C_1 + C_2 - T_1 - T_2$$

$$C_c = \frac{1}{2} x b E_c \varepsilon_c$$

$$C_1 = A_s E_s \left(\frac{x - d_3}{x}\right) \varepsilon_c$$

$$C_2 = A_s E_s \left(\frac{x - d_4}{x}\right) \varepsilon_c$$

$$T_1 = A_s E_s \left(\frac{h - x - d_1}{h - x}\right) \varepsilon_t = A_s E_s \left(\frac{h - x - d_1}{h - x}\right)\left(\frac{h - x}{x}\right) \varepsilon_c = A_s E_s \left(\frac{h - x - d_1}{x}\right) \varepsilon_c$$

And similarly,

$$T_2 = A_s E_s \left(\frac{h - x - d_2}{x}\right) \varepsilon_c$$

Substituting:

$$C = \frac{b}{2} E_c \varepsilon_c x + A_s E_s \left(\frac{x - d_3}{x}\right) \varepsilon_c + A_s E_s \left(\frac{x - d_4}{x}\right) \varepsilon_c - A_s E_s \left(\frac{h - x - d_1}{x}\right) \varepsilon_c - A_s E_s \left(\frac{h - x - d_2}{x}\right) \varepsilon_c$$

Gathering terms:

$$C = \frac{\varepsilon_c}{x} \left\{ \frac{b}{2} E_c x^2 + A_s E_s \left[x - d_3 + x - d_4 - (h - x - d_1) - (h - x - d_2) \right] \right\}$$

Canceling terms:

$$C = \frac{\varepsilon_c}{x}\left\{\frac{b}{2}E_c x^2 + A_s E_s\left[4x - 2h - d_3 - d_4 + d_1 + d_2\right]\right\}$$

Therefore:

$$0 = \frac{\varepsilon_c}{x}\left\{\frac{b}{2}E_c x^2 + A_s E_s\left[4x - 2h + d_{sum}\right]\right\} - C$$

Where:

$$d_{sum} = -d_3 - d_4 + d_1 + d_2$$

Multiply by $x \neq 0$, and expanding the terms:

$$0 = \varepsilon_c\left\{\frac{b}{2}E_c x^2 + 4A_s E_s x - 2A_s E_s h + A_s E_s d_{sum}\right\} - Cx$$

Now, bring the Cx term into the coefficient for x, and recognize that $\varepsilon_c \neq 0$,

$$0 = \varepsilon_c\left\{\frac{b}{2}E_c x^2 + \left(4A_s E_s - \frac{C}{\varepsilon_c}\right)x - 2A_s E_s h + A_s E_s d_{sum}\right\}$$

Therefore the { } must equal 0, since $\varepsilon_c \neq 0$,

$$0 = \left\{\frac{b}{2}E_c x^2 + \left(4A_s E_s - \frac{C}{\varepsilon_c}\right)x - 2A_s E_s h + A_s E_s d_{sum}\right\}$$

Now divide by $E_c \neq 0$,

$$0 = \left\{\frac{b}{2}x^2 + \left(4A_s \frac{E_s}{E_c} - \frac{C}{E_c \varepsilon_c}\right)x - 2A_s \frac{E_s}{E_c}h + A_s \frac{E_s}{E_c}d_{sum}\right\}$$

Let $\alpha = \frac{E_s}{E_c}$, and recognize that $E_c\varepsilon_c = \sigma_c$, the concrete stress:

$$0 = \left\{\frac{b}{2}x^2 + \left(4A_s\alpha - \frac{C}{\sigma_c}\right)x - 2A_s\alpha h + A_s\alpha d_{sum}\right\}$$

This is a quadratic for x, therefore:

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) \pm \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 - 4\frac{b}{2}\left(-2A_s\alpha h + A_s\alpha d_{sum}\right)}}{b}$$

Only the + sign for the radical makes physical sense, since the – sign will always lead to a negative value for x, since the radical will always be larger than the first term in the numerator. This is true because the sum of the terms $-2A_s\alpha h+A_s\alpha d_{sum}$ is always negative. ($d_{sum} \geq -2h$). Therefore:

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha\left(2h - d_{sum}\right)}}{b} \qquad \text{Equation (3)}$$

Also,

$$\sum M = 0$$

Therefore:

$$M = C_c\left(\frac{h}{2} - \frac{x}{3}\right) + C_1\left(\frac{h}{2} - d_3\right) + C_2\left(\frac{h}{2} - d_4\right) + T_1\left(\frac{h}{2} - d_1\right) + T_2\left(\frac{h}{2} - d_2\right) \qquad \text{Equation (4)}$$

Now, developing the equations for M and solving explicitly with the equations for C using x and σ_c as the unknowns will result in cubic equations for x. This becomes too complicated for an explicit solution. Therefore, the preferred method of solution is to just use the equation for C (Equation (1) or (3)) using an assumed value for σ_c to compute x. The values for x and the assumed value for σ_c are then used in Equations (2) or (4) to compute the internal moment. This is then compared to the applied moment. When the two values are close, the solution is considered to have converged. The equations were programmed using Excel. Only the final results are provided here.

Below is the evaluation of the section for the forces and moments due to curing from 1.125 days to 3.125 days. The sign convention established in Reference 5 and the signs used here are not the same. The sense of the moment as depicted in Figures 3 and 4 is the sense observed. The applied positive moment for the Z strips in Reference 5 produces tensile stresses on the bottom of the pad. However, the applied moments for the X strips in Reference 5 are positive when they produce compressive stresses on the bottom of the pad. This sign change results from the change in process where the transition is made from the finite element model, observing the right hand rule, to the concrete evaluation process.

Thus the sign conventions from Reference 5 will be interpreted using the sense of the applied moment shown on Figures 3 and 4. When the applied moment produces tensile stresses on the bottom of the pad Equations (1) and (2) will be utilized, and when the sense of the applied moment produces compressive stresses on the bottom of the pad Equations (3) and (4) will be used. The results are then tabulated in Table 1.

Numerical Evaluation: Time is 1.125 days NS (Z) Strip

b = 9 inches; h = 96 inches; d_1 = 3.45 inches; d_2 = 7.63 inches; d_3 = 3.45 inches; d_4 = 16.55 inches

A_s = 1.27 sq. in. #10 bar d_{sum} = -8.92

$f'_c = 1.054$ ksi Table 4, Reference 6:

E_c = 1964 ksi; E_s = 29000 ksi; $\alpha = \frac{29000}{1964} = 14.77$

$\sigma_c = 0.574$ ksi from off-line calculations; $\varepsilon_c = \frac{0.574}{1964} = 0.000292$ in/in

From Table 9 (4/11), Reference 6:

C = 4817.00 kip/17 foot section; C = 212.51 kip/9 inch section
M = 121000 in-kip/17 foot section; M = 5338.24 in-kip/9 inch section

Moment produces compression on the bottom of the pad, therefore use Equations (3) and (4).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \text{ x } 1.27 \text{ x } 14.77 - \frac{212.51}{0.574} = -295.22$$

$$2bA_s\alpha(2h - d_{sum}) = 2 \text{ x } 9 \text{ x } 1.27 \text{ x } 14.77 \text{ x } (2 \text{ x } 96 + 8.92) = 67819.71$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{sum})}}{b} = \frac{295.22 + \sqrt{-295.22^2 + 67819.71}}{9} = 76.54 \text{ inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{76.54 \text{ x } 9 \text{ x } 0.574}{2} = 197.71 \text{ kip}$$

$$C_1 = A_sE_s\left(\frac{x - d_3}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{76.54 - 3.45}{76.54}\right) \text{x } 0.000292 = 10.28 \text{ kip}$$

$$C_2 = A_s E_s \left(\frac{x - d_4}{x}\right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{76.54 - 16.55}{76.54}\right) \text{x } 0.000292 = 8.44 \text{ kip}$$

$$T_1 = A_s E_s \left(\frac{h - x - d_1}{x}\right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{96 - 76.54 - 3.45}{76.54}\right) \text{x } 0.000292 = 2.25 \text{ kip}$$

$$T_2 = A_s E_s \left(\frac{h - x - d_2}{x}\right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{96 - 76.54 - 7.63}{76.54}\right) \text{x } 0.000292 = 1.66 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$$\therefore \; C = C_c + C_1 + C_2 - T_1 - T_2$$

$$212.51 = 197.71 + 10.28 + 8.44 - 2.25 - 1.66 = 212.52 \text{ OK}$$

Now the internal moment is:

$$M = C_c\left(\frac{h}{2} - \frac{x}{3}\right) + C_1\left(\frac{h}{2} - d_3\right) + C_2\left(\frac{h}{2} - d_4\right) + T_1\left(\frac{h}{2} - d_1\right) + T_2\left(\frac{h}{2} - d_2\right) \quad \text{Equation (4)}$$

$$M = 197.71\left(\frac{96}{2} - \frac{76.54}{3}\right) + (10.28)\left(\frac{96}{2} - 3.45\right) + (8.44)\left(\frac{96}{2} - 16.55\right) + \ldots$$

$$(2.25)\left(\frac{96}{2} - 3.45\right) + (1.66)\left(\frac{96}{2} - 7.63\right) = 5336.31 \cong 5338.24 \text{ OK}$$

Concrete compressive stress of 0.574 ksi is 54% of f_c' and the maximum tensile steel stress is $\frac{2.25}{1.27} = 1.77$ ksi, therefore OK.

Numerical Evaluation: **Time is 1.125 days EW (X) Strip**

b = 9 inches; h = 96 inches; d_1 = 5.535 inches; d_2 = 9.705 inches; d_3 = 5.535 inches; d_4 = 18.635 inches

A_s = 1.27 sq. in. #10 bar d_{sum} = -8.93

$f_c' = 1.054$ ksi Table 4, Reference 6:

E_c = 1964 ksi; E_s = 29000 ksi; $\alpha = \frac{29000}{1964} = 14.77$

σ_c = 0.582 ksi from off-line calculations; $\varepsilon_c = \frac{0.582}{1964} = 0.000296$ in/in

From Table 9 (4/11), Reference 6:

C = 4319.608 kip/17 foot section; C = 190.57 kip/9 inch section
M = 123000 in-kip/17 foot section; M = 5426.47 in-kip/9 inch section

Moment produces compression on the bottom of the pad, therefore use Equations (3) and (4).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \text{ x } 1.27 \text{ x } 14.77 - \frac{190.57}{0.582} = -252.43$$

$$2bA_s\alpha(2h - d_{sum}) = 2 \text{ x } 9 \text{ x } 1.27 \text{ x } 14.77 \text{ x } (2 \text{ x } 96 + 8.93) = 67823.08$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{sum})}}{b} = \frac{252.43 + \sqrt{-252.43^2 + 67823.08}}{9} = 68.35 \text{ inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{68.35 \text{ x } 9 \text{ x } 0.582}{2} = 179.00 \text{ kip}$$

$$C_1 = A_sE_s\left(\frac{x - d_3}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{68.35 - 5.535}{68.35}\right) \text{x } 0.000296 = 10.03 \text{ kip}$$

$$C_2 = A_s E_s \left(\frac{x - d_4}{x} \right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{68.35 - 18.635}{68.35} \right) \text{x } 0.000296 = 7.94 \text{ kip}$$

$$T_1 = A_s E_s \left(\frac{h - x - d_1}{x} \right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{96 - 68.35 - 5.535}{68.35} \right) \text{x } 0.000296 = 3.53 \text{ kip}$$

$$T_2 = A_s E_s \left(\frac{h - x - d_2}{x} \right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{96 - 68.35 - 9.705}{68.35} \right) \text{x } 0.000296 = 2.87 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$\therefore\ C = C_c + C_1 + C_2 - T_1 - T_2$

$$190.57 = 179.00+10.03+7.94-3.53-2.87 = 190.57 \text{ OK}$$

Now the internal moment is:

$$M = C_c\left(\frac{h}{2} - \frac{x}{3}\right) + C_1\left(\frac{h}{2} - d_3\right) + C_2\left(\frac{h}{2} - d_4\right) + T_1\left(\frac{h}{2} - d_1\right) + T_2\left(\frac{h}{2} - d_2\right) \qquad \text{Equation (4)}$$

$$M = 179.00\left(\frac{96}{2} - \frac{68.35}{3}\right) + (10.03)\left(\frac{96}{2} - 5.535\right) + (7.94)\left(\frac{96}{2} - 18.635\right) + \ldots$$

$$(3.53)\left(\frac{96}{2} - 5.535\right) + (2.87)\left(\frac{96}{2} - 9.705\right) = 5432.77 \cong 5426.5 \text{ OK}$$

Concrete compressive stress of 0.582 ksi is 55% of $f_c^{'}$ and the maximum tensile steel stress is $\frac{3.53}{1.27} = 2.78 \text{ ksi}$, therefore OK.

Numerical Evaluation: Time is 1.125 days Shear NS (Z) and EW (X) Strips

Max Fy is 21.692 kip, Table 9 (4/11), Reference 6, largest of NS or EW values.

$$v_c = \frac{21.692}{204 \text{ x } (96 - 9.705)} = 0.00123 \text{ ksi} = 1.23 \text{ psi} \quad \text{OK by inspection.}$$

Numerical Evaluation: Time is 1.625 days NS (Z) Strip

b = 9 inches; h = 96 inches; d_1 = 3.45 inches; d_2 = 7.63 inches; d_3 = 3.45 inches; d_4 = 16.55 inches

A_s = 1.27 sq. in. #10 bar d_{sum} = -8.92

$f'_c = 1.523$ ksi Table 4, Reference 6:

E_c = 2358 ksi; E_s = 29000 ksi; $\alpha = \frac{29000}{2358} = 12.30$

$\sigma_c = 0.724$ ksi from off-line calculations; $\varepsilon_c = \frac{0.724}{2358} = 0.000307$ in/in

From Table 9 (5/11), Reference 6:

C = 5390.02 kip/17 foot section; C = 237.79 kip/9 inch section
M = 151000 in-kip/17 foot section; M = 6661.77 in-kip/9 inch section

Moment produces compression on the bottom of the pad, therefore use Equations (3) and (4).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \text{ x } 1.27 \text{ x } 12.30 - \frac{237.79}{0.724} = -265.969$$

$$2bA_s\alpha(2h - d_{sum}) = 2 \text{ x } 9 \text{ x } 1.27 \text{ x } 12.30 \text{ x } (2 \text{ x } 96 + 8.92) = 56487.66$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{sum})}}{b} = \frac{265.969 + \sqrt{-265.969^2 + 56487.66}}{9} = 69.18 \text{ inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{69.18 \text{ x } 9 \text{ x } 0.724}{2} = 225.40 \text{ kip}$$

$$C_1 = A_sE_s\left(\frac{x - d_3}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{69.18 - 3.45}{69.18}\right) \text{x } 0.000307 = 10.74 \text{ kip}$$

$$C_2 = A_s E_s \left(\frac{x - d_4}{x} \right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{69.18 - 16.55}{69.18} \right) \text{x } 0.000307 = 8.60 \text{ kip}$$

$$T_1 = A_s E_s \left(\frac{h - x - d_1}{x} \right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{96 - 69.18 - 3.45}{69.18} \right) \text{x } 0.000307 = 3.82 \text{ kip}$$

$$T_2 = A_s E_s \left(\frac{h - x - d_2}{x} \right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{96 - 69.18 - 7.63}{69.18} \right) \text{x } 0.000307 = 3.14 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$\therefore \; C = C_c + C_1 + C_2 - T_1 - T_2$

$$237.79 = 225.40 + 10.74 + 8.60 - 3.82 - 3.14 = 237.78 \text{ OK}$$

Now the internal moment is:

$$M = C_c\left(\frac{h}{2} - \frac{x}{3}\right) + C_1\left(\frac{h}{2} - d_3\right) + C_2\left(\frac{h}{2} - d_4\right) + T_1\left(\frac{h}{2} - d_1\right) + T_2\left(\frac{h}{2} - d_2\right) \qquad \text{Equation (4)}$$

$$M = 225.40\left(\frac{96}{2} - \frac{69.18}{3}\right) + (10.74)\left(\frac{96}{2} - 3.45\right) + (8.60)\left(\frac{96}{2} - 16.55\right) + \ldots$$

$$(3.82)\left(\frac{96}{2} - 3.45\right) + (3.14)\left(\frac{96}{2} - 7.63\right) = 6667.19 \cong 6661.77 \text{ OK}$$

Concrete compressive stress of 0.724 ksi is 48% of $f_c^{'}$ and the maximum tensile steel stress is $\frac{3.82}{1.27} = 3.01 \text{ ksi}$, therefore OK.

Numerical Evaluation: Time is 1.625 days EW (X) Strip

b = 9 inches; h = 96 inches; d_1 = 5.535 inches; d_2 = 9.705 inches; d_3 = 5.535 inches; d_4 = 18.635 inches

A_s = 1.27 sq. in. #10 bar d_{sum} = -8.93

$f_c^{'} = 1.523$ ksi Table 4, Reference 6:

$E_c = 2358$ ksi; $E_s = 29000$ ksi; $\alpha = \frac{29000}{2358} = 12.30$

$\sigma_c = 0.728$ ksi from off-line calculations; $\varepsilon_c = \frac{0.728}{2358} = 0.000309$ in/in

From Table 9 (5/11), Reference 6:

C = 4877.105 kip/17 foot section; C = 215.17 kip/9 inch section
M = 150000 in-kip/17 foot section; M = 6617.65 in-kip/9 inch section

Moment produces compression on the bottom of the pad, therefore use Equations (3) and (4).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \text{ x } 1.27 \text{ x } 12.30 - \frac{215.17}{0.728} = -233.08$$

$$2bA_s\alpha(2h - d_{sum}) = 2 \text{ x } 9 \text{ x } 1.27 \text{ x } 12.30 \text{ x } (2 \text{ x } 96 + 8.93) = 56490.47$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{sum})}}{b} = \frac{233.08 + \sqrt{-233.08^2 + 56490.47}}{9} = 62.89 \text{ inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{62.89 \text{ x } 9 \text{ x } 0.728}{2} = 206.01 \text{ kip}$$

$$C_1 = A_sE_s\left(\frac{x - d_3}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{62.89 - 5.535}{62.89}\right) \text{x } 0.000309 = 10.37 \text{ kip}$$

$$C_2 = A_s E_s \left(\frac{x - d_4}{x}\right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{62.89 - 18.635}{62.89}\right) \text{x } 0.000309 = 8.00 \text{ kip}$$

$$T_1 = A_s E_s \left(\frac{h - x - d_1}{x}\right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{96 - 62.89 - 5.535}{62.89}\right) \text{x } 0.000309 = 4.99 \text{ kip}$$

$$T_2 = A_s E_s \left(\frac{h - x - d_2}{x}\right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{96 - 62.89 - 9.705}{62.89}\right) \text{x } 0.000309 = 4.23 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$$\therefore C = C_c + C_1 + C_2 - T_1 - T_2$$

$$215.17 = 206.01 + 10.37 + 8.00 - 4.99 - 4.23 = 215.16 \text{ OK}$$

Now the internal moment is:

$$M = C_c\left(\frac{h}{2} - \frac{x}{3}\right) + C_1\left(\frac{h}{2} - d_3\right) + C_2\left(\frac{h}{2} - d_4\right) + T_1\left(\frac{h}{2} - d_1\right) + T_2\left(\frac{h}{2} - d_2\right) \qquad \text{Equation (4)}$$

$$M = 206.01\left(\frac{96}{2} - \frac{62.89}{3}\right) + (10.37)\left(\frac{96}{2} - 5.535\right) + (8.00)\left(\frac{96}{2} - 18.635\right) + \ldots$$

$$(4.99)\left(\frac{96}{2} - 5.535\right) + (4.23)\left(\frac{96}{2} - 9.705\right) = 6619.39 \cong 6617.65 \text{ OK}$$

Concrete compressive stress of 0.728 ksi is 48% of f_c' and the maximum tensile steel stress is $\frac{4.99}{1.27} = 3.93 \text{ ksi}$, therefore OK.

Numerical Evaluation: Time is 1.625 days Shear NS (Z) and EW (X) Strips

Max Fy is 22.181 kip, Table 9 (5/11), Reference 6, largest of NS or EW values.

$$v_c = \frac{22.181}{204 \text{ x } (96 - 9.705)} = 0.00126 \text{ ksi} = 1.26 \text{ psi} \quad \text{OK by inspection.}$$

Numerical Evaluation: **Time is 2.125 days** **NS (Z) Strip**

b = 9 inches; h = 96 inches; d_1 = 3.45 inches; d_2 = 7.63 inches; d_3 = 3.45 inches; d_4 = 16.55 inches

A_s = 1.27 sq. in. #10 bar d_{sum} = -8.92

$f_c' = 1.961$ ksi Table 4, Reference 6:

E_c = 2674 ksi; E_s = 29000 ksi; $\alpha = \frac{29000}{2674} = 10.85$

σ_c = 0.794 ksi from off-line calculations; $\varepsilon_c = \frac{0.794}{2674} = 0.000297$ in/in

From Table 9 (6/11), Reference 6:

C = 5574.946 kip/17 foot section; C = 245.95 kip/9 inch section
M = 163000 in-kip/17 foot section; M = 7191.18 in-kip/9 inch section

Moment produces compression on the bottom of the pad, therefore use Equations (3) and (4).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \text{ x } 1.27 \text{ x } 10.85 - \frac{245.95}{0.794} = -254.672$$

$$2bA_s\alpha(2h - d_{sum}) = 2 \text{ x } 9 \text{ x } 1.27 \text{ x } 10.85 \text{ x } (2 \text{ x } 96 + 8.92) = 49812.23$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{sum})}}{b} = \frac{254.672 + \sqrt{-254.672^2 + 49812.23}}{9} = 65.92 \text{ inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{65.92 \text{ x } 9 \text{ x } 0.794}{2} = 235.54 \text{ kip}$$

$$C_1 = A_sE_s\left(\frac{x - d_3}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{65.92 - 3.45}{65.92}\right) \text{ x } 0.000297 = 10.36 \text{ kip}$$

$$C_2 = A_s E_s \left(\frac{x - d_4}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{65.92 - 16.55}{65.92}\right) \text{x } 0.000297 = 8.19 \text{ kip}$$

$$T_1 = A_s E_s \left(\frac{h - x - d_1}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{96 - 65.92 - 3.45}{65.92}\right) \text{x } 0.000297 = 4.42 \text{ kip}$$

$$T_2 = A_s E_s \left(\frac{h - x - d_2}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{96 - 65.92 - 7.63}{65.92}\right) \text{x } 0.000297 = 3.72 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$$\therefore\ C = C_c + C_1 + C_2 - T_1 - T_2$$

$$245.95 = 235.54 + 10.36 + 8.19 - 4.42 - 3.72 = 245.95 \text{ OK}$$

Now the internal moment is:

$$M = C_c\left(\frac{h}{2} - \frac{x}{3}\right) + C_1\left(\frac{h}{2} - d_3\right) + C_2\left(\frac{h}{2} - d_4\right) + T_1\left(\frac{h}{2} - d_1\right) + T_2\left(\frac{h}{2} - d_2\right) \qquad \text{Equation (4)}$$

$$M = 235.54\left(\frac{96}{2} - \frac{65.92}{3}\right) + (10.36)\left(\frac{96}{2} - 3.45\right) + (8.19)\left(\frac{96}{2} - 16.55\right) + \ldots$$

$$(4.42)\left(\frac{96}{2} - 3.45\right) + (3.72)\left(\frac{96}{2} - 7.63\right) = 7196.57 \cong 7191.18 \text{ OK}$$

Concrete compressive stress of 0.794 ksi is 40% of $f_c^{'}$ and the maximum tensile steel stress is $\frac{4.42}{1.27} = 3.48 \text{ ksi}$, therefore OK.

Numerical Evaluation: Time is 2.125 days EW (X) Strip

b = 9 inches; h = 96 inches; d_1 = 5.535 inches; d_2 = 9.705 inches; d_3 = 5.535 inches; d_4 = 18.635 inches

A_s = 1.27 sq. in. #10 bar d_{sum} = -8.93

$f_c' = 1.961$ ksi Table 4, Reference 6:

E_c = 2674 ksi; E_s = 29000 ksi; $\alpha = \frac{29000}{2674} = 10.85$

$\sigma_c = 0.793$ ksi from off-line calculations; $\varepsilon_c = \frac{0.793}{2674} = 0.000297$ in/in

From Table 9 (6/11), Reference 6:

C = 5070.757 kip/17 foot section; C = 223.71 kip/9 inch section
M = 160000 in-kip/17 foot section; M = 7058.82 in-kip/9 inch section

Moment produces compression on the bottom of the pad, therefore use Equations (3) and (4).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \text{ x } 1.27 \text{ x } 10.85 - \frac{223.71}{0.793} = -227.01$$

$$2bA_s\alpha(2h - d_{sum}) = 2 \text{ x } 9 \text{ x } 1.27 \text{ x } 10.85 \text{ x } (2 \text{ x } 96 + 8.92) = 49814.71$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{sum})}}{b} = \frac{227.01 + \sqrt{-227.01^2 + 49814.71}}{9} = 60.60 \text{ inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{60.60 \text{ x } 9 \text{ x } 0.793}{2} = 216.24 \text{ kip}$$

$$C_1 = A_sE_s\left(\frac{x - d_3}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{60.60 - 5.535}{60.60}\right) \text{x } 0.000297 = 9.92 \text{ kip}$$

$$C_2 = A_s E_s\left(\frac{x - d_4}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{60.60 - 18.635}{60.60}\right) \text{x } 0.000297 = 7.56 \text{ kip}$$

$$T_1 = A_s E_s\left(\frac{h - x - d_1}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{96 - 60.60 - 5.535}{60.60}\right) \text{x } 0.000297 = 5.38 \text{ kip}$$

$$T_2 = A_s E_s\left(\frac{h - x - d_2}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{96 - 60.60 - 9.705}{60.60}\right) \text{x } 0.000297 = 4.63 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$$\therefore\ C = C_c + C_1 + C_2 - T_1 - T_2$$

$$223.71 = 216.24+9.92+7.56-5.38-4.63 = 223.71 \text{ OK}$$

Now the internal moment is:

$$M = C_c\left(\frac{h}{2} - \frac{x}{3}\right) + C_1\left(\frac{h}{2} - d_3\right) + C_2\left(\frac{h}{2} - d_4\right) + T_1\left(\frac{h}{2} - d_1\right) + T_2\left(\frac{h}{2} - d_2\right) \qquad \text{Equation (4)}$$

$$M = 216.24\left(\frac{96}{2} - \frac{60.60}{3}\right) + (9.92)\left(\frac{96}{2} - 5.535\right) + (7.56)\left(\frac{96}{2} - 18.635\right) + \ldots$$

$$(5.38)\left(\frac{96}{2} - 5.535\right) + (4.63)\left(\frac{96}{2} - 9.705\right) = 7061.24 \cong 7058.82 \text{ OK}$$

Concrete compressive stress of 0.793 ksi is 40% of $f_c^{'}$ and the maximum tensile steel stress is $\frac{5.38}{1.27} = 4.24$ ksi, therefore OK.

Numerical Evaluation: Time is 2.125 days Shear NS (Z) and EW (X) Strips

Max Fy is 27.83 kip, Table 9 (6/11), Reference 6, largest of NS or EW values.

$$v_c = \frac{27.83}{204 \text{ x } (96 - 9.705)} = 0.00158 \text{ ksi} = 1.58 \text{ psi} \quad \text{OK by inspection.}$$

Numerical Evaluation: **Time is 2.375 days** **NS (Z) Strip**

$b = 9$ inches; $h = 96$ inches; $d_1 = 3.45$ inches; $d_2 = 7.63$ inches; $d_3 = 3.45$ inches; $d_4 = 16.55$ inches

$A_s = 1.27$ sq. in. #10 bar $d_{sum} = -8.92$

$f'_c = 2.133$ ksi Table 4, Reference 6:

$E_c = 2788$ ksi; $E_s = 29000$ ksi; $\alpha = \dfrac{29000}{2788} = 10.40$

$\boldsymbol{\sigma_c} = 0.813$ ksi from off-line calculations; $\varepsilon_c = \dfrac{0.813}{2788} = 0.000292$ in/in

From Table 9 (7/11), Reference 6:

C = 5581.821 kip/17 foot section; C = 246.26 kip/9 inch section
M = 166000 in-kip/17 foot section; M = 7323.53 in-kip/9 inch section

Moment produces compression on the bottom of the pad, therefore use Equations (3) and (4).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \text{ x } 1.27 \text{ x } 10.40 - \frac{246.26}{0.813} = -250.058$$

$$2bA_s\alpha(2h - d_{sum}) = 2 \text{ x } 9 \text{ x } 1.27 \text{ x } 10.40 \text{ x } (2 \text{ x } 96 + 8.92) = 47775.43$$

$$x = \frac{-\left(4A_s\alpha - \dfrac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \dfrac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{sum})}}{b} = \frac{250.058 + \sqrt{-250.058^2 + 47775.43}}{9} = 64.69 \text{ inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{64.69 \text{ x } 9 \text{ x } 0.813}{2} = 236.66 \text{ kip}$$

$$C_1 = A_sE_s\left(\frac{x - d_3}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{64.69 - 3.45}{64.69}\right) \text{x } 0.000292 = 10.17 \text{ kip}$$

$$C_2 = A_s E_s \left(\frac{x - d_4}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{64.69 - 16.55}{64.69}\right) \text{x } 0.000292 = 7.99 \text{ kip}$$

$$T_1 = A_s E_s \left(\frac{h - x - d_1}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{96 - 64.69 - 3.45}{64.69}\right) \text{x } 0.000292 = 4.63 \text{ kip}$$

$$T_2 = A_s E_s \left(\frac{h - x - d_2}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{96 - 64.69 - 7.63}{64.69}\right) \text{x } 0.000292 = 3.93 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$$\therefore\ C = C_c + C_1 + C_2 - T_1 - T_2$$

$$246.26 = 236.66 + 10.17 + 7.99 - 4.63 - 3.93 = 246.26 \text{ OK}$$

Now the internal moment is:

$$M = C_c\left(\frac{h}{2} - \frac{x}{3}\right) + C_1\left(\frac{h}{2} - d_3\right) + C_2\left(\frac{h}{2} - d_4\right) + T_1\left(\frac{h}{2} - d_1\right) + T_2\left(\frac{h}{2} - d_2\right) \qquad \text{Equation (4)}$$

$$M = 236.66\left(\frac{96}{2} - \frac{64.69}{3}\right) + (10.17)\left(\frac{96}{2} - 3.45\right) + (7.99)\left(\frac{96}{2} - 16.55\right) + \ldots$$

$$(4.63)\left(\frac{96}{2} - 3.45\right) + (3.93)\left(\frac{96}{2} - 7.63\right) = 7325.79 \cong 7323.53 \text{ OK}$$

Concrete compressive stress of 0.809 ksi is 38% of f_c' and the maximum tensile steel stress is $\frac{4.63}{1.27} = 3.64\text{ ksi}$, therefore OK.

Numerical Evaluation: Time is 2.375 days EW (X) Strip

b = 9 inches; h = 96 inches; d_1 = 5.535 inches; d_2 = 9.705 inches; d_3 = 5.535 inches; d_4 = 18.635 inches

A_s = 1.27 sq. in. #10 bar d_{sum} = -8.93

$f'_c = 2.133$ ksi Table 4, Reference 6:

$E_c = 2788$ ksi; $E_s = 29000$ ksi; $\alpha = \frac{29000}{2788} = 10.40$

$\sigma_c = 0.814$ ksi from off-line calculations; $\varepsilon_c = \frac{0.814}{2788} = 0.000292$ in/in

From Table 9 (7/11), Reference 6:

C = 5119.114 kip/17 foot section; C = 225.84 kip/9 inch section
M = 163000 in-kip/17 foot section; M = 7191.18 in-kip/9 inch section

Moment produces compression on the bottom of the pad, therefore use Equations (3) and (4).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \text{ x } 1.27 \text{ x } 10.40 - \frac{225.84}{0.814} = -224.608$$

$$2bA_s\alpha(2h - d_{sum}) = 2 \text{ x } 9 \text{ x } 1.27 \text{ x } 10.40 \text{ x } (2 \text{ x } 96 + 8.93) = 47777.81$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{sum})}}{b} = \frac{224.608 + \sqrt{-224.608^2 + 47777.81}}{9} = 59.78 \text{ inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{59.78 \text{ x } 9 \text{ x } 0.814}{2} = 218.97 \text{ kip}$$

$$C_1 = A_sE_s\left(\frac{x - d_3}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{59.78 - 5.535}{59.78}\right) \text{x } 0.000292 = 9.76 \text{ kip}$$

$$C_2 = A_s E_s\left(\frac{x-d_4}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{59.78 - 18.635}{59.78}\right) \text{x } 0.000292 = 7.40 \text{ kip}$$

$$T_1 = A_s E_s\left(\frac{h-x-d_1}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{96 - 59.78 - 5.535}{59.78}\right) \text{x } 0.000292 = 5.52 \text{ kip}$$

$$T_2 = A_s E_s\left(\frac{h-x-d_2}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{96 - 59.78 - 9.705}{59.78}\right) \text{x } 0.000292 = 4.77 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$$\therefore C = C_c + C_1 + C_2 - T_1 - T_2$$

$$225.84 = 218.97+9.76+7.40-5.52-4.77 = 225.84 \text{ OK}$$

Now the internal moment is:

$$M = C_c\left(\frac{h}{2}-\frac{x}{3}\right)+C_1\left(\frac{h}{2}-d_3\right)+C_2\left(\frac{h}{2}-d_4\right)+T_1\left(\frac{h}{2}-d_1\right)+T_2\left(\frac{h}{2}-d_2\right) \qquad \text{Equation (4)}$$

$$M = 218.97\left(\frac{96}{2}-\frac{59.78}{3}\right)+(9.76)\left(\frac{96}{2}-5.535\right)+(7.40)\left(\frac{96}{2}-18.635\right)+\ldots$$

$$(5.51)\left(\frac{96}{2}-5.535\right)+(4.77)\left(\frac{96}{2}-9.705\right) = 7196.05 \cong 7191.18 \text{ OK}$$

Concrete compressive stress of 0.814 ksi is 38% of f_c' and the maximum tensile steel stress is $\frac{5.52}{1.27} = 4.35$ ksi, therefore OK.

