ENCLOSURE 3

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"EVALUATION OF STEEL STRUCTURES WITH THERMAL RESTRAINT,"
REVISION 1

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STRUCTURAL STEEL

Evaluation of Steel Structures

with Thermal Restraint

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STRUCTURAL STEEL Evaluation of Steel Structures with Thermal Restraint	REVISION LOG CIVIL DESIGN GUIDE DG-C1.6.12

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0	Original issue	10/26/88
1	Revised to incorporate outstanding revision notices and current recommended thermal evaluation techniques. General revision, changes are not marked due to extensive changes to the organization of the guide. Incorporated outstanding DGCN-CEB-89-01 and -02.	05/06/91

TABLE OF CONTENTS

APPENDIX A - EXAMPLE PROBLEMS

APPENDIX B - DERIVATION OF EQUATIONS

APPENDIX C - LINEAR ACCEPTANCE CRITERIA

APPENDIX D - NON-LINEAR ACCEPTANCE CRITERIA

1.0 GENERAL

The purpose of this design guide is to provide guidelines for the evaluation of thermal effects on existing steel structures.

1.1 APPLICABILITY

The use of this design guide is permitted only where invoked by specific plant design criteria.

The scope of this guide provides information on evaluation of thermal behavior for steel framing and attachment to concrete surfaces. The guide does not cover the evaluation of the thermal loads resulting from the heat input during the welding process.

Recommendations are provided for the development of an evaluation program for thermal effects on a large population of structures. The recommended evaluation program is based on evaluation of worst-case structures through a screening evaluation.

2.0 BACKGROUND

2.1 TERMS AND DEFINITIONS

Ancillary Member - An interconnecting member which is in the thermal load path and is capable of withstanding large nonlinear self-limiting deformations without loss of capacity to resist non-thermal load.

Ductility ratios (μ) - Ratio of the strain or displacement in a member being evaluated to the strain or displacement at yield in the member. There are three types of ductility ratios, they are defined as follows:

μ_{eb} - Ductility ratio based on the Energy Balance Equation from Appendix B.

μ_s - Ductility ratio based on strain determined by non-linear analysis.

μ_d - Ductility ratio based on displacements determined by non-linear analysis.

Ductile response - The ability of a structure to deform inelastically when displaced beyond its yield displacement. The relationship of loads and displacements can be idealized by a load/displacement curve.

Elastic range - Beginning portion of load/displacement curve which represents the member's or connection's initial elastic behavior. The slope of this portion of the load/displacement curve is equal to the member or connection's spring constant.

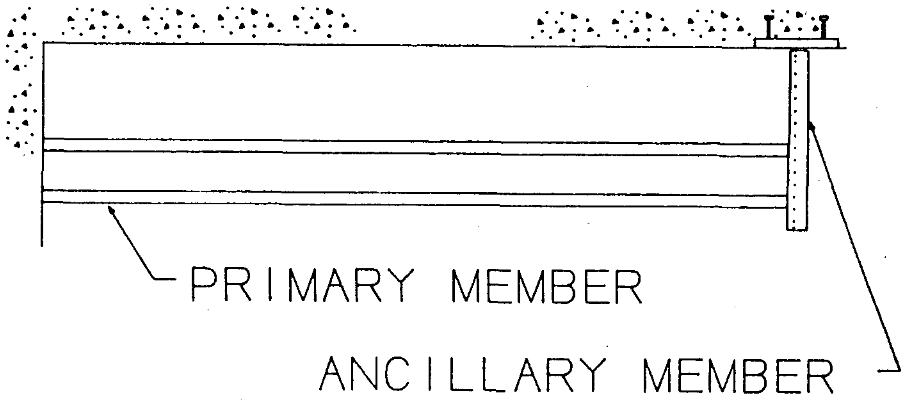
Linear Analysis - A structural analysis which establishes member behavior assuming a linear stress-strain curve.

Load/displacement curve - In the context of this guide, an idealized plot of the load on a structure (ordinate) plotted against a displacement in the structure (abscissa) under the load.

Non-linear Analysis - A structural analysis which establishes member behavior assuming a non-linear stress-strain curve.

Plastic behavior - Portion of load/displacement curve which represents the member's or connection's behavior beyond the elastic range.

Primary member - A thermally restrained member which may experience some loss of capacity to resist non-thermal loads due to large deformations. All members which are not classified as Ancillary members may conservatively be assumed to be primary members.



ANCILLARY AND PRIMARY MEMBERS

Spring constant (Ks) - Slope of the elastic range of a load/displacement curve.

Thermal coefficient (ε) - Linear change in length, per unit of length, for a change of one degree of temperature.

Thermal loads - Structural reactions induced in a restrained structure by thermal growth.

Thermal growth - The movement of a structure in response to a change in temperature.

Yield displacement - The displacement of a member at the point when the member transitions from elastic behavior to plastic behavior.

Yield load - The load on a member at the point when the member transitions from elastic behavior to plastic behavior.

Yield stress - The stress in a member at the point when the member transitions from elastic behavior to plastic behavior.

2.2 THERMAL BEHAVIOR INTRODUCTION

NRC Standard Review Plan, Reference 10.4, states that thermal loads can be neglected when it is shown that they are secondary and self-limiting in nature and where the material is ductile. This design guide provides guidelines for demonstration of ductile and self-limiting response under thermal load and recommended acceptance criteria.

This guide provides two methods for evaluation of steel structures for thermal loads combined with other loads:

1. Conventional linear analysis methods either using manual techniques, STRUDL or ANSYS

2. Non-linear analysis method using ANSYS

Recommended linear acceptance criteria are provided in Appendix C. These criteria are based generally on AISC allowable capacity with a factor of safety of 1.0 instead of 1.67. The use of a parabolic interaction curve for evaluation of combined bending and axial load is provided and allowable compression stresses in short stocky columns may be increased approximately 40 percent above the AISC stress. Since thermal stresses often go beyond yield, linear methods are most valuable for screening. Interaction values based on linear acceptance criteria are recommended for use in identification of worst case structures.

Acceptance criteria for members modeled using non-linear material properties are included in Appendix D. Primary members are limited to a ductility ratio, based on displacement, of 3. Ancillary members are limited to the maximum ductility ratios, based on either strain or displacement, given in Appendix A, Section 3.5.3, of Reference 10.4.

3.0 RECOMMENDED THERMAL EVALUATION PROGRAM

3.1 GENERAL

3.1.1 Thermal evaluations in accordance with this guide will generally be performed as part of a program for the investigation of thermal effects on a large population of steel structures. Therefore, a critical characteristic of thermal evaluations is the selection of structures and members for evaluation. This section of the guide provides recommendations for the selection of worst case structures based on experience from prior programs.

3.1.2 The identification of the structures most susceptible to thermal effects consist of 3 actions:

1. Initial screening of the entire population for identification of all structures which are thermally restrained.

2. Final screening to identify representative structures for rigorous evaluation. This screening considers the relative magnitude of nonthermal loads being applied to the structure.

3. Rigorous analysis of worst case structures which includes the application of the appropriate thermal loads in combination with nonthermal loads.

3.2 INITIAL SCREENING OF THERMALLY RESTRAINED STRUCTURES

3.2.1 All project structural drawings should be reviewed to find those structures which are restrained against thermal growth and are located in high temperature areas. Thermally restrained structures that are identified should be uniquely identified for tracking. The unique identifier and relevant data such as reference drawing numbers, ambient temperature, operating temperature, accident temperature, and broad classification of the restraint type should be tabulated for ease of reference. A sketch or duplication of a drawing detail depicting each structure should be incorporated into the project thermal evaluation calculations.

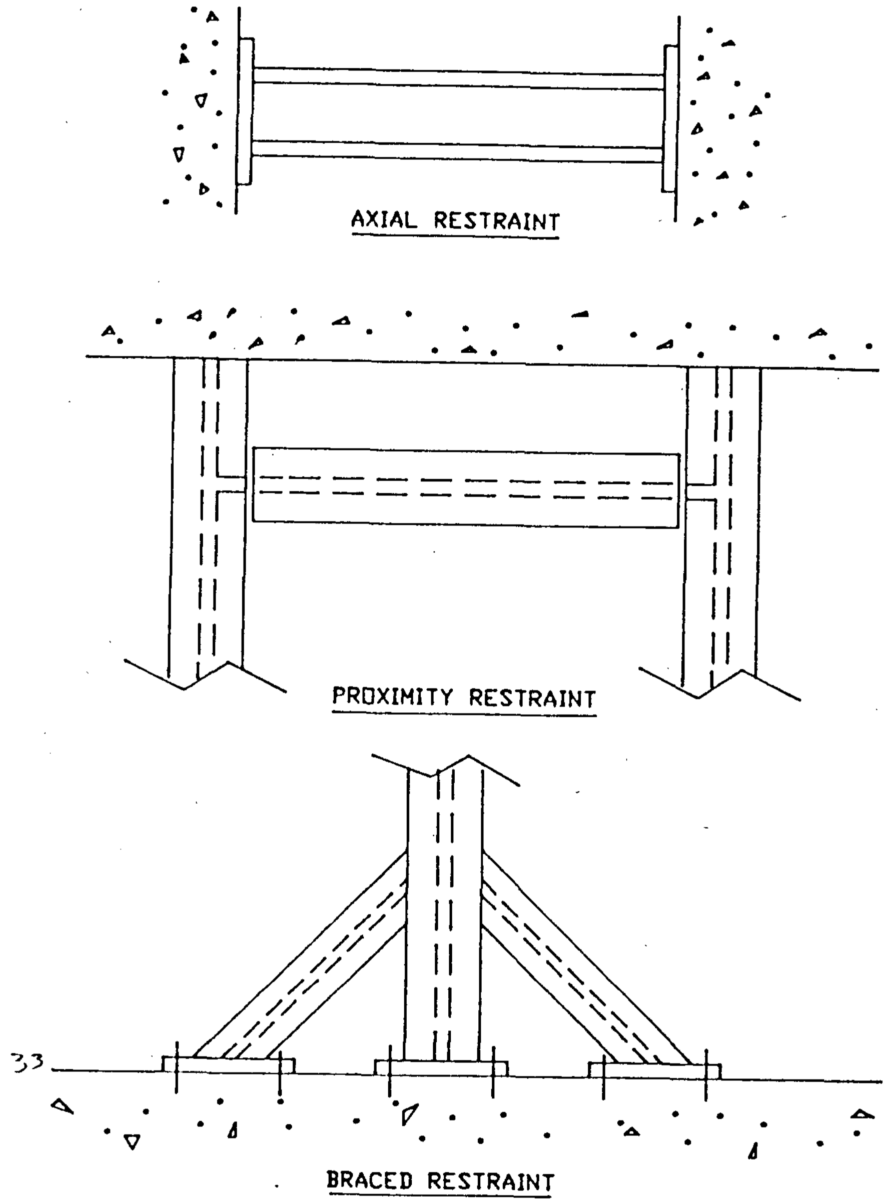


FIGURE 1 TYPICAL THERMAL RESTRAINT CONFIGURATIONS

3.2.2 Particular emphasis is needed when the following situations are encountered in high temperature areas:

(1) where both ends of the member attach directly to concrete surfaces (See Axial Restraint, Figure 1).

Note: As a member elongates or shortens due to temperature change, the axial load in the member will cause compression or tension in the end anchorage.

(2) where both ends of the member attach to relatively stiff adjoining steel members which are close to a concrete surface. (See Proximity Restraint, Figure 1)

(3) where both ends of the member attach to brace points in framing areas restrained by opposing concrete connections. (principally bracing members designed for tension and compression loads in frames with at least two concrete connections. See Braced Restraint, Figure 1)

(4) where both ends of the member attach to surface mounted or embedded plates. (See Header beam, Figure 2)

(5) where restraint can be classified as some combination of (1) through (4) above.

