Computech Engineering Services, Inc.

TORNADO RESISTANCE CRITERIA FOR REINFORCED MASONRY WALLS

San Onofre
Nuclear Generating Station, Unit 1

Prepared for:

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1 INTRODUCTION

This document describes criteria and methodology for the evaluation of reinforced masonry walls under tornado loads. These provisions are supplemental to the Tornado Resistance Design Review Criteria [Reference 1] developed for San Onofre Nuclear Generating Station, Unit 1 (SONGS-1) by Cygna Energy Services.

These criteria address the effects of tornado wind loads, pressure differentials and the global effects of missile impact. Penetration of the masonry walls by these missiles is a local effect and the effect on safety related equipment of these missiles after passing through the walls is outside the scope of these criteria.

The provisions of these criteria are to be used in conjunction with the overall criteria for tornado review of SONGS-1, i.e. for definition of loads and load combinations. These criteria provide the acceptance criteria and evaluation methods for the reinforced masonry portions of the plant.

The criteria are based largely on the methodology used for the SONGS-1 seismic evaluation of masonry walls. The concepts used are similar to those for reinforced concrete as specified in the ACI code 349-80, Appendix C (Reference 7) with their application modified to reflect the difference in material properties and behavior between reinforced concrete and reinforced masonry. Particular aspects of the criteria which are addressed in this code include the use of ductility for response to impulsive and impact loads, the use of dynamic load factors and the use of non-linear time history analysis.

2 LOADS

Tornado wind loads and pressure differentials shall be as provided by the project criteria. Missile load characteristics shall be those developed using the site specific missiles and plant data as described in the project criteria. The evaluation of global response to these loads shall be as described in Section 6 of these criteria.

3 ACCEPTANCE CRITERIA

The evaluation of reinforced masonry walls will use the load combinations specified in Section 5.3 of the project criteria. The acceptance criteria will first be based on allowable stress values as provided in ACI-531-79 (Reference 8) and using increase factors as provided in Section 5.3 of the project criteria. If these values are exceeded then ultimate strength criteria, which take account of the inherent ductility of reinforced masonry, will be used.

3.1 Inelastic Criteria

The inelastic criteria will be used only in conjunction with the results of an evaluation performed in accordance with the non-linear analysis methodology described in Section 4. Based on the results computed from these analyses the wall shall be judged capable of resisting tornado wind loads, pressure differentials and global missile effects if each of the following conditions is satisfied under specified load combinations:

1. The maximum strain in the steel rebar does not exceed one-half the ultimate strain.

2. The maximum compressive strain in the masonry face-shell does not exceed 0.004.

3. Maximum wall deflections do not exceed the stability limit of the wall.

4. Steel support members and connections meet the criteria for steel structures as given in Section 5.2 of the project criteria.

4 STRUCTURAL EVALUATION METHODOLOGY

If a wall is to be qualified using the alternate non-linear analysis criteria the methodology described in the following sections should be used to analyze the wall and evaluate the results.

The methodology described herein is based on that used for the SONGS-1 masonry wall seismic evaluation. A more detailed description may be found in the documentation provided for this seismic evaluation (References 2, 3 and 4). Some modifications have been made to the methodology to address the concerns expressed about some aspects by the NRC when they reviewed the seismic evaluation.

For illustrative purposes an example of an analysis for tornado loads is provided. This example wall has a height of 16'-10" and is assumed to be vertically spanning only. It has vertical rebars of #5 at 32" centers. This is not intended to represent any particular SONGS-1 wall and serves solely to clarify the methodology and provide estimates of the additional load carrying capacity available if non-linear response is computed.

4.1 Structural Model

The structural model used for the evaluation should be a two-dimensional representation of the actual wall with all components of the wall explicitly included, i.e. the masonry blocks, the face shells, and the rebar. The form of this model is as shown in Figure 1 for the example wall. This model includes the following features:

1. Individual masonry blocks represented by elastic plane stress elements.

2. Each joint explicitly modelled.

3. The rebar at each joint represented by a bi-linear truss element with a yield stress equal to that of the rebar.

4. The face shells at each joint represented by elements which fail at a specified stress in tension (equal to the rupture stress) and which remain elastic in compression.

5. At the points of maximum moment where yielding of the rebar will occur a longer effective length of rebar is used to model the actual plastic hinge length.

6. Elements appropriate to model the top support stiffness are included. In this example, an inclined truss member approximates the connection to the bottom flange of a steel section.

In the example wall the properties were based on a uniform wall width of 13'-0". Depending on the wall configuration these properties may be changed up the height of the wall if the net width is reduced due to openings. In such a case the steel and masonry face shell reductions may not be proportional if there are trimmer bars around openings.