Numerical Evaluation: Time is 2.375 days Shear NS (Z) and EW (X) Strips

Max Fy is 35.96 kip, Table 9 (7/11), Reference 6 largest of NS or EW values.

$$v_c = \frac{35.96}{204 \text{ x } (96 - 9.705)} = 0.00204 \text{ ksi} = 2.04 \text{ psi} \quad \text{OK by inspection.}$$

Numerical Evaluation: Time is 3.125 days NS (Z) Strip

$b = 9$ inches; $h = 96$ inches; $d_1 = 3.45$ inches; $d_2 = 7.63$ inches; $d_3 = 3.45$ inches; $d_4 = 16.55$ inches

$A_s = 1.27$ sq. in. #10 bar $d_{sum} = -8.92$

$f_c^{'} = 2.604$ ksi Table 4, Reference 6:

$E_c = 3080$ ksi; $E_s = 29000$ ksi; $\alpha = \frac{29000}{3080} = 9.42$

$\boldsymbol{\sigma_c} = 0.815$ ksi from off-line calculations; $\varepsilon_c = \frac{0.815}{3080} = 0.000265$ in/in

From Table 9 (8/11), Reference 6:

C = 5456.216 kip/17 foot section; C = 240.72 kip/9 inch section
M = 164000 in-kip/17 foot section; M = 7235.29 in-kip/9 inch section

Moment produces compression on the bottom of the pad, therefore use Equations (3) and (4).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \text{ x } 1.27 \text{ x } 9.42 - \frac{240.72}{0.815} = -247.525$$

$$2bA_s\alpha(2h - d_{sum}) = 2 \text{ x } 9 \text{ x } 1.27 \text{ x} 9.42 \text{ x } (2 \text{ x } 96 + 8.92) = 43246.07$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{sum})}}{b} = \frac{247.525 + \sqrt{-247.525^2 + 43246.07}}{9} = 63.42 \text{ inches}$$

Now,

$$C_c = \frac{1}{2} x b E_c \varepsilon_c = \frac{x b \sigma_c}{2} = \frac{63.42 \text{ x } 9 \text{ x } 0.815}{2} = 232.61 \text{ kip}$$

$$C_1 = A_s E_s \left(\frac{x - d_3}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{63.42 - 3.45}{63.42}\right) \text{x } 0.000265 = 9.22 \text{ kip}$$

$$C_2 = A_s E_s \left(\frac{x - d_4}{x} \right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{63.42 - 16.55}{63.42} \right) \text{x } 0.000265 = 7.20 \text{ kip}$$

$$T_1 = A_s E_s \left(\frac{h - x - d_1}{x} \right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{96 - 63.42 - 3.45}{63.42} \right) \text{x } 0.000265 = 4.48 \text{ kip}$$

$$T_2 = A_s E_s \left(\frac{h - x - d_2}{x} \right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{96 - 63.42 - 7.63}{63.42} \right) \text{x } 0.000265 = 3.83 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$$\therefore \; C = C_c + C_1 + C_2 - T_1 - T_2$$

$$240.72 = 232.61 + 9.22 + 7.20 - 4.48 - 3.83 = 240.72 \text{ OK}$$

Now the internal moment is:

$$M = C_c \left(\frac{h}{2} - \frac{x}{3} \right) + C_1 \left(\frac{h}{2} - d_3 \right) + C_2 \left(\frac{h}{2} - d_4 \right) + T_1 \left(\frac{h}{2} - d_1 \right) + T_2 \left(\frac{h}{2} - d_2 \right) \qquad \text{Equation (4)}$$

$$M = 232.61 \left(\frac{96}{2} - \frac{63.42}{3} \right) + (9.22) \left(\frac{96}{2} - 3.45 \right) + (7.20) \left(\frac{96}{2} - 16.55 \right) + \ldots$$

$$(4.48) \left(\frac{96}{2} - 3.45 \right) + (3.83) \left(\frac{96}{2} - 7.63 \right) = 7238.72 \cong 7235.29 \text{ OK}$$

Concrete compressive stress of 0.815 ksi is 31% of f_c' and the maximum tensile steel stress is $\frac{4.48}{1.27} = 3.52 \text{ ksi}$, therefore OK.

Numerical Evaluation: Time is 3.125 days EW (X) Strip

b = 9 inches; h = 96 inches; d_1 = 5.535 inches; d_2 = 9.705 inches; d_3 = 5.535 inches; d_4 = 18.635 inches

A_s = 1.27 sq. in. #10 bar d_{sum} = -8.93

$f_c^{'} = 2.604$ ksi Table 4, Reference 6:

$E_c = 3080$ ksi; $E_s = 29000$ ksi; $\alpha = \frac{29000}{3080} = 9.42$

$\sigma_c = 0.811$ ksi from off-line calculations; $\varepsilon_c = \frac{0.811}{3080} = 0.000263$ in/in

From Table 9 (8/11), Reference 6:

C = 4996.156 kip/17 foot section; C = 220.42 kip/9 inch section
M = 160000 in-kip/17 foot section; M = 7058.82 in-kip/9 inch section

Moment produces compression on the bottom of the pad, therefore use Equations (3) and (4).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \times 1.27 \times 9.42 - \frac{220.42}{0.811} = -223.96$$

$$2bA_s\alpha(2h - d_{sum}) = 2 \times 9 \times 1.27 \times 9.42 \times (2 \times 96 + 8.93) = 43248.23$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{sum})}}{b} = \frac{223.96 + \sqrt{-223.96^2 + 43248.23}}{9} = 58.84 \text{ inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{58.84 \times 9 \times 0.811}{2} = 214.74 \text{ kip}$$

$$C_1 = A_sE_s\left(\frac{x - d_3}{x}\right)\varepsilon_c = 1.27 \times 29000\left(\frac{58.84 - 5.535}{58.84}\right) \times 0.000263 = 8.79 \text{ kip}$$

$$C_2 = A_s E_s\left(\frac{x-d_4}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{58.84 - 18.635}{58.84}\right) \text{x } 0.000263 = 6.63 \text{ kip}$$

$$T_1 = A_s E_s\left(\frac{h-x-d_1}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{96 - 58.84 - 5.535}{58.84}\right) \text{x } 0.000263 = 5.21 \text{ kip}$$

$$T_2 = A_s E_s\left(\frac{h-x-d_2}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{96 - 58.84 - 9.705}{58.84}\right) \text{x } 0.000263 = 4.52 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$$\therefore\ C = C_c + C_1 + C_2 - T_1 - T_2$$

$$220.42 = 214.74 + 8.79 + 6.63 - 5.21 - 4.52 = 220.43 \text{ OK}$$

Now the internal moment is:

$$M = C_c\left(\frac{h}{2}-\frac{x}{3}\right) + C_1\left(\frac{h}{2}-d_3\right) + C_2\left(\frac{h}{2}-d_4\right) + T_1\left(\frac{h}{2}-d_1\right) + T_2\left(\frac{h}{2}-d_2\right) \qquad \text{Equation (4)}$$

$$M = 214.74\left(\frac{96}{2}-\frac{58.84}{3}\right) + (8.79)\left(\frac{96}{2}-5.535\right) + (6.63)\left(\frac{96}{2}-18.635\right) + \ldots$$

$$(5.21)\left(\frac{96}{2}-5.535\right) + (4.52)\left(\frac{96}{2}-9.705\right) = 7057.97 \cong 7058.82 \text{ OK}$$

Concrete compressive stress of 0.811 ksi is 31% of f_c' and the maximum tensile steel stress is $\frac{5.21}{1.27} = 4.10 \text{ ksi}$, therefore OK.

Numerical Evaluation: Time is 3.125 days Shear NS (Z) and EW (X) Strips

Max Fy is 49.868 kip, Table 9 (8/11), Reference 6 largest of NS or EW values.

$$v_c = \frac{49.868}{204 \text{ x } (96 - 9.705)} = 0.00283 \text{ ksi} = 2.83 \text{ psi} \quad \text{OK by inspection.}$$

The largest shear in Table 9 is 98.386 kip, see Table 9 (11/11). Results in v_c of 5.59 psi, also OK by inspection.

Numerical Evaluation: Shrinkage NS (Z) Strip

$b = 9$ inches; $h = 96$ inches; $d_1 = 3.45$ inches; $d_2 = 7.63$ inches; $d_3 = 3.45$ inches; $d_4 = 16.55$ inches

$A_s = 1.27$ sq. in. #10 bar $d_{sum} = -8.92$

$f'_c = 5.000$ ksi

$E_c = 2844$ ksi (see Concrete Properties, Ref.6); $E_s = 29000$ ksi; $\alpha = \frac{29000}{2844} = 10.20$

$\sigma_c = 0.459$ ksi from off-line calculations; $\varepsilon_c = \frac{0.459}{2844} = 0.000161$ in/in

From Table 13, Reference 6:

C = 39.019 kip/17 foot section; C = 1.72 kip/9 inch section
M = 85400 in-kip/17 foot section; M = 3767.65 in-kip/9 inch section

Moment produces compression on the bottom of the pad, therefore use Equations (3) and (4).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \text{ x } 1.27 \text{ x } 10.20 - \frac{1.72}{0.459} = 48.05$$

$$2bA_s\alpha(2h - d_{sum}) = 2 \text{ x } 9 \text{ x } 1.27 \text{ x } 10.20 \text{ x } (2 \text{ x } 96 + 8.92) = 46834.71$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{sum})}}{b} = \frac{-48.05 + \sqrt{48.05^2 + 46834.71}}{9} = 19.29 \text{ inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{19.29 \text{ x } 9 \text{ x } 0.459}{2} = 39.85 \text{ kip}$$

$$C_1 = A_sE_s\left(\frac{x - d_3}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{19.29 - 3.45}{19.29}\right) \text{x } 0.000161 = 4.88 \text{ kip}$$

$$C_2 = A_s E_s \left(\frac{x - d_4}{x}\right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{19.29 - 16.55}{19.29}\right) \text{x } 0.000161 = 0.85 \text{ kip}$$

$$T_1 = A_s E_s \left(\frac{h - x - d_1}{x}\right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{96 - 19.29 - 3.45}{19.29}\right) \text{x } 0.000161 = 22.57 \text{ kip}$$

$$T_2 = A_s E_s \left(\frac{h - x - d_2}{x}\right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{96 - 19.29 - 7.63}{19.29}\right) \text{x } 0.000161 = 21.28 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$$\therefore C = C_c + C_1 + C_2 - T_1 - T_2$$

$$1.72 = 39.85+4.88+0.85-22.57-21.28 = 1.73 \text{ OK}$$

Now the internal moment is:

$$M = C_c\left(\frac{h}{2} - \frac{x}{3}\right) + C_1\left(\frac{h}{2} - d_3\right) + C_2\left(\frac{h}{2} - d_4\right) + T_1\left(\frac{h}{2} - d_1\right) + T_2\left(\frac{h}{2} - d_2\right) \text{ Equation (4)}$$

$$M = 39.85\left(\frac{96}{2} - \frac{19.29}{3}\right) + (4.88)\left(\frac{96}{2} - 3.45\right) + (0.85)\left(\frac{96}{2} - 16.55\right) + \ldots$$

$$(22.57)\left(\frac{96}{2} - 3.45\right) + (21.28)\left(\frac{96}{2} - 7.63\right) = 3765.23 \cong 3767.65 \text{ OK}$$

Concrete compressive stress of 0.459 ksi is 9% of $f_c^{'}$ and the maximum tensile steel stress is $\frac{22.57}{1.27} = 17.78$ ksi, therefore OK.

Numerical Evaluation: Shrinkage EW (X) Strip

$b = 9$ inches; $h = 96$ inches; $d_1 = 5.535$ inches; $d_2 = 9.705$ inches; $d_3 = 5.535$ inches; $d_4 = 18.635$ inches

$A_s = 1.27$ sq. in. #10 bar $d_{sum} = -8.93$

$f'_c = 5.000\,\text{ksi}$

$E_c = 2844$ ksi (see Concrete Properties, Ref.6); $E_s = 29000$ ksi; $\alpha = \frac{29000}{2844} = 10.20$

$\sigma_c = 0.485\,\text{ksi}$ from off-line calculations; $\varepsilon_c = \frac{0.485}{2844} = 0.000171\,\text{in/in}$

From Table 13, Reference 6:

C = 489.971 kip/17 foot section; C = 21.62 kip/9 inch section
M = 84000 in-kip/17 foot section; M = 3705.88 in-kip/9 inch section

Moment produces compression on the bottom of the pad, therefore use Equations (3) and (4).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \times 1.27 \times 10.20 - \frac{21.62}{0.485} = 7.23$$

$$2bA_s\alpha(2h - d_{sum}) = 2 \times 9 \times 1.27 \times 10.20 \times (2 \times 96 + 8.93) = 46837.04$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{sum})}}{b} = \frac{-7.23 + \sqrt{7.23^2 + 46837.04}}{9} = 23.26\,\text{inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{23.26 \times 9 \times 0.485}{2} = 50.76\,\text{kip}$$

$$C_1 = A_sE_s\left(\frac{x - d_3}{x}\right)\varepsilon_c = 1.27 \times 29000\left(\frac{23.26 - 5.535}{23.26}\right) \times 0.000171 = 4.79\,\text{kip}$$

$$C_2 = A_s E_s\left(\frac{x-d_4}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{23.26-18.635}{23.26}\right)\text{x } 0.000171 = 1.25 \text{ kip}$$

$$T_1 = A_s E_s\left(\frac{h-x-d_1}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{96-23.26-5.535}{23.26}\right)\text{x } 0.000171 = 18.15 \text{ kip}$$

$$T_2 = A_s E_s\left(\frac{h-x-d_2}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{96-23.26-9.705}{23.26}\right)\text{x } 0.000171 = 17.02 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$$\therefore\ C = C_c + C_1 + C_2 - T_1 - T_2$$

$$21.62 = 50.76+4.79+1.25-18.15-17.02 = 21.63 \text{ OK}$$

Now the internal moment is:

$$M = C_c\left(\frac{h}{2}-\frac{x}{3}\right)+C_1\left(\frac{h}{2}-d_3\right)+C_2\left(\frac{h}{2}-d_4\right)+T_1\left(\frac{h}{2}-d_1\right)+T_2\left(\frac{h}{2}-d_2\right) \qquad \text{Equation (4)}$$

$$M = 50.76\left(\frac{96}{2}-\frac{23.26}{3}\right)+(4.79)\left(\frac{96}{2}-5.535\right)+(1.25)\left(\frac{96}{2}-18.635\right)+\ldots$$

$$(18.15)\left(\frac{96}{2}-5.535\right)+(17.02)\left(\frac{96}{2}-9.705\right) = 3705.48 \cong 3705.88 \text{ OK}$$

Concrete compressive stress of 0.485 ksi is 10% of $f_c^{'}$ and the maximum tensile steel stress is $\frac{18.15}{1.27} = 14.29 \text{ ksi}$, therefore OK.

Numerical Evaluation: Shrinkage Shear NS (Z) and EW (X) Strips

Max Fy is 63.084 kip, Table 13, Reference 6, largest of NS or EW values.

$$v_c = \frac{63.084}{204 \text{ x } (96-9.705)} = 0.00358 \text{ ksi} = 3.58 \text{ psi} \quad \text{OK by inspection.}$$

Detailed Evaluation of Allowable Stresses

The calculations above are tabulated in Table 1. This data shows that the percentage of applied concrete stress decreases over time while the steel stress peaks at 2.375 days.

Table 1 – Tabulation of Thermal/Shrinkage Stresses
Constrained Model Analyses

Time (days)	X (inches) NS/EW	f'_c (ksi)	σ_c (ksi) NS/EW	$\% f'_c$ NS/EW	σ_s (ksi) NS/EW
1.125	76.54/68.35	1.054	0.574/0.582	54/55	1.77/2.78
1.625	69.18/62.89	1.523	0.724/0.728	47/48	3.01/3.93
2.125	65.92/60.60	1.961	0.799/0.793	40/40	3.48/4.24
2.375	64.69/59.78	2.133	0.813/0.814	38/38	3.64/4.35
3.125	63.42/58.84	2.604	0.815/0.811	31/31	3.52/4.10
Shrinkage	19.29/23.26	5.000	0.459/0.485	9/10	17.77/14.29

Using the previously presented equations, the only difference between the approximate evaluation process and the detailed process is the assessment of β which is a function of the distance from the tensile face to the neutral axis. All of the evaluations above are for the applied moment producing compression on the bottom of the pad, hence tension on the top. Thus all the steel stresses are for the top steel. The equation for f_s shows that it is inversely proportional to β, which is inversely proportional to X, the distance to the neutral axis. Thus f_s is proportional to X. The lowest value for the f_s for the thermal conditions is from analysis for 3.125 days where NS X is 63.42 inches and EW X is 58.84 inches, see Table 1.

Thermal Conditions

For the NS top steel:

Allowing w to equal 0.013 inches
d_c from the surface to the CG of the closest NS bar is 2+2.9/2 = 3.45 inches
A is 2 x (2+2.9+1.27/2) x 9/2 = 49.815 sq. in. per bar (see Figure 1 for dimensions)
ß equals (63.42)/(63.42-(2+2.9+1.27/2)) = 1.10

$$f_s = \frac{w}{0.076\sqrt[3]{d_c A}\,\beta 10^{-3}} = \frac{0.013}{0.076\sqrt[3]{3.45x49.815}\ x\,1.10x10^{-3}} = 28.0 \text{ ksi for NS steel}$$

For the EW top steel:

d_c from the surface to the CG of the closest EW bar is 2+2.9+1.27/2 = 5.535 inches
A is 2 x (2+2.9+1.27+2.9/2) x 9/2 = 68.58 sq. in. per bar (see Figure 1 for dimensions)
ß equals (58.84)/(58.84-(2+2.9+1.27+2.9/2)) = 1.15

$$f_s = \frac{w}{0.076\sqrt[3]{d_c A}\,\beta 10^{-3}} = \frac{0.013}{0.076\sqrt[3]{5.535 \times 68.58} \times 1.15 \times 10^{-3}} = 20.5 \text{ ksi for EW steel}$$

Shrinkage Conditions

For the NS top steel:

Allowing w to equal 0.013 inches
d_c from the surface to the CG of the closest NS bar is 2+2.9/2 = 3.45 inches
A is 2 x (2+2.9+1.27/2) x 9/2 = 49.815 sq. in. per bar (see Figure 1 for dimensions)
ß equals (19.29)/(19.29-(2+2.9+1.27/2)) = 1.40

$$f_s = \frac{w}{0.076\sqrt[3]{d_c A}\,\beta 10^{-3}} = \frac{0.013}{0.076\sqrt[3]{3.45 \times 49.815} \times 1.40 \times 10^{-3}} = 22.0 \text{ ksi for NS steel}$$

For the EW top steel:

d_c from the surface to the CG of the closest EW bar is 2+2.9+1.27/2 = 5.535 inches
A is 2 x (2+2.9+1.27+2.9/2) x 9/2 = 68.58 sq. in. per bar (see Figure 1 for dimensions)
ß equals (23.26)/(23.26 -(2+2.9+1.27+2.9/2)) = 1.49

$$f_s = \frac{w}{0.076\sqrt[3]{d_c A}\,\beta 10^{-3}} = \frac{0.013}{0.076\sqrt[3]{5.535 \times 68.58} \times 1.49 \times 10^{-3}} = 15.9 \text{ ksi for EW steel}$$

Since the applied stresses are below the acceptable values, all crack widths are expected to be below the 0.013 inches which is considered acceptable, per ACI 207. Further, since all the steel stresses are well below yield, the cracks, if they occur, will all close.

Detailed Evaluation for Thermal Demands – Unconstrained Model Analysis

The evaluations above use the forces/moments from the constrained model presented in Reference 6. This modeling technique maximized the constraint of the rock on the pad, resulting in the highest values for the net internal compressive force. However, one can not guarantee that this compressive force will actually develop throughout the pad/rock interface in the field. Local high strain early life concrete creep and/or local shear failure at the rock/"mud" pad interface or at the "mud" pad/pad interface may all combine in some form to reduce the compressive forces resulting from the analytical constraint applied to the pad in the model. Therefore, it is prudent to consider a reduction in the compressive force in the concrete along with a reduction in the applied moments due to the possible slip between the pad and the rock. The unconstrained model analysis presented in Reference 6, bounds this condition in a very conservative manner. This analysis results in pad stress fields that are in equilibrium internally, since there are no external forces acting upon the pad. (The stress field results from Internal Restraint due to the non-uniform temperature distribution within the pad, see Reference 13, Section 4.4.)

The compressive stresses for all the time steps taken from Table 11 of Reference 6 show that $\sigma_{X\,Min} \cong \sigma_{Z\,Min} \cong \sigma_{3\,Min}$ and all three stresses are a fraction of $f_c^{'}$.

Table 2 – Pad Concrete Compressive Stresses (psi) Vs Time

Unconstrained Model

Time (days)	Load Case	$\sigma_{X\,Min}$	$\sigma_{Z\,Min}$	$\sigma_{3\,Min}$	$f_c^{'}$	$\frac{\sigma_{3min}}{f_c^{'}}$ (%)
0.25	1	-7	-7	-11	234	4.7
0.50	2	-67	-67	-68	469	14
0.625	3	-82	-82	-82	586	14
1.125	4	-190	-190	-190	1054	18
1.625	5	-261	-261	-262	1523	17
2.125	6	-284	-285	-285	1961	15
2.375	7	-283	-285	-285	2133	13
3.125	8	-256	-258	-259	2604	10
4.125	9	-218	-218	-219	2939	7.5
6.125	10	-154	-147	-160	3448	4.7
7.875	11	-220	-220	-232	3699	6.3

The evaluations below consider the top reinforcement and then the bottom.

Top Reinforcement- NS and EW Steel

After examining the path stress plots in Reference 6, the most demanding stress field for the top reinforcement occurs at time 1.625 days and is shown in Figure 28 of Reference 6. The stresses occur at the center (X and Z are both 0.0) of the pad. A portion of Figure 28 along with some construction added is shown in Figure 5 below. A conservative means to compute the force demand on the reinforcements is to compute the moment demand of the stress field at point "O" only from the tension side, and require the steel to be able to react the moment.

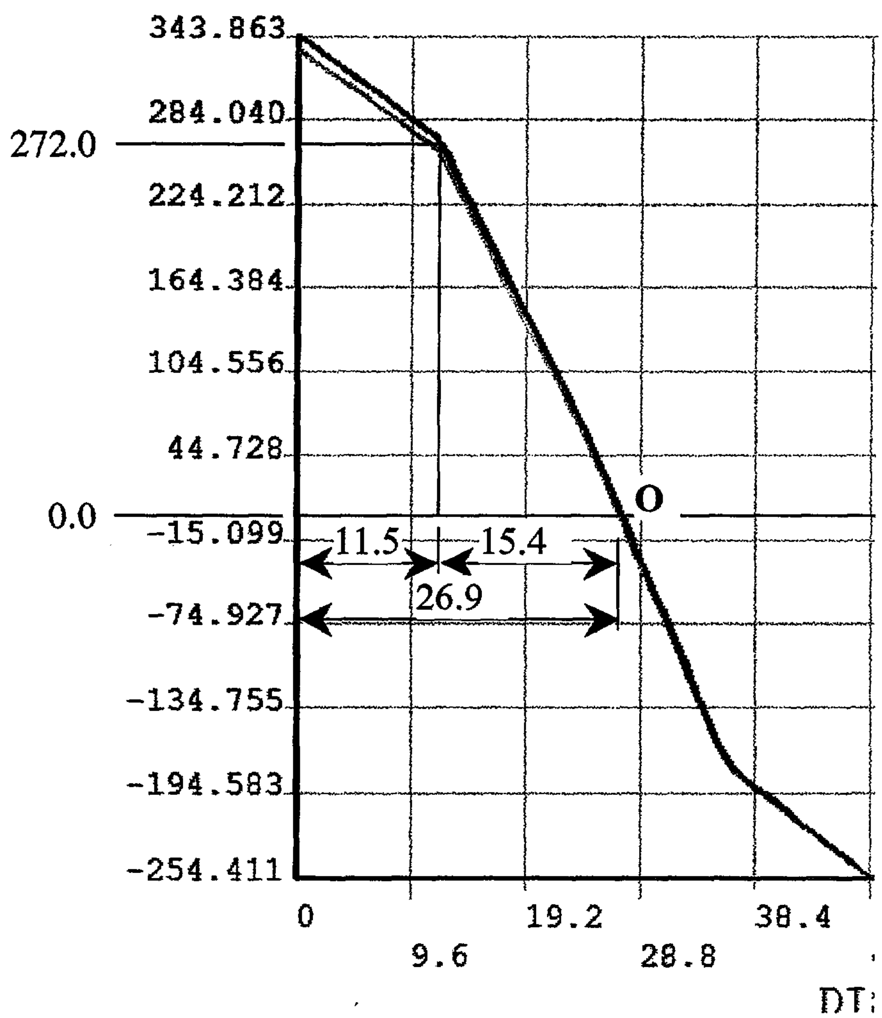


Figure 5 – Path plot for Sx and Sz at the top of the pad With additional construction for analysis

Thus, the applied moment of the tension stress field about point O is:

$$Mo = \left[\begin{array}{c}\left(\frac{1}{2}\right)(343.9-272.0)(11.5)\left(26.9-\frac{11.5}{3}\right)+ \\ (272.0)(11.5)\left(26.9-\frac{11.5}{2}\right)+\left(\frac{1}{2}\right)(272.0)(15.4)\left(\frac{2}{3}\right)(15.4)\end{array}\right](9) = 874{,}764 \text{ in - pounds}$$

For the NS steel, the center of gravity of the top steel is $\frac{3.45+7.62}{2} = 5.535$ inches (see Figure 1) from the concrete surface. Thus the necessary force to react the moment is $\left(\frac{874{,}764}{26.9-5.535}\right) = 40.94$ kip. And the bar stress is $\frac{40.94}{(2)(1.27)} = 16.12$ ksi. And, for the EW steel, the center of gravity of the top steel is $\frac{5.535+9.705}{2} = 7.62$ inches (see Figure 1) from the concrete surface. Thus the necessary force to react the moment is $\left(\frac{874{,}764}{26.9-7.62}\right) = 45.37$ kip. And the bar stress is $\frac{45.37}{(2)(1.27)} = 17.86$ ksi.

Bottom Reinforcement- NS Steel

The governing path for Sz occurs at 2.125 days and is shown in Figure 6, which is a portion of Figure 33 from Reference 6. Again as for the top steel, the sum of the moments about O is:

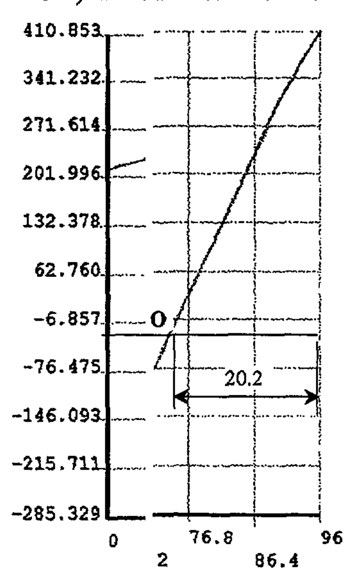


Figure 6 - Path plot for Sz at bottom of pad with additional construction for analysis

$Mo = \left(\frac{1}{2}\right)(410.9)(20.2)\left(\frac{2}{3}\right)(20.2)(9) = 502{,}991$ in pounds And the long (Z) steel (using only the bottom layer) is located 3.45 inches from the bottom of the pad. Thus the force in this bar is: $\frac{502.991}{20.2-3.45} = 30.03$ kip. The bar stress is therefore, $\frac{30.03}{1.27} = 23.7$ ksi

Bottom Reinforcement- EW Steel

The governing path for Sx occurs at 2.125 days and is shown in Figure 7, which is a portion of Figure 32 from Reference 6. Again as for the top steel, the sum of the moments about O is:

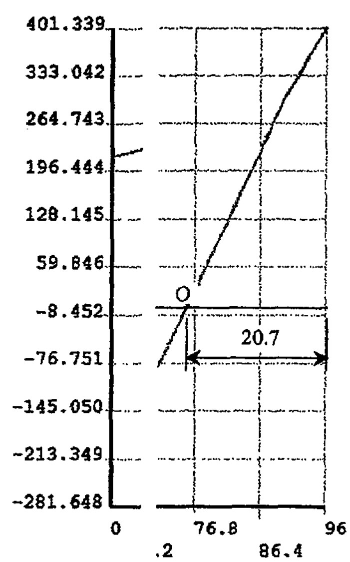


Figure 7 - Path plot for Sx at bottom of pad with additional construction for analysis

$Mo = \left(\frac{1}{2}\right)(401.3)(20.7)\left(\frac{2}{3}\right)(20.7)(9) = 515{,}859$ in pounds And the short (X) steel (using only the bottom layer) is located 5.535 inches from the bottom of the pad. Thus the force in this bar is: $\frac{515.859}{20.7-5.535} = 34.02$ kip. The bar stress is therefore, $\frac{34.02}{1.27} = 26.8$ ksi

Evaluation of the Unconstrained Model Stresses

Using the previous equations for crack width:

For the NS top steel:

d_c from the surface to the CG of the closest NS bar is 2+2.9/2 = 3.45 inches
A is 2 x (2+2.9+1.27/2) x 9/2 = 49.815 sq. in. per bar (see Figure 1 for dimensions)
ß equals (26.9/(26.9-(2+2.9+1.27/2)) = 1.26

$$w = 0.076\sqrt[3]{d_c A}\,\beta f_s 10^{-3} = 0.076\left(\sqrt[3]{3.45 \text{ x } 49.815}\right)1.26 \text{ x } 16.12 \text{ x} 10^{-3} = 0.0086 \text{ inches}$$

For the EW top steel:

d_c from the surface to the CG of the closest EW bar is 2+2.9+1.27/2 = 5.535 inches
A is 2 x (2+2.9+1.27+2.9/2) x 9/2 = 68.58 sq. in. per bar (see Figure 1 for dimensions)
ß equals (26.9)/(26.9-(2+2.9+1.27+2.9/2)) = 1.40

$$w = 0.076\sqrt[3]{d_c A}\,\beta f_s 10^{-3} = 0.076\left(\sqrt[3]{5.535 \text{ x } 68.58}\right)1.40 \text{ x } 17.86 \text{ x} 10^{-3} = 0.0138 \text{ inches}$$

For the NS bottom steel:

Consider only the bottom bar, the second bar is too high into the concrete to consider.

d_c from the surface to the CG of the closest NS bar is 2+2.9/2 = 3.45 inches
A is 2 x (2+2.9/2) x 9 = 62.1 sq. in. per bar (see Figure 2 for dimensions)
ß equals (20.2)/(20.2-(2+2.9/2)) = 1.21

$$w = 0.076\sqrt[3]{d_c A}\,\beta f_s 10^{-3} = 0.076\left(\sqrt[3]{3.45 \text{ x } 62.1}\right)1.21 \text{ x } 23.7 \text{ x} 10^{-3} = 0.0130 \text{ inches}$$

For the EW bottom steel:

d_c from the surface to the CG of the closest EW bar is 2+2.9+1.27/2 = 5.535 inches
A is 2 x (2+2.9+1.27/2) x 9 = 99.63 sq. in. per bar (see Figure 2 for dimensions)
ß equals (20.7)/(20.7-(2+2.9+1.27/2)) = 1.36

$$w = 0.076\sqrt[3]{d_c A}\,\beta f_s 10^{-3} = 0.076\left(\sqrt[3]{5.535 \text{ x } 99.63}\right)1.36 \text{ x } 26.8 \text{ x} 10^{-3} = 0.0227 \text{ inches}$$

Table 3 – Tabulation of Thermal Crack Widths
Unconstrained Model Analyses

Time (days)	Location/ Direction	Applied Stress (ksi)	Crack Width (mils)
1.625	Top NS	16.12	9
1.625	Top EW	17.86	14
2.125	Bot NS	23.7	13
2.125	Bot EW	26.8	23

The crack widths of 9 to 14 mils are acceptable since they are very small. The 23 mil crack width is also acceptable since: 1) the analysis that produced the applied forces is very conservative, 2) the mud mat protects the steel from water intrusion (also the water table is a considerable distance below the bottom of the pad) and 3) since the applied stress is well below yield, cracks, if they occur, will close in a matter of days.

The thermal and shrinkage evaluations presented above demonstrate that the #10 bar arrangement shown in Figures 1 and 2 is acceptable and not overly conservative. The expected cracking due to temperature and shrinkage is within the ACI recommended values for external exposure. Lastly, should a bond break occur during construction either between the rock and the "mud" pad, or the mud pad and the pad, the pad will not suffer any detrimental consequences.

Section 3 - Evaluation for Seismic Loads

The following calculation presents an evaluation of the concrete section with the #10 bar configuration shown in Figures 1 and 2 for the seismic loads. Actually, the evaluation simply evaluates ACI equation 4, U>D+L+Ess. As demonstrated above, this is the only equation that needs to be evaluated. The moments/forces from the seismic analysis, Reference 3, are the D+L+Ess terms. Thus, the following evaluation will demonstrate that the section capacity U bounds all of the moment/ force combinations provided in Reference 5.

The section capacities are computed for the entire range of anticipated loads. Thus the full moment/force interaction diagram for each strip direction, i.e., North-South (Z) and East-West (X) are computed and plotted. Since the steel arrangement is not symmetrical, the section capacities must be computed for the two senses of the applied moment, i.e., moment that produces tension on the bottom and moment that produces compression on the bottom. Figures 8 through 15 provide the graphical information needed to follow the calculations for the North-South section capacities. As, before, these are computed for a 9 inch width of pad.

The thermal calculations presented above assess the pad for the thickest the pad can be, which is 8 feet (96 inches) at its center. This is the basis for the calculation of the heat of hydration, the subsequent thermal demand computed in Reference 6 and the sizing of the reinforcement computed above. The seismic evaluation assesses the pad for the thinnest the pad can be, which is 7 feet 6 inches (90 inches) at its edges. This value is the thickness used in the computation of the seismic demand in Reference 5. This value will be used to evaluate the section for the seismic demand.