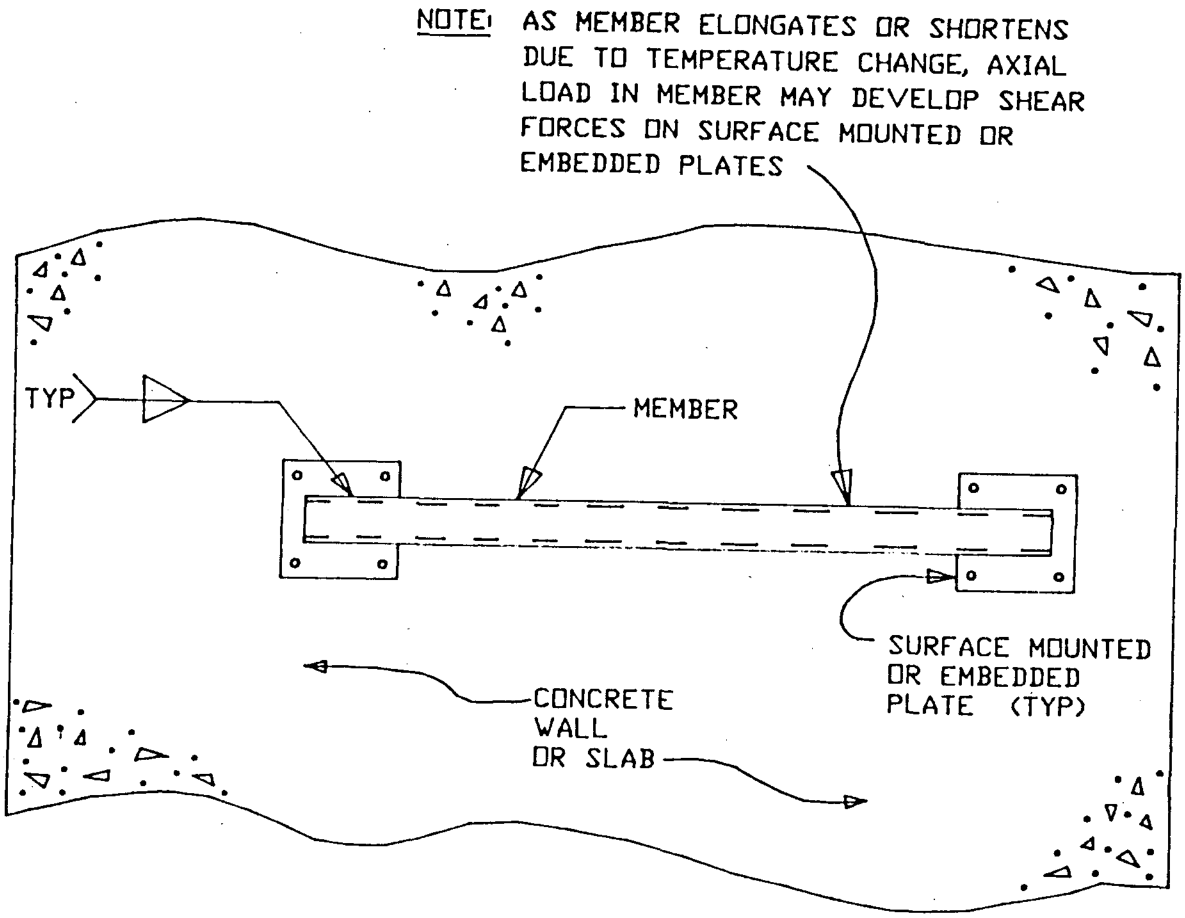


FIGURE 2 - HEADER BEAM

3.3 FINAL SCREENING OF REPRESENTATIVE CASES

3.3.1 The final screening evaluation is intended to group representative structures from which worst cases will be selected for rigorous evaluation. The worst case structures should be selected to envelop as many thermally restrained structures as possible.

3.3.2 Having identified, broadly classified, and tabulated all of the thermally restrained plant structures, the following information should be considered for the selection of the structures for rigorous evaluation.

A. Existence of details described in Section 3.3.3

B. Distance between thermal restraint points

C. Stiffness of restrained members and connections

3.3.3 Construction configuration details to be considered when loaded by thermal restraint either during heat-up or cool-down:

A. Welded member connections

B. Bolted friction or bearing member connections

C. Self-drilling concrete anchors under tension or shear

D. Wedge bolt anchors under tension or shear

E. Embedded anchor bolts under tension or shear

F. Headed concrete anchors under tension or shear

G. Axially restrained non-compact members

H. Existence of a non-thermal lateral load on axially restrained member

I. Existence of thermally induced lateral forces on axially restrained member

J. Thermal restraint provided by a concrete slab or wall which is vulnerable to punching shear

K. Restrained structures which form a partial or complete ring.

L. Proximity to free concrete edges for concrete anchors

Where necessary to establish this information abbreviated field assessments may be needed.

3.3.4 To simplify selection of the worst cases, spring stiffness may be estimated. The following approximate (order of magnitude) stiffness values are provided for that purpose (See Reference 10.16 for justifications):

A. Welds - shear: 10,000 kips/in

B. Bolts - shear: 10,000 kips/in per connection
tension: 10,000 kips/in per connection

Note: Where combinations of bolts and welds are found in the same connection, the overall connection would have a spring constant of 10,000 K/in.

C. Self-drilling concrete anchors -

shear: 1000 kips/in per anchor
tension: 400 kips/in per anchor

D. Wedge bolt anchors

shear: 1000 kips/in per anchor
tension: 400 kips/in per anchor

E. Embedded anchor bolts and headed concrete anchors

shear: 1000 kips/in per anchor
tension: 1000 kips/in per anchor

F. Steel members:700 x (beam size,lb/ft) / (Length, ft) (Note: The spring constant will be in kips/in)

G. Restraint produced by weak axis bending in any plate element of a connection: 50 kips/in

H. Restraint produced by bending in a structural member producing proximity restraint: 100 kips/in

I. Brace point resistance in a braced frame: 2000 kips/in

J. Baseplate rotational spring constant: 100,000 kip-in/radian (Note: This value can be translated to a linear spring constant for a specific configuration.)

WARNING: DO NOT USE THE ABOVE SPRING CONSTANTS IN FINAL THERMAL ANALYSIS. ORDER OF MAGNITUDE SPRING CONSTANT ESTIMATES ARE NOT SUFFICIENTLY ACCURATE FOR RIGOROUS ANALYSIS.

3.3.5 The spread sheet form which follows may be used to assemble estimates of the thermal restraining forces and information about configuration details for all restrained members.

3.3.6 Worst case structures should be selected such that each configuration detail present in the plant is evaluated. The determination of the worst case thermal structure should be based on consideration of the distance between thermal restraint points, temperature rise, and the stiffness of restrained members and connections. Although the spreadsheet information will prove useful in making the selection, the final identification of the worst case structures should be based on all relevant information and may be based partially on documented qualitative engineering judgments. A quantitative selection basis simplifies the demonstration that one structure will envelop a second structure and facilitates later comparisons to the worst case structure.

3.3.7 It is recommended that an abbreviated engineering field assessment inspection be conducted for all thermally restrained structures.

3.3.8 Where significant lateral loads are identified on engineering field assessments, they shall be considered in selection of the worst case thermal structures.

3.3.9 For any thermally restrained structure, an evaluation of each connection and member to assess existing stress risers is required. Extreme copes, a concentration of flange bolt holes at a potential plastic hinge points, or other details which could result in stress concentrations or local failure in plastic regions under thermal loading should be evaluated on a case by case basis. These factors must be considered as part of the engineering field assessment.

Structure ID#	______	______	______
Member ID#	______	______	______
Tamb, degree F	______	______	______
Tacc, degree F	______	______	______
C/M size	______	______	______
L, W/M length, ft	______	______	______
Lx, W/M length, ft	______	______	______
Ly, W/M length, ft	______	______	______
w, W/M wt/ft	______	______	______
rx, W/M r value	______	______	______
ry, W/M r value	______	______	______
K1, W/M spring - End 1	______	______	______
K2, W/M spring - Member=> 700*w/L	______	______	______
K3, W/M spring - End 2	______	______	______
Ktot, 1/(1/K1+1/K2+1/K3)	______	______	______

RESTRAINT FORCE & MEMBER ALLOWABLE:

Tδ = Tacc - Tamb	______	______	______
Pscreen = Tδ*L/(12800*Ktot)	______	______	______
φ min: K*Lx/(89*ry) or K*Lx/(89*rx)	______	______	______
Pallow	______	______	______

$0.00 \le \phi \le 0.15$: 15.0 * w

$0.15 \le \phi \le 0.40$: 16.9 * (1-φ) * w

$0.40 \le \phi \le \sqrt{2}$: 10.6 * $(1 - \phi^2/4)$ * w

$\sqrt{2} \le \phi \le 2$: 10.6 * w / ϕ^2

Interaction ratio=Pscreen/Pallow	______	______	______

ATTRIBUTES(yes/no):

Welded	______	______	______
Bolted	______	______	______
SSD	______	______	______
WB	______	______	______
AB	______	______	______
HCA	______	______	______
Non-compact section	______	______	______
Non-thermal S/R lateral load	______	______	______
Thermal S/R lateral load	______	______	______
Concrete slab or wall	______	______	______
Ring	______	______	______
Free edge	______	______	______

W/M - Worst case member   S/R - Safety-related

4.0 GENERAL ANALYSIS DATA

4.1 DETERMINATION OF SPRING CONSTANTS

The following sections provide values or methods for determining values of constants used in thermal evaluations.

4.1.1 Axial Spring Constants for Steel Members

The axial spring constant for a steel member is equal to:

Equation 1:

$$Ks = \frac{(\text{Member area}, As \times \text{Modulus of elasticity}, Es)}{\text{Member length}, L}$$

4.1.2 Bending Spring Constants for Steel Members

Equation 2:

Ks = Load which causes a unit translational displacement at the members attachment point.

See example problem 1 in Appendix A.

4.1.3 Connection Spring Constants for Steel Framing

Spring constant data is not generally available. Connection springs can conservatively be neglected (*).

4.1.4 Spring Constants for Concrete Walls and Slabs

Equation 3:

Ks = Load which causes a unit displacement in the supporting wall or slab at the members attachment point using the effective moment of inertia (Ie) in accordance with ACI 318 (Reference 10.2).

(*) Friction connections must be assumed to have no free movement in the context of thermal ductility evaluations unless the slip load is exceeded.

4.1.5 Concrete Attachment Spring Constants

All spring constants for attachments to concrete surfaces shall be in accordance with Appendix A of Design Standard DS-C1.7.1 (Reference 10.5).

Springs for thermal loads which result in a bearing load being placed directly on a concrete surface may conservatively be neglected.

4.1.6 Rotational Spring Constants for Baseplates

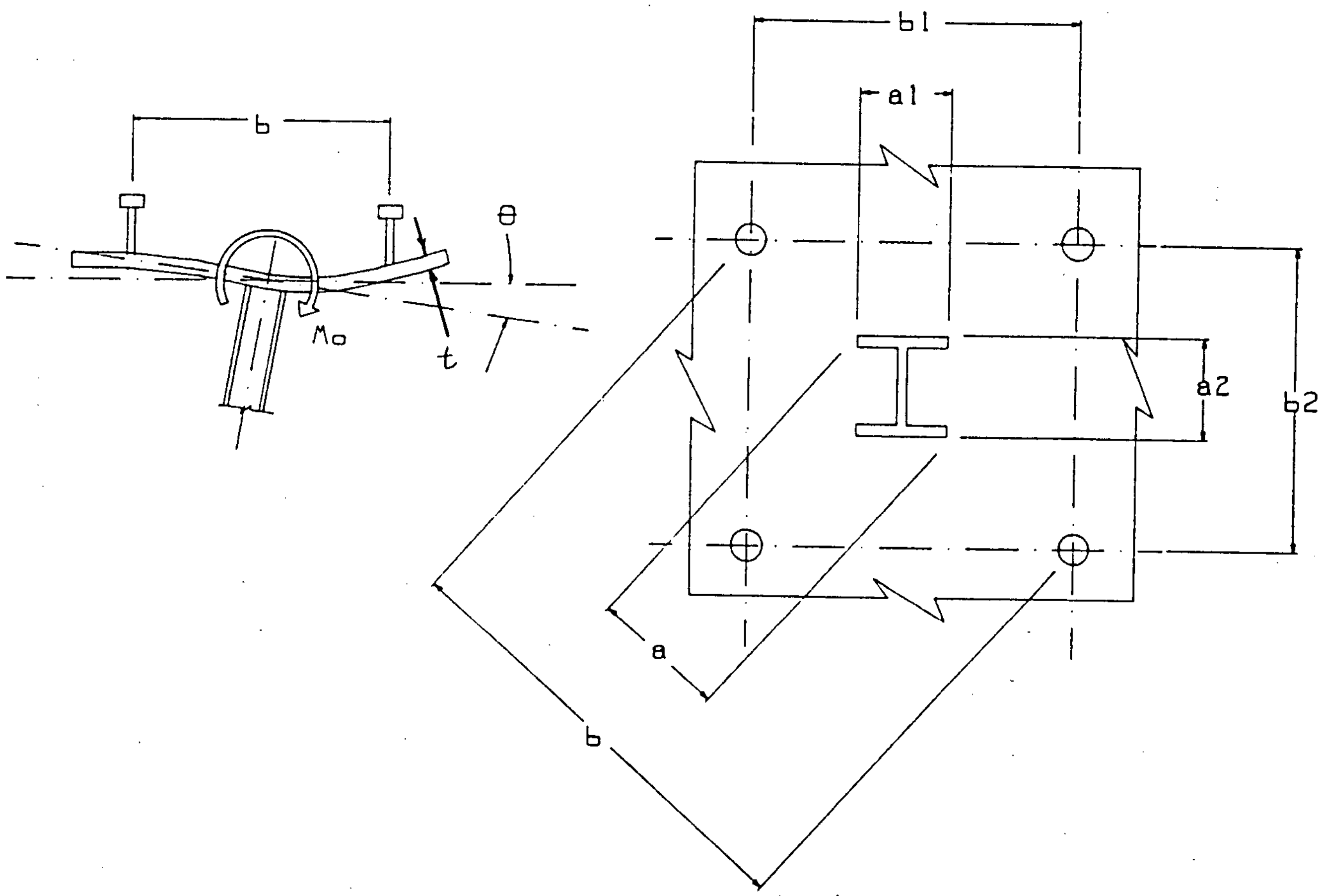


FIGURE 3 - BASEPLATE ROTATION SPRING CONSTANT DETERMINATION

Figure 3 provides a method for approximating the stiffness for the rotational spring associated with an embedded or surface mounted plate (See Reference 10.15, Table 24, Case 20)