4.2 Boundary Conditions

The model boundary conditions should be those appropriate for the SONGS-1 walls. Typically, the SONGS-1 walls have a top support detail with negligible stiffness in the vertical direction and so this support should be modelled as free to move in the vertical direction. This effectively precludes stress stiffenening effects in the rebar (i.e. from catenary action) under large displacements.

The typical detail at the bottom of the SONGS-1 walls provides for dowels from the foundation lapping with the wall vertical reinforcement. The seismic tests demonstrated that this detail was sufficient to develop the ultimate moment at the wall base. For the earlier seismic evaluation the model assumed a pinned base condition. This was later modified when it was realized that this assumption introduced undue conservatism (e.g. in the Fuel Storage Building analysis). However, use of a fixed base for other buildings with strip footings may require that the stiffness of the footing be included in the model. This could be done by inclusion of a linear spring at the wall base, similar to the top detail. In this case the actual base fixity would be partial.

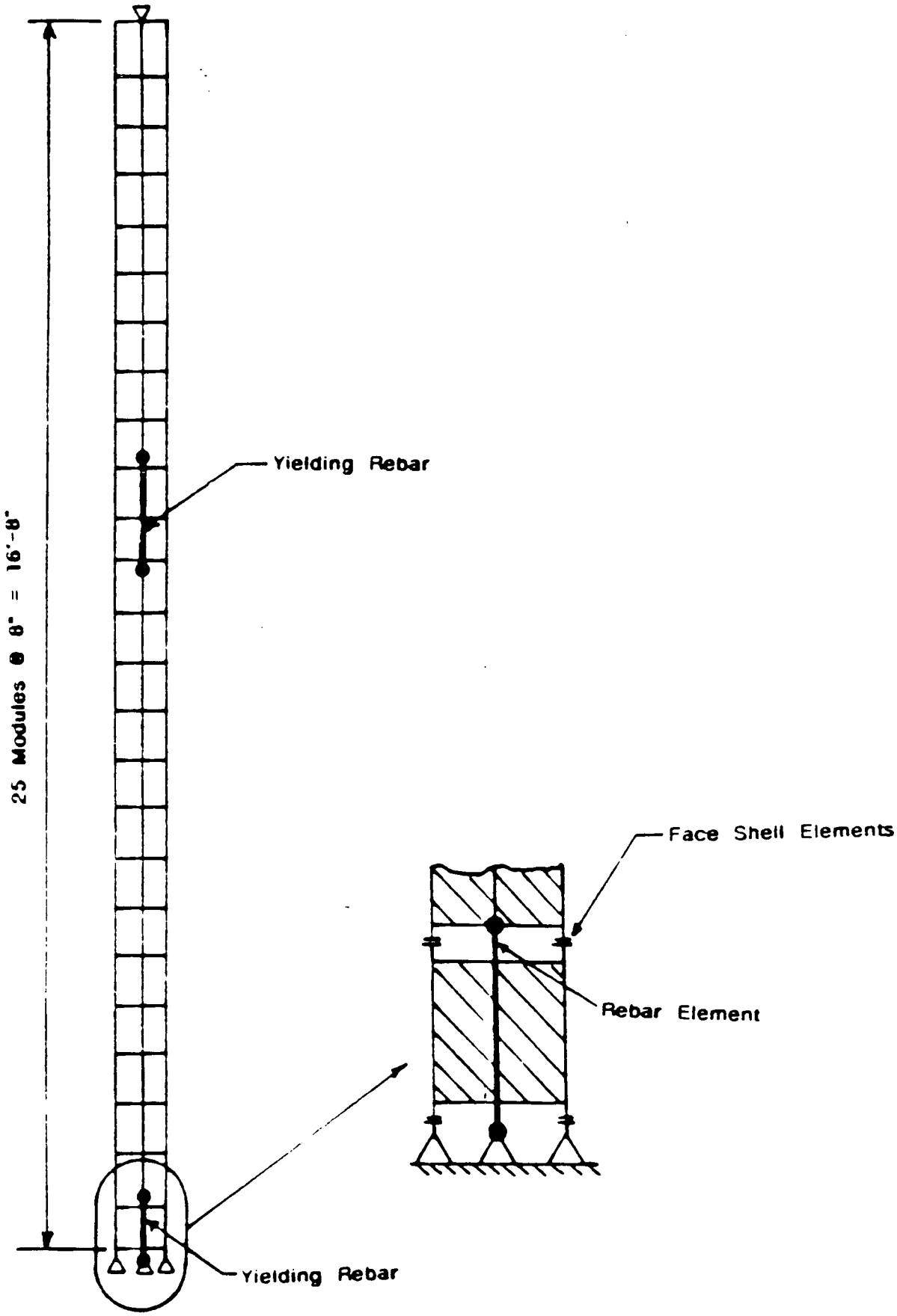

FIGURE 1 : ANALYTICAL MODEL FOR EXAMPLE WALL

4.3 Load Specification

4.3.1 Lateral Loads

The tornado pressure and wind loads are represented by an equivalent uniform static load as provided by the project criteria. This is applied to the model as nodal loads at each block element. For non-linear analysis this will typically be specified in a number of increments so as to achieve numerical stability as the non-linear force deflection curve is tracked. For the example wall the load was applied in 2 psf increments.