Basic data for Strength Method

Concrete strength: $f'_c = 5000\,\text{psi}$

Whitney stress: $0.85f'_c = 4250\,\text{psi} = 4.25\,\text{ksi}$

Depth of stress block: $\beta_1 X$ where β_1 is 0.80 for 5000 psi concrete
and X is the distance from the compression fiber to the NA

Concrete strain: $\varepsilon_c = 0.003\,\text{in/in}$

Steel yield stress: $f_y = 60000\,\text{psi} = 60.0\,\text{ksi}$

Steel yield strain: $\varepsilon_y = \dfrac{60}{29E3} = 0.002069\,\text{in/in}$

Net section depth for evaluation is 90 inches (7'-6")

North-South Section Concrete Capacity

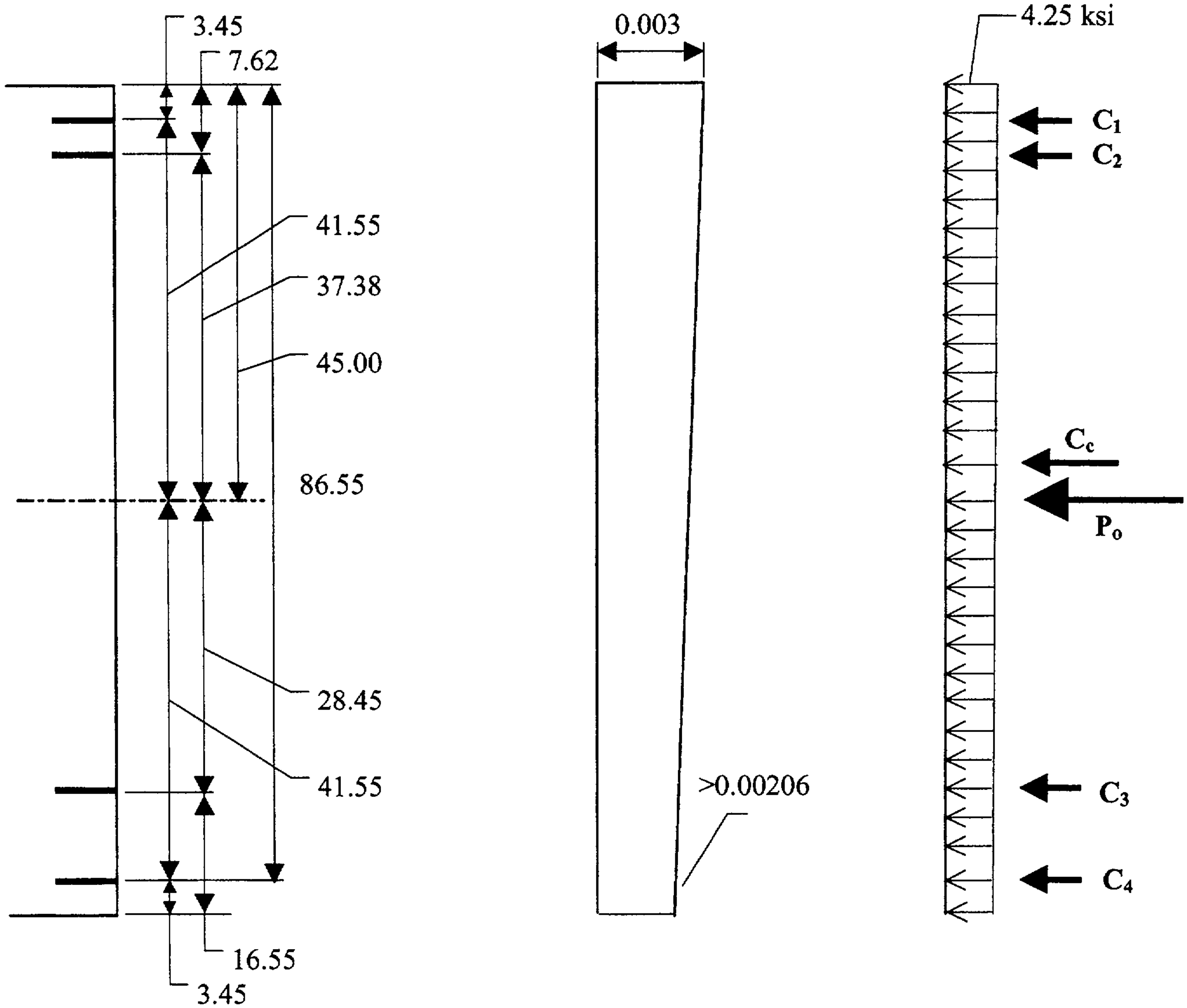


Figure 8 – North-South (Z strip) Section – Net Axial Compression No Moment

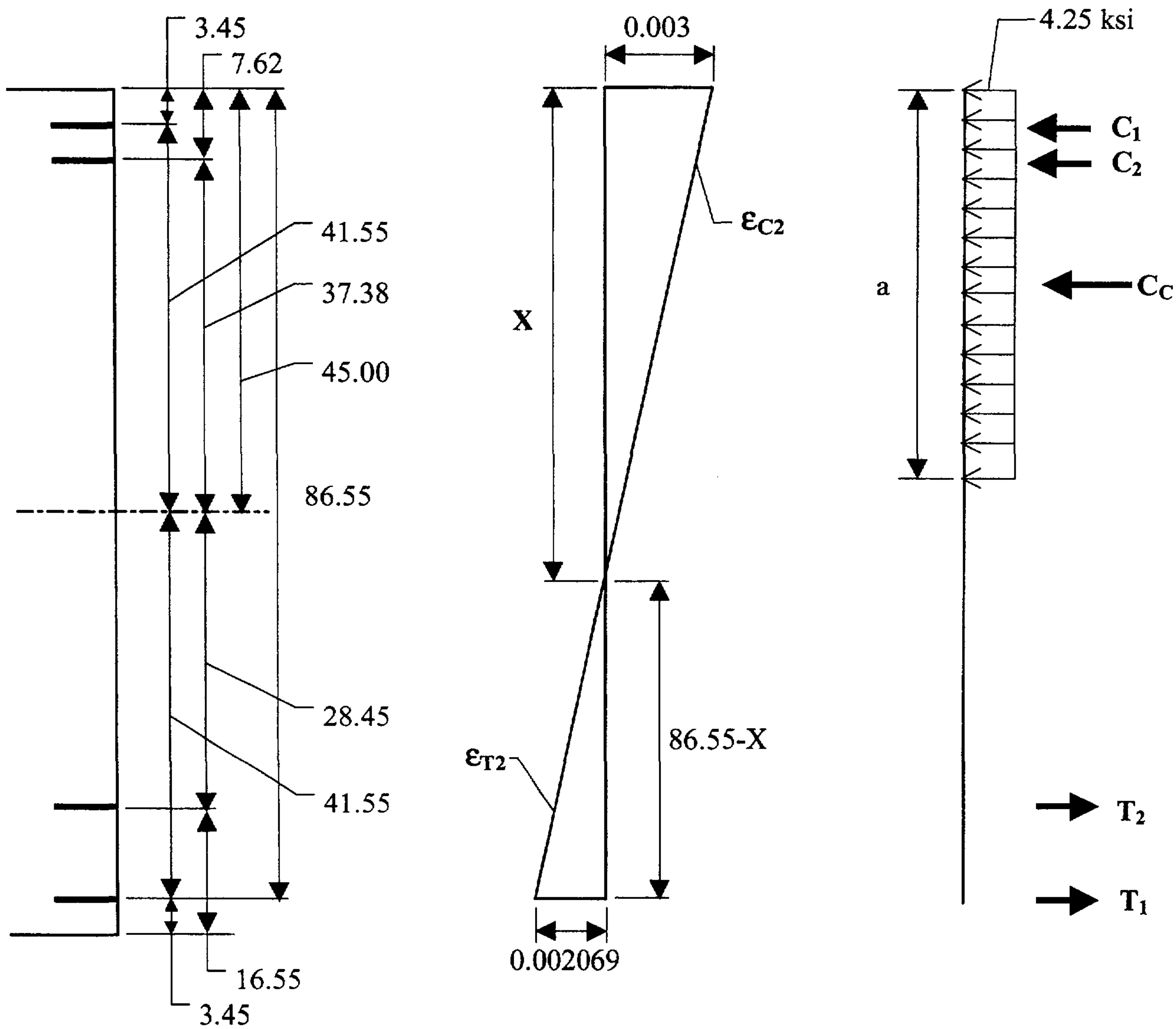


Figure 9 – North-South (Z strip) Section – Balanced Condition
Tension on the Bottom
This is +Mx, see Reference 5

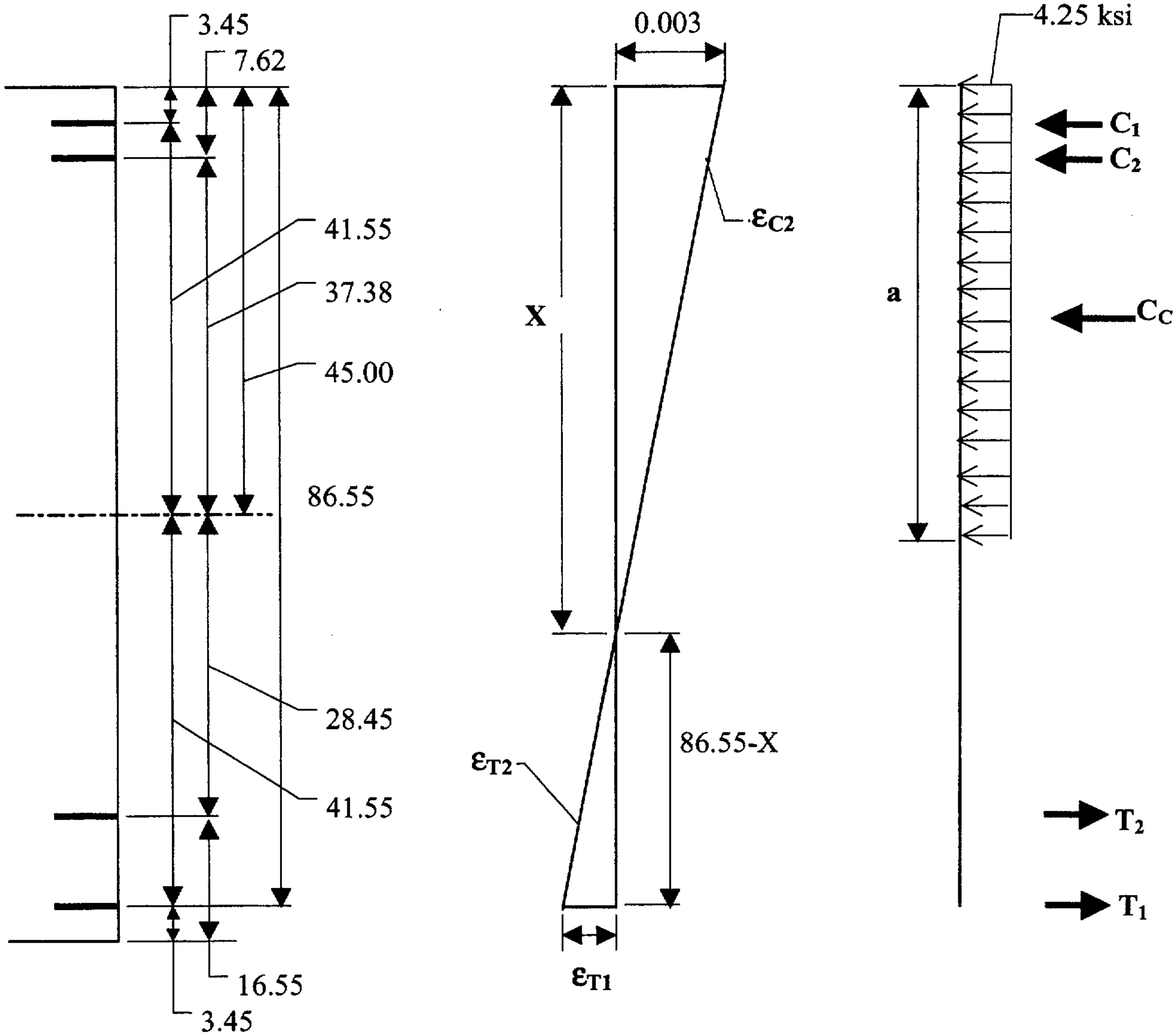


Figure 10 – North-South (Z strip) Section – Compression Controls
Tension on the Bottom
This is +Mx, see Reference 5

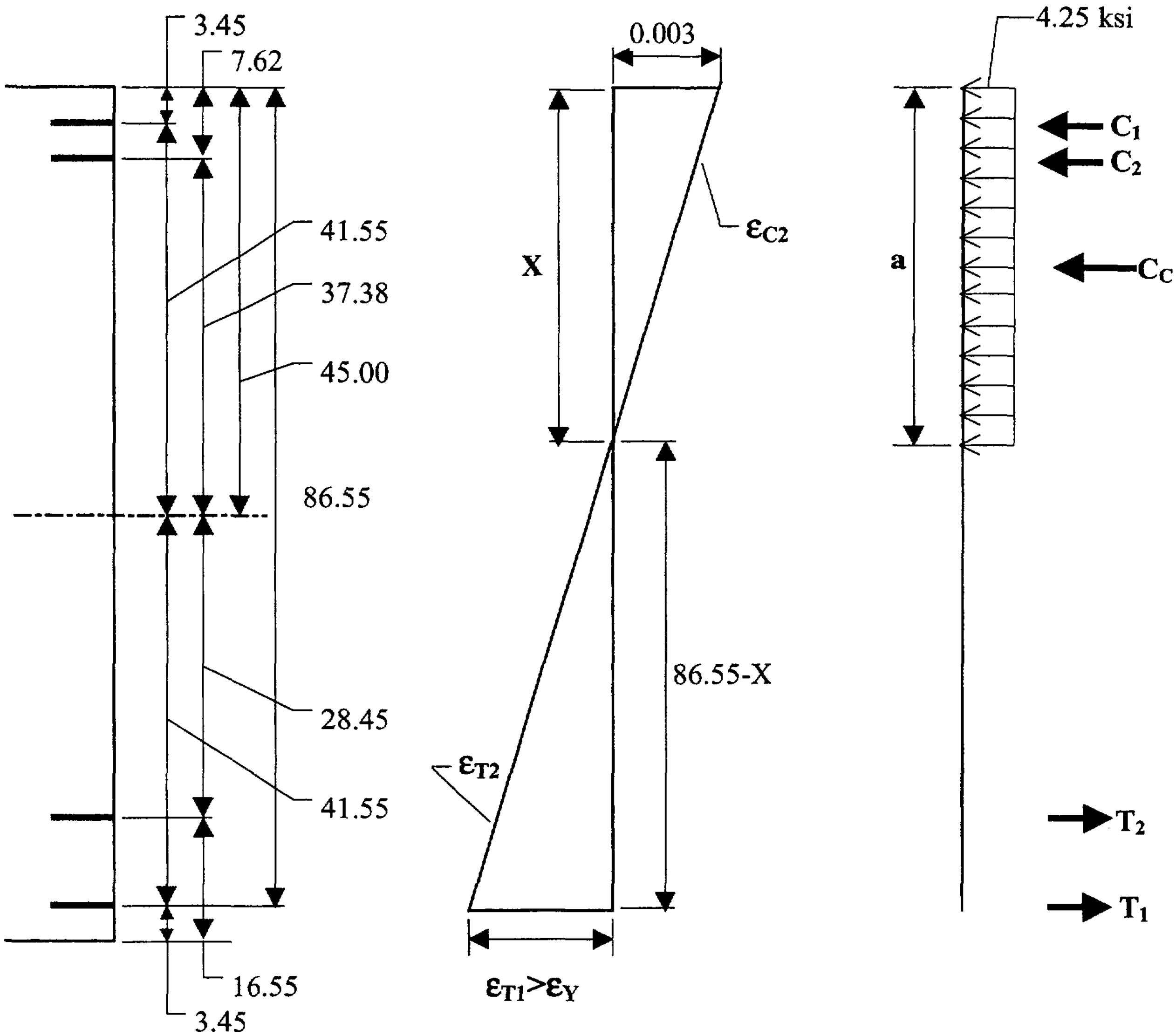


Figure 11 – North-South (Z strip) Section – Tension Controls

Tension on the Bottom

This is +Mx, see Reference 5

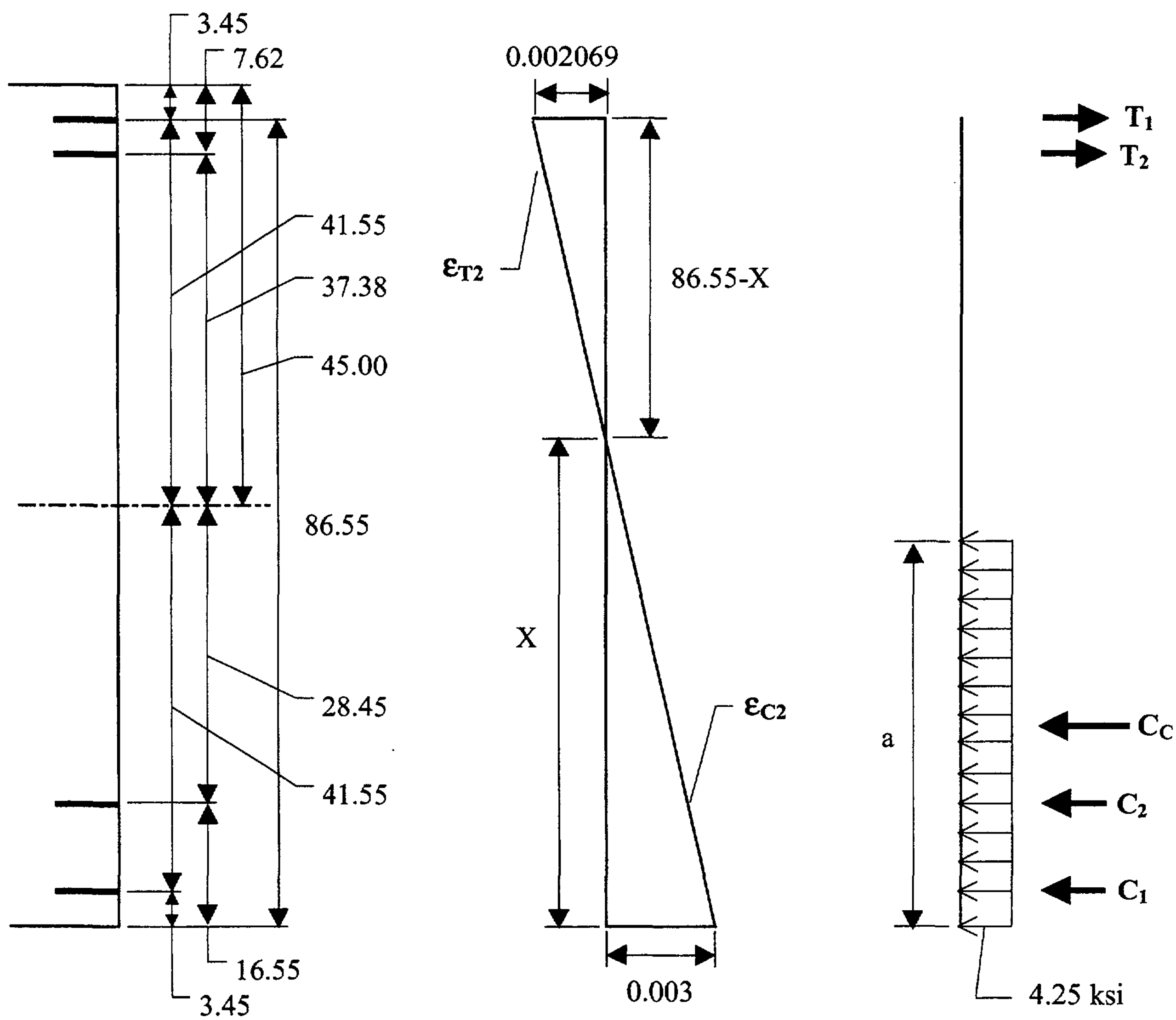


Figure 12 – North-South (Z strip) Section – Balanced Condition
Compression on the Bottom
This is -Mx, see Reference 5

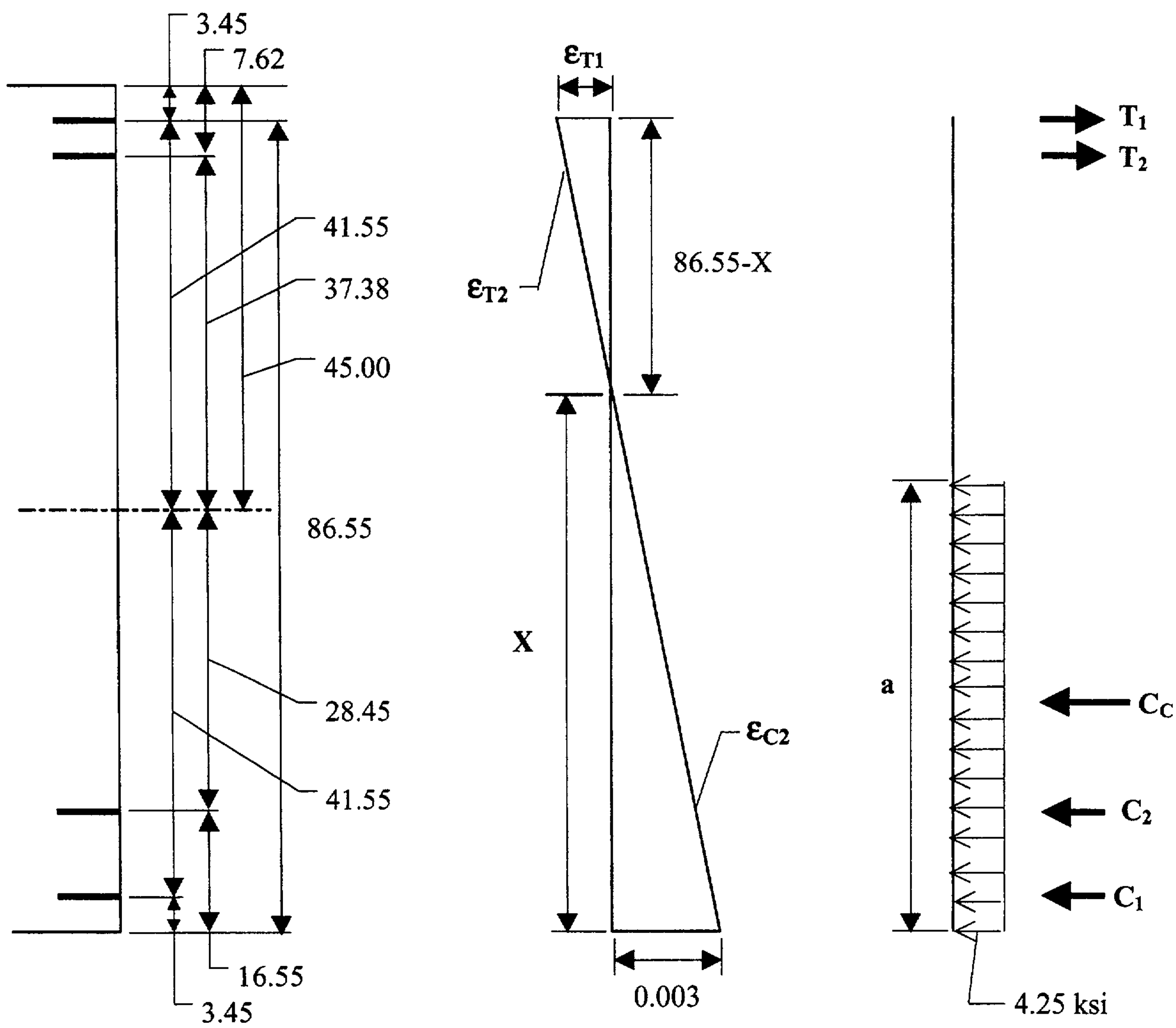


Figure 13 – North-South (Z strip) Section – Compression Controls
Compression on the Bottom
This is -Mx, see Reference 5

JOB. NO.	PGE-009	DATE	March 11, 2003
PROJECT	DCPP ISFSI		
SUBJECT	ISFSI Cask Storage Pad Steel Reinforcement		
CLIENT	PG&E-DCPP	ORIGINATOR	S. C. Tumminelli
REVIEWER	K. L. Whitmore	APPROVED	R. F. Evers
CALCULATION NO.	PGE-009-CALC-007	REVISION	0

ENERCON SERVICES, INC.

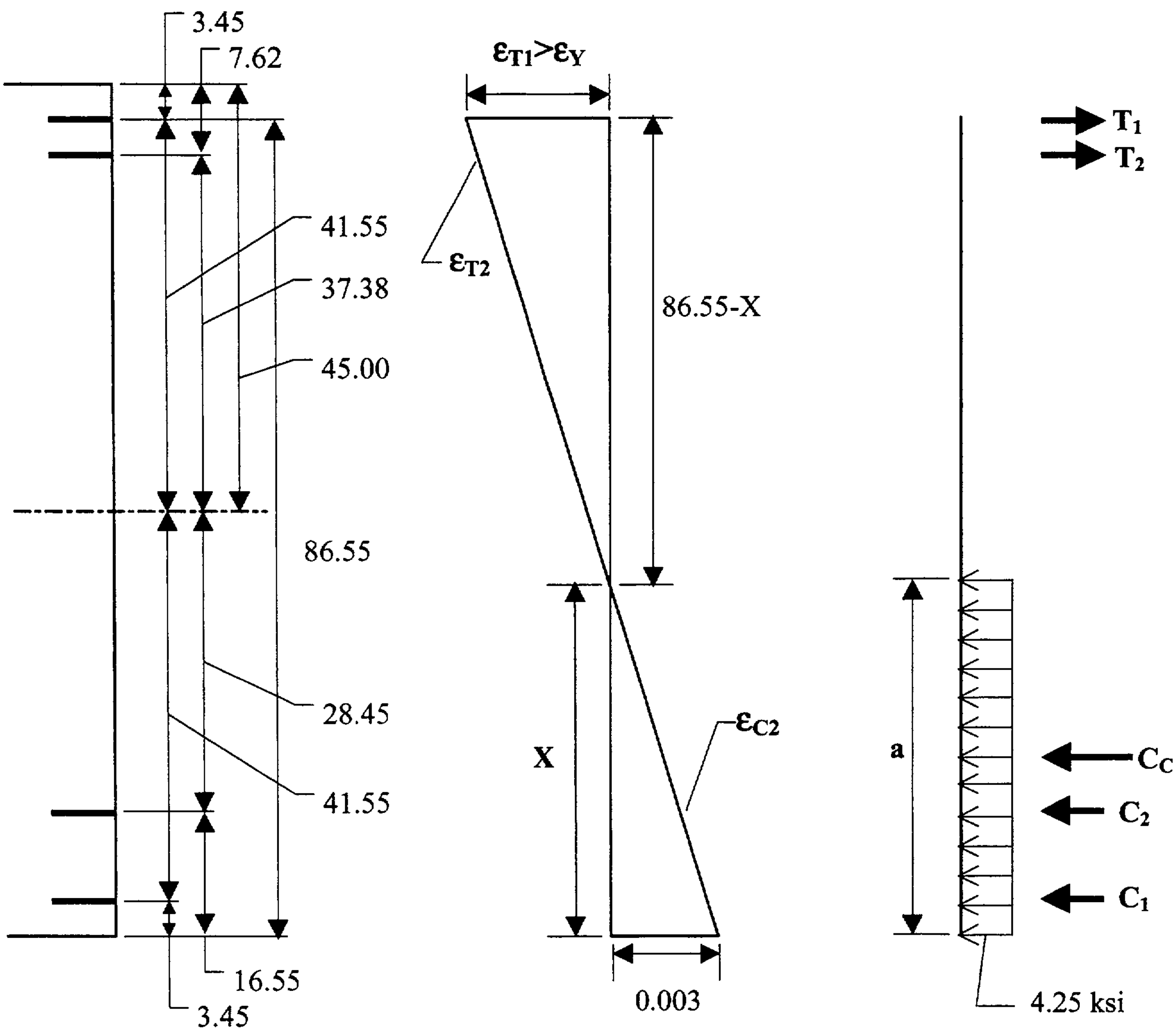


Figure 14 – North-South (Z strip) Section – Tension Controls
Compression on the Bottom
This is -Mx, see Reference 5

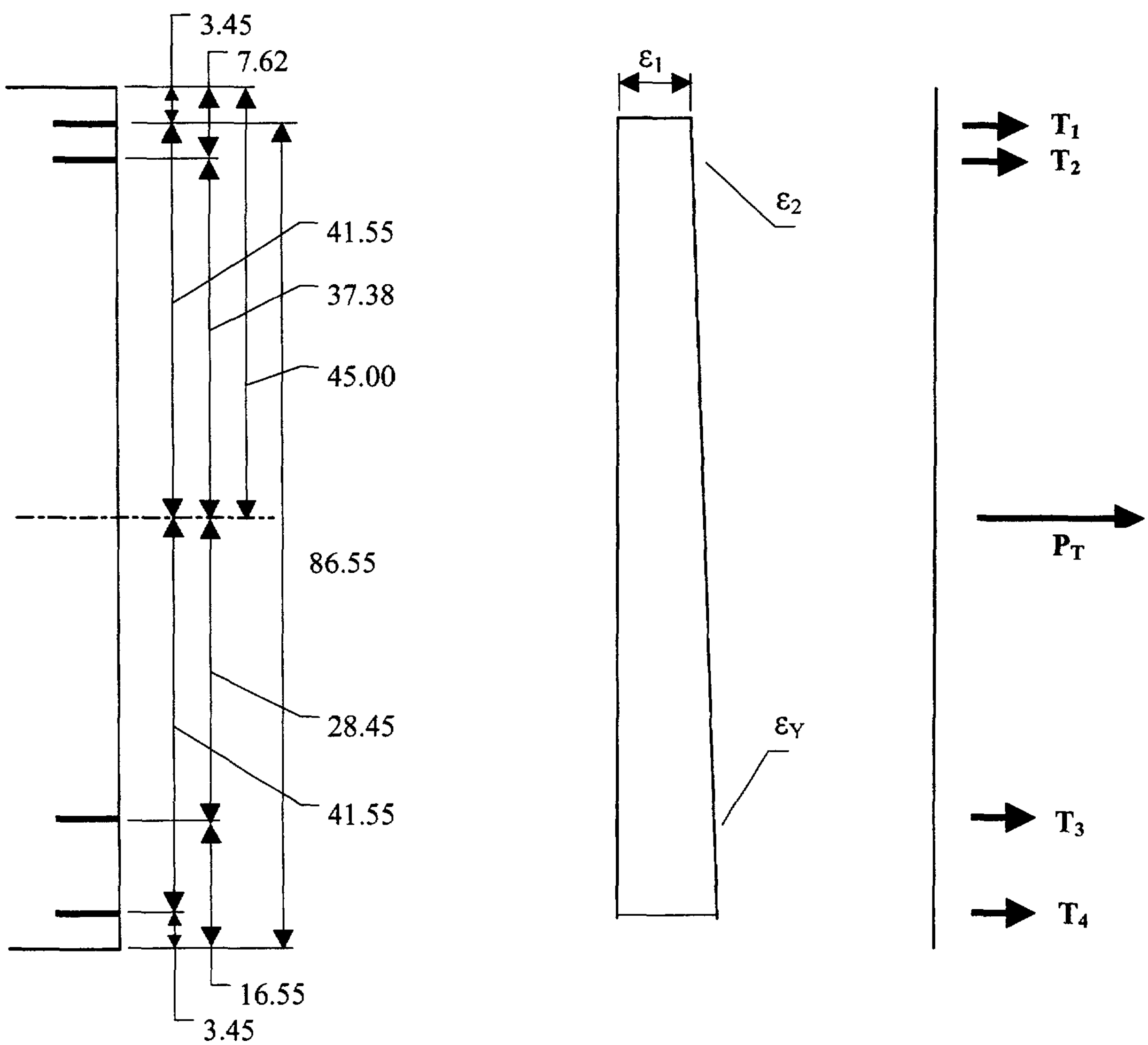


Figure 15 – North-South (Z strip) Section – Net Axial Tension
No Moment

Compute the axial force/moment interaction diagram.