$$a = \sqrt{(a1^2 + a2^2)} \quad \& \quad b = \sqrt{(b1^2 + b2^2)}$$

a/b	0.1	0.15	0.2	0.25	0.3	0.4	0.5	0.6	0.7	0.8
α	1.403	1.058	0.820	0.641	0.500	0.301	0.169	0.084	0.035	0.010

$$K_{plate\text{-}rot} = E * t^3 / \alpha$$

Alternately this spring constant may be determined using unit loads applied to a BASEPLATE II model.

4.2 VARIATION IN STEEL PROPERTIES WITH TEMPERATURE

There is a decrease in the modulus of elasticity (E_s) with increased temperature. Figure 4 describes the reduction.

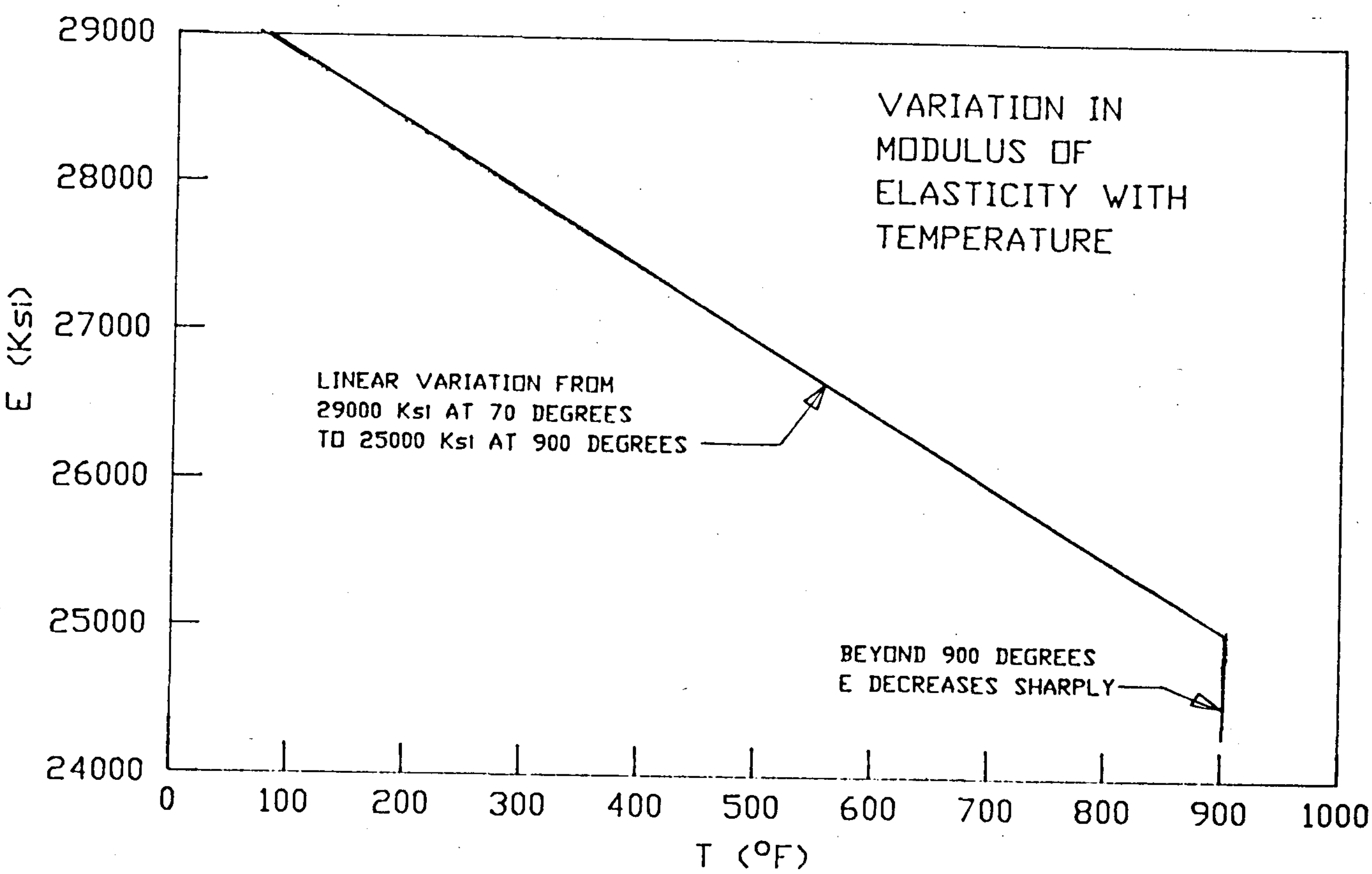


FIGURE 4 - VARIATION IN MODULUS OF ELASTICITY

4.3 TEMPERATURE RISE AND THERMAL COEFFICIENT

For temperatures (t) exceeding 100 degrees fahrenheit up to 1200 degrees fahrenheit:

$$\varepsilon_{stl} = (6.1+0.0019\ t) \times 10^{-6}\ \text{in/(in} \times {}^{\circ}\text{F)}^{*}$$

(*) Inch units (in) are shown, however unit length is to be consistent with the unit length used in other computations.

The temperature rise used for thermal evaluations shall account for differential thermal growth which takes place between the plant criteria ambient temperature and the accident or operating temperature being evaluated. Normally evaluations shall be based on the coefficient of expansion of steel, ε_{steel}, and the temperature difference between ambient and either the operating or accident temperature.

Where documented justification demonstrates that the rise to the operating temperature takes place slowly, and that concrete restraining surfaces can move in response to temperature change, the thermal coefficient used for operating temperature evaluation may be based on differential of the thermal coefficients for concrete and steel.

$$\varepsilon_{delta} = \varepsilon_{steel} - \varepsilon_{concrete}$$

where: $\varepsilon_{concrete} = 5.5 \times 10^{-6}$ in/in-F°

from Reference 10.18, page 187.

The rise from the operating temperature to the accident temperature is relatively rapid and therefore the coefficient of expansion for the structure must be assumed to be the coefficient for steel.

The temperature of the member used for determination of temperature rise is normally assumed to be the peak compartment temperature from the project environmental drawings. This temperature may be reduced by performing a heat flow analysis to determine the rise taking boundary conditions and the members response over time into consideration. The analysis is normally performed by the mechanical engineering organization. The calculation can be either a manual or a finite element analysis.

5.0 RIGOROUS ANALYSIS

Detailed documented walk-downs are needed for each worst case for rigorous analysis. For worst case analysis, all significant extreme design event attachment loads shall be determined by appropriate calculation.

5.1 LINEAR ANALYSIS - MANUAL

Where this method is used for final acceptance, the stresses from non-thermal loads must be superimposed.

Worst case structures can be analyzed using conventional linear analysis techniques. Acceptance of structures analyzed by linear analysis shall be based on project design criteria. Acceptance criteria are recommended in Appendix C.

A flow diagram of the normal analysis steps is shown in Figure 5.

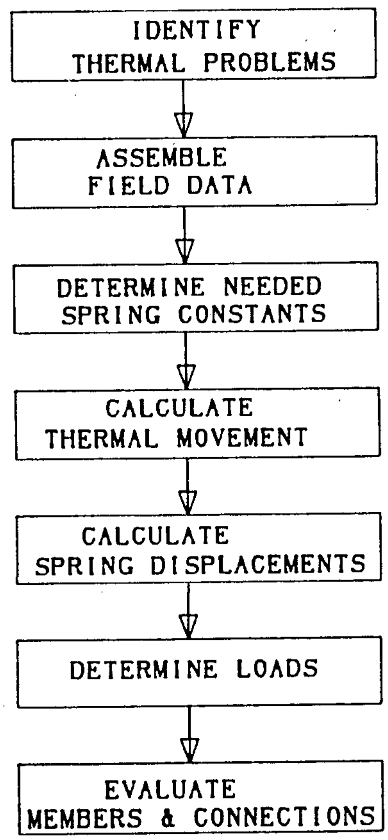
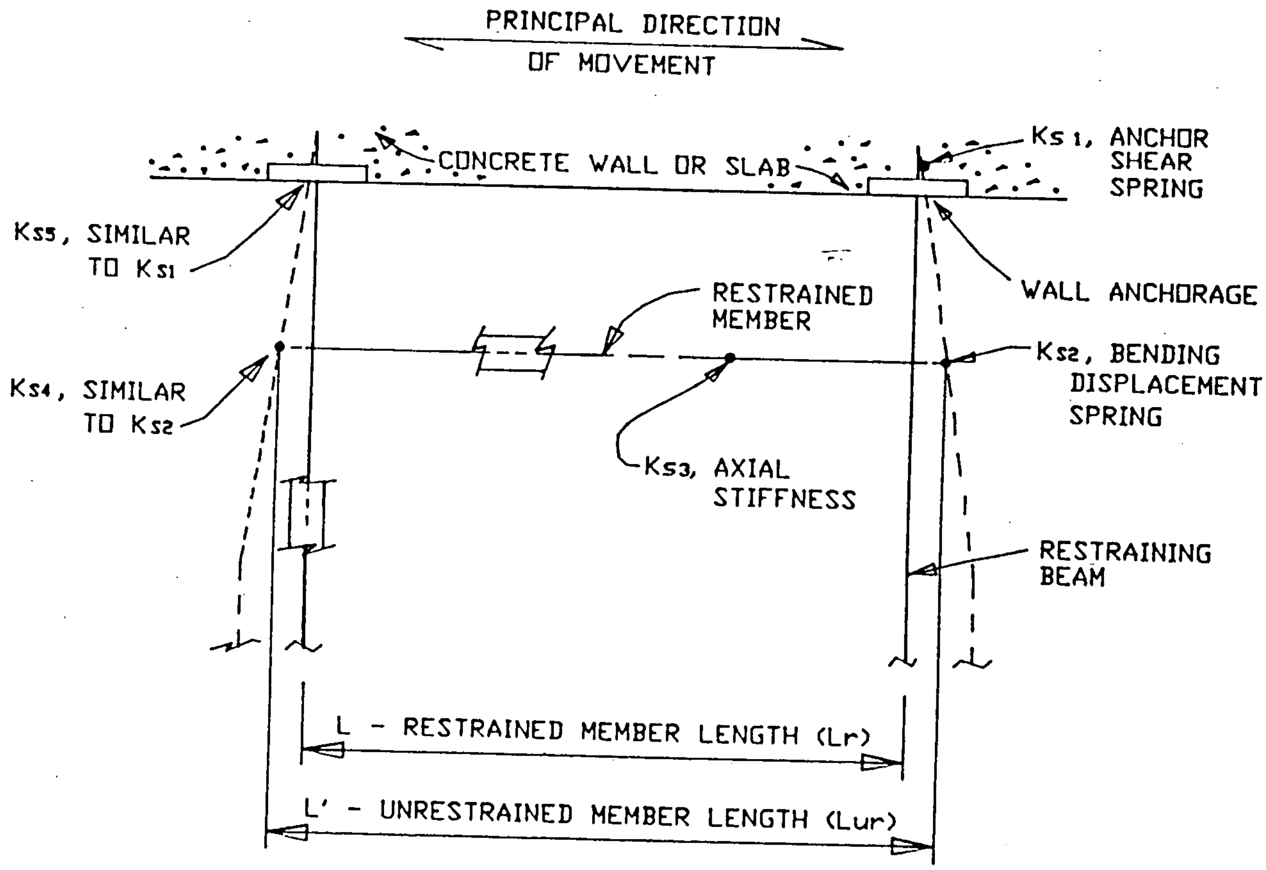


FIGURE 5 - ANALYSIS FLOW CHART

5.1.1 Develop Single Degree of Freedom Model

For simple structures, a single degree of freedom model can be used to evaluate thermal loads. The thermal movement can be calculated, spring constants can be established for each member and connection, and the ductility ratio or capacity of each member can then be assessed.