Tornado missile loads will be applied as time varying point loads applied to the wall at the position which provides the maximum response - see Section 6.

4.3.2 Vertical Loads

In addition to the tornado lateral loads the model should include the self-weight of the wall. As there are no vertical accelerations due to tornado the dead load will act vertically downward throughout the analysis and will have a beneficial effect in that it will delay the onset of cracking and yielding. As deflections increase the wall rotates about the outer edge of one faceshell and the dead weight of the wall continues to resist the applied load until the center of mass of the wall moves beyond the point of rotation at the wall base. This point will theoretically occur when the wall deflection equals the wall thickness, i.e. 7.625". Beyond this point geometric instability will occur and therefore this deflection forms a limiting value in the criteria.

The dead load should include the weight of the wall itself plus any roof loads and equipment loads carried by the wall. In cases of equipment loads eccentric to the wall it may be necessary to include an element normal to the wall to model the eccentricity of the mass.

4.4 Method of Analysis

The analysis for wind and pressure loads should be performed in a step-by-step procedure which will provide the force-deflection response for a monotonically increasing lateral load. This load should extend at least to the point at which one or more of the acceptance criteria are exceeded, i.e. steel strain ratio limit, face shell strain limit or stability limit. The solution procedure should include a large-displacement formulation to ensure that the stability limits on deflection are obtained from the analysis.

The methodology used to date requires a transformation on the face shell force to obtain face shell strains, as discussed in the following sub-section. Therefore, the analysis would be continued beyond the point at which this strain limit is expected to be exceeded. The actual limiting load is then determined by evaluating the results.

Figure 2 shows the load-deflection curve obtained for the example wall for a uniform load increasing from 0 to 90 psf. The upper curve demonstrates the beneficial effect on load capacity of including the self-weight of the wall in the analysis at loads less than the stability limit, which in this example occurred at slightly more than 6" displacement. When dead load is not included the analysis does not determine the stability limit.

For missile loads determined as described in Section 6 the analysis will be either by the Dynamic Load Factor method (Reference 6) if the loads are less than the elastic limit of the wall or by a non-linear time history analysis if the loads exceed the elastic limit. The analysis shall be performed with a sufficiently short time step to ensure numerical stability under the action of the time dependent load.

PROJECT : SONGS-1 TORNADO EVALUATION
CLIENT : SOUTHERN CALIFORNIA EDISON
SUBJECT : EVALUATION OF MASONRY WALLS - NON-LINEAR ANALYSIS
DISPLACEMENT UNDER STATIC LOAD