North-South Net Axial Compression - No Moment – Figure 8

Since the reinforcement is not symmetrical, in order to achieve a net axial moment at the mid-height of the section, the concrete stress must vary slightly over the height of the section so that the unbalanced moment produced by the steel at a stress of Fy is offset by the concrete. Thus, the unbalanced moment is ΔM = AsFy(Net moment arm difference) = 1.27 x 60 x (37.38-28.45) = 680.5 in kips. Therefore the Δ concrete stress ($\Delta\sigma$) is computed from the equilibrium equation $\Delta M = \Delta\sigma \times S$ where S is the section modulus. $S = \frac{9x90^2}{6} = 12150\,\text{inches}^3$. And, $\Delta\sigma = \frac{\Delta M}{S} = \frac{680.5}{12150} = 0.056\,\text{ksi}$. Thus, the concrete stress varies linearly from 4.25 ksi at the top to 4.25-(2)(0.056) = 4.138 at the bottom of the section. And,

$$P_o = \left(4.138 + \left(\frac{1}{2}\right)(2)0.056\right)(9)(90) + (60)(4)(1.27) = 3701.9\,\text{kips}$$

Check on the moment:

$$M = (4.25\text{-}4.138)(0.5)(9)(90)\,((2/3)(90)\text{-}45))\text{-}76.2(37.38\text{-}28.45) = -0.1 \text{ OK}$$

North-South Balanced Condition – Tension on the Bottom - Figure 9

Compute X (distance to NA):

$$X = \left(\frac{0.003}{0.003 + 0.002069}\right)86.55 = 51.22\,\text{inches}$$

$$\varepsilon_{T1} = \varepsilon_Y = 0.002069$$

$$\varepsilon_{T2} = \left(\frac{35.33 + 3.45 - 16.55}{35.33}\right)0.002069 = 0.001302$$

$$\varepsilon_{C2} = \left(\frac{51.22 - 7.62}{51.22}\right)0.003 = 0.002554 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:

Depth of stress block: $a = 0.80 \times 51.22 = 40.98\,\text{inches}$

$$C_c = 0.85f_c^{'}ab = 4.25 \text{ x } 40.98 \text{ x } 9 = 1567.5 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85f_c^{'}\right) = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_2 = E\varepsilon A_s = 29E3 \text{ x } 0.001302 \text{ x } 1.27 = 47.9 \text{ kips}$$

$$T_1 = A_s f_Y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1567.5 + 70.8 + 70.8 - 76.2 - 47.9 = 1585.0 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)37.38 + (T_2)28.45$$

$$M_n = 1567.5\left(45 - \frac{40.98}{2}\right) + (70.8 + 76.2)41.55 + (70.8)37.38 + (47.9)28.45 = 48{,}536.5 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Tension on Bottom - Compression Controls – Figure 10

Set $\varepsilon_{T1} = \frac{\varepsilon_Y}{2} = \frac{0.002069}{2} = 0.001035$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.001035}\right) 86.55 = 64.35 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 64.35 + 3.45 - 16.55}{86.55 - 64.35}\right) 0.001035 = 0.000424$$

$$\varepsilon_{C2} = \left(\frac{64.35 - 7.62}{64.35}\right) 0.003 = 0.002645 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 64.35 = 51.48$ inches

$$C_c = 0.85 f_c' ab = 4.25 \times 51.48 \times 9 = 1969.1 \text{ kips}$$

$$C_1 = A_s (f_Y - 0.85 f_c') = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_1 = E\varepsilon A_s = 29E3 \times 0.001035 \times 1.27 = 38.1 \text{ kips}$$

$$T_2 = E\varepsilon A_s = 29E3 \times 0.000424 \times 1.27 = 15.6 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1969.1 + 70.8 + 70.8 - 38.1 - 15.6 = 2057.0 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)37.38 + (T_2)28.45$$

$$M_n = 1969.1\left(45 - \frac{51.48}{2}\right) + (70.8 + 38.1)\ 41.55 + (70.8)\ 37.38 + (15.6)28.45 = 45{,}540.0 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Tension on Bottom - Compression Controls – Figure 10

Set $\varepsilon_{T1} = 0.0$, therefore:

$$X = 86.55 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{3.45 - 16.55}{86.55}\right)0.003 = -0.000454 \quad \text{(-) indicates compression}$$

$$\varepsilon_{C2} = \left(\frac{86.55 - 7.62}{86.55}\right)0.003 = 0.002736 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:

Depth of stress block: $a = 0.80 \text{ x } 86.55 = 69.24 \text{ inches}$

$$C_c = 0.85f_c^{'}ab = 4.25 \text{ x } 69.24 \text{ x } 9 = 2648.4 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85f_c^{'}\right) = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_1 = 0.0 \text{ kips}$$

$$T_2 = E\varepsilon A_s = 29E3 \text{ x } -0.000454 \text{ x } 1.27 = -16.7 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 2648.4 + 70.8 + 70.8 - (-16.7) = 2806.7 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)37.38 + (T_2)28.45$$

$$M_n = 2648.4\left(45 - \frac{69.24}{2}\right) + (70.8)\ 41.55 + (70.8)\ 37.38 - (16.7)28.45 = 32{,}603.5 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Tension on Bottom - Tension Controls – Figure 11

Set $\varepsilon_{T2} = \varepsilon_Y = 0.002069$, (Note the change from ε_{T1} to ε_{T2}), therefore:

$$X = \left(\frac{0.003}{0.003 + 0.002069}\right)(86.55 + 3.45 - 16.55) = 43.47 \text{ inches}$$

$$\varepsilon_{T1} = \left(\frac{86.55 - 43.47}{86.55 - 43.47 + 3.45 - 16.55}\right)0.002069 = 0.002973 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{43.47 - 7.62}{43.47}\right)0.003 = 0.002474 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 43.47 = 34.78$ inches

$$C_c = 0.85 f_c' ab = 4.25 \times 34.78 \times 9 = 1330.2 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85 f_c'\right) = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1330.2 + 70.8 + 70.8 - 76.2 - 76.2 = 1319.4 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)37.38 + (T_2)28.45$$

$$M_n = 1330.2\left(45 - \frac{34.78}{2}\right) + (70.8 + 76.2)\ 41.55 + (70.8)\ 37.38 + (76.2)28.45 = 47{,}649.1 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Tension on Bottom - Tension Controls – Figure 11

Set $\varepsilon_{T1} = 2.0\varepsilon_Y = 2.0 \times 0.002069 = 0.004138$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.004138}\right)86.55 = 36.38 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 36.38 + 3.45 - 16.55}{86.55 - 36.38}\right)0.004138 = 0.003058 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{36.38 - 7.62}{36.38}\right)0.003 = 0.002372 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 36.38 = 29.10$ inches

$$C_c = 0.85f_c^{'}ab = 4.25 \times 29.10 \times 9 = 1113.1 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85f_c^{'}\right) = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1113.1 + 70.8 + 70.8 - 76.2 - 76.2 = 1102.3 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)37.38 + (T_2)28.45$$

$$M_n = 1113.1\left(45 - \frac{29.10}{2}\right) + (70.8 + 76.2)\ 41.55 + (70.8)\ 37.38 + (76.2)28.45 = 44{,}816.1 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Tension on Bottom - Tension Controls – Figure 11

Set $\varepsilon_{T1} = 4.0\varepsilon_Y = 4.0 \times 0.002069 = 0.008276$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.008276}\right)86.55 = 23.03 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 23.03 + 3.45 - 16.55}{86.55 - 23.03}\right)0.008276 = 0.006569 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{23.03 - 7.62}{23.03}\right)0.003 = 0.002007 < 0.002069$$

$$\varepsilon_{C1} = \left(\frac{23.03 - 3.45}{23.03}\right)0.003 = 0.002551 > 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 23.03 = 18.42$ inches

$$C_c = 0.85 f_c' ab = 4.25 \times 18.42 \times 9 = 704.6 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85 f_c'\right) = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = \left(E\varepsilon_{C2} - 4.25\right)1.27 = \left(29E3 \times 0.002007 - 4.25\right)1.27 = 68.5 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 704.6 + 70.8 + 68.5 - 76.2 - 76.2 = 691.5 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + \left(C_1 + T_1\right)41.55 + \left(C_2\right)37.38 + \left(T_2\right)28.45$$

$$M_n = 704.6\left(45 - \frac{18.42}{2}\right) + (70.8 + 76.2)\ 41.55 + (68.5)\ 37.38 + (76.2)28.45 = 36{,}053.9 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction - - Tension on Bottom - Tension Controls – Figure 11

Set $\varepsilon_{T1} = 8.0\varepsilon_Y = 8.0 \times 0.002069 = 0.016552$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.016552}\right)86.55 = 13.28 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 13.28 + 3.45 - 16.55}{86.55 - 13.28}\right)0.016552 = 0.013593 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{13.28 - 7.62}{13.28}\right)0.003 = 0.001279 < 0.002069$$

$$\varepsilon_{C1} = \left(\frac{13.28 - 3.45}{13.28}\right)0.003 = 0.002221 > 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 13.28 = 10.62$ inches

$$C_c = 0.85f'_c ab = 4.25 \times 10.62 \times 9 = 406.2 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85f'_c\right) = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = A_s\left(E\varepsilon_{C2} - 0.85f'_c\right) = 1.27 \times (29E3 \times 0.001279 - 4.25) = 41.7 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 406.2 + 70.8 + 41.7 - 76.2 - 76.2 = 366.3 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)37.38 + (T_2)28.45$$

$$M_n = 406.2\left(45 - \frac{10.62}{2}\right) + (70.8 + 76.2)\ 41.55 + (41.7)\ 37.38 + (76.2)28.45 = 25{,}956.6 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Tension on Bottom - Tension Controls – Figure 11

Set $\varepsilon_{T1} = 16\varepsilon_Y = 16 \text{ x } 0.002069 = 0.033104$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.033104}\right)86.55 = 7.19 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 7.19 + 3.45 - 16.55}{86.55 - 7.19}\right)0.033104 = 0.027640 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{7.19 - 7.62}{7.19}\right)0.003 = -0.000179 \text{ (-) indicates bar in tension}$$

$$\varepsilon_{C1} = \left(\frac{7.19 - 3.45}{7.19}\right)0.003 = 0.001561 < 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 7.19 = 5.75$ inches

$$C_c = 0.85f_c^{'}ab = 4.25 \text{ x } 5.75 \text{ x } 9 = 219.9 \text{ kips}$$

$$C_1 = A_s\left(E\varepsilon_{C1} - 0.85f_c^{'}\right) = 1.27 \text{ x } (29E3 \text{ x } 0.001561 - 4.25) = 52.1 \text{ kips}$$

$$C_2 = A_sE\varepsilon_{C2} = 1.27 \text{ x } 29E3 \text{ x } - 0.000179 = -6.60 \text{ kips (bar in tension)}$$

$$T_1 = A_sf_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

$$T_2 = A_sf_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 219.9 + 52.1 - 6.60 - 76.2 - 76.2 = 113.0 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)37.38 + (T_2)28.45$$

$$M_n = 219.9\left(45 - \frac{5.75}{2}\right) + (52.1 + 76.2)\ 41.55 + (-6.60)\ 37.38 + (76.2)28.45 = 16{,}515.3 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Tension on Bottom - Tension Controls – Figure 11

Set $\varepsilon_{T1} = 32\varepsilon_Y = 32 \text{ x } 0.002069 = 0.066208$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.066208}\right)86.55 = 3.75 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 3.75 + 3.45 - 16.55}{86.55 - 3.75}\right)0.066208 = 0.055733 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{3.75 - 7.62}{3.75}\right)0.003 = -0.003096 > \varepsilon_Y \text{ (-) indicates bar in tension}$$

$$\varepsilon_{C1} = \left(\frac{3.75 - 3.45}{3.75}\right)0.003 = 0.000240 < 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 3.75 = 3.00$ inches

$$C_c = 0.85f_c^{'}ab = 4.25 \text{ x } 3.00 \text{ x } 9 = 114.8 \text{ kips}$$

$$C_1 = A_s\left(E\varepsilon_{C1} - 0.85f_c^{'}\right) = 1.27 \text{ x } (29E3 \text{ x } 0.000240 - 4.25) = 3.4 \text{ kips}$$

$$C_2 = A_s f_Y = 1.27 \text{ x } (-60) = -76.2 \text{ kips (bar in tension)}$$

$$T_1 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 114.8 + 3.4 - 76.2 - 76.2 - 76.2 = -110.4 \text{ kips (-) indicates tension}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)37.38 + (T_2)28.45$$

$$M_n = 114.8\left(45 - \frac{3.00}{2}\right) + (3.4 + 76.2)\ 41.55 + (-76.2)\ 37.38 + (76.2)28.45 = 7620.7 \text{ in - kips}$$

North-South Net Axial Tension - No Moment – Figure 15

As with the net compression case, the strain across the section must vary slightly in order to achieve zero moment at mid-height.

Set $\varepsilon_3 = \varepsilon_y = 0.002069$ and therefore $T_3 = T_4 = 76.2$ kips

Moment equilibrium requires:

$$M = 0 = (T_1)\,41.55 + (T_2)\,37.38 - (T_3)\,28.45 - (T_4)\,41.55$$

Use the strain:

$$\varepsilon_2 = \varepsilon_1 + \left(\frac{7.62 - 3.45}{41.55 + 28.45}\right)(\varepsilon_y - \varepsilon_1)$$

$$\varepsilon_2 = \varepsilon_1 + 0.000123 - 0.05957\varepsilon_1$$

$$\varepsilon_2 = 0.9404\varepsilon_1 + 0.000123$$

Now, use the stress-strain relation and the bar areas:

$$T_1 = A_s E\varepsilon_1 = 1.27 \text{ x } 29\text{E}3 \text{ x } \varepsilon_1 = 36{,}830\varepsilon_1$$

and

$$T_2 = A_s E\varepsilon_2 = 1.27 \text{ x } 29\text{E}3 \text{ x } (0.9404\varepsilon_1 + 0.000123) = 34{,}634.9\varepsilon_1 + 4.53$$

Substitute in to equation for M

$$0 = (36{,}830\varepsilon_1)41.55 + (34{,}634\varepsilon_1 + 4.53)37.38 - (76.2)28.45 - (76.2)41.55$$

$$0 = 2{,}824{,}905\varepsilon_1 - 5{,}164.7$$

$$\varepsilon_1 = \frac{5164.7}{2{,}824{,}905} = 0.001828$$

$$\varepsilon_2 = (0.9404)0.001828 + 0.000123 = 0.001842$$

And, $T_1 = 1.27 \text{ x } 29\text{E}3 \text{ x } 0.001828 = 67.3$ kip

And, $T_2 = 1.27 \times 29E3 \times 0.001842 = 67.8$ kip

Thus, $T = P_t = 67.3+67.8+76.2+76.2=287.5$ kip

Check on the moment:

$0 = (67.3)41.55 + (67.8)37.38 - (76.2)28.45 - (76.2)41.55 = -3.3$ OK

North-South Calculation of φ per ACI – Tension on Bottom

$\varphi = 0.90$ for tension plus flexure

$\varphi = 0.70$ for compression plus flexure

$0.1f_c^{'}A_g = 0.1 \times 5.00 \times 9 \times 90 = 405 \text{ kip} > \varphi P_b = \varphi\,(1585.0)$

$$C_n = 366.3 \text{ kips},\quad \varphi = \frac{0.90}{1+\frac{2C_n}{f_c^{'}A_g}} = \frac{0.90}{1+\frac{2 \times 366.3}{5 \times 9 \times 90}} = 0.76$$

$$C_n = 113.0 \text{ kips},\quad \varphi = \frac{0.90}{1+\frac{2C_n}{f_c^{'}A_g}} = \frac{0.90}{1+\frac{2 \times 113.0}{5 \times 9 \times 90}} = 0.85$$

North-South Balanced Condition – Compression on the Bottom - Figure 12

Compute X (distance to NA):

$$X = \left(\frac{0.003}{0.003 + 0.002069}\right) 86.55 = 51.22 \text{ inches}$$

$$\varepsilon_{T1} = \varepsilon_Y = 0.002069$$

$$\varepsilon_{T2} = \left(\frac{35.33 + 3.45 - 7.62}{35.33}\right) 0.002069 = 0.001825$$

$$\varepsilon_{C2} = \left(\frac{51.22 - 16.55}{51.22}\right) 0.003 = 0.002031 < 0.002069$$

$$\varepsilon_{C1} = \left(\frac{51.22 - 3.45}{51.22}\right) 0.003 = 0.002798 > 0.002069$$

Compute individual internal forces:

Depth of stress block: $a = 0.80 \text{ x } 51.22 = 40.98 \text{ inches}$

$$C_c = 0.85 f_c' ab = 4.25 \text{ x } 40.98 \text{ x } 9 = 1567.5 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85 f_c'\right) = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = A_s\left(E\varepsilon - 0.85 f_c'\right) = 1.27 \text{ x } ((29E3)(0.002031) - 4.25) = 69.4 \text{ kips}$$

$$T_2 = E\varepsilon A_s = 29E3 \text{ x } 0.001825 \text{ x } 1.27 = 67.2 \text{ kips}$$

$$T_1 = A_s f_Y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1567.5 + 70.8 + 69.4 - 76.2 - 67.2 = 1564.3 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)28.45 + (T_2)37.38$$

$$M_n = 1567.5\left(45 - \frac{40.98}{2}\right) + (70.8 + 76.2)41.55 + (69.4)28.45 + (67.2)37.38 = 49{,}013.6 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Compression on Bottom - Comp Controls – Figure 13

Set $\varepsilon_{T1} = \frac{\varepsilon_Y}{2} = \frac{0.002069}{2} = 0.001035$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.001035}\right) 86.55 = 64.35 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 64.35 + 3.45 - 7.62}{86.55 - 64.35}\right) 0.001035 = 0.000840$$

$$\varepsilon_{C2} = \left(\frac{64.35 - 16.55}{64.35}\right) 0.003 = 0.002228 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 64.35 = 51.48$ inches

$$C_c = 0.85 f_c' ab = 4.25 \times 51.48 \times 9 = 1969.1 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85 f_c'\right) = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_1 = E\varepsilon A_s = 29E3 \times 0.001035 \times 1.27 = 38.1 \text{ kips}$$

$$T_2 = E\varepsilon A_s = 29E3 \times 0.000840 \times 1.27 = 30.9 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1969.1 + 70.8 + 70.8 - 38.1 - 30.9 = 2041.7 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)28.45 + (T_2)37.38$$

$$M_n = 1969.1\left(45 - \frac{51.48}{2}\right) + (70.8 + 38.1)\ 41.55 + (70.8)\ 28.45 + (30.9)37.38 = 45{,}619.0 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Compression on Bottom - Comp Controls – Figure 13

Set $\varepsilon_{T1} = 0.0$, therefore:

$$X = 86.55 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{3.45 - 7.62}{86.55}\right)0.003 = -0.000145 \quad \text{(-) indicates compression}$$

$$\varepsilon_{C2} = \left(\frac{86.55 - 16.55}{86.55}\right)0.003 = 0.002426 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:

Depth of stress block: $a = 0.80 \text{ x } 86.55 = 69.24$ inches

$$C_c = 0.85 f_c' ab = 4.25 \text{ x } 69.24 \text{ x } 9 = 2648.4 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85 f_c'\right) = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_1 = 0.0 \text{ kips}$$

$$T_2 = E\varepsilon A_s = 29E3 \text{ x } -0.000145 \text{ x } 1.27 = -5.3 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 2648.4 + 70.8 + 70.8 - (-5.3) = 2795.3 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)28.45 + (T_2)37.38$$

$$M_n = 2648.4\left(45 - \frac{69.24}{2}\right) + (70.8)\ 41.55 + (70.8)\ 28.45 - (5.3)37.38 = 32{,}248.3 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Compression on Bottom - Tension Controls – Figure 14

Set $\varepsilon_{T1} = 1.5\,\varepsilon_Y = 1.5 \text{ x } 0.002069 = 0.003104$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.003104}\right)(86.55) = 42.54 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 42.54 + 3.45 - 7.62}{86.55 - 42.54}\right)0.003104 = 0.002811 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{42.54 - 16.55}{42.54}\right)0.003 = 0.001833 < 0.002069$$

$$\varepsilon_{C1} = \left(\frac{42.54 - 3.45}{42.54}\right)0.003 = 0.002757 > 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 42.54 = 34.03$ inches

$$C_c = 0.85f_c^{'}ab = 4.25 \text{ x } 34.03 \text{ x } 9 = 1301.6 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85f_c^{'}\right) = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = A_s\left(E\varepsilon - 0.85f_c^{'}\right) = 1.27 \text{ x } ((29E3)(0.001833) - 4.25) = 62.1 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1301.6 + 70.8 + 62.1 - 76.2 - 76.2 = 1282.1 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)28.45 + (T_2)37.38$$

$$M_n = 1301.6\left(45 - \frac{34.03}{2}\right) + (70.8 + 76.2)\ 41.55 + (62.1)\ 28.45 + (76.2)37.38 = 47{,}148.2 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Compression on Bottom - Tension Controls – Figure 14

Set $\varepsilon_{T1} = 2.0\varepsilon_Y = 2.0 \times 0.002069 = 0.004138$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.004138}\right) 86.55 = 36.38 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 36.38 + 3.45 - 7.62}{86.55 - 36.38}\right) 0.004138 = 0.003794 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{36.38 - 16.55}{36.38}\right) 0.003 = 0.001635 < 0.002069$$

$$\varepsilon_{C1} = \left(\frac{36.38 - 3.45}{36.38}\right) 0.003 = 0.002716 > 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 36.38 = 29.10$ inches

$$C_c = 0.85 f_c' ab = 4.25 \times 29.10 \times 9 = 1113.1 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85 f_c'\right) = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = A_s\left(E\varepsilon - 0.85 f_c'\right) = 1.27 \times ((29E3)(0.001635) - 4.25) = 54.8 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1113.1 + 70.8 + 54.8 - 76.2 - 76.2 = 1086.3 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1) 41.55 + (C_2) 28.45 + (T_2) 37.38$$

$$M_n = 1113.1\left(45 - \frac{29.10}{2}\right) + (70.8 + 76.2)\ 41.55 + (54.8)\ 28.45 + (76.2) 37.38 = 44{,}409.2 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Compression on Bottom - Tension Controls – Figure 14

Set $\varepsilon_{T1} = 4.0\varepsilon_Y = 4.0 \times 0.002069 = 0.008276$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.008276}\right)86.55 = 23.03 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 23.03 + 3.45 - 7.62}{86.55 - 23.03}\right)0.008276 = 0.007733 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{23.03 - 16.55}{23.03}\right)0.003 = 0.000844 < 0.002069$$

$$\varepsilon_{C1} = \left(\frac{23.03 - 3.45}{23.03}\right)0.003 = 0.002551 > 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 23.03 = 18.42$ inches

$$C_c = 0.85f_c'ab = 4.25 \times 18.42 \times 9 = 704.6 \text{ kips}$$

$$C_1 = A_s(f_Y - 0.85f_c') = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = (E\varepsilon_{C2} - 4.25)1.27 = (29E3 \times 0.000844 - 4.25)1.27 = 25.7 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 704.6 + 70.8 + 25.7 - 76.2 - 76.2 = 648.7 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)28.45 + (T_2)37.38$$

$$M_n = 704.6\left(45 - \frac{18.42}{2}\right) + (70.8 + 76.2)\ 41.55 + (25.7)\ 28.45 + (76.2)37.38 = 34{,}905.0 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Compression on Bottom - Tension Controls – Figure 14

Set $\varepsilon_{T1} = 8.0\varepsilon_Y = 8.0 \text{ x } 0.002069 = 0.016552$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.016552}\right)86.55 = 13.28 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 13.28 + 3.45 - 7.62}{86.55 - 13.28}\right)0.016552 = 0.015610 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{13.28 - 16.55}{13.28}\right)0.003 = -0.000739 > -0.002069 \text{ (-) indicates bar in tension}$$

$$\varepsilon_{C1} = \left(\frac{13.28 - 3.45}{13.28}\right)0.003 = 0.002221 > 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 13.28 = 10.62 \text{ inches}$

$$C_c = 0.85f'_c ab = 4.25 \text{ x } 10.62 \text{ x } 9 = 406.2 \text{ kips}$$

$$C_1 = A_s(f_Y - 0.85f'_c) = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = A_s(E\varepsilon_{C2}) = 1.27 \text{ x } (29E3 \text{ x}(-0.000739)) = -27.2 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 406.2 + 70.8 - 27.2 - 76.2 - 76.2 = 297.4 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)28.45 + (T_2)37.38$$

$$M_n = 406.2\left(45 - \frac{10.62}{2}\right) + (70.8 + 76.2)\ 41.55 + (-27.2)\ 28.45 + (76.2)37.38 = 24{,}304.4 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Compression on Bottom - Tension Controls – Figure 14

Set $\varepsilon_{T1} = 16\varepsilon_Y = 16 \text{ x } 0.002069 = 0.033104$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.033104}\right)86.55 = 7.19 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 7.19 + 3.45 - 7.62}{86.55 - 7.19}\right)0.033104 = 0.031365 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{7.19 - 16.55}{7.19}\right)0.003 = -0.003905 < -0.002069 \text{ (-) indicates bar in tension}$$

$$\varepsilon_{C1} = \left(\frac{7.19 - 3.45}{7.19}\right)0.003 = 0.001561 < 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 7.19 = 5.75$ inches

$$C_c = 0.85f_c^{'}ab = 4.25 \text{ x } 5.75 \text{ x } 9 = 219.9 \text{ kips}$$

$$C_1 = A_s\left(E\varepsilon_{C1} - 0.85f_c^{'}\right) = 1.27 \text{ x } (29E3 \text{ x } 0.001561 - 4.25) = 52.1 \text{ kips}$$

$$C_2 = -A_s f_y = -1.27 \text{ x } 60 = -76.2 \text{ kips (bar in tension)}$$

$$T_1 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 219.9 + 52.1 - 76.2 - 76.2 - 76.2 = 43.4 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + \left(C_1 + T_1\right)41.55 + \left(C_2\right)28.45 + \left(T_2\right)37.38$$

$$M_n = 219.9\left(45 - \frac{5.75}{2}\right) + (52.1 + 76.2)\ 41.55 + (-76.2)\ 28.45 + (76.2)37.38 = 15{,}274.6 \text{ in - kips}$$

N-S Nominal Force/Moment Interaction – Compression on Bottom - Tension Controls – Figure 14

Set $\varepsilon_{T1} = 32\varepsilon_Y = 32 \text{ x } 0.002069 = 0.066208$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.066208}\right)86.55 = 3.75 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{86.55 - 3.75 + 3.45 - 7.62}{86.55 - 3.75}\right)0.066208 = 0.062874 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{3.75 - 16.55}{3.75}\right)0.003 = -0.01024 < -\varepsilon_Y \text{ (-) indicates bar in tension}$$

$$\varepsilon_{C1} = \left(\frac{3.75 - 3.45}{3.75}\right)0.003 = 0.000240 < 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 3.75 = 3.00 \text{ inches}$

$$C_c = 0.85f_c'ab = 4.25 \text{ x } 3.00 \text{ x } 9 = 114.8 \text{ kips}$$

$$C_1 = A_s\left(E\varepsilon_{C1} - 0.85f_c'\right) = 1.27 \text{ x } (29E3 \text{ x } 0.000240 - 4.25) = 3.4 \text{ kips}$$

$$C_2 = A_s f_Y = 1.27 \text{ x } (-60) = -76.2 \text{ kips (bar in tension)}$$

$$T_1 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 114.8 + 3.4 - 76.2 - 76.2 - 76.2 = -110.4 \text{ kips (-) indicates tension}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)41.55 + (C_2)28.45 + (T_2)37.38$$

$$M_n = 114.8\left(45 - \frac{3.00}{2}\right) + (3.4 + 76.2)\ 41.55 + (-76.2)\ 28.45 + (76.2)37.38 = 8981.6 \text{ in - kips}$$

North-South Calculation of φ per ACI – Compresion on Bottom

$\varphi = 0.90$ for tension plus flexure

$\varphi = 0.70$ for compression plus flexure

$$0.1f_c^{'}A_g = 0.1 \text{ x } 5.00 \text{ x } 9 \text{ x } 90 = 405 \text{ kip} > \varphi P_b = \varphi\ (1585.0)$$

$$C_n = 297.4 \text{ kips}, \quad \varphi = \frac{0.90}{1+\frac{2C_n}{f_c^{'}A_g}} = \frac{0.90}{1+\frac{2 \text{ x } 297.4}{5 \text{ x } 9 \text{ x } 90}} = 0.78$$

$$C_n = 43.4 \text{ kips}, \quad \varphi = \frac{0.90}{1+\frac{2C_n}{f_c^{'}A_g}} = \frac{0.90}{1+\frac{2 \text{ x } 43.4}{5 \text{ x } 9 \text{ x } 90}} = 0.88$$

North-South Maximum Design Axial Load Strength

Per Reference 11, 10.3.5.2, the maximum design axial load strength shall not be greater than $0.80\varphi P_o$. Therefore, φC_n is limited to 0.80 x 0.70 x 3703.2 = 2073.8 in Table 4 below.

Table 4 below presents the North-South (Z Strip) section capacity data computed above. The signs of the moment are adjusted to conform to the sign convention established in Reference 5. Therefore, the moments that produce tension on the bottom are (+) and those that produce compression on the bottom are (-).

Table 4 – North-South Section Capacity Data

Moment Sign (±) and Condition	M_n in-kips	C_n kips	φ	φ M_n in-kips	φ C_n kips	e inches
Compression no moment	0	3701.9	0.7	0	2591.3*	0
ACI Code Maximum				18,848.8**	2073.8	9.089
(+) Compression controls	32,603.5	2806.7	0.7	22,822.5	1964.7	11.616
(+) Compression controls	45,540.0	2057.0	0.7	31,878.0	1439.9	22.139
(+) Balanced Condition	48,536.5	1585.0	0.7	33,975.6	1109.5	30.622
(+) Tension controls	47,649.1	1319.4	0.7	33,354.4	923.6	36.113
(+) Tension controls	44,816.1	1102.3	0.7	31,371.3	771.6	40.657
(+) Tension controls	36,053.9	691.5	0.7	25,237.7	484.1	52.133
(+) Tension controls	25,956.6	366.3	0.76	19,727.0	278.4	70.858
(+) Tension controls	16,515.3	113.0	0.85	14,038.0	96.1	146.077
(+) Tension controls	7620.7	-110.4	0.9	6858.6	-99.4	-69.000
Tension no moment	0	-287.6	0.9	0	-258.8	0
ACI Code Maximum				-18,408.4***	2073.8	-8.877
(-) Compression controls	-32,248.3	2795.3	0.7	-22,573.8	1956.7	-11.537
(-) Compression controls	-45,619.0	2041.7	0.7	-31933.3	1429.2	-22.343
(-) Balanced Condition	-49,013.6	1564.3	0.7	-34,309.5	1095.0	-31.333
(-) Tension controls	-47,148.2	1282.1	0.7	-33,003.7	897.5	-36.773
(-) Tension controls	-44,409.2	1086.3	0.7	-31,086.4	760.4	-40.882
(-) Tension controls	-34,905.0	648.7	0.7	-24,433.5	454.1	-53.806
(-) Tension controls	-24,304.4	297.4	0.78	-18,957.4	232.0	-81.713
(-) Tension controls	-15,274.6	43.4	0.88	-13,441.6	38.2	-351.874
(-) Tension controls	-8981.6	-110.4	0.9	-8083.4	-99.4	81.322

* This value is used only for linear interpolation for the allowable moment associated with the Code maximum compressive load, see notes ** and *** below.

** Linearly interpolated from the values above and below using 2591.3 = 0.70 x 3701.9 for the φ C_n with the zero moment.

*** Linearly interpolated as above.

East-West Section Concrete Capacity

The calculations below present the section capacity data for the EW section. Figures 16 to 23 provide the graphical information needed to follow the calculations.