Often the most critical situations involve only a few members in close proximity to concrete surfaces and only a single direction of thermal movement needs to be analyzed (parallel to the axis of the member being evaluated). See Figure 6 for a typical example of how spring constants can be identified.



K_{s1}, K_{s2}, K_{s3} ARE SPRING CONSTANTS IN THE PRINCIPAL DIRECTION OF GROWTH FOR EACH CONTRIBUTING MEMBER AND CONNECTION. K_{s3} IS THE AXIAL SPRING CONSTANT OF THE RESTRAINED MEMBER. K_{s2} IS THE BENDING SPRING CONSTANT FOR THE RESTRAINING BEAM AT THE ATTACHMENT POINT. K_{s1} IS THE SHEAR SPRING CONSTANT FOR THE CONCRETE WALL ANCHORAGE.

FIGURE 6 - SPRING IDENTIFICATION EXAMPLE

5.1.2 Quantify Thermal Movement

The ambient, operating and accident temperatures (t) must be in accordance with project design criteria and environmental drawings.

The thermal movement of a steel member (L_δ) experiencing a temperature change (t_δ) is calculated as follows:

Equation 4:

$$L_\delta = \varepsilon \times t_\delta \times L$$

See Section 4.3 for determination of ε.

The thermal movement of multiple members is the vector sum of each contributing member in the direction under consideration.

Where connections meet inherent thermal growth provisions (See appendix C) L_δ may be reduced.

5.1.3 Calculate Linear Displacements and Loads

For single degree of freedom models the amount of displacement in each component (L_δ) is equal to:

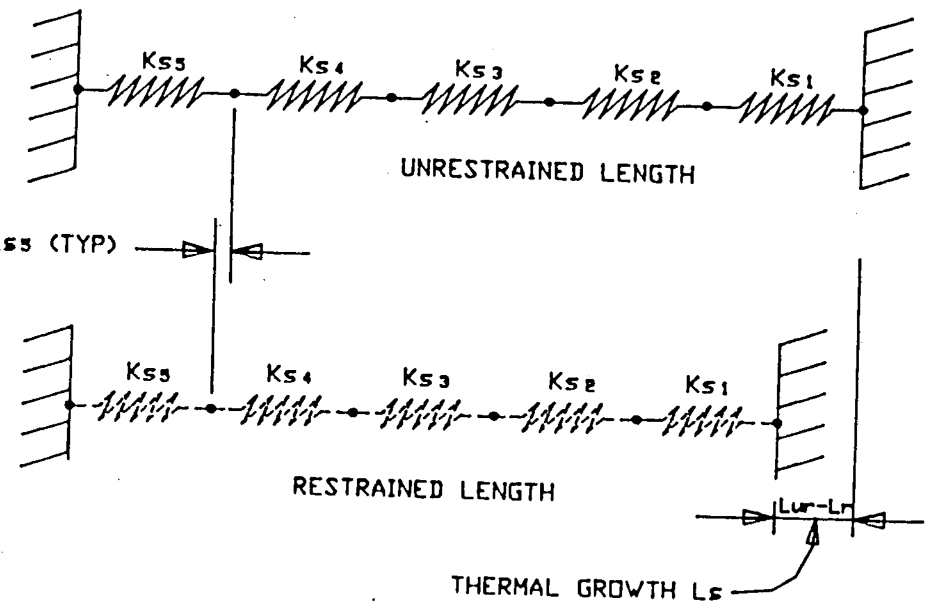
Equation 5: For member or connection (i) of (n) members and connections:(Derivation in Appendix B1.0)

$$L_\delta = L_\delta \,/\, [\, Ks_i \times \sum_{i=1}^{i=n} (1/Ks_i) \,]$$

Methods for combining parallel and orthogonal springs are provided in Appendix B.

Figure 7 provides a graphic representation of this equation for the spring constant example given in Figure 6.

$n = 5$ AND $Ks_4 \leq Ks_2$ AND $Ks_5 \leq Ks_1$



$$Ls_i = Ls \Big/ \left[Ks_i \times \sum_{i=1}^{i=n} \left(1 / Ks_i \right) \right]$$

$$Ls_1 = Ls \Big/ \left[Ks_1 \times \left(1 / Ks_1 + 1 / Ks_2 + 1 / Ks_3 \right) \right]$$

$$Ls_2 = Ls \Big/ \left[Ks_2 \times \left(1 / Ks_1 + 1 / Ks_2 + 1 / Ks_3 \right) \right]$$

$$Ls_3 = Ls \Big/ \left[Ks_3 \times \left(1 / Ks_1 + 1 / Ks_2 + 1 / Ks_3 \right) \right]$$

FIGURE 7 - THERMAL DISPLACEMENT DETERMINATION

5.1.4 Determine Reactions and Acceptability

Once displacements based on linear response are determined, each displacement can be converted to a reaction by multiplying the spring constant by the displacement.

Recommended acceptance criteria for members and connections which are modeled using linear material properties are provided in Appendix C.

5.2 LINEAR ANALYSIS - STRUDL/ANSYS

If a multiple degree of freedom or indeterminate system must be modeled, a computer model incorporating realistic spring constants will allow determination of linear forces and displacements.

5.2.1 STRUDL is an appropriate tool to use only when response is anticipated to be predominantly in the linear range with one or two specific locations in the structure going somewhat over yield. If this behavior cannot be predicted on the basis of hand calculations as described in section 5.1, a linear ANSYS model is recommended. Although ANSYS does not have a code check capability, it can be modified to model non-linear behavior in portions of the structure which go beyond yield.

5.2.2 For members analyzed with STRUDL, acceptance criteria given in Appendix C may be used.

5.3 NON-LINEAR ANALYSIS - ANSYS

The ANSYS computer program has the capability to perform non-linear analysis of thermally restrained structures. There are both advantages and disadvantages to using this capability. Reference 10.1 provides general non-linear steel member behavior information.

Advantages: (1) Non-linear acceptance criteria permit qualification of thermally restrained structures which could not be qualified using linear analysis. (2) The actual member behavior can be predicted beyond the structures yield point. (3) Locations with plastic deformation and corresponding strains and displacements can be determined directly.

Disadvantage: (1) Time required for analysis is greater than would be required for a hand solution or to develop a STRUDL model. (2) ANSYS does not have a built-in capability to perform member checks. (3) A simple manual calculation is needed to determine whether a structure is loaded beyond yield.

The ANSYS 3-D thin wall beam element, STIF24, is normally the most appropriate element to use for thermally restrained beams. This element responds in a linear manner in all directions except along the members axis. The axial deformations can be linear or non-linear depending on the forces in the member.

The members cross-section should be modeled with about 5 segment points in each flange and the web. If the member has some initial curvature, either due to the geometry modeled or due to a lateral load, beam-column behavior will be modeled. The large displacement option must be used to get good results.

5.3.1 Primary and Ancillary Members

The acceptance criteria for members depends on the type of member being evaluated. Inter-connecting members referred to as Ancillary members, such as stub beams actually tend to relieve thermal forces in primary members but may experience large deformations because of their flexibility. Recommended Ancillary Member acceptance criteria is provided in Appendix D.

5.3.2 Loading Sequence

Deformations in structures loaded beyond yield are load path dependent. In other words, the deformation experienced by the structure may differ depending on the sequence and signed direction of load application. The most conservative loading sequence will normally be application of all criteria loads followed by the thermal forces, but other sequences should be considered.

6.0 EVALUATION OF CONNECTIONS

6.1 GENERAL INFORMATION

6.1.1 Flexible Connection Plates

The rotations and bending ductility of the plate elements of connections and baseplates do not need to be evaluated unless that bending is critical to overall structural stability (i.e. a plate supporting a cantilever). Behavior is analogous to a clip angle which, although it may bend inelastically, is acceptable in accordance with Section 1.2 of Reference 10.3.

6.1.2 Inherent Thermal Growth Capacity

Recommended criteria for inherent thermal growth capacity are included in Appendix C.

6.1.3 Punching Shear

It is possible to calculate spring constants for the restraining concrete members and to take credit for their movement. Where this flexibility is insufficient to relieve thermal reactions and the yield loads of all members and connections are high, concrete punching shear capacity of supporting walls must be evaluated.

See Appendix C for recommended load combinations and acceptance criteria.

6.2 CONCRETE ANCHORAGE

6.2.1 Under shear loading, attachments to concrete using embedded (or grouted) bolts, wedge bolt anchors, self drilling anchors, and Nelson studs (or equal), all respond in a ductile manner under thermal shear loading. The test results included in References 10.8 and 10.9 demonstrate that, for all of these anchors, the bolt fails prior to concrete failure as long as the bolt is in confined concrete. Reference 10.5 provides edge distance requirements which insure proper concrete confinement.

6.2.2 Recommended acceptance criteria for concrete anchorage is provided in Appendix C. The following information may be used to conservatively establish the maximum reaction that could exist at an anchor.

For embedded bolts and expansion anchors, the shear yield load (Py) is based on the ultimate tensile capacity of the bolt material (Fu) and the bolt tensile stress area (Ab)*. It may be calculated as follows:

Equation 6:

$$Py = Ab \times Fu$$

(*) See Reference 10.3, Fastener Data section of Part 4, onnections for definition of tensile stress area

Equation 6 is based on the test results in Reference 10.8 and 10.9. Anchor bolts loaded in shear plastically rotate just below the concrete surface, spalling some concrete, and then elongate and ultimately fail primarily in tension.

The above capacity may be used to reduce the reaction at an anchor for evaluations of the restrained member or other connections. This reaction may be over-estimated since spring constants for concrete anchors are conservatively established. See Example Problem A2 in Appendix A.

6.3 BOLTED CONNECTIONS

6.3.1 Thermal Modifications

The effects of thermal movements in steel structures are best minimized by modifying connections to allow axial movement. When slotted connections are added to an existing structure, the constructability of the connections is a prime consideration. Often the existing member must be supported by adjacent structural members temporarily in order to perform the modification. For large members this can be a significant problem that must be considered in designing the modification.

Slotted connections installed to relieve thermal loadings must be detailed such that preloaded bolts cannot be installed. Friction bolts are unacceptable for connections modified to prevent thermal stress although methods are provided for evaluation of existing slotted connections with friction bolts.

Where slotted connections are added to existing structures the seismic response of the structure must be reviewed to ensure that the seismic design is consistent with the modified structure.

6.3.2 Allowable Bolt Stress and Slip Resistance

Recommended criteria for upper and lower bound bolt strength are provided in Appendix C.

6.4 WELDED CONNECTIONS

The recommended criteria for allowable weld stress is provided in Appendix C. The maximum allowable stress may not be increased by using load combination stress increase factors.

Base metal stress at the weld to base metal interface does not need to be checked unless the weld electrode is more than two strength categories greater than the matching electrode for the base metal. Tests have demonstrated that the fusion zone is not critical in determining the shear strength of fillet welds. (See References 10.10 and 10.11)

7.0 ACCEPTANCE BY COMPARISON TO PREVIOUS EVALUATIONS

Structures which are evaluated after the initial worst case evaluation is completed must be compared to previous controlling worst cases to demonstrate acceptance.

Where previous selection screening procedures were explicit enough to be reproduced, comparisons to the controlling cases can be made using the same procedure that was originally used for selection.

If previous screening procedures cannot be reproduced reliably, a structure that is geometrically similar to a previously evaluated structure can be accepted by computing the interaction ratio given below. An interaction value of unity or less demonstrates acceptability.