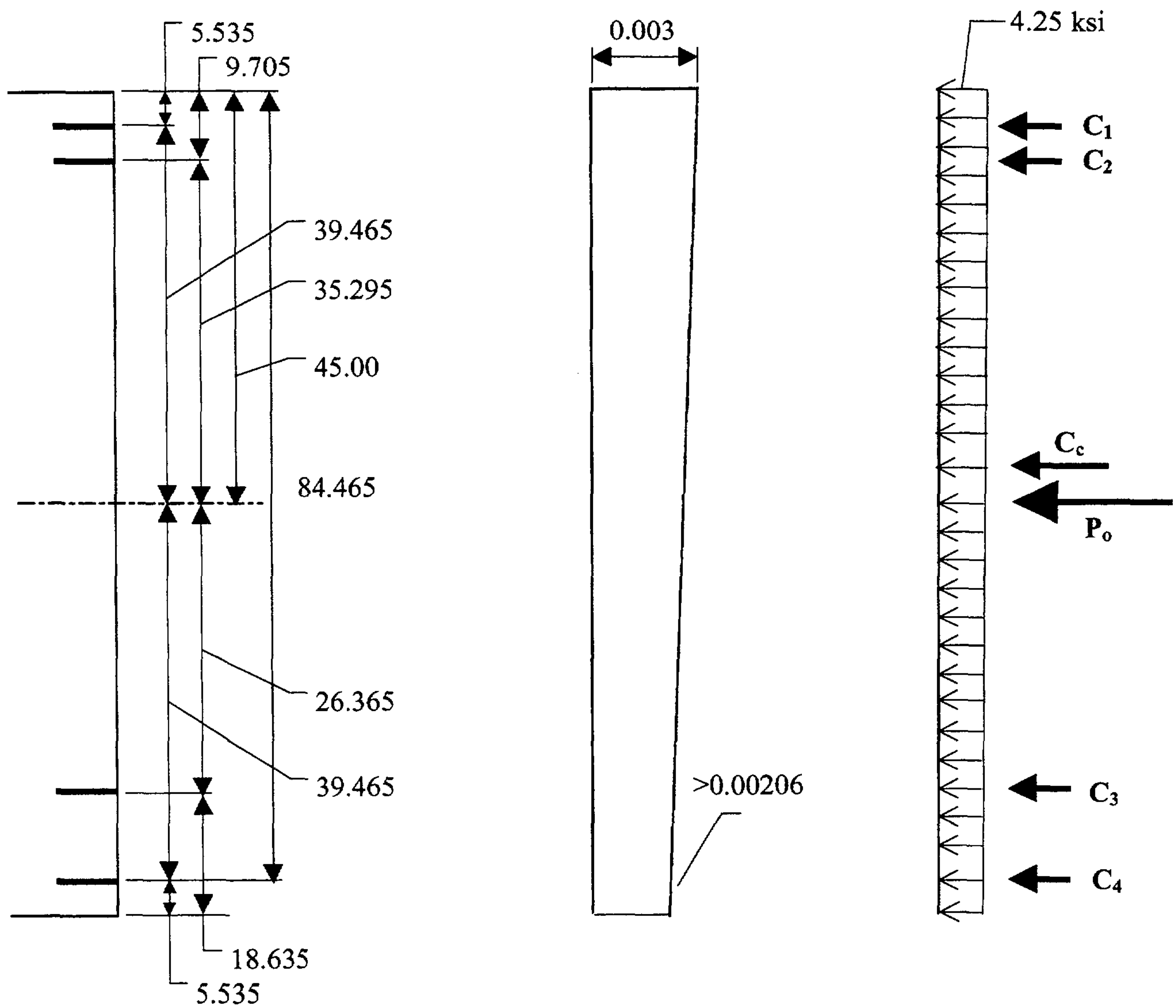


Figure 16 – East-West (X strip) Section – Net Axial Compression No Moment

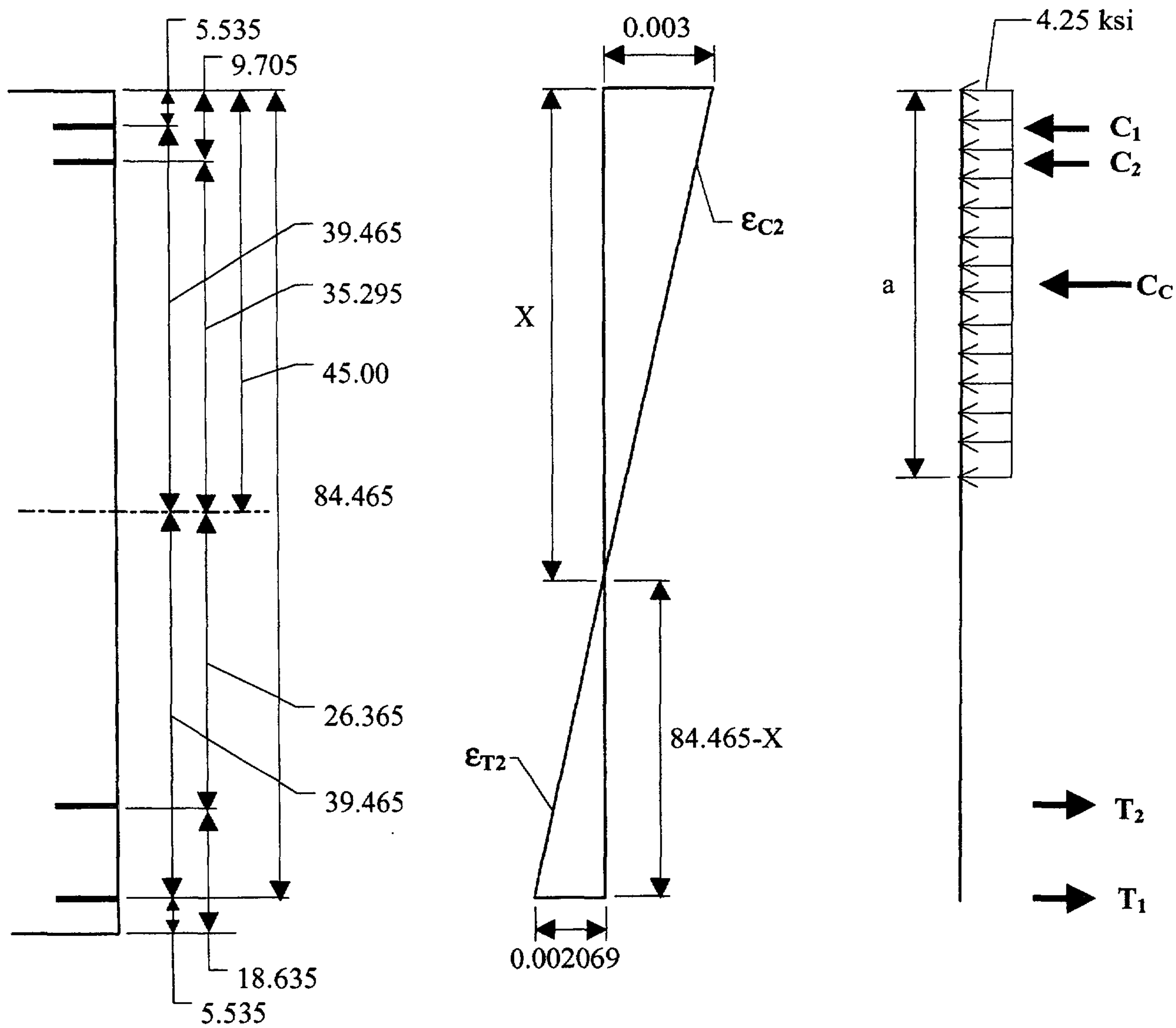


Figure 17 – East-West (X strip) Section – Balanced Condition
Tension on the Bottom
This is -Mz, see Reference 5

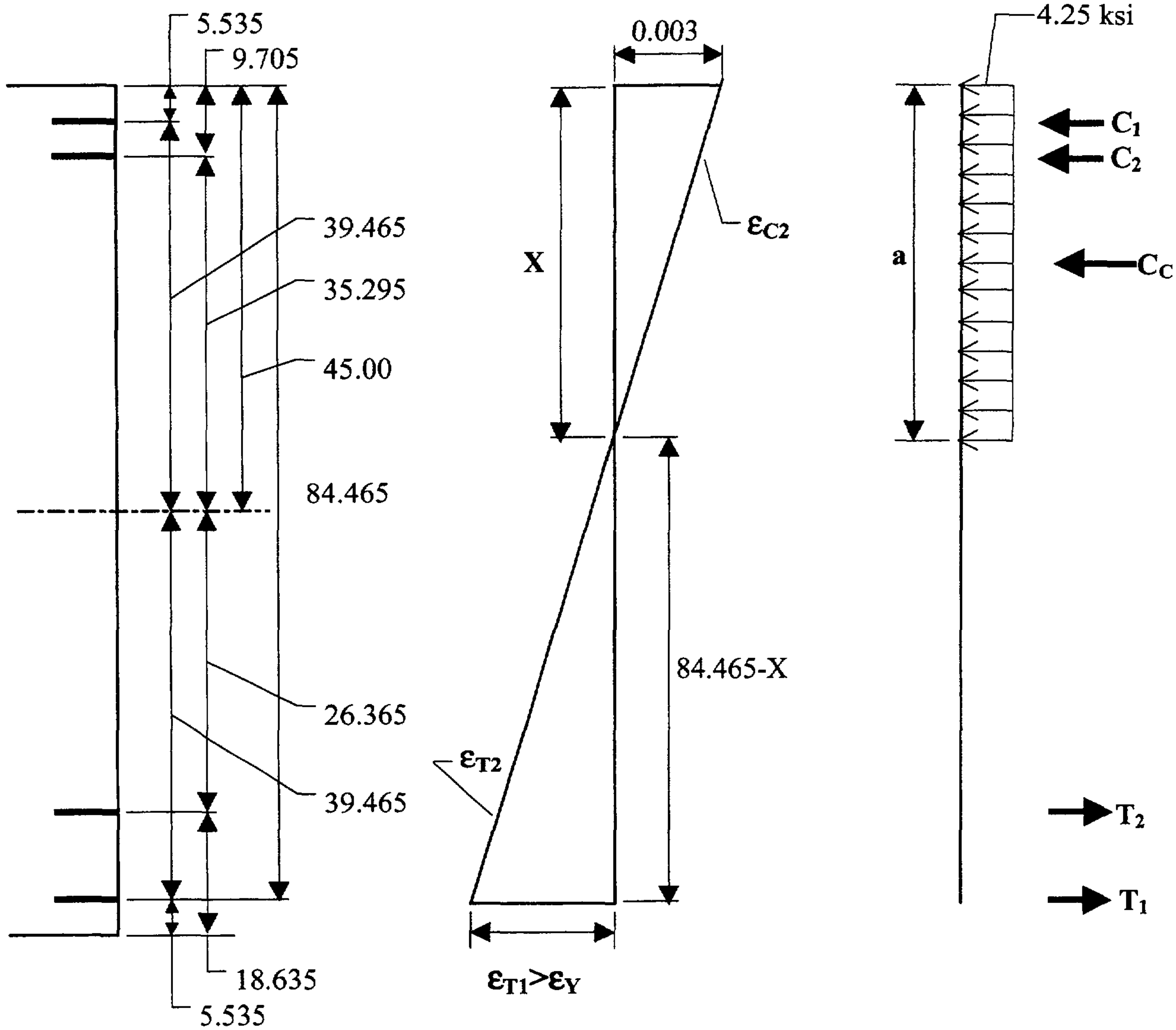


Figure 18 – East-West (X strip) Section – Compression Controls
Tension on the Bottom
This is -Mz, see Reference 5

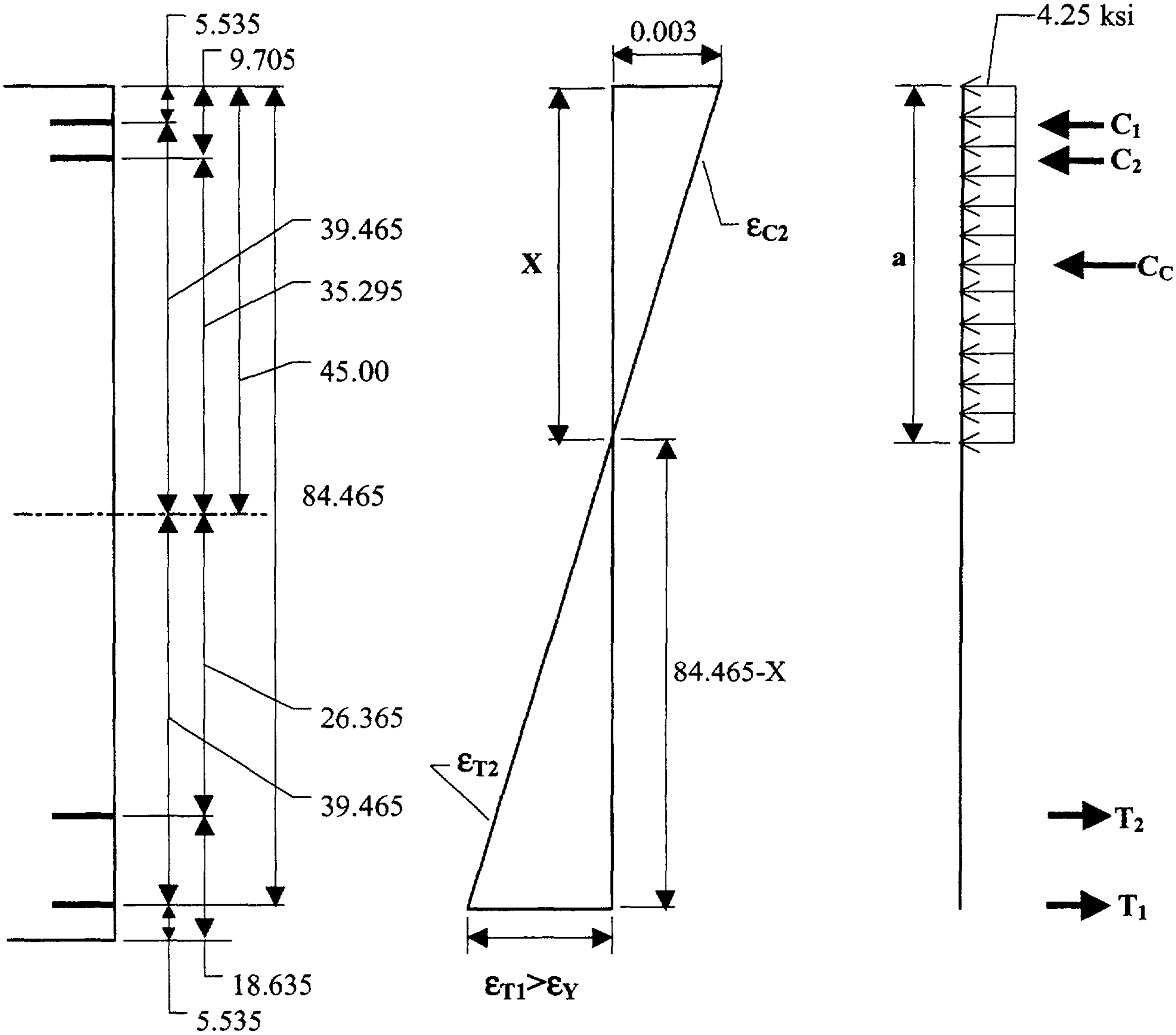


Figure 19 – East-West (X strip) Section – Tension Controls
Tension on the Bottom
This is -Mz, see Reference 5

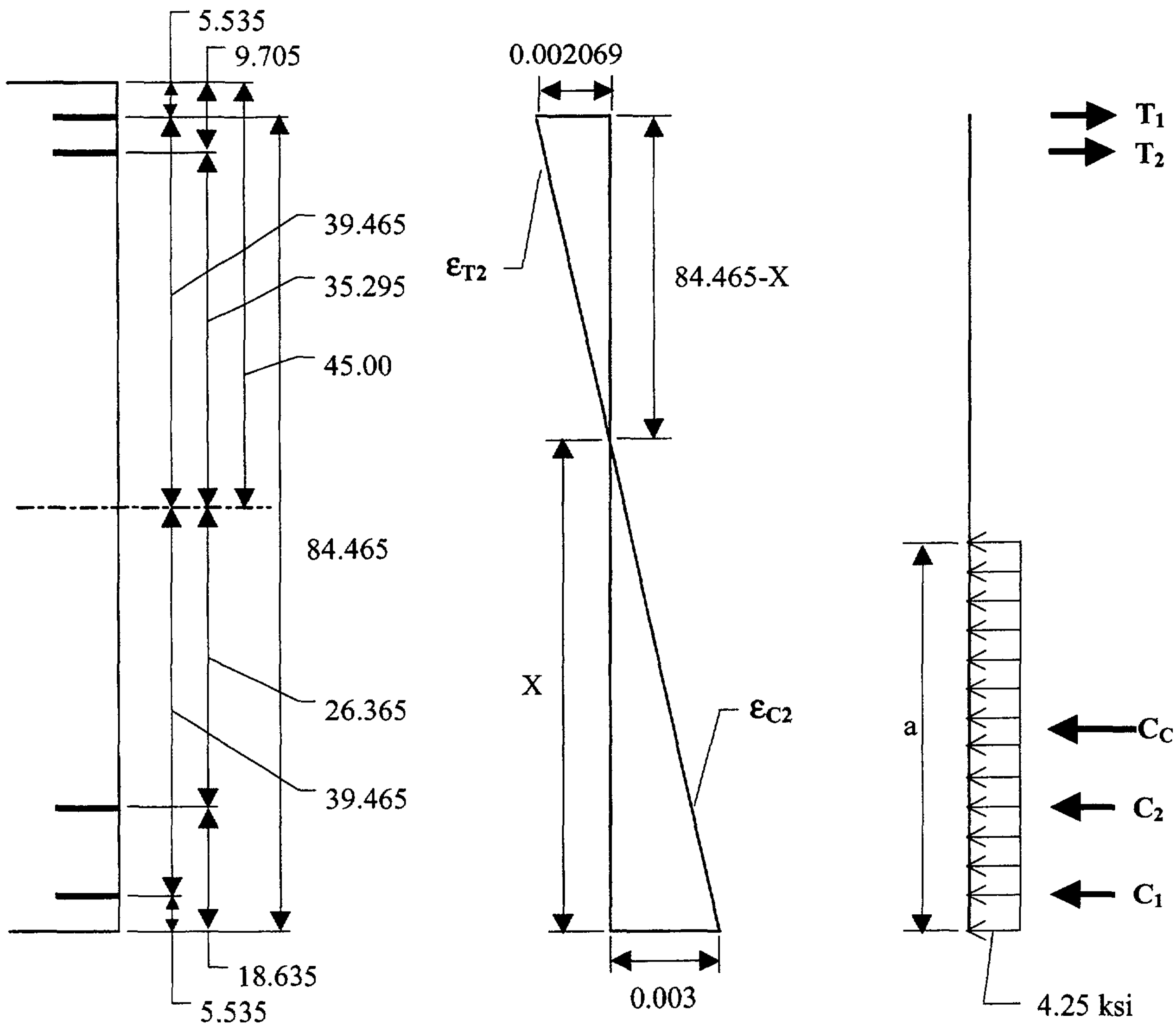


Figure 20 – East-West (X strip) Section – Balanced Condition
Compression on the Bottom
This is Mz, see Reference 5

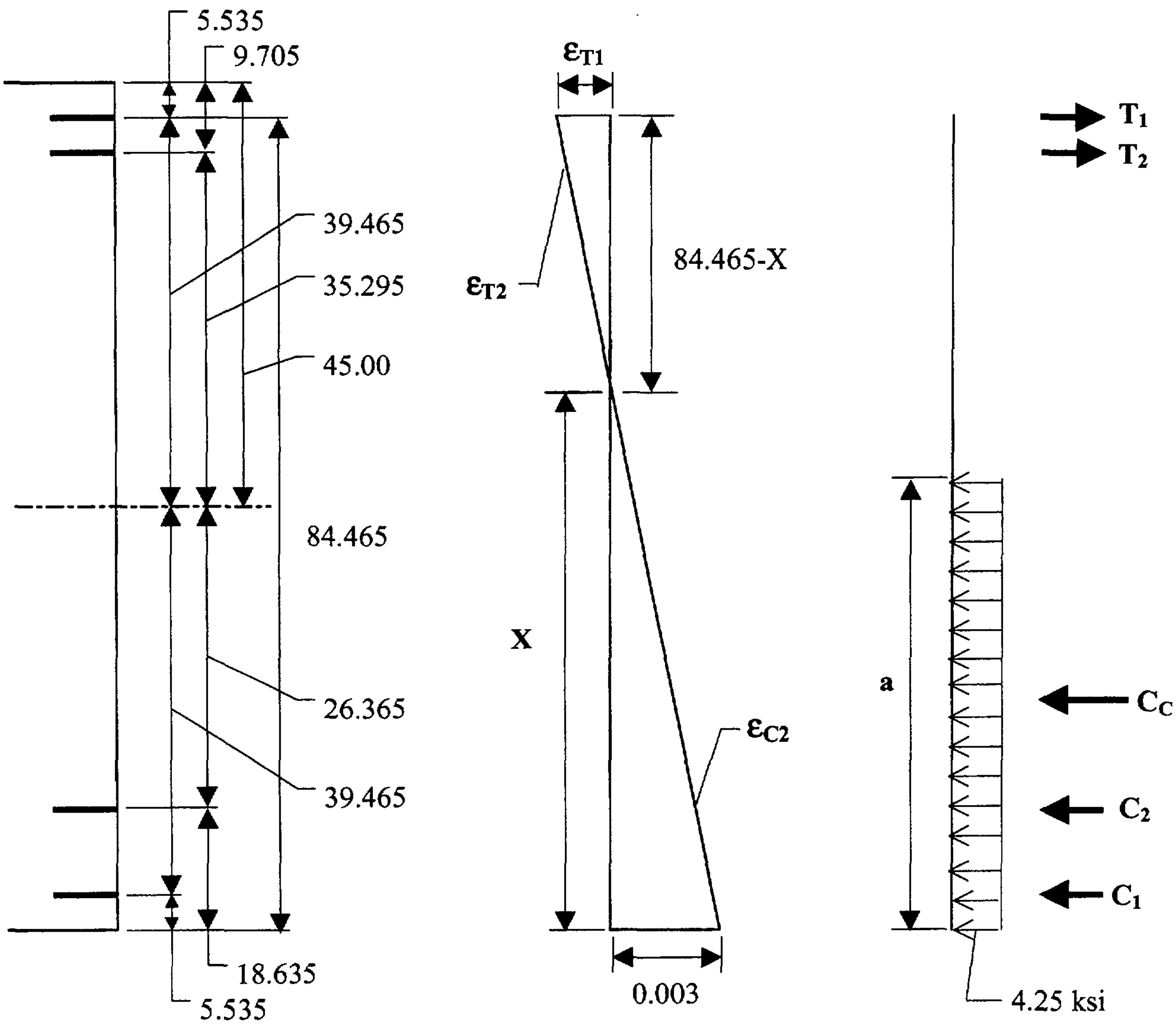


Figure 21 – East-West (X strip) Section – Compression Controls
Compression on the Bottom
This is Mz, see Reference 5

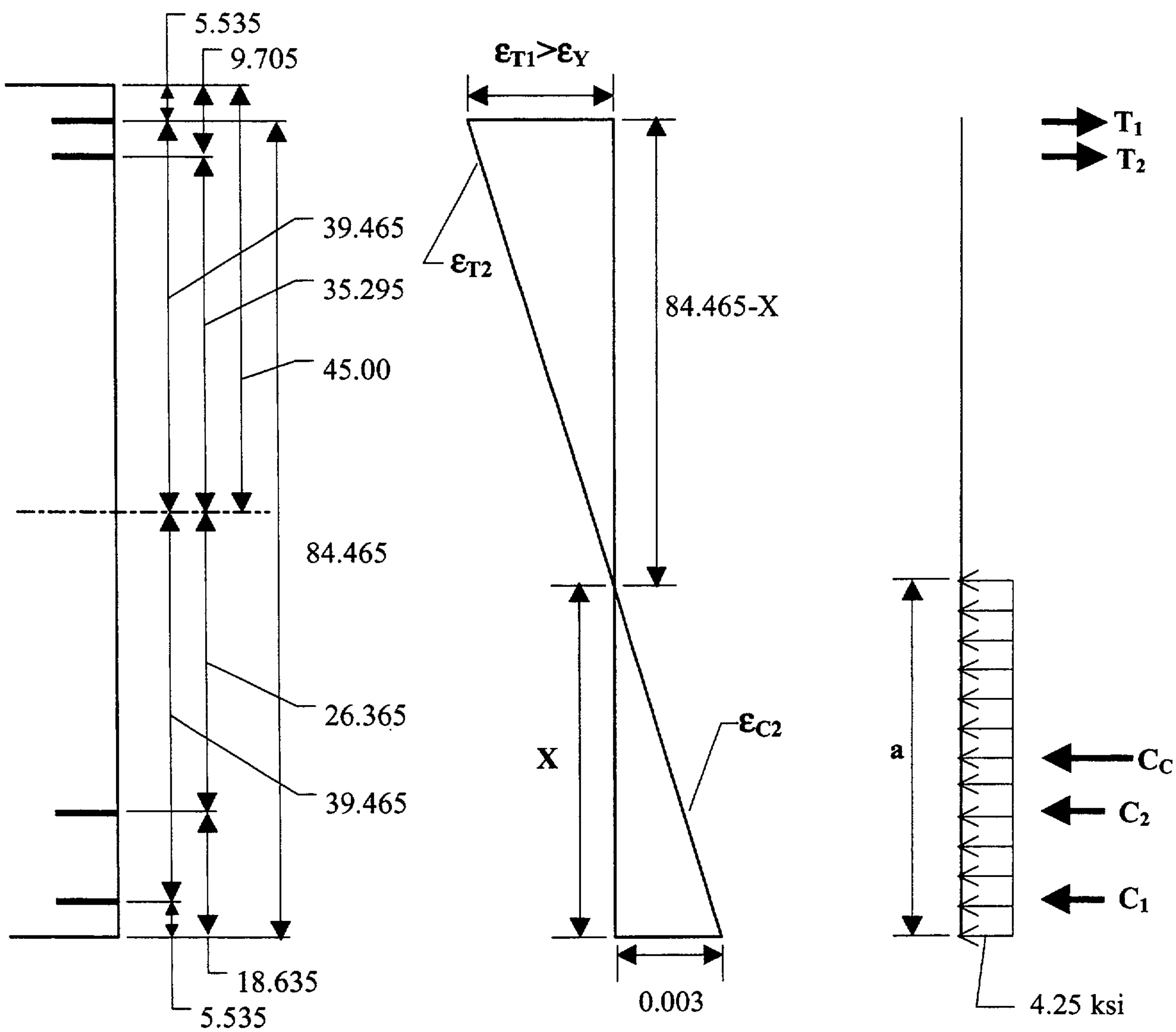


Figure 22 – East-West (X strip) Section – Tension Controls
Compression on the Bottom
This is Mz, see Reference 5

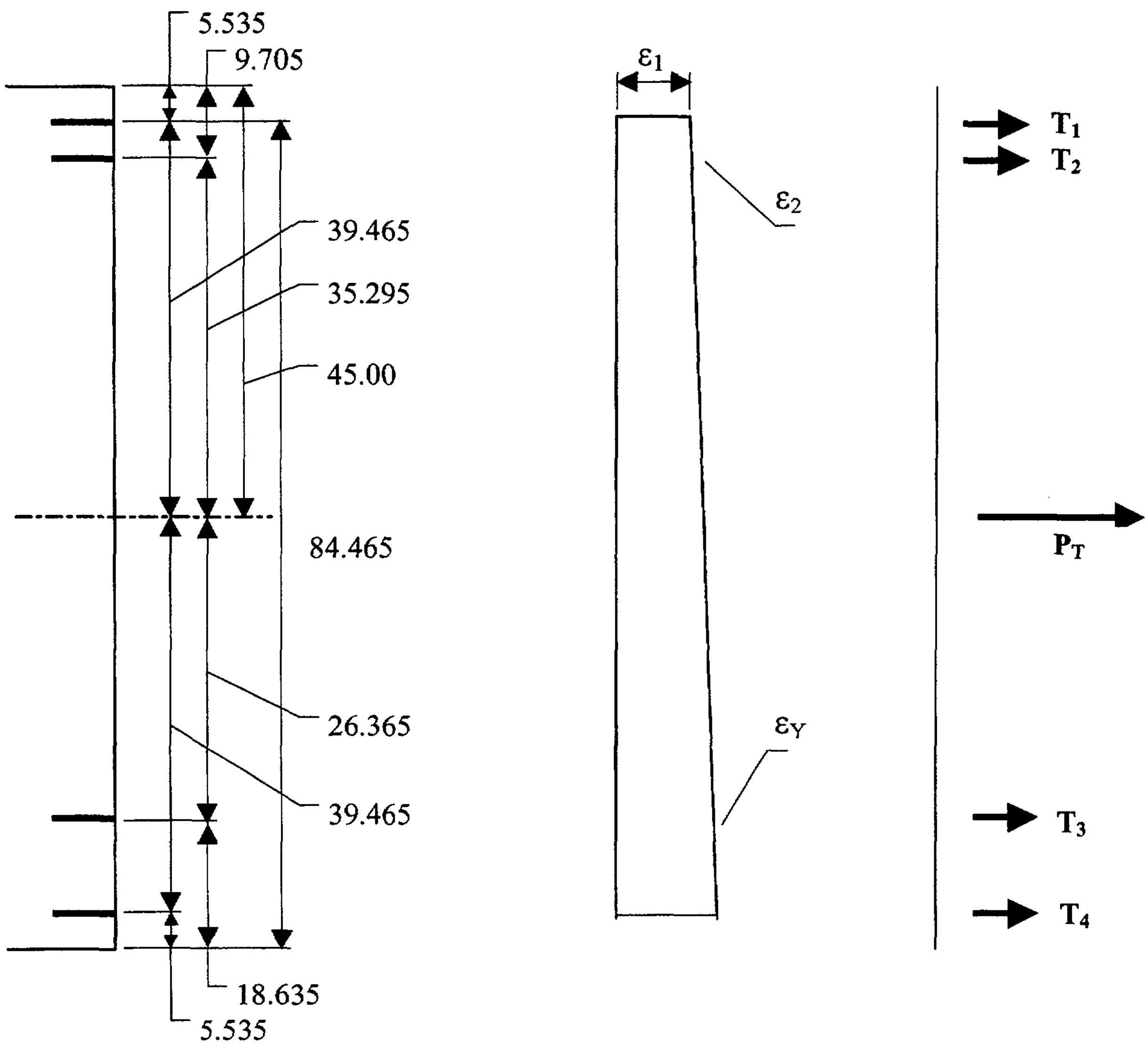


Figure 23 – East-West (Z strip) Section – Net Axial Tension
No Moment

Compute the axial force/moment interaction diagram.

East-West Net Axial Compression - No Moment – Figure 16

Since the reinforcement is not symmetrical, in order to achieve a net axial moment at the mid-height of the section, the concrete stress must vary slightly over the height of the section so that the unbalanced moment produced by the steel at a stress of Fy is offset by the concrete. Thus, the unbalanced moment is ΔM = AsFy(Net moment arm difference) = 1.27 x 60 x (35.295-26.365) = 680.5 in kips. Therefore the Δ concrete stress ($\Delta\sigma$) is computed from the equilibrium equation $\Delta M = \Delta\sigma \times S$ where S is the section modulus. $S = \frac{9x90^2}{6} = 12150\,\text{inches}^3$. And, $\Delta\sigma = \frac{\Delta M}{S} = \frac{680.5}{12150} = 0.056\,\text{ksi}$. Thus, the concrete stress varies linearly from 4.25 ksi at the top to 4.25-(2)(0.056) = 4.138 at the bottom of the section. And,

$$P_o = \left(4.138 + \left(\frac{1}{2}\right)(2)0.056\right)(9)(90) + 60 \times 4 \times 1.27 = 3701.9\ \text{kips}$$

Check on the moment:

$$M = (4.25-4.138)(0.5)(9)(90)\ ((2/3)(90)-45))-76.2(37.38-28.45) = -0.1\ \text{OK}$$

East-West Balanced Condition – Tension on Bottom - Figure 17

Compute X (distance to NA):

$$X = \left(\frac{0.003}{0.003+0.002069}\right)84.465 = 49.989\ \text{inches}$$

$$\varepsilon_{T1} = \varepsilon_Y = 0.002069$$

$$\varepsilon_{T2} = \left(\frac{34.476+5.535-18.635}{34.476}\right)0.002069 = 0.001283$$

$$\varepsilon_{C2} = \left(\frac{49.989-9.705}{49.989}\right)0.003 = 0.002418 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:

Depth of stress block: $a = 0.80 \times 49.989 = 39.991\ \text{inches}$

$$C_c = 0.85f_c'ab = 4.25 \times 39.991 \times 9 = 1529.7\ \text{kips}$$

$$C_1 = A_s(f_Y - 0.85f_c') = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_2 = E\varepsilon A_s = 29E3 \text{ x } 0.001283 \text{ x } 1.27 = 47.3 \text{ kips}$$

$$T_1 = A_s f_Y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1529.7 + 70.8 + 70.8 - 76.2 - 47.3 = 1547.8 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)35.295 + (T_2)26.365$$

$$M_n = 1529.7\left(45 - \frac{39.991}{2}\right) + (70.8 + 76.2)39.465 + (70.8)35.295 + (47.3)26.365 = 47{,}796.7 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Tension on Bottom - Comp Controls – Figure 18

Set $\varepsilon_{T1} = \frac{\varepsilon_Y}{2} = \frac{0.002069}{2} = 0.001035$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.001035}\right)84.465 = 62.799 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 62.799 + 5.535 - 18.635}{84.465 - 62.799}\right)0.001035 = 0.000410$$

$$\varepsilon_{C2} = \left(\frac{62.799 - 9.705}{62.799}\right)0.003 = 0.002536 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 62.799 = 50.239$ inches

$$C_c = 0.85f'_c ab = 4.25 \text{ x } 50.239 \text{ x } 9 = 1921.6 \text{ kips}$$

$$C_1 = A_s(f_Y - 0.85f'_c) = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_1 = E\varepsilon A_s = 29E3 \text{ x } 0.001035 \text{ x } 1.27 = 38.1 \text{ kips}$$

$$T_2 = E\varepsilon A_s = 29E3 \text{ x } 0.000410 \text{ x } 1.27 = 15.1 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1921.6 + 70.8 + 70.8 - 38.1 - 15.1 = 2010.0 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)35.295 + (T_2)26.365$$

$$M_n = 1921.6\left(45 - \frac{50.239}{2}\right) + (70.8 + 38.1)\ 39.465 + (70.8)\ 35.295 + (15.1)26.365 = 45{,}397.1 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Tension on Bottom - Comp Controls – Figure 18

Set $\varepsilon_{T1} = 0.0$, therefore:

$$X = 84.465 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{5.535 - 18.635}{84.465}\right)0.003 = -0.000465 \quad \text{(-) indicates compression}$$

$$\varepsilon_{C2} = \left(\frac{84.465 - 9.705}{84.465}\right)0.003 = 0.002655 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 84.465 = 67.572$ inches

$$C_c = 0.85f_c'ab = 4.25 \text{ x } 67.572 \text{ x } 9 = 2584.6 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85f_c'\right) = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_1 = 0.0 \text{ kips}$$

$$T_2 = E\varepsilon A_s = 29E3 \text{ x } -0.000465 \text{ x } 1.27 = -17.1 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 2584.6 + 70.8 + 70.8 - (-17.1) = 2743.3 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)35.295 + (T_2)26.365$$

$$M_n = 2584.6\left(45 - \frac{67.572}{2}\right) + (70.8)\ 39.465 + (70.8)\ 35.295 - (17.1)26.365 = 33{,}825.9 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Tension on Bottom - Tension Controls – Figure 19

Set $\varepsilon_{T2} = \varepsilon_Y = 0.002069$, (Note the change from ε_{T1} to ε_{T2}), therefore:

$$X = \left(\frac{0.003}{0.003 + 0.002069}\right)(84.465 + 5.535 - 18.635) = 42.473 \text{ inches}$$

$$\varepsilon_{T1} = \left(\frac{84.465 - 42.473}{84.465 - 42.473 + 5.535 - 18.635}\right)0.002069 = 0.003007 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{42.473 - 9.705}{42.473}\right)0.003 = 0.002315 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 42.473 = 33.978$ inches

$$C_c = 0.85 f_c' ab = 4.25 \times 33.978 \times 9 = 1299.7 \text{ kips}$$

$$C_1 = A_s(f_Y - 0.85 f_c') = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1299.7 + 70.8 + 70.8 - 76.2 - 76.2 = 1288.9 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)35.295 + (T_2)26.365$$

$$M_n = 1299.7\left(45 - \frac{33.978}{2}\right) + (70.8 + 76.2)\ 39.465 + (70.8)\ 35.295 + (76.2)26.365 = 46{,}715.2 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Tension on Bottom - Tension Controls – Figure 19

Set $\varepsilon_{T1} = 2.0\varepsilon_Y = 2.0 \text{ x } 0.002069 = 0.004138$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.004138}\right)84.465 = 35.499 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 35.499 + 5.535 - 18.635}{84.465 - 35.499}\right)0.004138 = 0.003031 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{35.499 - 9.705}{35.499}\right)0.003 = 0.002180 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 35.499 = 28.399$ inches

$$C_c = 0.85f_c'ab = 4.25 \text{ x } 28.399 \text{ x } 9 = 1086.3 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85f_c'\right) = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1086.3 + 70.8 + 70.8 - 76.2 - 76.2 = 1075.5 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)35.295 + (T_2)26.365$$

$$M_n = 1086.3\left(45 - \frac{28.399}{2}\right) + (70.8 + 76.2)\ 39.465 + (70.8)\ 35.295 + (76.2)26.365 = 43{,}767.8 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Tension on Bottom Tension Controls – Figure 19

Set $\varepsilon_{T1} = 4.0\varepsilon_Y = 4.0 \times 0.002069 = 0.008276$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.008276}\right)84.465 = 22.472 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 22.472 + 5.535 - 18.635}{84.465 - 22.472}\right)0.008276 = 0.006527 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{22.472 - 9.705}{22.472}\right)0.003 = 0.001704 < 0.002069$$

$$\varepsilon_{C1} = \left(\frac{22.472 - 5.535}{22.472}\right)0.003 = 0.002261 > 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 22.472 = 17.978$ inches

$$C_c = 0.85f_c'ab = 4.25 \times 17.978 \times 9 = 687.7 \text{ kips}$$

$$C_1 = A_s(f_Y - 0.85f_c') = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = (E\varepsilon_{C2} - 4.25)1.27 = (29E3 \times 0.001704 - 4.25)1.27 = 57.4 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 687.7 + 70.8 + 57.4 - 76.2 - 76.2 = 663.5 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)35.295 + (T_2)26.365$$

$$M_n = 687.7\left(45 - \frac{17.978}{2}\right) + (70.8 + 76.2)39.465 + (57.4)35.295 + (76.2)26.365 = 34{,}601.1 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Tension on Bottom - Tension Controls – Figure 19

Set $\varepsilon_{T1} = 8.0\varepsilon_Y = 8.0 \text{ x } 0.002069 = 0.016552$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.016552}\right)84.465 = 12.960 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 12.960 + 5.535 - 18.635}{84.465 - 12.960}\right)0.016552 = 0.013520 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{12.960 - 9.705}{12.960}\right)0.003 = 0.000753 < 0.002069$$

$$\varepsilon_{C1} = \left(\frac{12.960 - 5.535}{12.960}\right)0.003 = 0.001719 < 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 12.960 = 10.368 \text{ inches}$

$$C_c = 0.85f_c'ab = 4.25 \text{ x } 10.368 \text{ x } 9 = 396.6 \text{ kips}$$

$$C_1 = A_s(E\varepsilon_{C1} - 0.85f_c') = 1.27 \text{ x } (29E3 \text{ x } 0.001719 - 4.25) = 57.9 \text{ kips}$$

$$C_2 = A_s(E\varepsilon_{C2} - 0.85f_c') = 1.27 \text{ x } (29E3 \text{ x } 0.000753 - 4.25) = 22.3 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 396.6 + 57.9 + 22.3 - 76.2 - 76.2 = 324.4 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)35.295 + (T_2)26.365$$

$$M_n = 396.6\left(45 - \frac{10.368}{2}\right) + (57.9 + 76.2)\ 39.465 + (22.3)\ 35.295 + (76.2)26.365 = 23{,}879.4 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Tension on Bottom - Tension Controls – Figure 19

Set $\varepsilon_{T1} = 16\varepsilon_Y = 16 \text{ x } 0.002069 = 0.033104$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.033104}\right) 84.465 = 7.018 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 7.018 + 5.535 - 18.635}{84.465 - 7.018}\right) 0.033104 = 0.027505 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{7.018 - 9.705}{7.018}\right) 0.003 = -0.001149 \text{ (-) indicates bar in tension}$$

$$\varepsilon_{C1} = \left(\frac{7.018 - 5.535}{7.018}\right) 0.003 = 0.000634 < 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 7.018 = 5.614$ inches

$$C_c = 0.85f_c^{'}ab = 4.25 \text{ x } 5.614 \text{ x } 9 = 214.7 \text{ kips}$$

$$C_1 = A_s\left(E\varepsilon_{C1} - 0.85f_c^{'}\right) = 1.27 \text{ x } (29E3 \text{ x } 0.000634 - 4.25) = 18.0 \text{ kips}$$

$$C_2 = A_s E\varepsilon_{C2} = 1.27 \text{ x } (29E3 \text{ x } (-0.001149)) = -42.3 \text{ kips (bar in tension)}$$

$$T_1 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 214.7 + 18.0 - 42.2 - 76.2 - 76.2 = 38.1 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)35.295 + (T_2)26.365$$

$$M_n = 214.7\left(45 - \frac{5.614}{2}\right) + (18.0 + 76.2)\ 39.465 + (-42.2)35.295 + (76.2)26.365 = 13{,}296.0 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Tension on Bottom - Tension Controls – Figure 19

Set $\varepsilon_{T1} = 32\varepsilon_Y = 32 \times 0.002069 = 0.066208$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.066208}\right)84.465 = 3.661 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 3.661 + 5.535 - 18.635}{84.465 - 3.661}\right)0.066208 = 0.055474 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{3.661 - 9.705}{3.661}\right)0.003 = -0.004953 > \varepsilon_Y \text{ (-) indicates bar in tension}$$

$$\varepsilon_{C1} = \left(\frac{3.661 - 5.535}{3.661}\right)0.003 = -0.001536 < 0.002069 \text{ (-) indicates bar in tension}$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 3.661 = 2.929$ inches

$$C_c = 0.85f_c^{'}ab = 4.25 \times 2.929 \times 9 = 112.0 \text{ kips}$$

$$C_1 = A_s E\varepsilon_{C1} = 1.27 \times (29E3 \times -0.001536) = -56.6 \text{ kips (bar in tension)}$$

$$C_2 = A_s f_Y = 1.27 \times (-60) = -76.2 \text{ kips (bar in tension)}$$

$$T_1 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 112.0 - 56.6 - 76.2 - 76.2 - 76.2 = -173.2 \text{ kips (-) indicates tension}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)35.295 + (T_2)26.365$$

$$M_n = 112.0\left(45 - \frac{2.929}{2}\right) + (-56.6 + 76.2)\ 39.465 + (-76.2)\ 35.295 + (76.2)26.365 = 4969.0 \text{ in - kips}$$

East-West Net Axial Tension - No Moment – Figure 23

As with the net compression case, the strain across the section must vary slightly in order to achieve zero moment at mid-height.

Set $\varepsilon_3 = \varepsilon_y = 0.002069$ and therefore $T_3 = T_4 = 76.2$ kips

Moment equilibrium requires:

$$M = 0 = (T_1)\,39.465 + (T_2)\,35.295 - (T_3)\,26.365 - (T_4)\,39.465$$

Use the strain:

$$\varepsilon_2 = \varepsilon_1 + \left(\frac{9.705 - 5.535}{39.465 + 26.365}\right)(\varepsilon_y - \varepsilon_1)$$

$$\varepsilon_2 = \varepsilon_1 + 0.000131 - 0.063345\varepsilon_1$$

$$\varepsilon_2 = 0.936655\varepsilon_1 + 0.000131$$

Now, use the stress-strain relation and the bar areas:

$$T_1 = A_s E \varepsilon_1 = 1.27 \text{ x } 29\text{E}3 \text{ x } \varepsilon_1 = 36{,}830\varepsilon_1$$

and

$$T_2 = A_s E \varepsilon_2 = 1.27 \text{ x } 29\text{E}3 \text{ x } (0.936655\varepsilon_1 + 0.000131) = 34{,}497.0\varepsilon_1 + 4.825$$

Substitute in to equation for M

$$0 = (36{,}830\varepsilon_1)39.465 + (34{,}497\varepsilon_1 + 4.825)35.295 - (76.2)26.365 - (76.2)39.465$$

$$0 = 2{,}671{,}068\varepsilon_1 - 4{,}845.9$$

$$\varepsilon_1 = \frac{4845.9}{2{,}671{,}068} = 0.001814$$

$$\varepsilon_2 = (0.93665)0.001814 + 0.000131 = 0.0018303$$

And, $T_1 = 1.27 \text{ x } 29\text{E}3 \text{ x } 0.001814 = 66.8$ kip

And, $T_2 = 1.27 \times 29E3 \times 0.001830 = 67.4$ kip

Thus, $T = P_t = 66.8+67.4+76.2+76.2=286.6$ kip

Check on the moment:

$0 = (66.8)39.465 + (67.4)35.295 - (76.2)26.365 - (76.2)39.465 = -1.1$ OK

East-West Calculation of φ per ACI – Tension on Bottom

$\varphi = 0.90$ for tension plus flexure

$\varphi = 0.70$ for compression plus flexure

$$0.1 f_c^{'} A_g = 0.1 \text{ x } 5.00 \text{ x } 9 \text{ x } 90 = 405 \text{ kip}$$

$$C_n = 324.4 \text{ kips}, \quad \varphi = \frac{0.90}{1+\frac{2C_n}{f_c^{'}A_g}} = \frac{0.90}{1+\frac{2 \text{ x } 324.4}{5 \text{ x } 9 \text{ x } 90}} = 0.78$$

$$C_n = 38.1 \text{ kips}, \quad \varphi = \frac{0.90}{1+\frac{2C_n}{f_c^{'}A_g}} = \frac{0.90}{1+\frac{2 \text{ x } 38.1}{5 \text{ x } 9 \text{ x } 90}} = 0.88$$

East-West Balanced Condition – Compression on Bottom - Figure 20

Compute X (distance to NA):

$$X = \left(\frac{0.003}{0.003 + 0.002069}\right)84.465 = 49.989 \text{ inches}$$

$$\varepsilon_{T1} = \varepsilon_Y = 0.002069$$

$$\varepsilon_{T2} = \left(\frac{34.476 + 5.535 - 9.705}{34.476}\right)0.002069 = 0.001819$$

$$\varepsilon_{C2} = \left(\frac{49.989 - 18.635}{49.989}\right)0.003 = 0.001882 < 0.002069$$

$$\varepsilon_{C1} = \left(\frac{49.989 - 5.535}{49.989}\right)0.003 = 0.002668 > 0.002069$$

Compute individual internal forces:

Depth of stress block: $a = 0.80 \text{ x } 49.989 = 39.991 \text{ inches}$

$$C_c = 0.85f_c^{'}ab = 4.25 \text{ x } 39.991 \text{ x } 9 = 1529.7 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85f_c^{'}\right) = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = A_s\left(E\varepsilon - 0.85f_c^{'}\right) = 1.27 \text{ x } ((29E3)(0.001882) - 4.25) = 63.9 \text{ kips}$$

$$T_2 = E\varepsilon A_s = 29E3 \text{ x } 0.001819 \text{ x } 1.27 = 67.0 \text{ kips}$$

$$T_1 = A_s f_Y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1529.7 + 70.8 + 63.9 - 76.2 - 67.0 = 1521.2 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)26.365 + (T_2)35.295$$

$$M_n = 1529.7\left(45 - \frac{39.991}{2}\right) + (70.8 + 76.2)39.465 + (63.9)26.365 + (67.0)35.295 = 48{,}100.2 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Compression on Bottom - Comp Controls – Figure 21

Set $\varepsilon_{T1} = \frac{\varepsilon_Y}{2} = \frac{0.002069}{2} = 0.001035$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.001035}\right) 84.465 = 62.799 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 62.799 + 5.535 - 9.705}{84.465 - 62.799}\right) 0.001035 = 0.000836$$

$$\varepsilon_{C2} = \left(\frac{62.799 - 18.635}{62.799}\right) 0.003 = 0.002110 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 62.799 = 50.239$ inches

$$C_c = 0.85 f'_c ab = 4.25 \times 50.239 \times 9 = 1921.6 \text{ kips}$$

$$C_1 = A_s \left(f_Y - 0.85 f'_c\right) = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_1 = E\varepsilon A_s = 29E3 \times 0.001035 \times 1.27 = 38.1 \text{ kips}$$

$$T_2 = E\varepsilon A_s = 29E3 \times 0.000836 \times 1.27 = 30.8 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1921.6 + 70.8 + 70.8 - 38.1 - 30.8 = 1994.3 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)26.365 + (T_2)35.295$$

$$M_n = 1921.6\left(45 - \frac{50.239}{2}\right) + (70.8 + 38.1)\ 39.465 + (70.8)26.365 + (30.8)\ 35.295 = 45{,}453.8 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Compresison on Bottom - Comp Controls – Figure 21

Set $\varepsilon_{T1} = 0.0$, therefore:

$$X = 84.465 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{5.535 - 9.705}{84.465}\right)0.003 = -0.000148 \quad \text{(-) indicates compression}$$

$$\varepsilon_{C2} = \left(\frac{84.465 - 18.635}{84.465}\right)0.003 = 0.002338 > 0.002069 \text{ and } \varepsilon_{C1} > 0.002069 \text{ also.}$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 84.465 = 67.572$ inches

$$C_c = 0.85 f_c' ab = 4.25 \text{ x } 67.572 \text{ x } 9 = 2584.6 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85 f_c'\right) = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = 70.8 \text{ kips}$$

$$T_1 = 0.0 \text{ kips}$$

$$T_2 = E\varepsilon A_s = 29\text{E}3 \text{ x } -0.000148 \text{ x } 1.27 = -5.5 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 2584.6 + 70.8 + 70.8 - (-5.5) = 2731.7 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)26.365 + (T_2)35.295$$

$$M_n = 2584.6\left(45 - \frac{67.572}{2}\right) + (70.8)\ 39.465 + (70.8)26.365 + (-5.5)\ 35.295 = 33{,}450.3 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Compression on Bottom - Tension Controls - Figure 22

Set $\varepsilon_{T1} = 1.5\varepsilon_Y = 1.5 \times 0.002069 = 0.003104$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.003104}\right) 84.465 = 41.513 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 41.513 + 5.535 - 9.705}{84.465 - 41.513}\right) 0.003104 = 0.002803 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{41.513 - 18.635}{41.513}\right) 0.003 = 0.001653 < 0.002069$$

$$\varepsilon_{C1} = \left(\frac{41.513 - 5.535}{41.513}\right) 0.003 = 0.002600 > 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 41.513 = 33.210$ inches

$$C_c = 0.85 f'_c ab = 4.25 \times 33.210 \times 9 = 1270.3 \text{ kips}$$

$$C_1 = A_s(f_Y - 0.85 f'_c) = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = A_s(E\varepsilon - 0.85 f'_c) = 1.27 \times ((29E3)(0.001653) - 4.25) = 55.5 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1270.3 + 70.8 + 55.5 - 76.2 - 76.2 = 1244.2 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)26.365 + (T_2)35.295$$

$$M_n = 1270.3\left(45 - \frac{33.210}{2}\right) + (70.8 + 76.2)\ 39.465 + (55.5)26.365 + (76.2)\ 35.295 = 46{,}024.3 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Compression on Bottom - Tension Controls - Figure 22

Set $\varepsilon_{T1} = 2.0\varepsilon_Y = 2.0 \times 0.002069 = 0.004138$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.004138}\right)84.465 = 35.499 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 35.499 + 5.535 - 9.705}{84.465 - 35.499}\right)0.004138 = 0.003786 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{35.499 - 18.635}{35.499}\right)0.003 = 0.001425 < 0.002069$$

$$\varepsilon_{C2} = \left(\frac{35.499 - 5.535}{35.499}\right)0.003 = 0.002532 > 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 35.499 = 28.399$ inches

$$C_c = 0.85f_c^{'}ab = 4.25 \times 28.399 \times 9 = 1086.3 \text{ kips}$$

$$C_1 = A_s\left(f_Y - 0.85f_c^{'}\right) = 1.27 \times (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = A_s\left(E\varepsilon - 0.85f_c^{'}\right) = 1.27 \times ((29E3)(0.001425) - 4.25) = 47.1 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 1086.3 + 70.8 + 47.1 - 76.2 - 76.2 = 1051.8 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)26.365 + (T_2)35.295$$

$$M_n = 1086.3\left(45 - \frac{28.399}{2}\right) + (70.8 + 76.2)\ 39.465 + (47.1)26.365 + (76.2)\ 35.295 = 43{,}191.2 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Compression on Bottom - Tension Controls - Figure 22

Set $\varepsilon_{T1} = 4.0\varepsilon_Y = 4.0 \text{ x } 0.002069 = 0.008276$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.008276}\right) 84.465 = 22.472 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 22.472 + 5.535 - 9.705}{84.465 - 22.472}\right) 0.008276 = 0.007719 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{22.472 - 18.635}{22.472}\right) 0.003 = 0.000512 < 0.002069$$

$$\varepsilon_{C1} = \left(\frac{22.472 - 5.535}{22.472}\right) 0.003 = 0.002261 > 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 22.472 = 17.978 \text{ inches}$

$$C_c = 0.85 f_c^{'} ab = 4.25 \text{ x } 17.978 \text{ x } 9 = 687.7 \text{ kips}$$

$$C_1 = A_s \left(f_Y - 0.85 f_c^{'}\right) = 1.27 \text{ x } (60 - 4.25) = 70.8 \text{ kips}$$

$$C_2 = \left(E\varepsilon_{C2} - 4.25\right)1.27 = \left((29E3)(0.000512) - 4.25\right)1.27 = 13.5 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 687.7 + 70.8 + 13.5 - 76.2 - 76.2 = 619.6 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)26.365 + (T_2)35.295$$

$$M_n = 687.7\left(45 - \frac{17.978}{2}\right) + (70.8 + 76.2)\ 39.465 + (13.5)26.365 + (76.2)\ 35.295 = 33{,}611.5 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Compression on Bottom - Tension Controls - Figure 22

Set $\varepsilon_{T1} = 8.0\varepsilon_Y = 8.0 \times 0.002069 = 0.016552$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.016552}\right) 84.465 = 12.960 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 12.960 + 5.535 - 9.705}{84.465 - 12.960}\right) 0.016552 = 0.015587 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{12.960 - 18.635}{12.960}\right) 0.003 = -0.001314 > -0.002069 \text{ (-) indicates bar in tension}$$

$$\varepsilon_{C1} = \left(\frac{12.960 - 5.535}{12.960}\right) 0.003 = 0.001719 < 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \times 12.960 = 10.368$ inches

$$C_c = 0.85 f'_c ab = 4.25 \times 10.368 \times 9 = 396.6 \text{ kips}$$

$$C_1 = A_s\left(E\varepsilon_{C1} - 0.85 f'_c\right) = 1.27 \times (29E3 \times 0.001719 - 4.25) = 57.9 \text{ kips}$$

$$C_2 = A_s\left(E\varepsilon_{C2}\right) = 1.27 \times ((29E3)(-0.001314)) = -48.4 \text{ kips}$$

$$T_1 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \times 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 396.6 + 57.9 - 48.4 - 76.2 - 76.2 = 253.7 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)26.365 + (T_2)35.295$$

$$M_n = 396.6\left(45 - \frac{10.368}{2}\right) + (57.9 + 76.2)\ 39.465 - (48.4)26.365 + (76.2)\ 35.295 = 22{,}496.7 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Compression on Bottom - Tension Controls - Figure 22

Set $\varepsilon_{T1} = 16\varepsilon_Y = 16 \text{ x } 0.002069 = 0.033104$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.033104}\right)84.465 = 7.018 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 7.018 + 5.535 - 9.705}{84.465 - 7.018}\right)0.033104 = 0.031322 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{7.018 - 18.635}{7.018}\right)0.003 = -0.004966 < -0.002069 \text{ (-) indicates bar in tension}$$

$$\varepsilon_{C1} = \left(\frac{7.018 - 5.535}{7.018}\right)0.003 = 0.000634 < 0.002069$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 7.018 = 5.614$ inches

$$C_c = 0.85f_c^{'}ab = 4.25 \text{ x } 5.614 \text{ x } 9 = 214.7 \text{ kips}$$

$$C_1 = A_s\left(E\varepsilon_{C1} - 0.85f_c^{'}\right) = 1.27 \text{ x } (29E3 \text{ x } 0.000634 - 4.25) = 18.0 \text{ kips}$$

$$C_2 = -A_s f_y = -1.27 \text{ x } 60 = -76.2 \text{ kips (bar in tension)}$$

$$T_1 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 214.7 + 18.0 - 76.2 - 76.2 - 76.2 = 4.1 \text{ kips}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)26.365 + (T_2)35.295$$

$$M_n = 214.7\left(45 - \frac{5.614}{2}\right) + (18.0 + 76.2)\ 39.465 - (76.2)26.365 + (76.2)\ 35.295 = 13{,}456.9 \text{ in - kips}$$

E-W Nominal Force/Moment Interaction – Compression on Bottom - Tension Controls - Figure 22

Set $\varepsilon_{T1} = 32\varepsilon_Y = 32 \text{ x } 0.002069 = 0.066208$, therefore:

$$X = \left(\frac{0.003}{0.003 + 0.066208}\right)84.465 = 3.661 \text{ inches}$$

$$\varepsilon_{T2} = \left(\frac{84.465 - 3.661 + 5.535 - 9.705}{84.465 - 3.661}\right)0.066208 = 0.062791 > \varepsilon_y$$

$$\varepsilon_{C2} = \left(\frac{3.661 - 18.635}{3.661}\right)0.003 = -0.012270 < -\varepsilon_Y \text{ (-) indicates bar in tension}$$

$$\varepsilon_{C1} = \left(\frac{3.661 - 5.535}{3.661}\right)0.003 = -0.001536 > -0.002069 \text{ (-) indicates bar in tension}$$

Compute individual internal forces:
Depth of stress block: $a = 0.80 \text{ x } 3.661 = 2.929$ inches

$$C_c = 0.85f'_c ab = 4.25 \text{ x } 2.929 \text{ x } 9 = 112.0 \text{ kips}$$

$$C_1 = A_s E\varepsilon_{C1} = 1.27 \text{ x } (29E3 \text{ x } -0.001536) = -56.6 \text{ kips (bar in tension)}$$

$$C_2 = A_s f_Y = 1.27 \text{ x } (-60) = -76.2 \text{ kips (bar in tension)}$$

$$T_1 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

$$T_2 = A_s f_y = 1.27 \text{ x } 60 = 76.2 \text{ kips}$$

Net nominal compression is:

$$C_n = C_c + C_1 + C_2 - T_1 - T_2 = 112.0 - 56.6 - 76.2 - 76.2 - 76.2 = -173.2 \text{ kips (-) indicates tension}$$

Net nominal moment is:

$$M_n = C_c\left(45 - \frac{a}{2}\right) + (C_1 + T_1)39.465 + (C_2)26.365 + (T_2)35.295$$

$$M_n = 112.0\left(45 - \frac{2.929}{2}\right) + (-56.6 + 76.2)\ 39.465 - (76.2)26.365 + (76.2)\ 35.295 = 6330.0 \text{ in - kips}$$

East-West Calculation of φ per ACI – Compression on Bottom

$\varphi = 0.90$ for tension plus flexure

$\varphi = 0.70$ for compression plus flexure

$$0.1f'_cA_g = 0.1 \text{ x } 5.00 \text{ x } 9 \text{ x } 90 = 405 \text{ kip}$$

$$C_n = 253.7 \text{ kips}, \quad \varphi = \frac{0.90}{1+\frac{2C_n}{f'_cA_g}} = \frac{0.90}{1+\frac{2 \text{ x } 253.7}{5 \text{ x } 9 \text{ x } 90}} = 0.80$$

$$C_n = 4.1 \text{ kips}, \quad \varphi = \frac{0.90}{1+\frac{2C_n}{f'_cA_g}} = \frac{0.90}{1+\frac{2 \text{ x } 4.1}{5 \text{ x } 9 \text{ x } 90}} = 0.90$$

East-West Maximum Design Axial Load Strength

Per Reference 11, 10.3.5.2, the maximum design axial load strength shall not be greater than $0.80\varphi P_o$. Therefore, φC_n is limited to 0.80 x 0.70 x 3701.9 = 2073.1 in Table 5 below.

Table 5 below presents the East-West (X Strip) section capacity data computed above. The signs of the moment are adjusted to conform to the sign convention established in Reference 5. Therefore, the moments that produce tension on the bottom are (-) and those that produce compression on the bottom are (+).

JOB. NO.	PGE-009	DATE	March 11, 2003
PROJECT	DCPP ISFSI		
SUBJECT	ISFSI Cask Storage Pad Steel Reinforcement		
CLIENT	PG&E-DCPP	ORIGINATOR	S. C. Tumminelli
REVIEWER	K. L. Whitmore	APPROVED	R. F. Evers
CALCULATION NO.	PGE-009-CALC-007	REVISION	0

ENERCON SERVICES, INC.

Table 5 – East-West Section Capacity Data

Moment Sign (±) and Condition	M_n in-kips	C_n kips	φ	φ M_n in-kips	φ C_n kips	e inches
Compression no moment	0	3701.9	0.7	0	2591.3*	0
(-) ACI Code Maximum				-18,283.4**	2073.1	-8.886
(-) Compression controls	-33,825.9	2743.3	0.7	-23,678.1	1920.2	-12.331
(-) Compression controls	-45,397.1	2010.0	0.7	-31,778.0	1407.0	-22.586
(-) Balanced Condition	-47,796.7	1547.8	0.7	-33,457.7	1083.5	-30.879
(-) Tension controls	-46,715.2	1288.9	0.7	-32,700.6	902.2	-36.245
(-) Tension controls	-43,767.8	1075.5	0.7	-30,637.5	752.9	-40.693
(-) Tension controls	-34,601.1	663.5	0.7	-24,220.8	464.5	-52.144
(-) Tension controls	-23,879.4	324.4	0.78	-18,625.9	253.0	-73.620
(-) Tension controls	-13,296.0	38.1	0.88	-11,700.5	33.5	-349.269
(-) Tension controls	-4969.0	-173.2	0.9	-4472.1	-155.9	28.686
Tension no moment	0	-286.6	0.9	0	-257.9	0
(+) ACI Code Maximum				17,867.4***	2073.1	8.619
(+) Compression controls	33,450.3	2731.7	0.7	23,415.2	1912.2	12.245
(+) Compression controls	45,453.8	1994.3	0.7	31,817.7	1396.0	22.792
(+) Balanced Condition	48,100.2	1521.2	0.7	33,670.1	1064.8	31.621
(+) Tension controls	46,024.3	1244.2	0.7	32,217.0	870.9	36.993
(+) Tension controls	43,191.2	1051.8	0.7	30,233.8	736.3	41.067
(+) Tension controls	33,611.5	619.6	0.7	23,528.1	433.7	54.250
(+) Tension controls	22,496.7	253.7	0.8	17,997.4	203.0	88.657
(+) Tension controls	13,456.9	4.1	0.9	12,111.2	3.7	3273.3
(+) Tension controls	6330	-173.2	0.9	5697	-155.9	-36.543

* This value is used only for linear interpolation for the allowable moment associated with the Code maximum compressive load, see notes ** and *** below.

** Linearly interpolated from the values above and below using 2591.3 = 0.70 x 3701.9 for the $\varphi\ C_n$ with the zero moment.

*** Linearly interpolated as above.

North-South and East-West Section Evaluations

The section capacity data from Tables 4 and 5 is plotted in Figures 24 and 29 below. The applied seismic forces necessary to qualify the concrete and reinforcement are taken from Table 11 (1/2) for the N-S (Z) sections and Table 11 (2/2) for the E-W (X) sections, see Reference 5, and are shown in Tables 6 (1/2) and (2/2). This data is referred to as "Selected Forces" since they were selected to be the values from the entire data set to bound all the points in the data set. Thus they are the only points that require evaluation to qualify the design.

The forces from the seismic calculation are in pounds (lbs) and inch-pounds (in-lbs) for a 17-foot (204 inches) wide section (strip) of the pad, and are presented in the 2nd and 3rd columns in Tables 6 (1/2) and (2/2) below. This reinforcement calculation uses kips and in-kips and evaluates a 9 inch wide concrete section. Thus the forces from the seismic calculation are converted. Further, the seismic calculation identified a factor of 1.15 to be applied to all the forces to provide for the variability in Young's modulus and Poisson's ratio. Thus the values in the 2nd and 3rd columns are factored by:

$$\text{Factor} = \left(\frac{9}{204}\right)\left(\frac{1}{1000}\right)1.15 = 5.074\text{E} - 5$$

Therefore, the values in the 4th and 5th columns in Tables 6 (1/2) and (2/2) are the values in the 2nd and 3rd columns factored by 5.074E-5. These are plotted in Figures 25 and 30 below. Presenting this data graphically, the selected forces are plotted on the section capacity curves in Figures 26 and 31. This data graphically presented shows that the applied forces are well within the allowable ACI factored capacity envelope. The graphical presentation of the data is then presented using all the applied forces from the seismic analysis calculation, Reference 5. Thus, Figures 27 and 32 present all the applied force data points factored by 5.074E-5, and Figures 28 and 33 present these same data points plotted with the ACI factored capacity envelope as before. Again, the graphical presentation shows that all the calculated applied forces are well with the envelope.

Table 6 (1/2) – N-S (Z) Strips – Selected Forces for Design

Quadrant (1)	Mx (17 feet) in-lbs (2)	Fz (17 feet) lbs (3)	Mx (9 inches) in-kips (4)	Fz (9 inches) kips (5)	e in (6)
+Mx Ten. On Bot. +Fz Comp. on Sect.	0.0449E8 0.184E8 0.336E8 0.345E8	0.971E6 0.638E6 0.224E6 0.00812E6	227.8 933.5 1704.7 1750.4	49.3 32.4 11.4 0.4	4.621 28.812 149.535 4376.00
-Mx Comp. On Bot. +Fz Comp. on Sect.	-0.443E8	0.984E6	-2247.6	49.9	-45.042
-Mx Comp. On Bot. -Fz Ten. On Sect.	-0.00187E8	-0.224E6	-9.5	-11.4	0.833
+Mx Ten. on Bot. -Fz Ten. on Sect.	0.333E8 0.272E8	-0.097E6 -0.606E6	1689.5 1380.0	-4.9 -30.7	-344.796 -44.951
Shear Force	Fy	-731078	Fy	-37.1	NA

Table 6 (2/2) – E-W (X) Strips – Selected Forces for Design

Quadrant (1)	Mz (17 feet) in-lbs (2)	Fx (17 feet) in-lbs (3)	Mz (9 inches) in-kips (4)	Fx (9 inches) kips (5)	e in (6)
+Mz Comp. on Bot. +Fx Comp. on Sect.	0.4165E8	0.8925E6	2113.1	45.3	46.647
-Mz Ten. On Bot. +Fx Comp. on Sect.	-0.00231E8 -0.238E8 -0.476E8 -0.481E8	0.7565E6 0.5355E6 0.177E6 0.171E6	-11.7 -1207.5 -2415.0 -2440.4	38.4 27.2 9.0 8.7	-0.305 -44.393 -268.333 –280.506
-Mz Ten. On Bot. -Fx Ten. On Sect.	-0.370E8 -0.2516E8 -0.2482E8	-0.08415E6 -0.5185E6 -0.5372E6	-1877.2 -1276.5 -1259.3	-4.3 -26.3 -27.3	436.558 48.536 46.128
+Mz Comp. on Bot. -Fx Ten. On Sect.	0.00255E8	-0.2397E6	12.9	-12.2	-1.057
Shear Force	Fy	-653720	Fy	-33.2	NA

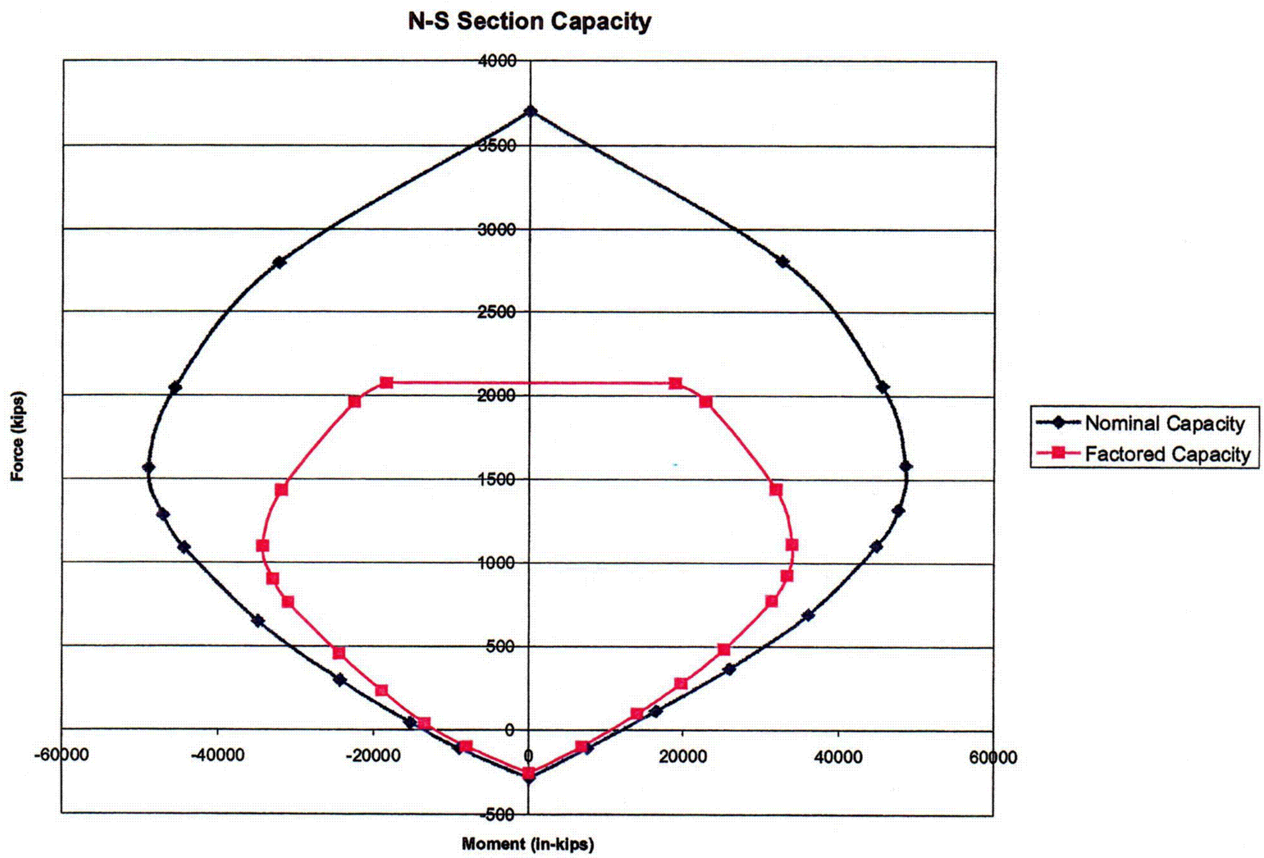


Figure 24 – North-South Section Capacities
From Table 4

C01

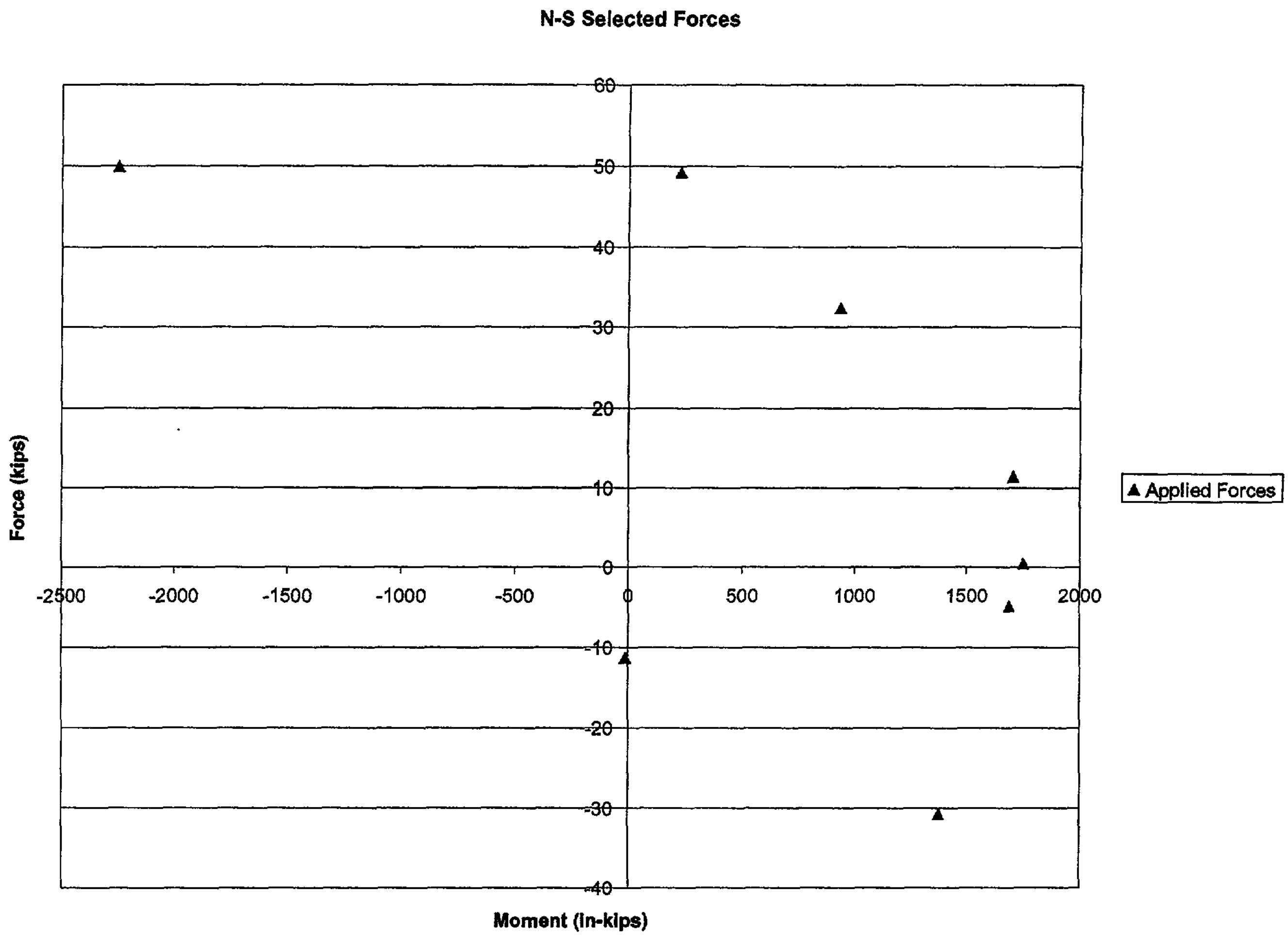


Figure 25 - North-South Selected Forces
From Table 6 (1/2)
Compare to Figure 33, Reference 5