The basis for geometric similarity must be clearly documented in the calculation. If no similar structures are identified, a rigorous analysis in accordance with section 5.0 must be performed.

An interaction check should be made for each of the most critical members and connections which are classified as thermally restrained structures.

$$IR/IR_o \times dT/dT_o \times Ks/Ks_o \times L/L_o <= 1.0$$

where:

IR - Interaction ratio, ratio of the actual stress level to allowable stress level, for the extreme non-thermal load case for the member or connection being evaluated. This ratio must be based on previous analysis.

IR_o - Interaction ratio for the extreme load case for the corresponding member or connection previously evaluated.

dT - Design temperature change for structure to be evaluated.

dT_O - Design temperature change for structure previously evaluated.

Ks - Summation of spring constants between the controlling thermal restraint boundary points for the structure being evaluated.

Ks_O - Summation of spring constants between the controlling thermal restraint boundary points for the structure evaluated previously.

L - Straight line distance between the controlling thermal restraint boundary points for structure being evaluated.

L_O - Straight line distance between the controlling thermal restraint boundary points for the structure evaluated previously.

8.0 EXAMPLE PROBLEMS

Two simple structural frames have been selected as example problems to illustrate the concepts presented in this guide. These example calculations are included as figures A1 and A2.

9.0 TECHNICAL JUSTIFICATIONS

The equations and recommended values found in this guide are justified in Reference 10.16.

10.0 REFERENCES

10.1 ASCE--Manuals and Reports on Engineering Practice--No. 41, "Plastic Design In Steel, A Guide And Commentary." New York: American Society of Civil Engineers, 1971.

10.2 ACI 318, "Building Code Requirements For Reinforced Concrete." Detroit: American Concrete Institute, Code of Record Date.

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10.4 NUREG-0800, U.S.Nuclear Regulatory Commission, Standard Review Plan, Rev. 1, July 1981.

10.5 Civil Design Standard DS-C1.7.1, General Anchorage to Concrete

10.6 TVA Calculation SCG-CSG-87-193. [RIMS B04 89 0505 200]

10.7 Howland, F. L. and Newmark, N.M. Static Load Deflection Tests of Beam-Columns University of Illinois Civil Engineering Studies, Structural Research Service, No. 65, Dec. 1953.

10.8 TVA Concrete Anchorage Tests [RIMS B41880930001]

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10.11 F. R. Preece, "AWS-AISC Fillet Weld Study --Longitudinal and Transverse Shear Tests", Testing Engineers, Inc. Oakland, CA, May 31, 1968.

10.12 Springfield, Design of Steel Columns Subject to Biaxial Bending, Engineering Journal of the American Institute of Steel Construction, 3rd quarter, 1975, page 73.

10.13 "Guide to Design Criteria for Bolted and Riveted Joints" by G.L. Kulak, T.W.Fisher, and H.A.Sturic (Second edition, Wiley 1987)

10.14 Calculation for Thermal Effects on Concrete Anchors, [RIMS B04 890505 200], CD-Q0303-890897

10.15 "Roark's Formulas for Stress and Strain, Warren C. Young, 6th edition, McGraw-Hill, New York, 1989.

10.16 "Technical Justifications for Thermal Calculation Procedures", CSG-91-001

10.17 TVA Design Standard, "Temperature and Shrinkage Reinforcement", DS C1.5.4

10.18 "Handbook of Concrete Engineering", Edited by Mark Fintel, 2nd Edition, Van Nostrand Reinhold, New York, 1985.

A1.0 EXAMPLE PROBLEM 1

Referring to figure A1, calculate the unrestrained thermal travel in the horizontal direction and check member adequacy:

$\varepsilon = (6.1 + 0.0019 \times 270) \times 10^{-6}$ (Ambient 70° and t = 270°)

$= 0.0000066\ 1/F^{\circ}$

$L\delta = 22 \times 12 \times 0.0000066 \times 200 = 0.348$ in

Deduct inherent thermal growth capacity: $L\delta = 0.348 - 0.0312 = 0.317$ in.

W10x19 properties:

$I(x\text{-}x) = 96.3\ in^4$ $\qquad I(y\text{-}y) = 4.28\ in^4$

$S(x\text{-}x) = 18.8\ in^3$ $\qquad S(y\text{-}y) = 2.13\ in^3$

$A = 5.62\ in^2$

Calculate spring constant at attachment point:

Displacement $= \delta = P\,L^3 / 48EI \Rightarrow P = \delta\, 48EI / L^3$

Calculate the force for a unit displacement of 1.0 inch:

$Ks2 = 1 \times 48 \times 28000 \times 4.28 / (14 \times 12)^3$

$= 1.213$ K/in

Determine other spring constants:

$Ks1 = AE/L = 5.62 \times 28000 / (22 \times 12) = 596.1$ K/in

$Ks3 = Ks4 = Ks5 = 1000 \times 4 = 4000$ K/in
(Reference: Appendix A, DS-C1.7.1)

Combine the effects of springs 3 and 4 (Refer to Appendix B3.0 for the development of the equation used below):

$L = 14 \times 12 = 168$ in

$a = 7 \times 12 = 84$ in

$$Ks\ 34 = \frac{L^2}{[\ (L-a)^2/Ks3 + a^2/Ks4\ ]}$$

$$= \frac{168^2}{[\ (168-84)^2/4000 + 84^2/4000\ ]}$$

$$Ks34 = 8000\ K/in$$

Note: The use of the Appendix B equation was done to illustrate the equations use, obviously adding springs Ks3 and Ks4 would have been simpler for the geometry given for this example.

Calculate the displacement of each spring using equation 5:

$$L\delta 1 = L\delta/[Ks1 \times (1/Ks1 + 1/Ks2 + 1/Ks34 + 1/Ks5)]$$

$$= 0.317\ /\ [596.1 \times (1/596.1 + 1/1.213 + 1/8000 + 1/4000)]$$

$$= 0.000643\ in$$

$$L\delta 2 = 0.317\ /\ (1.213 \times 0.8264) = 0.3160\ in$$

$$L\delta 34 = 0.317\ /\ (8000 \times 0.8264) = 0.0000429\ in$$

$$L\delta 5 = 0.317\ /\ (4000 \times 0.8264) = 0.0000958\ in$$

Check the capacity of member 2:

Moment => $P = 0.3160 \times 1.213 = 0.383\ K$

$M = PL/4 = 0.383 \times 168\ /\ 4 = 16.1\ K\text{-}in$

$fs = 16.1\ /\ 2.13 = 7.56\ K/in^2$

Since 7.56 ksi is < 36 ksi - OK

Therefore member 2 is responding in the elastic range and is acceptable as long as it meets all applicable steel design requirements in accordance with project criteria.

Member 1 is OK by inspection.

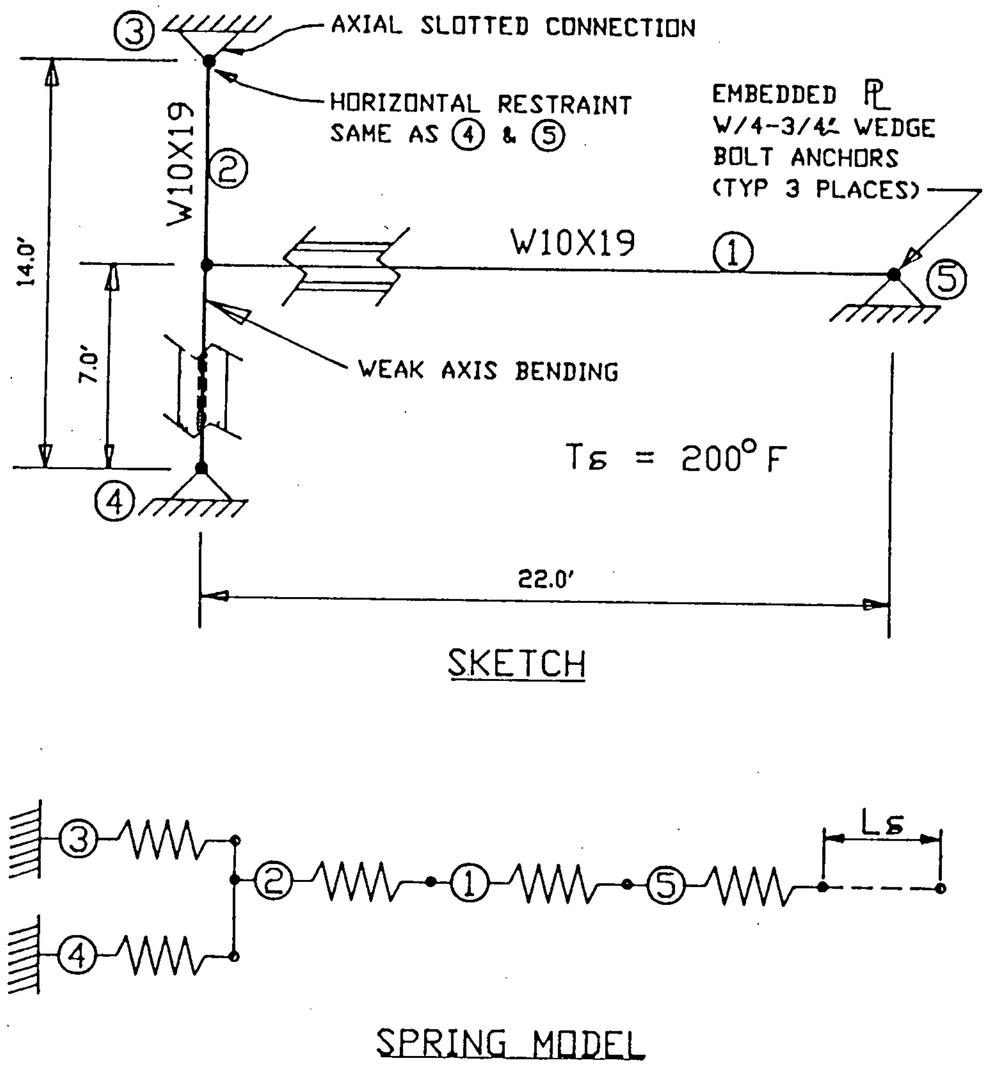


FIGURE A1 - EXAMPLE PROBLEM 1

A2.0 EXAMPLE PROBLEM 2

Evaluate member 3 for thermal overload (See Figure A2).

Calculate member length for 3:

$$L = \sqrt{(10^2 + 8^2)} = 12.81 \text{ ft}$$

$$= 153.7 \text{ in}$$

Tensile stress area of 3/4 in anchor: $Ab = 0.334 \text{ in}^2$

Properties W8x35: $A = 10.3 \text{ in}^2$

$r_{xx} = 3.51$ in

$r_{yy} = 2.03$ in

Calculate the spring constants for Ks1, Ks2, Ks3, Ks4, and Ks5:

Ks2 = Ks4 = No. bolts x Ks = 6 x 1000 = 6000 K/in

Ks3 = A E / L = 10.3 x 28000 / 153.7 = 1876 K/in

Ks1, Ks5 => RIGID

Combine springs Ks1 and Ks2 (also Ks4 and Ks5) to determine the spring constant of an equivalent spring on the axis of member 3.

Referring to the equation derivation in Appendix B4.0 calculate the angles between the bearing surface and the load at each end of member 3.

$$\phi_L = \text{Arc Sin } (96/153.7) = 38.6^\circ$$

$$\phi_R = \text{Arc Sin } (120/153.7) = 51.3^\circ$$

Calculate the equivalent springs at each end of member 3.

Ks_L = 1 / ($\cos^2$ 38.6/6000)

= 15563 K/in

Ks_R = 1 / ($\cos^2$ 51.3/6000)

= 22982 K/in

Calculate total displacement for member 3:

ε = 0.0000066 1/F° (Reference example problem 1)

$L\delta$ = 153.7 x 0.0000066 x 200 - 0.0312*2 = 0.1405 in

Solve for each spring displacement:

$L\delta_L$ = 0.1405 / [15563 x (1/15563 + 1/1876 + 1/22982)]

= 0.0141 in

$L\delta_3$ = 0.1405 / [1876 x 0.0006408]

= .117 in

$L\delta_R$ = 0.1405 / [22982 x 0.0006408]

= 0.00954 in

The reaction in each of the three springs: .117 x 1876 = 219.5 K

CHECK CONCRETE ANCHOR:

Unrestrained growth per anchor = 0.1405/2 = 0.0702"

0.0702" < 0.2 * 0.75 = 0.15 OK

DETERMINE MAXIMUM POSSIBLE ANCHOR REACTION:

The controlling concrete anchor for shear loading is spring 2.