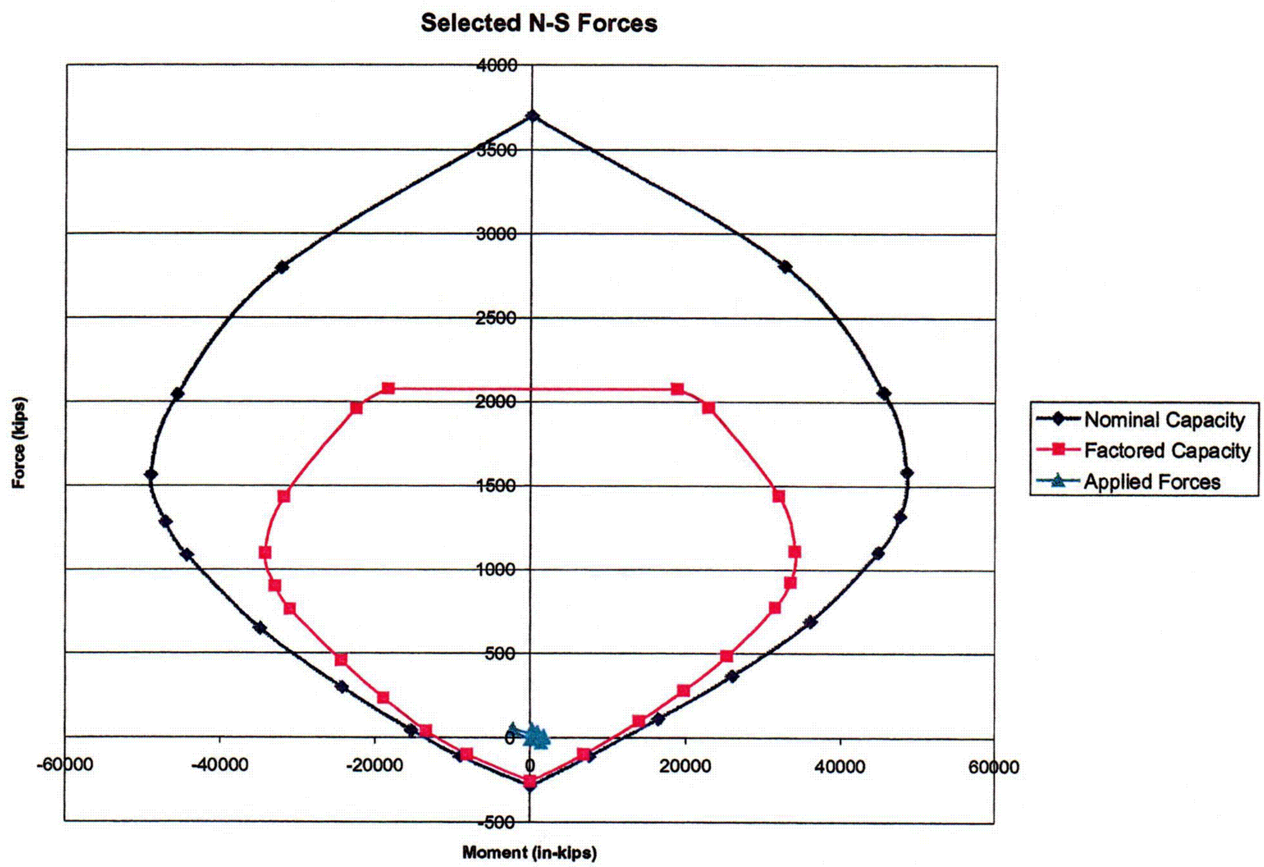


Figure 26 – North-South Section
Selected Applied Forces
From Table 6 (1/2)
Compare to Figure 33, Reference 5

C02

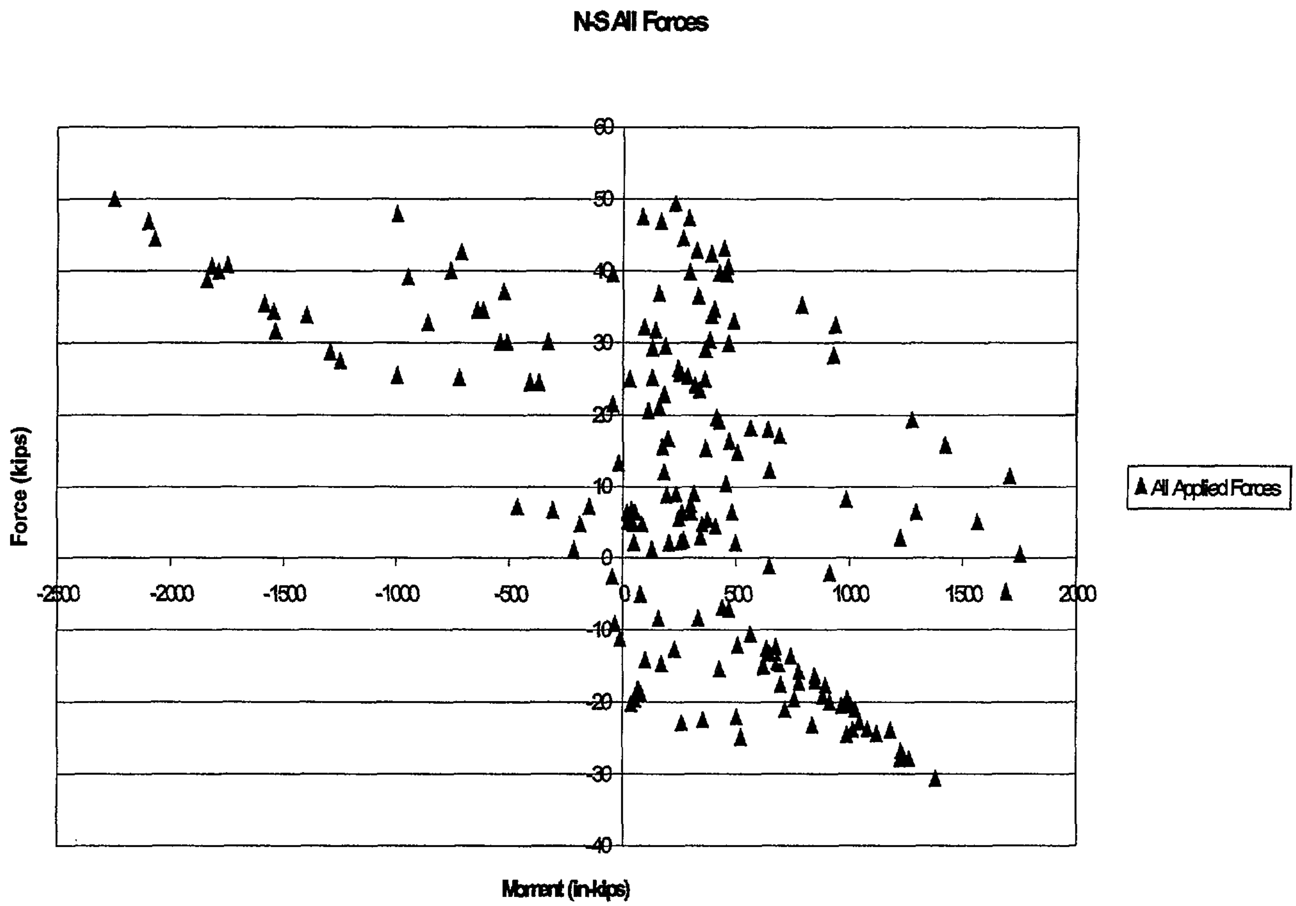


Figure 27 – North-South
All Applied Forces
Compare to Figure 33, Reference 5

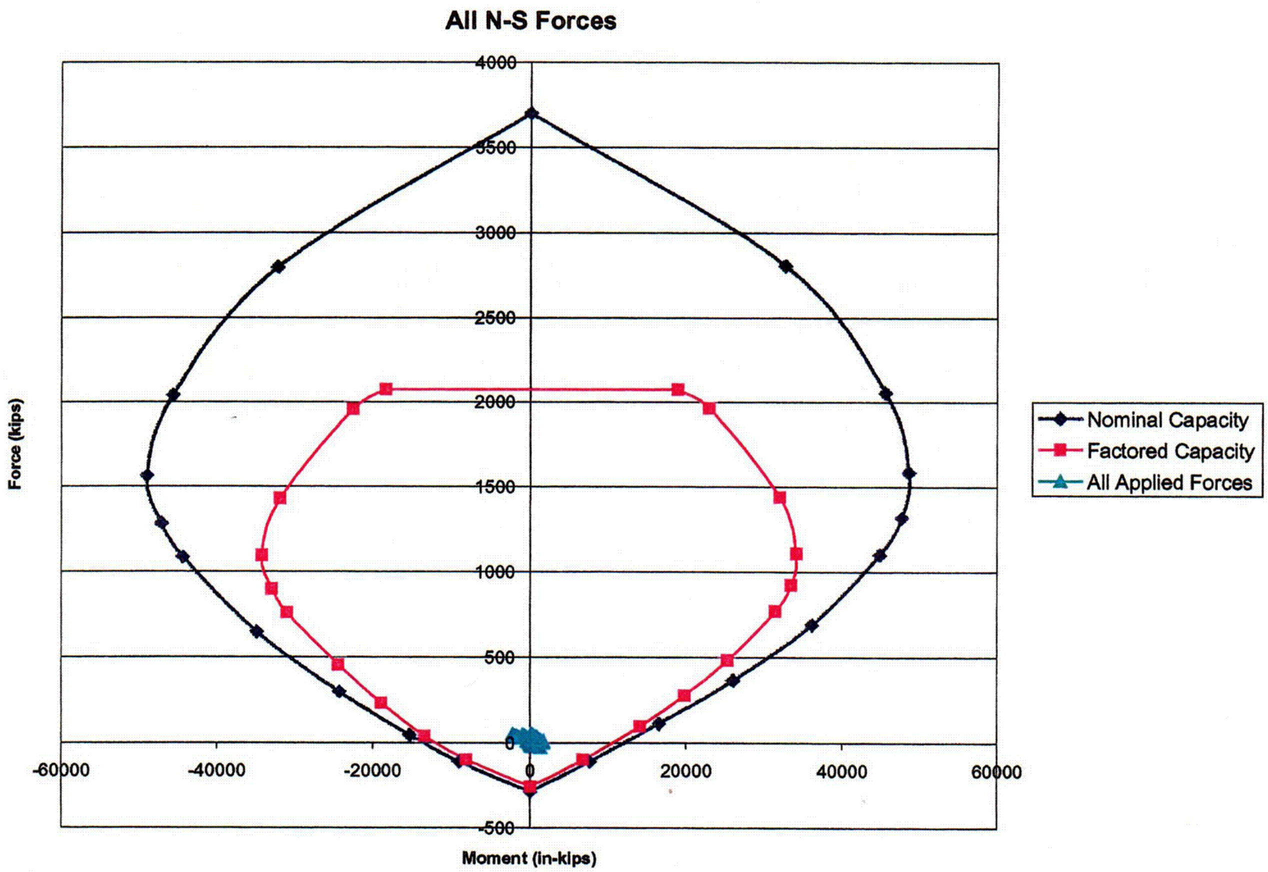


Figure 28 – North-South Section
All Applied Forces
Compare to Figure 33, Reference 5

C03

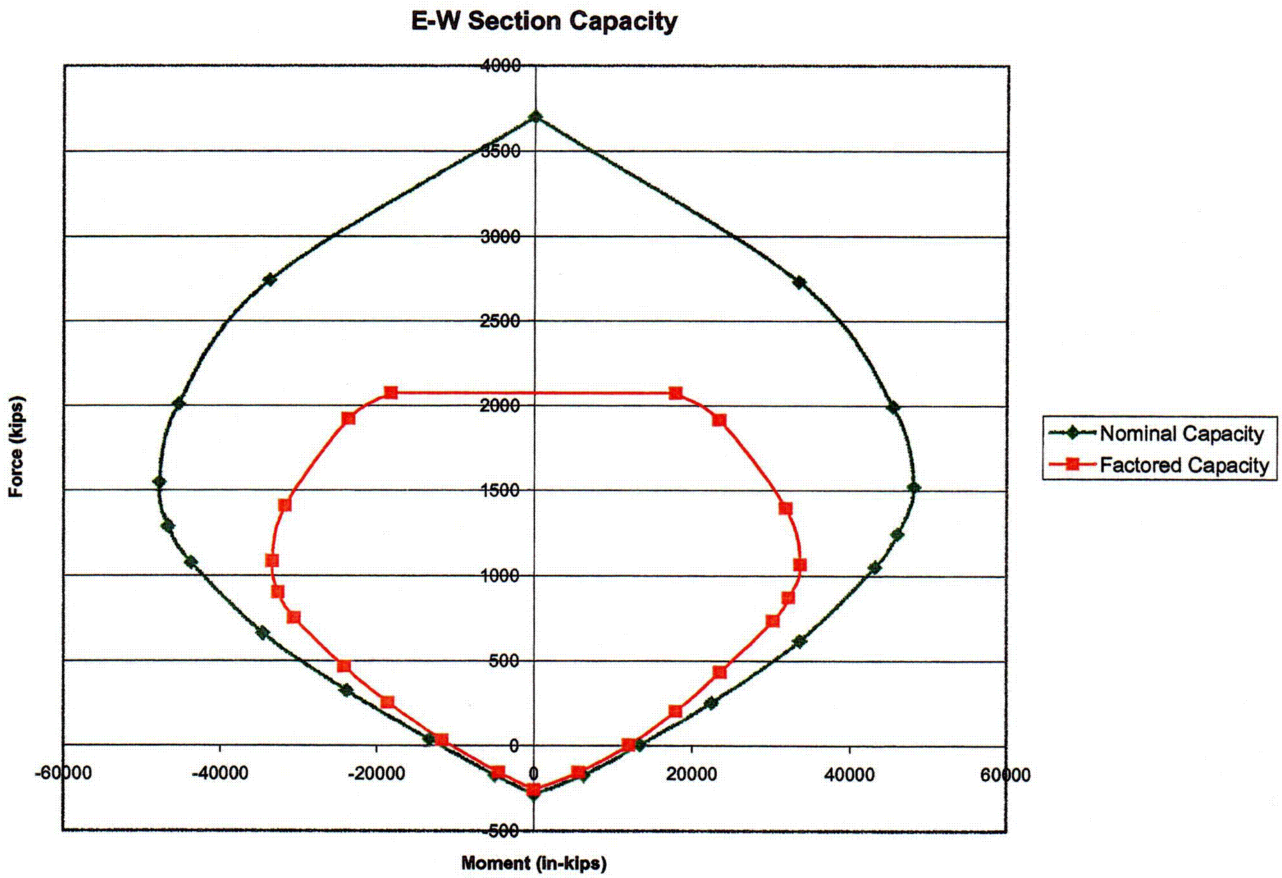


Figure 29 - East-West Section Capacities
From Table 5

C04

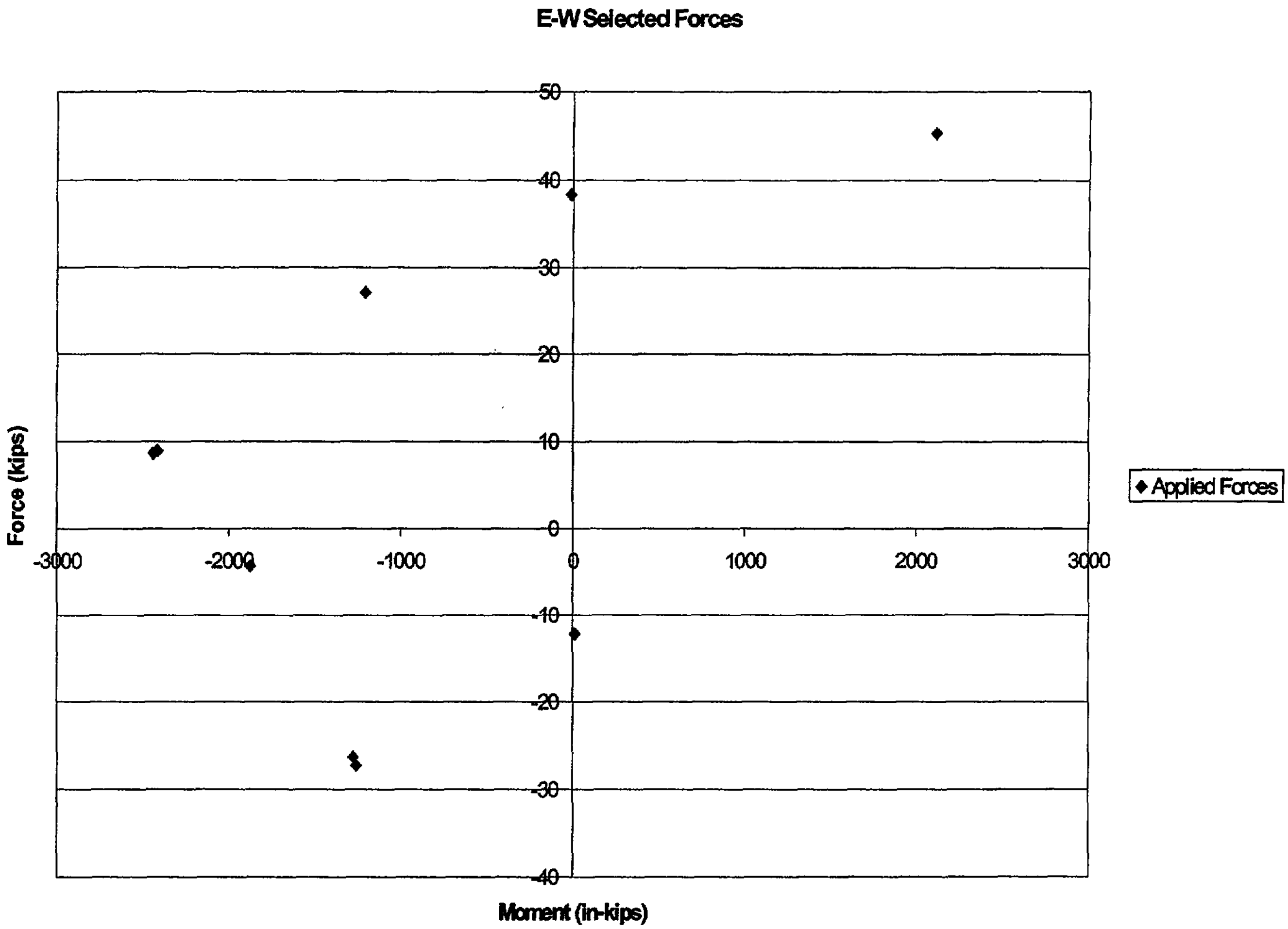


Figure 30 - East-West Selected Forces
From Table 6 (2/2)
Compare to Figure 34, Reference 5

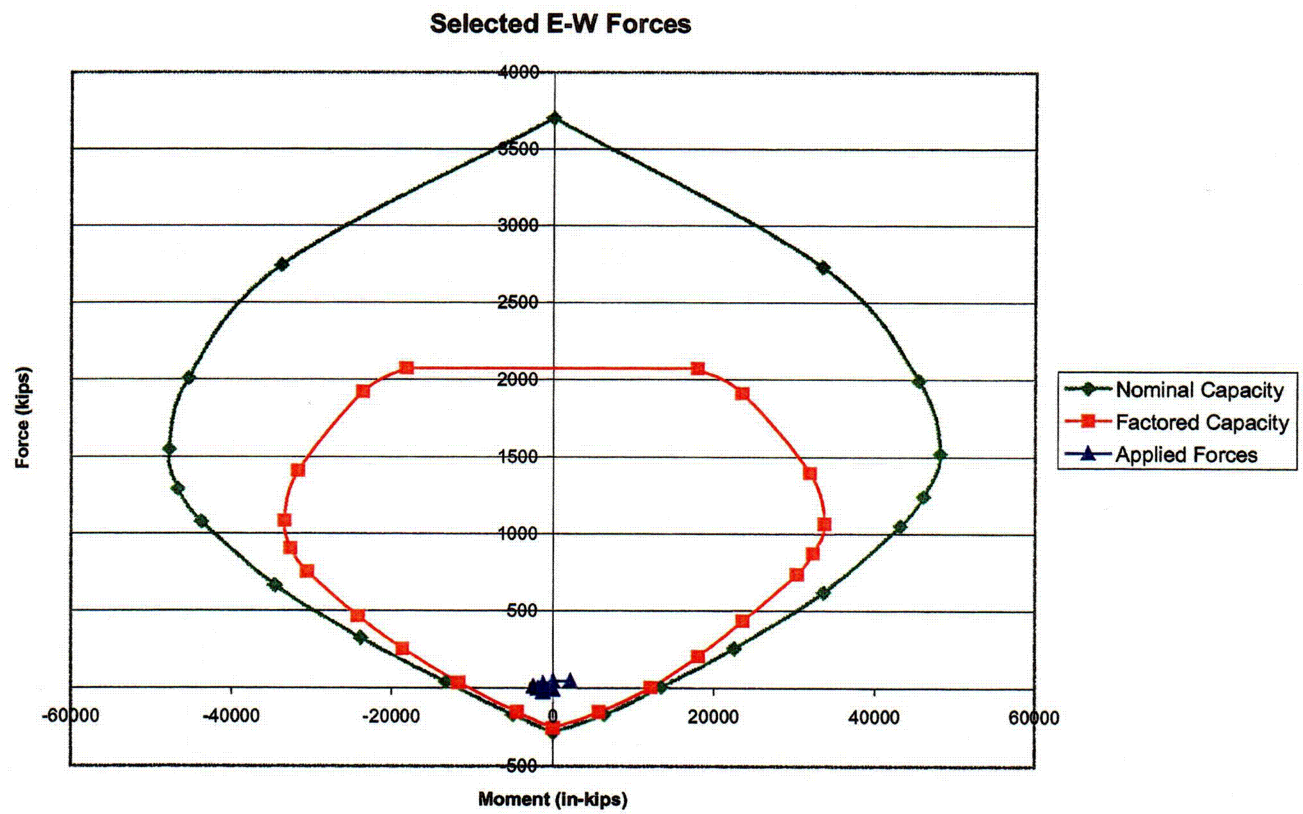


Figure 31 – East-West Section
Selected Applied Forces
From Table 6 (2/2)
Compare to Figure 34, Reference 5

C05

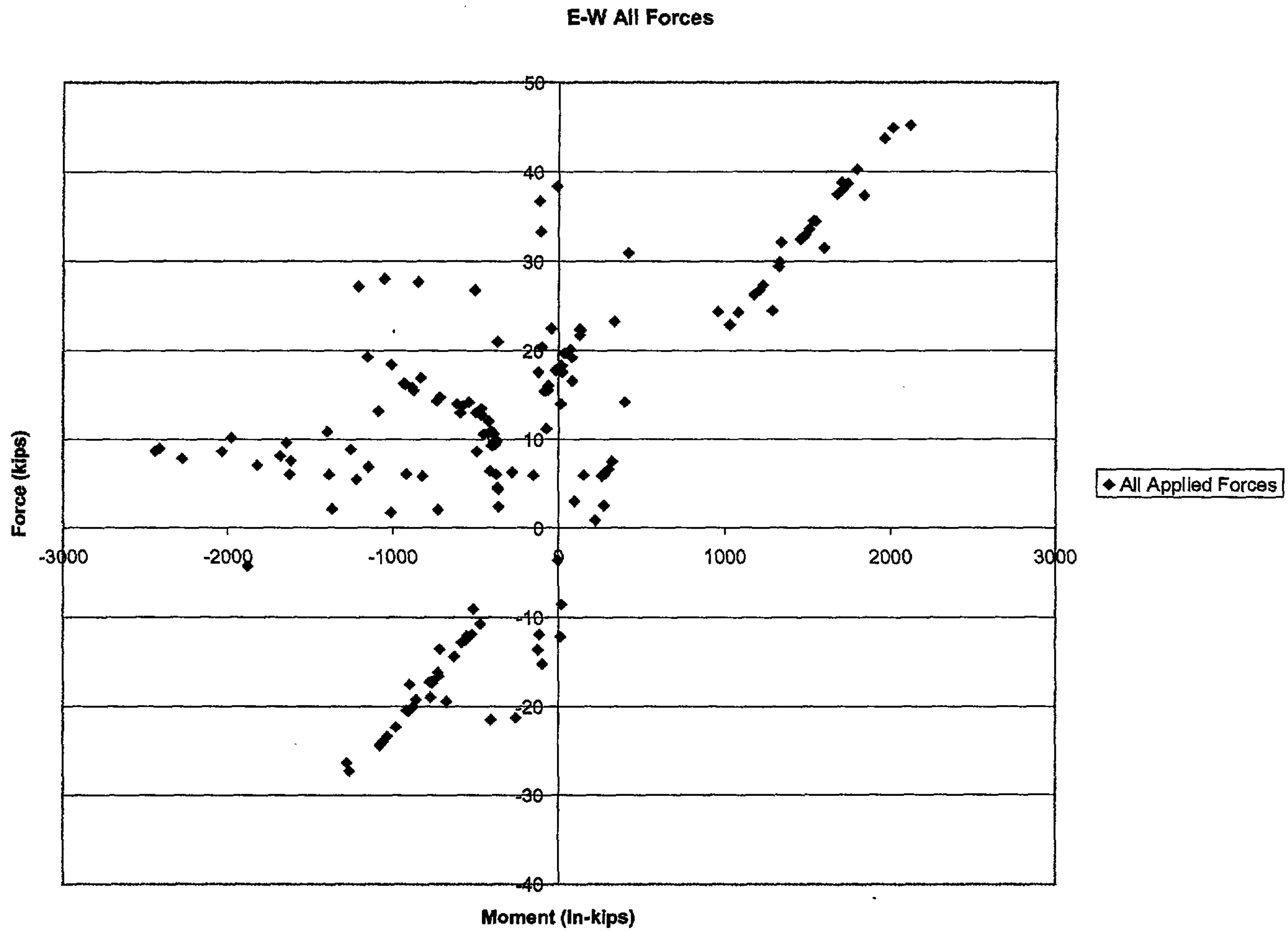


Figure 32 – East-West
All Applied Forces
Compare to Figure 34, Reference 5

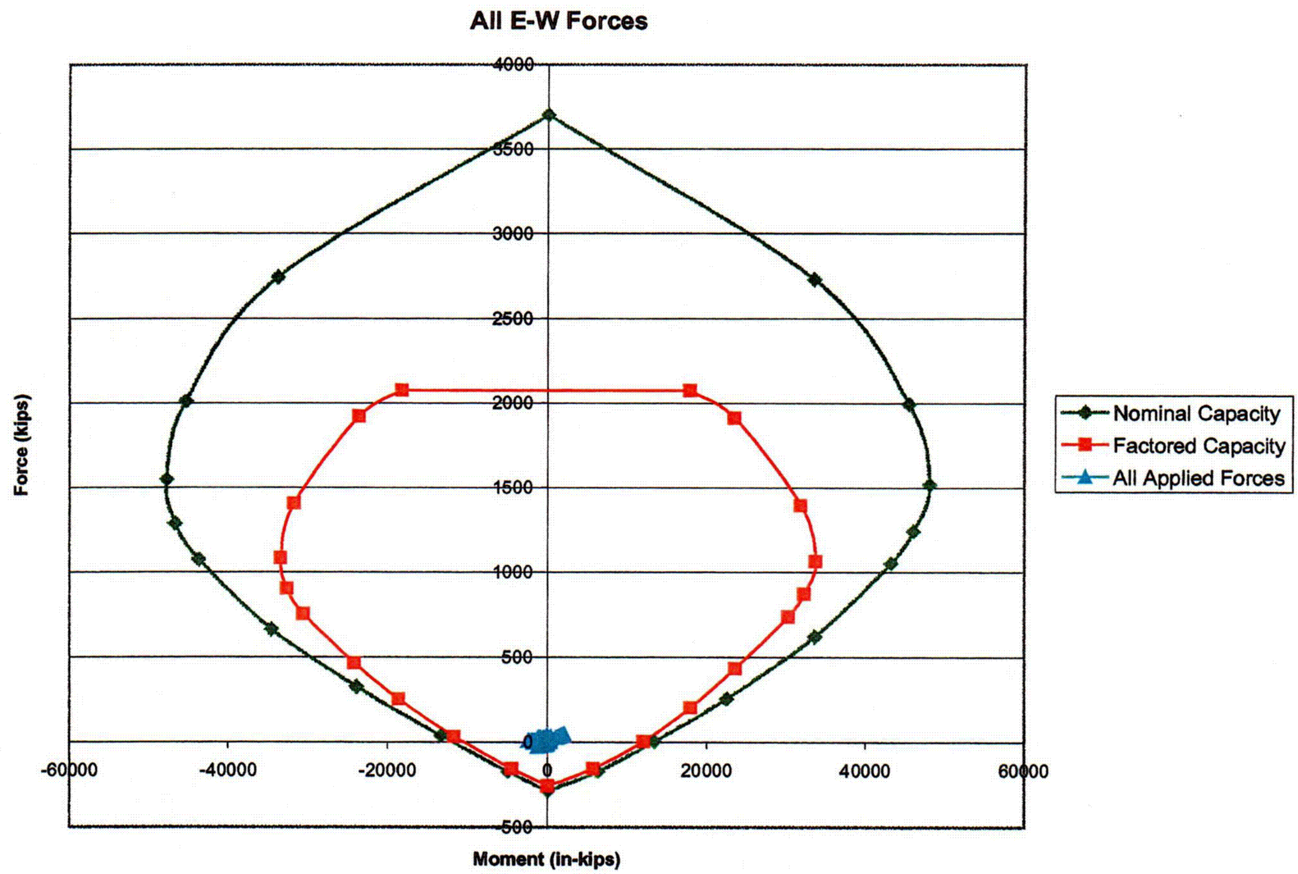


Figure 33 – East-West Section
All Applied Forces
Compare to Figure 34, Reference 5

C06

The applied seismic forces (Selected Data) are evaluated numerically relative to the ACI factored capacity data by extending the vector from the origin to the applied force data point (Ms,Ps) to the ACI factored capacity curve holding the eccentricity e constant. The process does not hit the capacity curve exactly since the intersection is for two straight lines. Since the capacity curve is always concave inward, the straight line approximation will always yield conservative results.

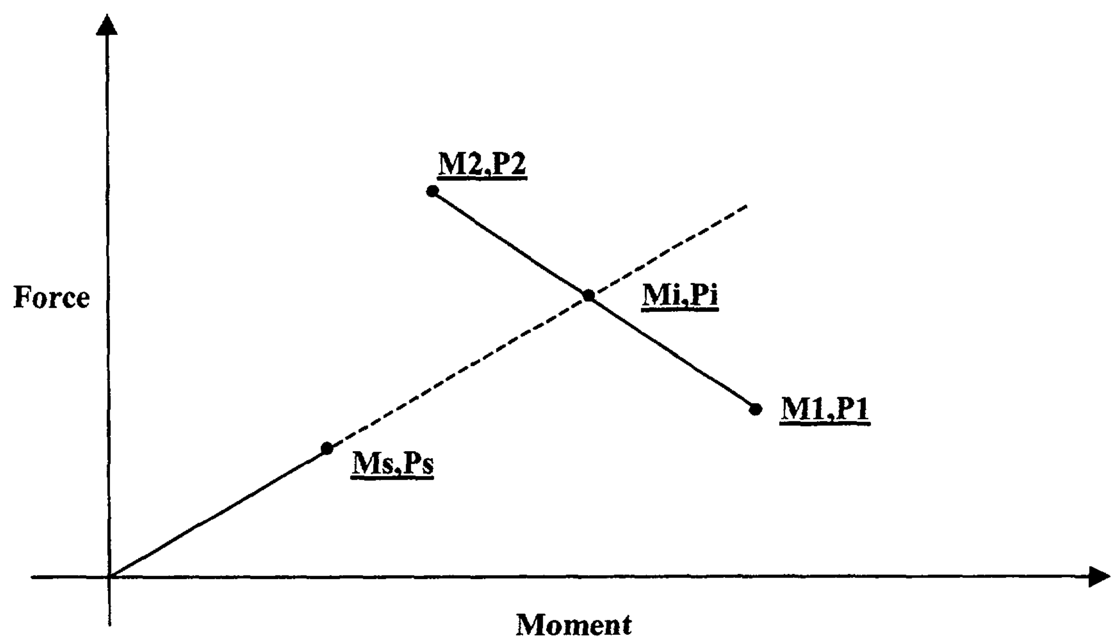


Figure 34 – Intersection of Applied Force Vector with the ACI Factored Capacity Curve.

Developing the equation for a numerical evaluation, the following definitions are used (see Figure 20):

The applied moment/force is Ms, Ps

The ACI factored moment/force values are M1, P1 and M2, P2

The intersection of the applied force line with the line on the ACI factored line (linearly interpolated between the two closest points is) Mi, Pi.

And, the capacity beyond the ACI Code allowable is computed as either Mi/Ms or Pi/Ps. Both will yield the same numerical value.

The algebra to solve for Mi and Pi is as follows:

In Figure 34, the equation for the line passing through Ms, Ps is:

$$P = \frac{Ps}{Ms} M$$

Also, in Figure 34, the equation for the line passing through M1, P1 and M2, P2 is:

$$P = P1 + \left(\frac{P2 - P1}{M2 - M1}\right)(M - M1)$$

Now, at the intersection of the two lines P = P, therefore,

$$\frac{Ps}{Ms} M = P1 + \left(\frac{P2 - P1}{M2 - M1}\right)(M - M1) = P1 + \left(\frac{P2 - P1}{M2 - M1}\right) M - \left(\frac{P2 - P1}{M2 - M1}\right) M1$$

Therefore,

$$\left(\frac{Ps}{Ms} - \frac{P2 - P1}{M2 - M1}\right) M = P1 - \left(\frac{P2 - P1}{M2 - M1}\right) M1$$

Thus,

$$M = Mi = \left[P1 - \left(\frac{P2 - P1}{M2 - M1}\right) M1\right] \div \left[\frac{Ps}{Ms} - \frac{P2 - P1}{M2 - M1}\right]$$

And,

$$Pi = \frac{Ps}{Ms} Mi$$

Further, the ratios of the allowable Code values divided by the applied values is:

$$R = \frac{Mi}{Ms} = \frac{Pi}{Ps}$$

The values of R are computed for all the forces in Tables 6 (1/2) and (2/2). The values of factored capacity used are those where the eccentricity of the seismic loads e_s is between the values of the capacity eccentricities e_c. As an example, for the N-S applied seismic forces (Table 6 (1/2), Ps of 32.4 kips and Ms of 933.5 in-kips, the e_s is 28.812 inches. Therefore, the values for the calculation of Mi and Pi are M1 = 31,878.0 in-kips and P1 = 1439.9 kips (e_c = 22.139 inches) and M2 = 33,975.6 in-kips and P2 = 1109.5 kips (e_c = 30.622 inches), see Table 4. Thus:

$$M = Mi = \left[P1 - \left(\frac{P2-P1}{M2-M1}\right)M1\right] \div \left[\frac{Ps}{Ms} - \frac{P2-P1}{M2-M1}\right]$$

$$\therefore Mi = \left[1439.9 - \left(\frac{1109.5-1439.9}{33{,}975.6-31{,}878.0}\right)31{,}878.0\right] \div \left[\frac{32.4}{933.5} - \frac{1109.5-1439.9}{33{,}975.6-31{,}878.0}\right] = 33{,}612.9$$

And,

$$R = \frac{33{,}612.9}{933.5} = 36.0$$

All of the values of Mi, Pi and the corresponding R are provided in Tables 7 (1/2) for the N-S (Z) sections and in Table 7(2/2) for the E-W (X) sections.

Table 7 (1/2) – N-S (Z) Sections
Numerical Evaluation of the Selected Forces for Design*

Ms/Ps e_s from Table 6 (1/2)	M1/P1 e_c from Table 4	M2/P2 e_c from Table 4	Mi/Pi	R
227.8 / 49.3 4.621	0 / 2073.8 0	18,848.8 / 2073.8 9.089	9582.4 / 2073.8	42.1
933.5 / 32.4 28.812	31,878.0 / 1439.9 22.139	33,975.6 / 1109.5 30.622	33,612.9 / 1166.6	36.0
1704.7 / 11.4 149.535	14,038.0 / 96.1 146.077	6858.6 / -99.4 –69.000	13,929.8 / 93.2	8.2
1750.5 / 0.4 4376.0	14,038.0 / 96.1 146.077	6858.6 / -99.4 -69.000	10,597.8 / 2.4	6.1
-2247.6 / 49.9 -45.042	-31,086.4 / 760.4 -40.882	-24,433.5 / 454.1 -53.806	-28,140.1 / 624.8	12.5
-9.5 / -11.4 0.833	-8083.4 / -99.4 81.322	0 / -258.8 0	-212.2 / -254.6	22.3
1689.5 / -4.9 -344.796	14,038.0 / 96.1 146.077	6858.6 / -99.4 -69.000	9497.0 / -27.5	5.6
1380.0 / -30.7 -44.951	6858.6 / -99.4 -69.000	0 / -258.8 0	5689.5 / -126.6	4.1

* Units are: in-kips for Ms, M1, M2 and Mi, kips for Ps, P1, P2 and Pi, and inches for e_s and e_c.

Table 7 (2/2) – E-W (X) Sections
Numerical Evaluation of the Selected Forces for Design*

Ms/Ps e_s from Table 6 (2/2)	M1/P1 e_c from Table 5	M2/P2 e_c from Table 5	Mi/Pi	R
2113.1 /45.3 46.647	30,233.8 / 736.3 41.067	23,528.1 / 433.7 54.250	26,508.5 / 568.3	12.5
-11.7 / 38.4 -0.305	0 / 2073.1 0	-23,678.1 / 1920.2 -12.331	-631.6 / 2073.1	54.0
-1207.5 / 27.2 –44.393	-30,637.5 / 752.9 -40.693	-24,220.8 / 464.5 -52.144	-27,838.0 / 627.1	23.1
-2415.0 / 9.0 –268.333	-18,625.9 / 253.0 –73.620	-11,700.5 / 33.5 -349.269	-12,061.8 / 45.0	5.0
-2440.4 / 8.7 -280.506	-18,625.9 / 253.0 -73.620	-11,700.5 / 33.5 -349.269	-11,992.4 / 42.8	4.9
-1877.2 / -4.3 436.558	-11,700.5 / 33.5 -349.269	-4472.1 / -155.9 28.686	-9584.1 / -22.0	5.1
-1276.5 / -26.3 48.536	-11,700.5 / 33.5 -349.269	-4472.1 / -155.9 28.686	-5834.3 / -120.2	4.6
-1259.3 / -27.3 46.128	-11,700.5 / 33.5 -349.269	-4472.1 / -155.9 28.686	-5703.3 / -123.6	4.5
12.9 / -12.2 -1.057	5697.0 / -155.9 -36.543	0 / -257.9 0	267.3 / -253.1	20.7

* Units are: in-kips for Ms, M1, M2 and Mi, kips for Ps, P1, P2 and Pi, and inches for e_s and e_c.

The N-S assessment is shown graphically in Figures 35 and 36; the E-W data is shown in Figures 37 and 38.

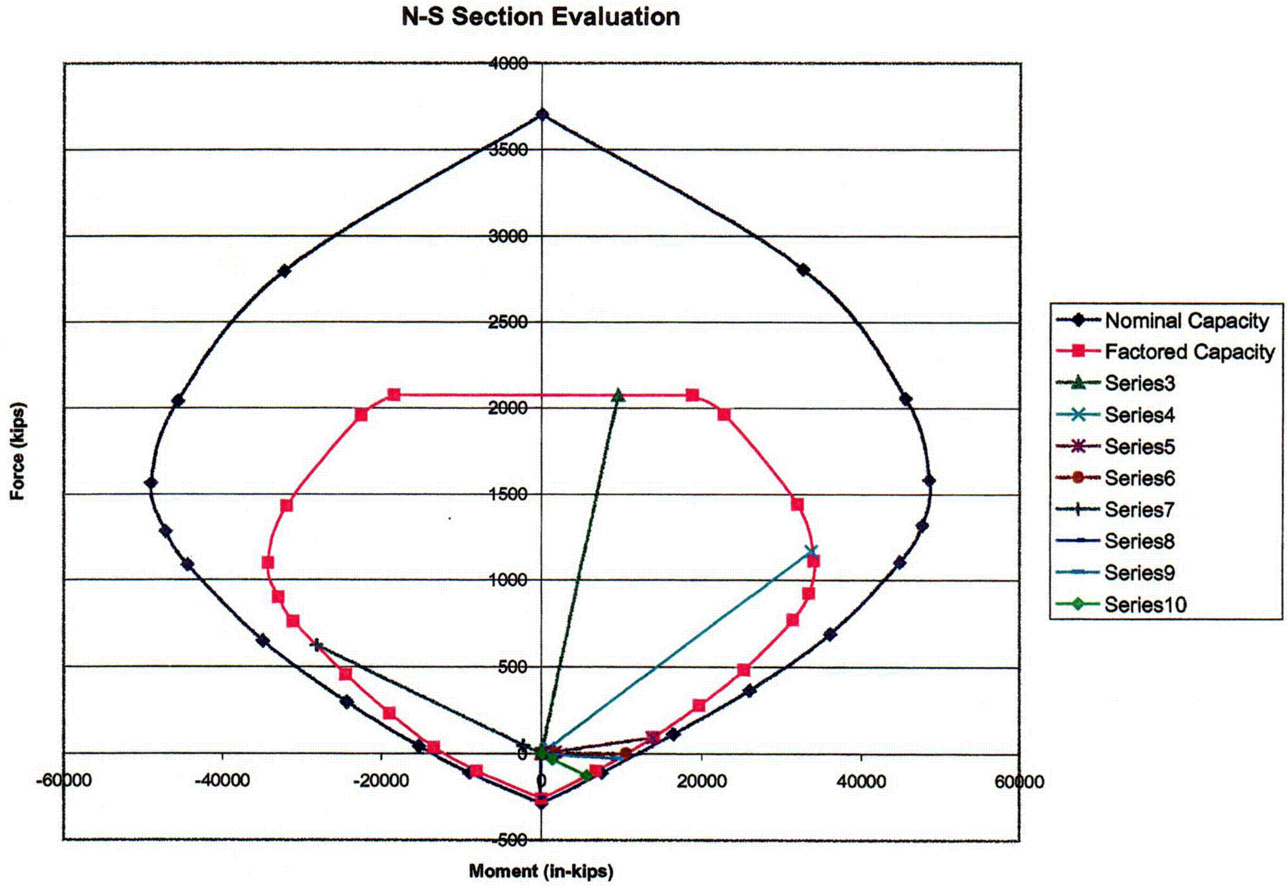


Figure 35 – Graphical Presentation of the N-S Numerical Evaluation data from Table 7 (1/2)

C07

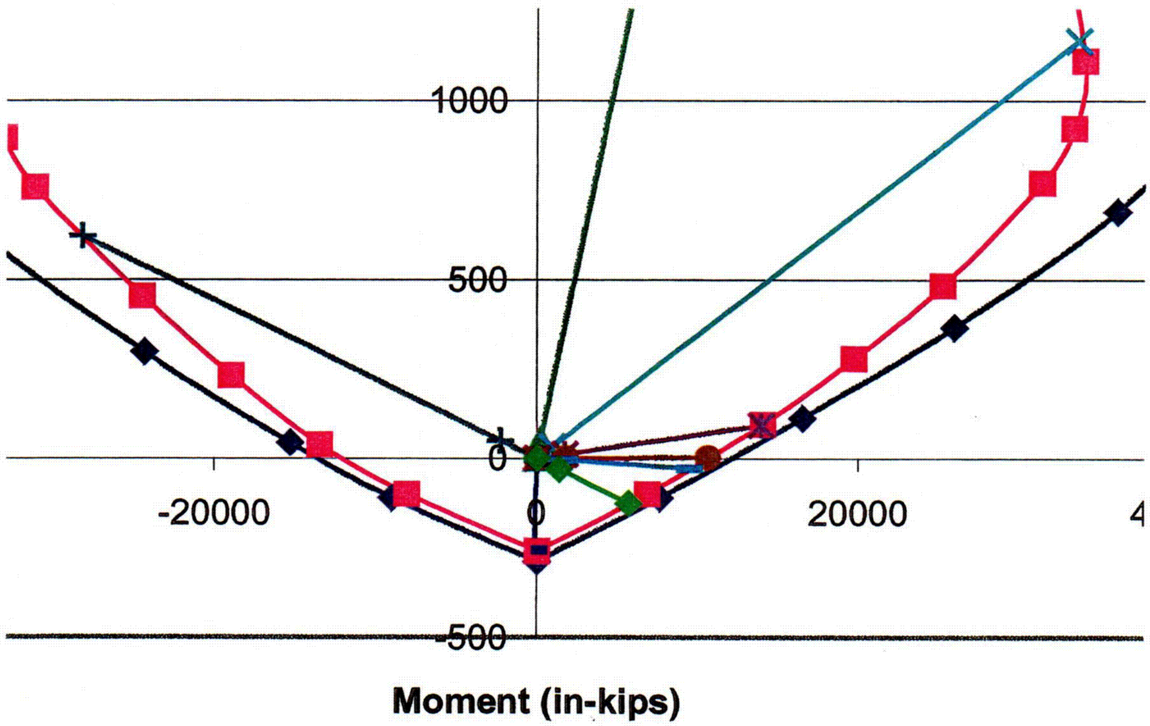


Figure 36 – Graphical Presentation of the N-S Numerical Evaluation data from Table 7 (1/2) zoom at the origin to show detail

C08

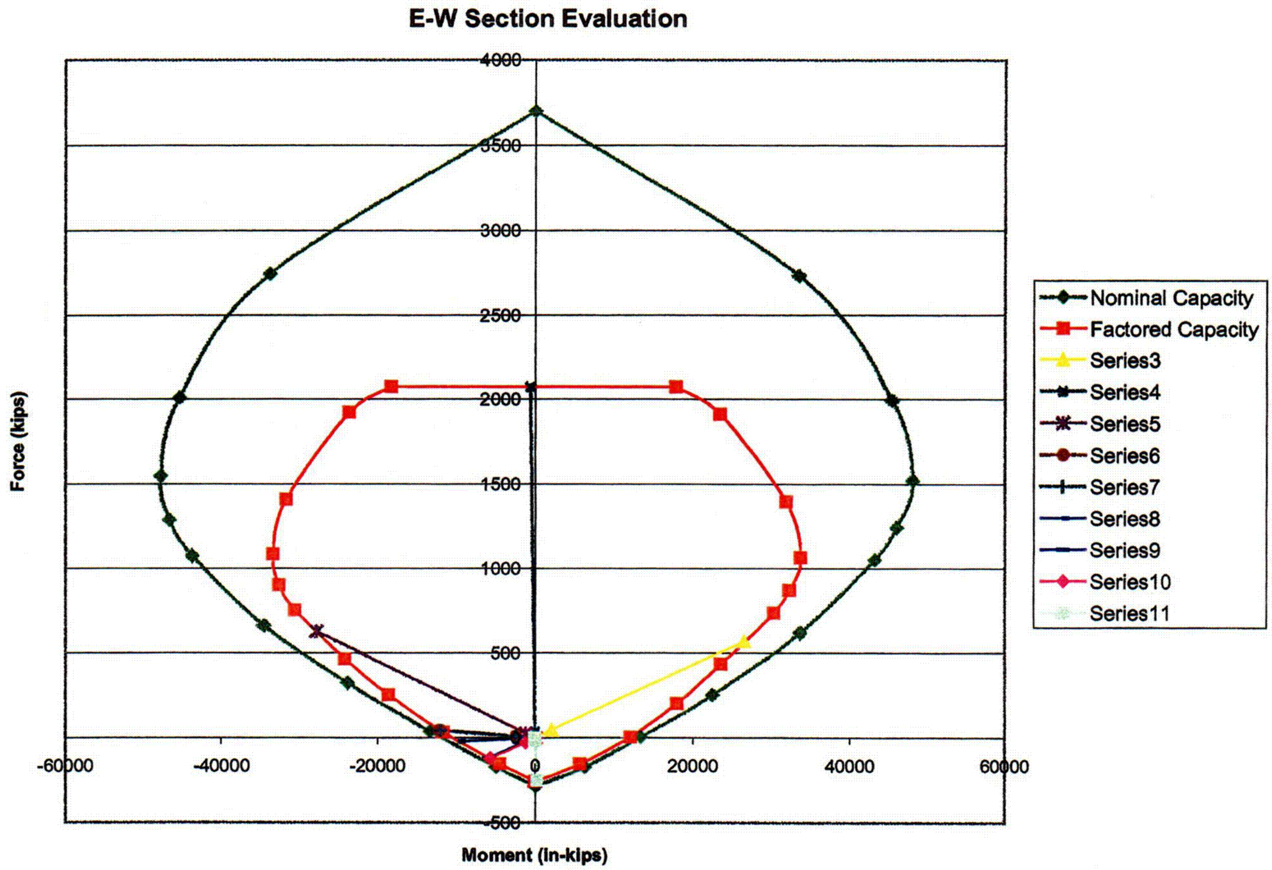


Figure 37 – Graphical Presentation of the E-W Numerical Evaluation data from Table 7 (2/2)

C09

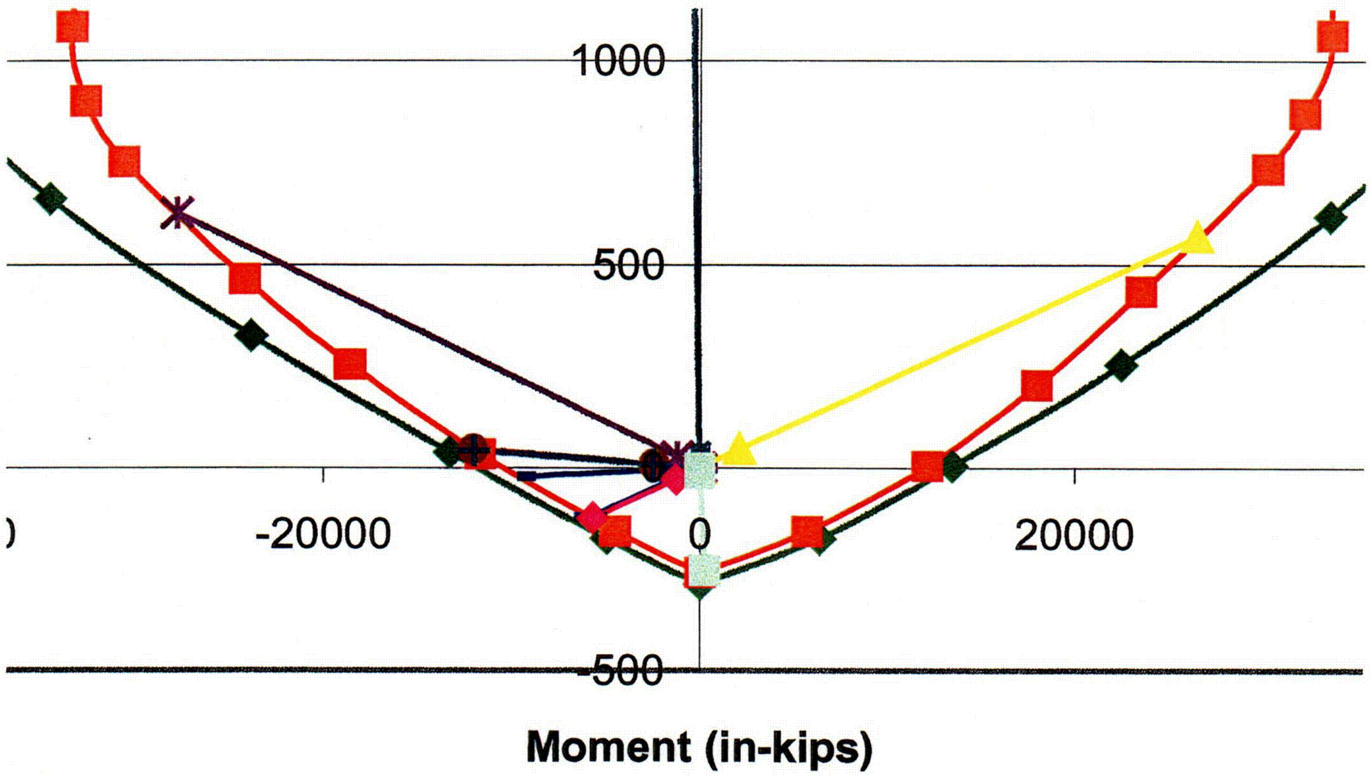


Figure 38 – Graphical Presentation of the E-W Numerical Evaluation data from Table 7 (2/2) zoom at the origin to show detail

Shear Evaluation

Evaluation of the applied shears per ACI.

The applied shears are:

$V_{ns} = 37.1$ kips see Table 6 (1/2)
$V_{ew} = 33.2$ kips see Table 6 (2/2)

Calculate the effective depths (since the bending capacities of both the NS and EW sections are greater than twice the applied loads, the inner layers of reinforcement do not need to be included in the calculation for d):

$$d_{ns} = 90.00 - 3.45 = 86.55 \text{ inches}$$

$$d_{ew} = 90.00 - 5.535 = 84.465 \text{ inches}$$

and, calculate the Code strength and compare to the applied forces:

$$\varphi Vc = 2\varphi\sqrt{f_c'}bd \quad \text{see Reference 11, 11.3.1.1}$$

Therefore,

$$\varphi Vc_{ns} = 2 \times 0.85 \times \sqrt{5000} \times 9 \times 86.55 = 93{,}636.1 \text{ lbs} = 93.6 \text{ kips} > 37.1 \text{ kips OK}$$

and

$$\varphi Vc_{ew} = 2 \times 0.85 \times \sqrt{5000} \times 9 \times 84.465 = 91{,}380.4 \text{ lbs} = 91.4 \text{ kips} > 33.2 \text{ kips OK}$$

Therefore, the applied shears are well within ACI Code limits.

Development Length and Lap Splice Requirements

All reinforcement in the horizontal plane will be spliced using mechanical connectors. However, at the edges, the bars will be bent to the vertical and lap spliced.

Compute the basic development length l_d per ACI (Reference 11, 12.2) for the #10 bars.

The basic length is computed from either 12.2.2 or 12.2.3. Compute spacing values to determine the appropriate equation for l_d. The clear distance is nominally 9-1.27 = 7.73 inches. If bars are moved say

½ inch and lap spliced in a horizontal plane, then the clear distance becomes 8.5 – 2 x 1.27 = 5.96 inches. Now, 5.96 inches is greater than $2d_b = 2.54$ inches. Also, the clear cover will be at least 2 inches (exposed to weather), (See Reference 11, 7.7.1). Since 2 inches is greater than $d_b = 1.27$ inches, the equation to use for l_d from 12.2.2.is:

$$\frac{l_d}{d_b} = \frac{f_y \alpha \beta}{20\sqrt{f_c'}}$$

Now,

$$f_y = 60{,}000 \text{ psi}$$
$$f_c' = 5000 \text{ psi}$$
$$\sqrt{f_c'} = 70.7 \text{ psi} < 100 \text{ psi (see 12.1.2)}$$
$$\alpha = 1.0 \text{ (not a top bar)}$$
$$\beta = 1.0 \text{ bars are uncoated}$$

$$\frac{l_d}{d_b} = \frac{f_y \alpha \beta}{20\sqrt{f_c'}} = \frac{60{,}000 \times 1 \times 1}{20\sqrt{5000}} = 42.4$$

and

$$l_b = 42.4 \times d_b = 42.4 \times 1.27 = 53.9 \text{ inches from 12.2.2}$$

The other ACI method is Eq. (12-1)

$$\frac{l_d}{d_b} = \frac{3}{40} \frac{f_y}{\sqrt{f_c'}} \frac{\alpha \beta \gamma}{\left(\frac{c + K_{tr}}{d_b}\right)}$$

where newly introduced terms are:

$\gamma = 1.0$ size factor
c = spacing or cover dimension (smallest of center to surface or ½ center to center)
K_{tr} transverse reinforcement index = 0 (conservative)

and the term:

$$\frac{c + K_{tr}}{d_b} \text{ must be less than 2.5}$$

Smallest c (yields largest l_d) is 2+1.27/2 = 2.635.
And,

$$\frac{c+K_{tr}}{d_b}=\frac{2.635+0}{1.27}=2.075 \text{ less than } 2.5$$

Therefore,

$$\frac{l_d}{d_b}=\frac{3}{40}\frac{f_y}{\sqrt{f_c'}}\frac{\alpha\beta\gamma}{\left(\frac{c+K_{tr}}{d_b}\right)}=\frac{3}{40}\frac{60{,}000}{\sqrt{5000}}\frac{1\text{ x }1\text{ x }1}{2.075}=30.7$$

$$l_d = 30.7\text{ x }d_b = 30.7\text{ x }1.27 = 39.0\text{ inches from 12.2.3}$$

Therefore the basic development length is 39 inches = 3'-3".

Class B splice, factor by 1.3, 1.3 x 39.0 = 51 inches = 4'-3"

If, in the development of the details, bundled bars result, the splice lengths shall be increased by 20% for 3 bar bundles to 1.2 x 51 = 61 inches and by 33% for 4 bar bundles to 1.33 x 51 = 68 inches.

Seismic Forces in Reinforcement

The design of the reinforcement that is located just above the embedment structure anchor plates has additional requirement beyond the bending response to the thermal, shrinkage and seismic. These are addressed in the embedment structure calculation, see Reference 4. An input to that calculation is the set of forces in the reinforcement due to the seismic demand. There will be no appreciable forces in these bars due to the thermal and shrinkage demand since considerable time will have elapsed between the construction of the pad and the placement of the fuel casks.

As noted in the details below, this calculation uses 90 inches for the depth of the pad while making use of the working stress equations developed for the thermal analyses. Use is made of equations 1 and 2 since the applied moments produce tension on the bottom of the pad.

Forces in Reinforcement: Bounding Seismic - Tension on Bottom NS (Z) Strip

$b = 9$ inches; $h = 90$ inches; $d_1 = 3.45$ inches; $d_2 = 7.63$ inches; $d_3 = 3.45$ inches; $d_4 = 16.55$ inches

$A_s = 1.27$ sq. in. #10 bar $d_{net} = -8.92$

$f'_c = 5.000\,\text{ksi}$

$E_c = 4030$ ksi; $E_s = 29000$ ksi; $\alpha = \frac{29000}{4030} = 7.196$

$\sigma_c = 0.221\,\text{ksi}$ from off-line calculations; $\varepsilon_c = \frac{0.221}{4030} = 5.48\text{E-5 in/in}$

Bounding Seismic Force/Moment Combination from Table 6 (1/2).
This is the largest positive Mx combined with the largest tension:

C = -695.867 kip/17 foot section; C = -30.7 kip/9 inch section (-) indicates tension
M = 39675.73 in-kip/17 foot section; M = 1750.4 in-kip/9 inch section

Moment produces tension on the bottom of the pad, therefore use Equations (1) and (2).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \times 1.27 \times 7.196 - \frac{-30.7}{0.221} = 175.470$$

$$2bA_s\alpha(2h - d_{net}) = 2 \times 9 \times 1.27 \times 7.196 \times (2 \times 90 - 8.92) = 28{,}142.76$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{net})}}{b} = \frac{-175.470 + \sqrt{175.470^2 + 28{,}142.76}}{9} = 7.477 \text{ inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{7.477 \text{ x } 9 \text{ x } 0.221}{2} = 7.436 \text{ kip}$$

$$C_1 = A_sE_s\left(\frac{x-d_1}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{7.477 - 3.45}{7.477}\right) \text{x } 5.48\text{E-}5 = 1.088 \text{ kip}$$

$$C_2 = A_sE_s\left(\frac{x-d_2}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{7.477 - 7.63}{7.477}\right) \text{x } 5.48\text{E-}5 = -0.041 \text{ kip} \quad \text{(-) indicates tension}$$

$$T_1 = A_sE_s\left(\frac{h-x-d_3}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{90 - 7.477 - 3.45}{7.477}\right) \text{x } 5.48\text{E-}5 = 21.344 \text{ kip}$$

$$T_2 = A_sE_s\left(\frac{h-x-d_4}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{90 - 7.477 - 16.55}{7.477}\right) \text{x } 5.48\text{E-}5 = 17.808 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$$\therefore \; C = C_c + C_1 + C_2 - T_1 - T_2$$

-30.7 = 7.436+1.088-0.041-21.344-17.808 = -30.67 OK

Now the internal moment is:

$$M = C_c\left(\frac{h}{2} - \frac{x}{3}\right) + C_1\left(\frac{h}{2} - d_1\right) + C_2\left(\frac{h}{2} - d_2\right) + T_1\left(\frac{h}{2} - d_3\right) + T_2\left(\frac{h}{2} - d_4\right) \qquad \text{Equation (2)}$$

$$M = 7.436\left(\frac{90}{2} - \frac{7.477}{3}\right) + (1.088)\left(\frac{90}{2} - 3.45\right) + (-0.041)\left(\frac{90}{2} - 7.63\right) + \ldots$$

$$(21.344)\left(\frac{90}{2} - 3.45\right) + (17.808)\left(\frac{90}{2} - 16.55\right) = 1753.2 \cong 1750.4 \text{ OK}$$

Forces in Reinforcement: Bounding Seismic – Tension on Bottom EW (X) Strip

b = 9 inches; h = 90 inches; d_1 = 5.535 inches; d_2 = 9.705 inches; d_3 = 5.535 inches; d_4 = 18.635 inches

A_s = 1.27 sq. in. #10 bar d_{net} = -8.93

$f_c' = 5.000\,\text{ksi}$

E_c = 4030 ksi; E_s = 29000 ksi; $\alpha = \frac{29000}{4030} = 7.196$

$\sigma_c = 0.398$ ksi from off-line calculations; $\varepsilon_c = \frac{0.398}{4030} = 9.88\text{E-5 in/in}$

Bounding Seismic Force/Moment Combination from Table 6 (2/2).
This is the largest negative Mz combined with the largest tension:

C = -618.8 kip/17 foot section; C = -27.3 kip/9 inch section
M = 55315.73 in-kip/17 foot section; M = 2440.4 in-kip/9 inch section

Moment produces tension on the bottom of the pad, therefore use Equations (1) and (2).

$$\left(4A_s\alpha - \frac{C}{\sigma_c}\right) = 4 \text{ x } 1.27 \text{ x } 7.196 - \frac{-27.3}{0.398} = 105.149$$

$$2bA_s\alpha(2h - d_{net}) = 2 \text{ x } 9 \text{ x } 1.27 \text{ x } 7.196 \text{ x } (2 \text{ x } 90 - 8.93) = 28141.111$$

$$x = \frac{-\left(4A_s\alpha - \frac{C}{\sigma_c}\right) + \sqrt{\left(4A_s\alpha - \frac{C}{\sigma_c}\right)^2 + 2bA_s\alpha(2h - d_{net})}}{b} = \frac{-105.149 + \sqrt{105.149^2 + 28141.111}}{9} = 10.315\,\text{inches}$$

Now,

$$C_c = \frac{1}{2}xbE_c\varepsilon_c = \frac{xb\sigma_c}{2} = \frac{10.315 \text{ x } 9 \text{ x } 0.398}{2} = 18.474\,\text{kip}$$

$$C_1 = A_sE_s\left(\frac{x - d_1}{x}\right)\varepsilon_c = 1.27 \text{ x } 29000\left(\frac{10.315 - 5.535}{10.315}\right) \text{x } 9.88\text{E-5} = 1.686\,\text{kip}$$

$$C_2 = A_s E_s \left(\frac{x - d_2}{x} \right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{10.315 - 9.705}{10.315} \right) \text{x } 9.88\text{E-}5 = 0.215 \text{ kip}$$

$$T_1 = A_s E_s \left(\frac{h - x - d_3}{x} \right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{90 - 10.315 - 5.535}{10.315} \right) \text{x } 9.88\text{E-}5 = 26.158 \text{ kip}$$

$$T_2 = A_s E_s \left(\frac{h - x - d_4}{x} \right) \varepsilon_c = 1.27 \text{ x } 29000 \left(\frac{90 - 10.315 - 18.635}{10.315} \right) \text{x } 9.88\text{E-}5 = 21.536 \text{ kip}$$

Check on equilibrium $\sum F = 0$

$$\therefore\ C = C_c + C_1 + C_2 - T_1 - T_2$$

$$-27.3 = 18.474 + 1.686 + 0.215 - 26.158 - 21.536 = -27.32 \text{ OK}$$

Now the internal moment is:

$$M = C_c \left(\frac{h}{2} - \frac{x}{3} \right) + C_1 \left(\frac{h}{2} - d_1 \right) + C_2 \left(\frac{h}{2} - d_2 \right) + T_1 \left(\frac{h}{2} - d_3 \right) + T_2 \left(\frac{h}{2} - d_4 \right) \qquad \text{Equation (2)}$$

$$M = 18.474 \left(\frac{90}{2} - \frac{10.315}{3} \right) + (1.686) \left(\frac{90}{2} - 5.535 \right) + (0.215) \left(\frac{90}{2} - 9.705 \right) + \ldots$$

$$(26.158) \left(\frac{90}{2} - 5.535 \right) + (21.536) \left(\frac{90}{2} - 18.635 \right) = 2442.1 \cong 2440.4 \text{ OK}$$

Thus, the largest N-S or E-W force in a bar just above the anchor plates (T_2) is 21.536 kip.

Summary and Conclusions

This calculation has provided the determination of the evaluation required by the governing documents for the design, i.e., the PG&E specification and all of its relevant subordinate documents and the regulatory requirements. Further, the selected reinforcement, #10 bars at 9 inches, Figures 1 and 2, has been shown to limit the expected crack widths due to curing stresses (temperature and shrinkage) to within the recommended values prescribed by ACI 207. The seismic evaluation shows that the pad is compliant with all the required design parameters. Finally, forces in reinforcement are computed for use in the embedment structure design.