Load per bolt = 219.5/6 = 36.6 K

Shear load per bolt = 36.6 x cos 38.6 = 28.6 K

Shear yield load per bolt = 0.334 sqin x 58 ksi* = 19.37 K

Maximum possible load prior to yield = 19.4 K

CHECK STEEL MEMBER:

Yield load : 36 x 10.3 = 370.8 K

$\phi = (K\ L\ /\ \pi\ r\ ) \times \sqrt{(Fy/E)}$

$= (1 \times 157.3\ /[\pi \times 3.51]) \times \sqrt{(36/28000)}$

$= 0.511$

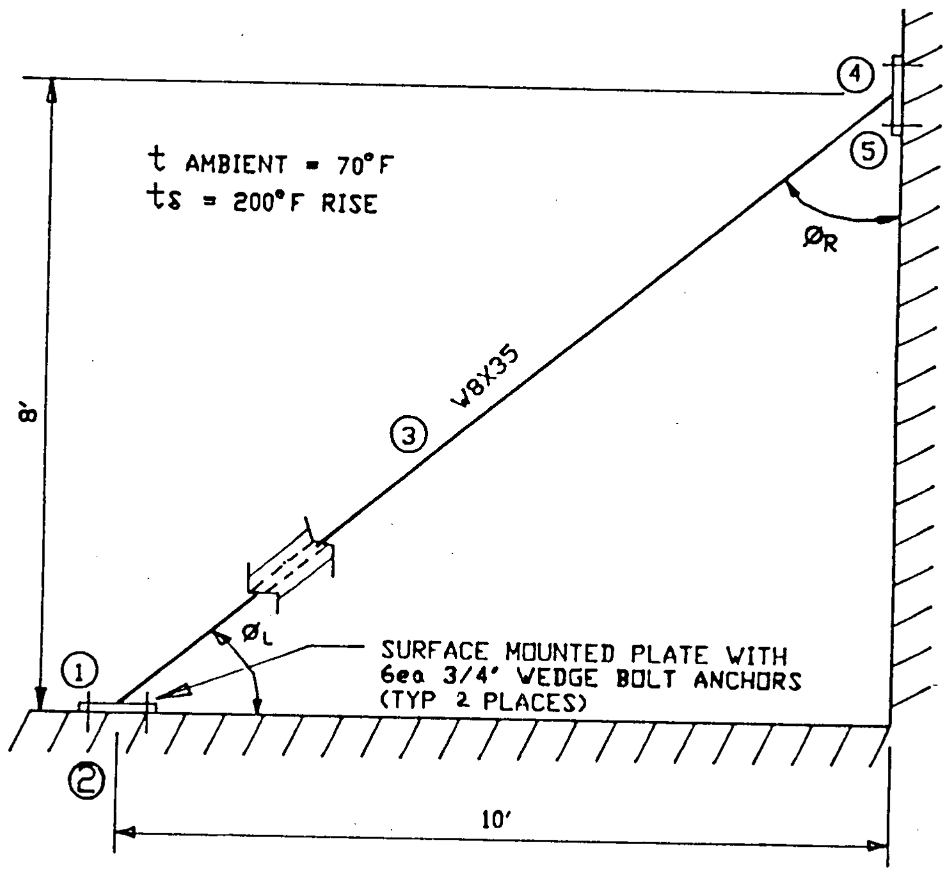
$Pu = (1.0 - 0.511^2/4) * 10.3 * 36 = 346.6$ K (See C2.3.2)

Calculate actual member load:

Since the concrete anchor is loaded beyond its shear yield point the shear reaction at spring Ks2 exceeds the actual reaction in the anchor. The actual spring reaction is 19.4 since the anchor deforms plastically at that load.

Ptot = 6 x 19.4 / cos 38.6 = 148.9 K < 346.6 K - OK

(*) A36 wedge bolt material, shear yield is based on bolt ultimate tensile capacity.



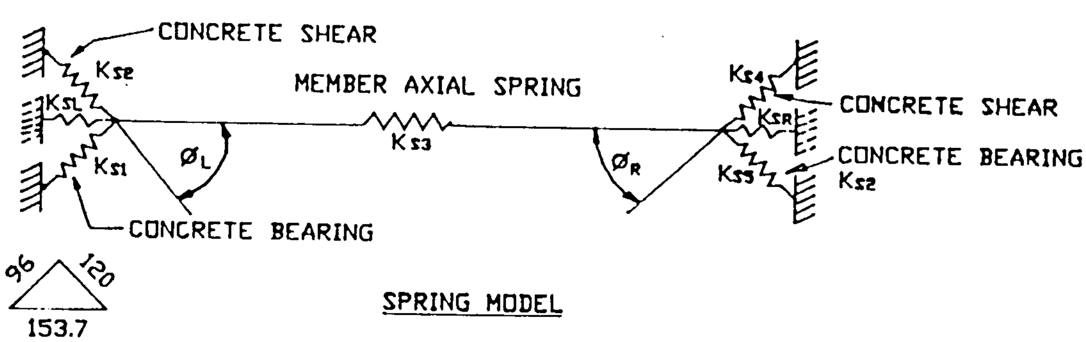


FIGURE A2 - EXAMPLE PROBLEM 2

APPENDIX B - EQUATION DERIVATIONS

B1.0 DERIVATION OF EQUATION 5

The following derivation refers to figure 7. The derivation is for the special case for 5 linear springs. The basis of the general equation in the guide is apparent on inspection of the 5 spring derivation.

Since the force in each of the five springs is equal by inspection:

$$L_{\delta 1} \times K_{s1} = L_{\delta 2} \times K_{s2} = L_{\delta 3} \times K_{s3} \qquad (a)$$

$$= L_{\delta 4} \times K_{s4} = L_{\delta 5} \times K_{s5} \qquad (b)$$

Therefore:

$$L_{\delta 2} = (L_{\delta 1} \times K_{s1})/K_{s2} \qquad (c)$$

$$L_{\delta 3} = (L_{\delta 1} \times K_{s1})/K_{s3} \qquad (d)$$

$$L_{\delta 4} = (L_{\delta 1} \times K_{s1})/K_{s4} \qquad (e)$$

$$L_{\delta 5} = (L_{\delta 1} \times K_{s1})/K_{s5} \qquad (f)$$

The following relation is apparent by inspection of figure 3

$$L_{\delta} = L_{\delta 1} + L_{\delta 2} + L_{\delta 3} + L_{\delta 4} + L_{\delta 5} \qquad (g)$$

Substituting (c) through (f) into (g):

$$L_{\delta} = L_{\delta 1} + (L_{\delta 1} \times K_{s1})/K_{s2} + (L_{\delta 1} \times K_{s1})/K_{s3} + (L_{\delta 1} \times K_{s1})/K_{s4} + (L_{\delta 1} \times K_{s1})/K_{s5} \qquad (h)$$

Combining terms:

$$L_{\delta} = L_{\delta 1} \times [\ K_{s1} \times (1/K_{s1}+1/K_{s2}+1/K_{s3}+1/K_{s4}+1/K_{s5})\ ] \qquad (i)$$

By inspection and rearranging terms the following equation is verified:

$$L_{\delta i} = L_{\delta} / [K_{si} \times \sum_{i=1}^{i=n} (1/K_{si})]$$

where n = 5 or the number of springs being evaluated in the general case, and i is the number of the spring being evaluated.

B2.0 DERIVATION OF ENERGY-BALANCE EQUATION

The following derivation is with reference to figure B1.

Equal energy concepts imply that the potential energy stored by the elastic system at maximum deflection (at point D, figure B1) must be the same as that stored by the elastoplastic system at maximum deflection (at point G, figure B1). Stated another way area OCD must equal area OEFG.

Thus:

$$OA \times OD / 2 = (OB \times \delta_y)/2 + [\delta_r - \delta_y] \times OB \qquad (a)$$

Since OB = Py and OA = Pr

$$(Pr \times OD)/2 = (Py \times \delta_y)/2 + [\delta_r - \delta_y] \times Py \qquad (b)$$

By similar triangles OE δ_y and OCD

$$OD = OA \times \delta_y/OB = Pr \times \delta_y/ Py \qquad (c)$$

Substituting OD from (c) into (b), combining and solving for δ_r/δ_y which is equal to the ductility ratio:

$$\mu = \delta_r/\delta_y = 1/2\ [(Pr^2/Py^2) + 1]$$

where, referring to Figure B1,

δ_r = deflection at load being evaluated

δ_y = deflection at load causing yield

Pr = maximum load assuming elastic response of the member or structure

Py = load on member or structure at yield

A more detailed derivation is found in TVA calculation SCG-CSG-87-193 (Reference 10.6).

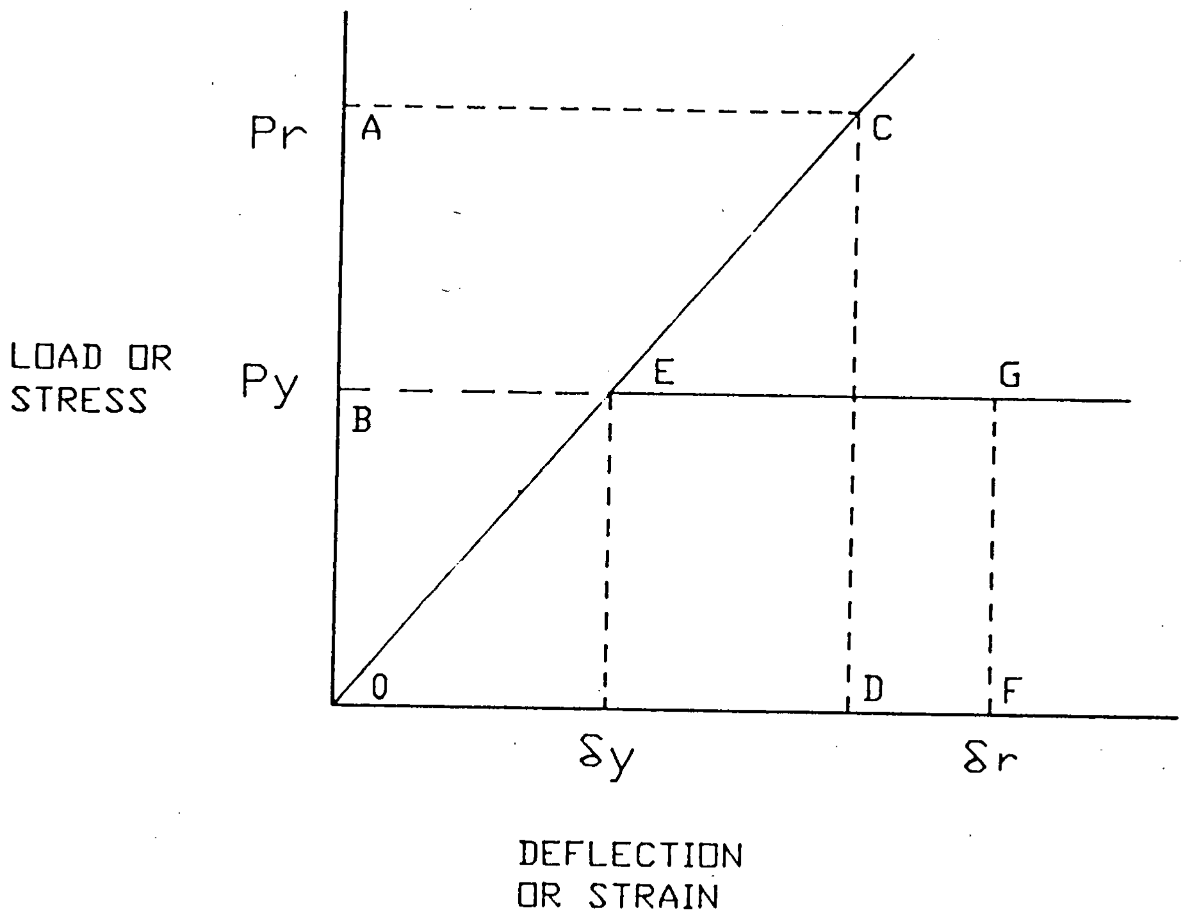


FIGURE B1 - ILLUSTRATION OF CONCEPTS USED TO DERIVE DUCTILITY RATIO

B3.0 DERIVATION OF EQUATION TO COMBINE PARALLEL SPRINGS

Referring to figure B2, the spring 1 and 2 reactions are equal to:

$$R1 = P(L - a) / L \quad \text{and} \quad R2 = P\,a / L$$

The displacements at springs 1 and 2 are:

$$L\delta 1 = R1 / Ks1 = P\,(L - a) / (L\,Ks1)$$

$$L\delta 2 = R2 / Ks2 = P\,a / (L\,Ks2)$$

The displacement at the end of the attachment member is:

$$L\delta_{member} = L\delta 1 + a/L\,(L\delta 2 - L\delta 1)$$

$$= (L\,L\delta 1 + a\,L\delta 2 - a\,L\delta 1) / L$$

$$= [\,(L-a)\,L\delta 1 + a\,L\delta 2\,] / L$$

$$= \frac{(L-a)\,[P(L-a)/(L\,Ks1)] + a\,[P\,a/(L\,Ks2)]}{L}$$

$$= \frac{P}{L^2} \times [\,(L-a)^2/Ks1 + a^2/Ks2\,]$$

Therefore an equivalent spring for the member can be computed as follows:

$$P = L\delta_{member} \times K_{eqiv}$$

$$K_{eqiv} = P / L\delta_{member}$$

$$Ks\ equiv = \frac{P}{\frac{P}{L^2} \times [\,(L-a)^2/Ks1 + a^2/Ks2\,]}$$

$$Ks\ equiv = \frac{L^2}{[\,(L-a)^2/Ks1 + a^2/Ks2\,]}$$

Where Ks2 is a rigid spring the equation simplifies to:

$$Ks\ equiv = \frac{L^2}{[\ (L - a)^2 / Ks1\ ]}$$

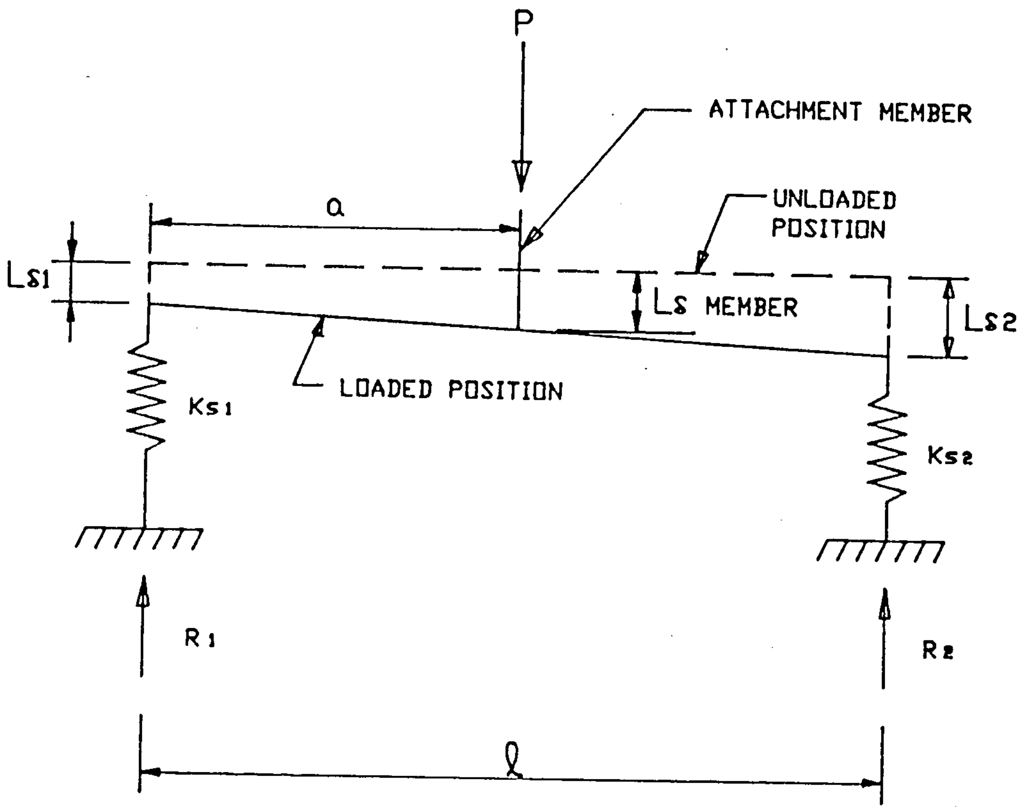


FIGURE B2 - Combining Parallel Springs

B4.0 DERIVATION OF EQUATIONS TO COMBINE ORTHOGONAL SPRINGS

Referring to figure B3, ϕ is the acute angle between the bearing surface and the applied load axis.

$$\delta 1 = P \times \sin\phi / Ks1$$

$$\delta 2 = P \times \cos\phi / Ks2$$

$$\delta = \delta 1 \times \sin\phi + \delta 2 \times \cos\phi$$

$$Ks\ equiv = P / \delta$$

$$= P / [(P \times \sin^2\phi / Ks1) + (P \times \cos^2\phi / Ks2)]$$

$$= 1 / [\ (\sin^2\phi / Ks1) + (\cos^2\phi / Ks2)\ ]$$

Where Ks1 is a rigid spring the equation simplifies to:

$$Ks\ equiv = 1 / (\ \cos^2\phi / Ks2)$$

Where Ks2 is a rigid spring the equation simplifies to:

$$Ks\ equiv = 1 / (\ \sin^2\phi / Ks1)$$

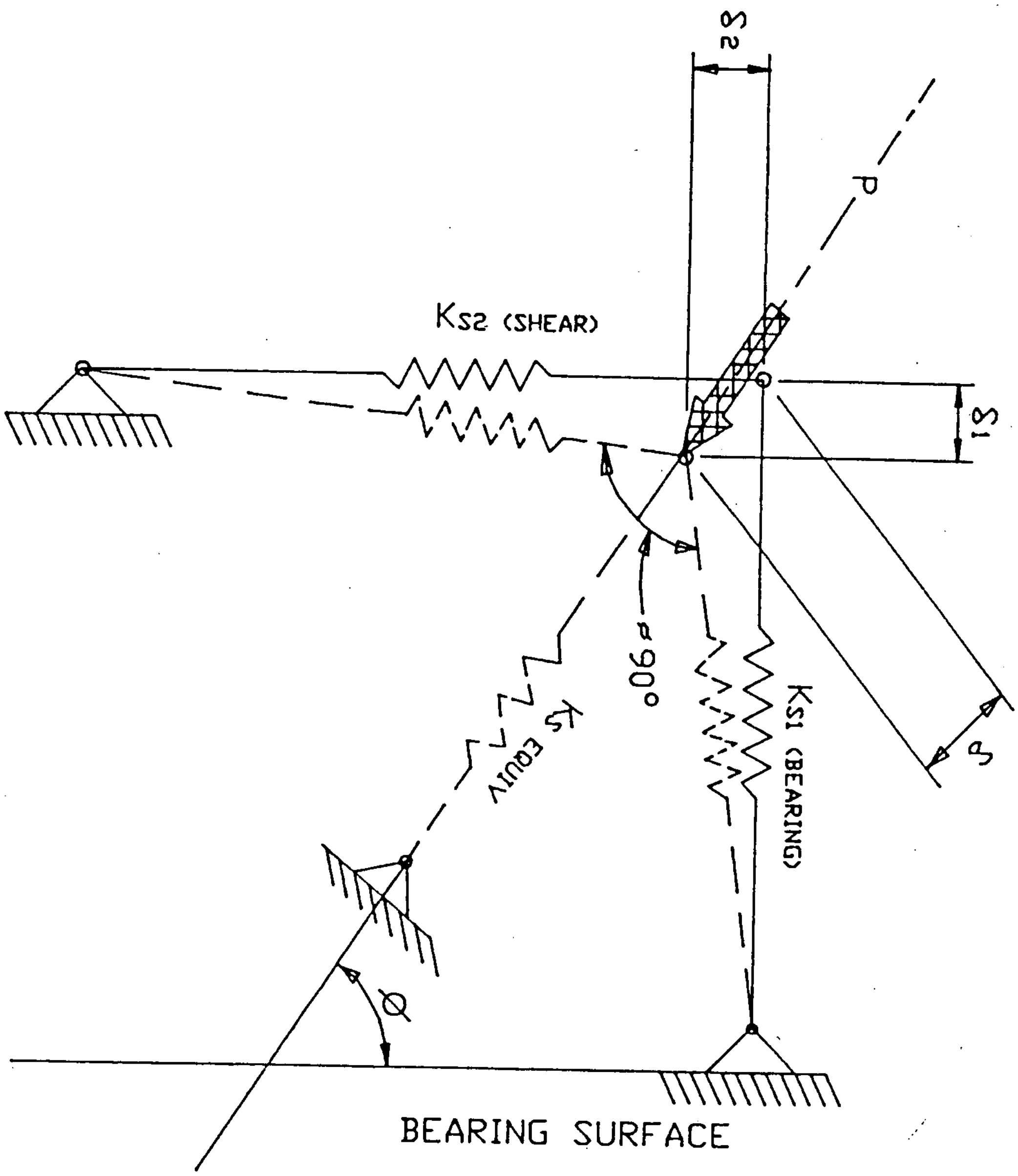


FIGURE B3 - Combining Orthogonal Springs

C1.0 GENERAL

This appendix provides recommendations for the acceptance criteria for thermal analyses for inclusion in project design criteria. This criteria is to be applied to all members modeled assuming linear behavior. Acceptance is generally based on the energy balance method for shear and tension and AISC for compression and combined bending.

C1.1 LOAD COMBINATIONS

Final acceptance calculations for evaluations of thermal behavior must include all loads in combination as specified in applicable project design criteria unless they are negligible in comparison to thermal forces. Compliance with the recommended criteria given below for the worst case structures constitutes demonstration of ductile and self-limiting behavior for the entire population of thermally restrained structures.

Allowable stresses and loads for unfactored (i.e. normal operating) load cases shall be in accordance with project design criteria.

Allowable stresses in this Appendix apply to the factored load combinations which include the Ta load term (i.e. SSE or DBE in combinations with Ta). The stress and force requirements recommended here are ultimate capacities and are not to be factored upward in accordance with project criteria.

C2.0 MEMBERS

C2.1 Minimum Width-Thickness Ratio Requirements

C2.1.2 It is assumed that plastic hinges may form at locations where the combined axial and bending stresses sum to greater than Fy. Plastic design width to thickness ratios (C2.1.4 and C2.1.5) and lateral bracing requirements (C2.2) apply at these locations.

C2.1.3 The width-thickness ratio for flanges of rolled C, W, M, or S shapes and similar built-up single-web shapes that are subjected to compression but do not involve plastic hinge rotation under ultimate loading shall not exceed the following values:

$b/t \leq 95/\sqrt{Fy}$ for unstiffened plate elements

$b/t \leq 253/\sqrt{Fy}$ for stiffened plate elements

where: b - element length
t - element thickness

C2.1.4 If the same members are subjected to compression involving stresses above yield at ultimate loading, width-thickness shall not exceed the following values:

Fy	b/t
36	8.5
42	8.0
45	7.4
50	7.0
55	6.6
60	6.3
65	6.0

The thickness of sloping flanges may be taken as their average thickness.

The width-thickness ratio of similarly compressed flange plates in box sections and cover plates shall not exceed $190/\sqrt{Fy}$. For this purpose, the width of a cover plate shall be taken as the distance between longitudinal lines of connecting high-strength bolts or welds.

C2.1.5 The depth-thickness ratio of webs of members subjected to stresses above yield shall not exceed the value given by Equation (C1A or C1B), as applicable.

Equation C1A:

$$d/t = (412/\sqrt{Fy})\ (1 - 1.4 \times P/Py)\ \text{when}\ \ P/Py \le 0.27$$

Equation C1B:

$$d/t = 257/\sqrt{Fy} \quad \text{when} \quad P/Py > 0.27$$

C2.2 Lateral Bracing Requirements

Members shall be adequately braced to resist lateral and torsional displacements at locations where stresses are beyond yield. If braces are not provided locally at the yielded region, the laterally unsupported distance, lcr, between braced locations on the member or frame shall not exceed the value determined from Equation (C2A) or (C2B), as applicable.

Equation C2A:

$$lcr/ry = 1375/Fy + 25$$

$$\text{when} \quad +1.0 > M/Mp > -0.5$$

Equation C2B:

$$lcr/ry = 1375/Fy$$

when $- 0.5 > M/Mp > - 1.0$

where

ry = radius of gyration of the member about its weak axis, inches

M = lesser of the moments at the ends of the unbraced segment, kip-feet

M/Mp = end moment ratio, positive when the segment is bent in reverse curvature and negative when bent in single curvature

If stresses through out the member are below yield, the maximum distance between points of lateral support shall satisfy the requirements of Sections 1.5.1.4 of the AISC Specification (Reference 10.3).

C2.3 Load Capacity and Ductility Determination

The following acceptance criteria are recommended for thermally restrained structures which have been analyzed using linear analysis methods:

C2.3.1 Shear and tension in all members shall be limited by the energy balance equation ductility as follows:

$\mu_{eb} \leq 1.5$

where:

if ($Pr \leq Py$) then:

$\mu_{eb} = Pr/Py$

Note: The above is not actually a ductility ratio since it is less than 1.

if ($Pr > Py$) then:

$\mu_{eb} = 1/2 * (Pr^2/Py^2 + 1)$

Note: Py shall be shear yield based on $Fy/\sqrt{3}$ for shear evaluations.

C2.3.2 For compression members, the allowable ultimate compression force, Pu, shall be determined as follows:

$\phi = Kl/(\pi r) * \sqrt{(Fy/E)}$

If ($\phi \leq 0.15$) then:

$Pu = \sqrt{2} * Ag * Fy$ (Based on Energy balance equation set equal to 1.5 and solving for Pr)

If ($0.15 < \phi \leq 0.40$) then:

$$Pu = 1.6 * (1 - \phi) * Ag * Fy$$

(Based on data that shows that columns with Kl/r ratios < 40 will yield before buckling, $\phi = 0.40$ is equivalent to Kl/r = 36. This is selected to be conservatively less than 40.)

If ($0.40 < \phi \leq \sqrt{2}$) then:

$$Pu = [\ 1 - \phi^2/4\ ] * Ag * Fy$$

(Based on AISC equation 1.5-1 with safety factor of 1)

If ($\sqrt{2} < \phi \leq 2$) then:

$$Pu = Fy * Ag / \phi^2$$

(Based on AISC equation 1.5-2 with factor of safety of 1)

C2.3.3 Combined bending and axial load capacity should be based on the allowable capacity of the AISC specification 1.6.1 using an allowable compression stress based on the above ultimate axial loads and an allowable bending stress of 1.7 times the AISC allowable bending stress (not to exceed the plastic moment capacity of the section).

Alternately acceptance can be based on the following equations from Reference 10.12:

At brace points:

$$(Mx/M_{pcx})^{\pounds} + (My/M_{pcy})^{\pounds} \leq 1.0$$

Between brace points:

$(CmxMx/M_{ucx})^{\eta} + (CmyMy/M_{ucy})^{\eta} \leq 1.0$

where:

$Pyld = Py * \sqrt{2}$ at brace points

$£ = 1.6 - (P/Pyld) / [2 * \ln(P/Pyld)]$

if bf/d is greater than or equal to 0.3:

$\eta = 0.4 + P/Pyld + bf/d \geq 1.0$

otherwise: $\eta = 1.0$

At brace point:

$M_{ucx} = M_{pcx} = 1.18 * M_{px} (1 - (P/Pyld)$

$M_{ucy} = M_{pcy} = 1.19 * M_{py} (1 - (P/Pyld)^2$

Between brace points:

$M_{ucy} = M_{uy} [1 - (P/Pu)] \;\; [1- (P/P_{ey})]$

$M_{ucx} = M_{ux} [1 - (P/Pu)] \;\; [1- (P/P_{ex})]$

where:

$M_{ux} = 1.7*Fbx*Sx \leq M_{px}$

$M_{ux} = 1.7*Fby*Sy \leq M_{py}$

$P_{ex} = \pi^2 E/[Klx/rx]^2$

$P_{ey} = \pi^2 E/[Kly/ry]^2$

C2.4 Definition of Terms

The following data values are needed for the above evaluations for equations in Section C2.3:

Ag = Gross area of member, in^2

bf = flange width of C, W, M or S section, in

Cmx = Equivalent moment factors about the x-axis used in the AISC specification formula (Section 1.6.1)

Cmy = Equivalent moment factors about the y-axis used in the AISC specification formula (Section 1.6.1)

d = Depth of C, W, M or S section, in

E = Modulus of elasticity, $kips/in^2$

Fbx, y = Allowable bending stress in accordance with AISC Specification, section 1.5.1.4, $kips/in^2$

Fy = Specified yield stress, kip/in^2

K = Effective length factor

l = Unbraced length of member, in

lx = Unbraced length about x-axis, in

ly = Unbraced length about y-axis, in

Mx = Moment applied about the x-axis, kip-in

M_{px} = Plastic moment capacity about the x-axis, Fy*Zx, kip-in

M_{py} = Plastic moment capacity about the y-axis, Fx*Zy, kip-in

My = Moment applied about the y-axis, kip-in

P = Applied axial load including thermal restraint force, kips. Shall not exceed Pu, Pex, Pey, or Pyld.

Pr = Linear response force, kips

Pu = Ultimate allowable axial strength, kips

Py = Compressive yield strength, Ag x Fy, kips

r = Radius of gyration about plane of buckling, in

rx = Radius of gyration about x-axis plane of buckling, in

ry = Radius of gyration about y-axis plane of buckling, in

Sx = Section modulus about the x-axis, in^3

Zy = Plastic section modulus about y-axis, in^3

C2.5 Alternate Acceptance Criteria

Alternately member acceptance can be based on AISC allowable stresses multiplied by 1.7. No reduction for upper bound stress criteria (i.e. 0.9 x Fy) is required.

C3.0 ACCEPTANCE OF CONCRETE ANCHOR DISPLACEMENTS

Concrete anchors must meet the following requirements or alternately they must meet load capacity requirements in Reference 10.5.

Concrete self drilling anchors may be considered acceptable for thermal shear loading if unrestrained thermal growth at the anchor is less than or equal to 0.1 times the nominal anchor diameter.

Concrete anchors other than self drilling anchors may be considered acceptable for thermal shear loading if unrestrained thermal growth at the anchor is less than or equal to 0.2 times the nominal anchor diameter.

Under tensile loading, ductile concrete anchors (as defined in Reference 10.5) must meet ductility ratio requirements for tension members (see C2.3) and the concrete pullout capacity of the anchor must exceed the ultimate tensile capacity of the bolt. Anchors not meeting the above concrete pullout capacity requirements shall not be qualified for thermal loads in excess of the capacity calculated using Reference 10.5.

Under tensile loading, non ductile concrete anchors (as defined in Reference 10.5) shall not be qualified for thermal loads in excess of the capacity calculated using Reference 10.5.

C4.0 ACCEPTANCE CRITERIA FOR WELD CAPACITY

Welds must be qualified based on an allowable stress level of two thirds of the nominal tensile strength of the weld metal.

C5.0 ACCEPTANCE CRITERIA FOR BOLT CAPACITY

Bolted connections must be qualified based on allowable loads (Pt,Pv,Ptv,Pb) given below. These allowables are based on two thirds of the ultimate connection capacities determined in accordance with recommendations of Reference 10.13. In the equations shown below, Ab is the nominal area of the bolt and Fu is the ultimate tensile stress of the bolt. (Applicable sections of Reference 10.13 are shown)

C5.1 Bolt Tensile Capacity: (4.10.1)

$$Pt = 0.5 * Ab * Fu$$

C5.2 Bolt Shear Capacity: (5.4.2.ii)

For a shear plane passing through bolt shank:

$$Pv = 0.3 * Ab * Fu$$

For a shear plane passing through bolt threads:

$$Pv = 0.225 * Ab * Fu$$

C5.3 Combined Tension & Shear Capacity: (4.10.3)

$$Ptv = (X/0.62)^2 + Y^2 <= 1.0$$

where X is the ratio of the shear stress on the shear plane to the ultimate tensile strength and Y is the ratio of the tensile stress to the ultimate tensile strength. The shear stress and tensile stress are based on the stress area. The stress area may be conservatively taken as 0.75 times the nominal bolt area.

C5.4 Plate Bearing Capacity: (5.4.3.iii)

$$Pb = 0.79 * (L-(d/2)) * t * Fup$$

where L is the distance from the center of the bolt hole to the free edge, d is the diameter of the bolt hole, t is the thickness of the plate, and Fup is the ultimate tensile stress of the plate. Bearing capacity is not considered to be a critical limit state for loads directed away from free edges.

C6.0 SLIP RESISTANCE Of BOLTED CONNECTIONS

The minimum design bolt reaction determination of thermal reactions for a slotted bolted friction connection for the initial slip from the as-installed bolt position to its elevated temperature position shall be:

$Tb = 4.0 \times Fv$ (in Kips for an ASTM A325, 5/8" to 1" diameter)

Where Fv = Allowable AISC friction slip capacity (Kips) as identified on Table 2a, 8th Edition AISC Manual, page 5-213.

Tb = Minimum design bolt reaction, kips

The slip resistance of the bolted connection returning from the elevated temperature bolt position to the original position shall be:

$Tb = 2.0 \times Fv$

C7.0 CONNECTION PLATE ELEMENTS

The ductility of plate elements of connections (i.e. the leg of a clip angle) bent about their minor axis under thermal loading do not have be evaluated since plates bent about that axis are very ductile and safely relieve thermal forces.

C8.0 INHERENT THERMAL GROWTH CAPACITY

Each thermal restraint point at a concrete surface may be assumed to have one thirty second inch (1/32") available free travel due to normal construction tolerances except where thermal load reactions act within 10 degrees of normal to an embedded plate and the structural connection is welded. Assumed travel may be in any direction.

C9.0 CONCRETE STRUCTURE ACCEPTANCE

For all concrete evaluations, acceptance should be based on criteria Ultimate Strength Capacity (ACI 318) reduced by the appropriate capacity reduction factor. All load factors may be assumed equal to unity. If project criteria requires a more conservative practice, it shall govern.

D1.0 GENERAL

The criteria in this section are recommended for acceptance of structural members stressed beyond yield and analyzed using non-linear analytical methods.

D2.0 LOAD COMBINATIONS

Same load combinations given in Section C1.0.

The sequence of load application must be considered for non-linear analysis. The most conservative sequence shall be used for analysis.

D3.0 MEMBERS

D3.1 Ductility Requirements

Each member shall be classified as either an ancillary or a primary member in accordance with Section 2.1 definitions. The acceptance criteria differ for each type.

Both primary and ancillary members can alternately be accepted on the basis of the linear acceptance criteria presented in Appendix C.

D3.2 Primary Member Acceptance

Primary members can be accepted on the basis of displacement based ductility ratios, μ_d. The following acceptance criteria must be applied to each plastic region of a member.

$$\mu_d \leq 3$$

where:

$$\mu_d = \frac{\sqrt{(\delta_{xu}^2 + \delta_{yu}^2 + \delta_{zu}^2)}}{\sqrt{(\delta_{xy}^2 + \delta_{yy}^2 + \delta_{zy}^2)}}$$

where: δ_{xu}, δ_{yu}, δ_{zu} = The ultimate x, y, z translational displacement at the centroid of the member's cross-section. Acceptance shall be based on the maximum calculated μ_d for all cross-section centroid points along the member's axis.

δ_{xy}, δ_{yy}, δ_{zy} = The initial yield x, y, z translational displacements at the same centroidal point identified for the ultimate displacement.

D3.3 Ancillary Members

A demonstration that combined stresses are below yield is sufficient to accept a member as long as no compression forces are present.

Otherwise ancillary members must meet the following acceptance criteria when analyzed using non-linear methods:

$$\mu_d \leq \mu_{sec}$$

or

$$\mu_s \leq \mu_{sec}$$

where:

μ_s and μ_d are as defined in Section 2.1

μ_{sec} is limited as follows:

Structural Steel Tension Members

$$\mu_{sec} = 0.5 \quad \varepsilon_\mu / \varepsilon_y$$

where

ε_μ = Percentage elongation at failure (rupture)

ε_y = Strain corresponding to yield stress

Tension due to flexure:

$$\mu_{sec} = 10$$

Structural Steel Compression Members:

if Kl/r less than 20

$\mu_{sec} = 1.3$

if Kl/r is greater than 20

$\mu_{sec} = 1$

μ_d shall be determined in accordance with Section D3.3 and μ_s shall be the ultimate strain in the member divided by the yield strain for the member's material.

D4.0 CONNECTIONS

Connections shall meet the requirements of Appendix C.