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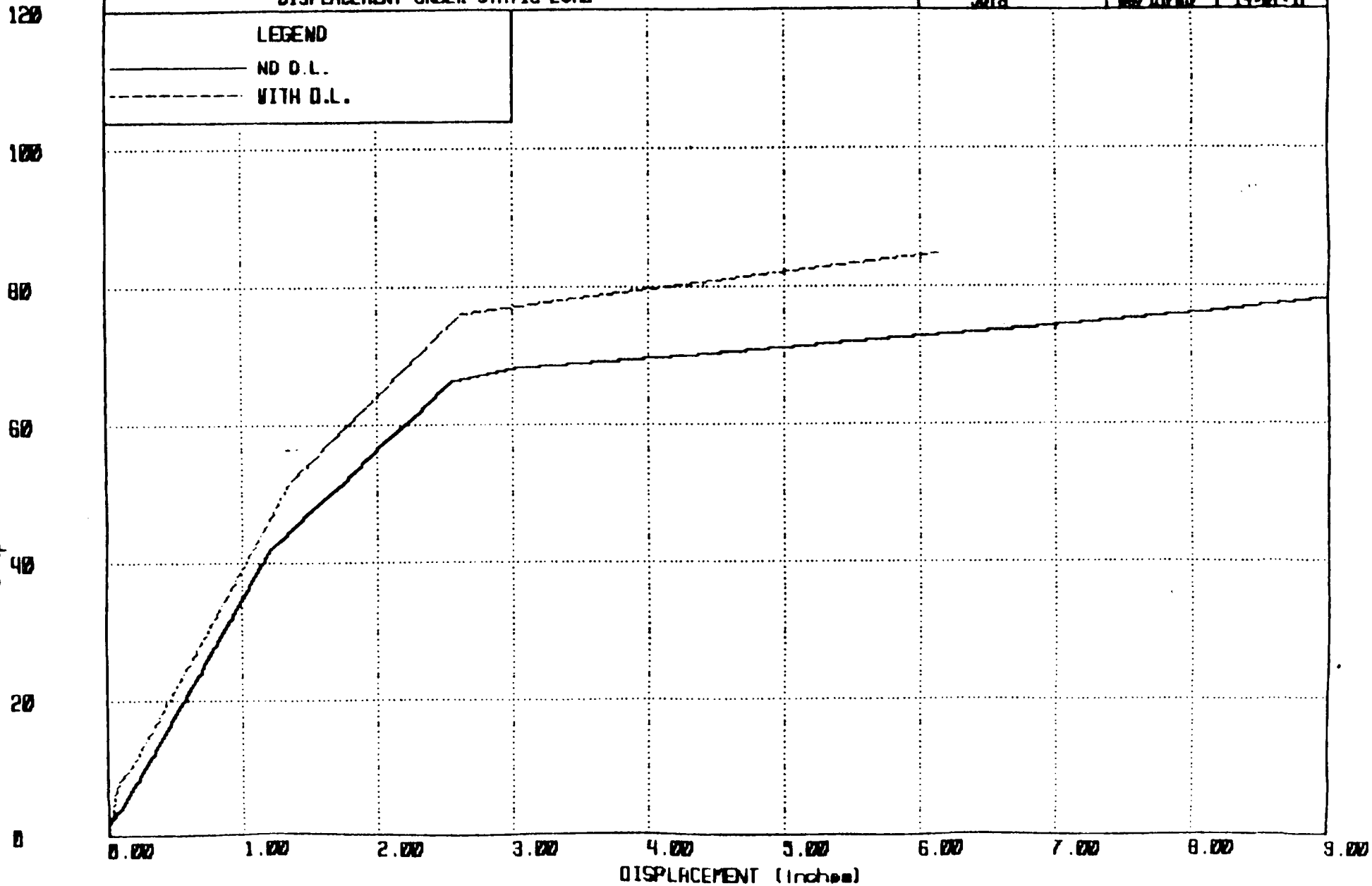


FIGURE 2 : FORCE-DEFLECTION CURVE : STATIC LOAD

4.5 Evaluation of Results

The results from the non-linear analysis are evaluated by determining the load corresponding to the point at which steel strains exceed allowables, masonry strains exceed allowable, support forces exceed allowables or the stability limit is reached. The lowest of these loads will then be the limiting force on the masonry wall. The governing loads for each limiting criterion are computed as described below.

4.5.1 Support Forces

The support capacity is dependent on the actual structural configuration For the example wall the top support is provided by embedded 3/4" diameter bolts at 16" centers in the bond beam. Using an allowable load for these bolts of 1100 lbs provides a total capacity of 10725 lbs over the 13'-0" length of wall in the model. In accordance with increase factors used for other component evaluations for extreme loads this strength is increased by 1.5, providing an ultimate load capacity of 16,080 lbs.

The support forces obtained from the analysis are 10,620 lbs at 76 psf load increasing to 11,200 lbs at 85 psf, the stability limit. Extrapolating from these values indicates a support strength adequate for loads up to 160 psf, well beyond the stability limit.

4.5.2 Stability Limit

With dead load included in the analysis the solution procedure could not achieve convergence beyond a load of 85 psf, which occurred at 6.1" displacement. Beyond this point the self-weight of the wall tended to increase the overturning thereby reducing the load carrying capacity of the wall. Under applied loads equilibrium cannot be achieved for increasing loads beyond this point. Therefore, 85 psf is the limiting loads for stability.

4.5.3 Steel Strain Ratios

The steel strain ratios are obtained by dividing the maximum extension

in the yielding rebar elements by the yield extension in these elements. At the base the yield extension is 0.0158" and at the upper hinge 0.0295". With no dead load the limiting strain ratio of 45 would not be reached until a load level of 92 psf. When dead load is included the maximum steel strain ratios reach a maximum value of 14 at the stability limit of 85 psf and so do not form a governing load limit.

4.5.4 Masonry Strains

The wall model used for the evaluation produces face shell forces. Based on the work done for the seismic evaluation strains are a more appropriate parameter for assessing the capacity of the masonry face shells. In Reference 4 a procedure for determining face shell strains was developed. This procedure uses the maximum compression force, the plastic hinge configuration and masonry stress-strain properties to compute the distribution of stresses and strains in the face shell for given wall deflections.

This procedure has been followed for the example walls and the face shell stress-strain distributions computed at the base and upper hinges for deflections ranging from 2" to 10". Figures 3 and 4 show these distributions. At each section the stress distribution is linear up to deflections of approximately 2.5" and strains reach the criteria limit of 0.004 at deflections of 7 inches. However the stability limit is reached at a lower deflection of 6.1" and so the masonry strains do not form the critical parameter for this wall.

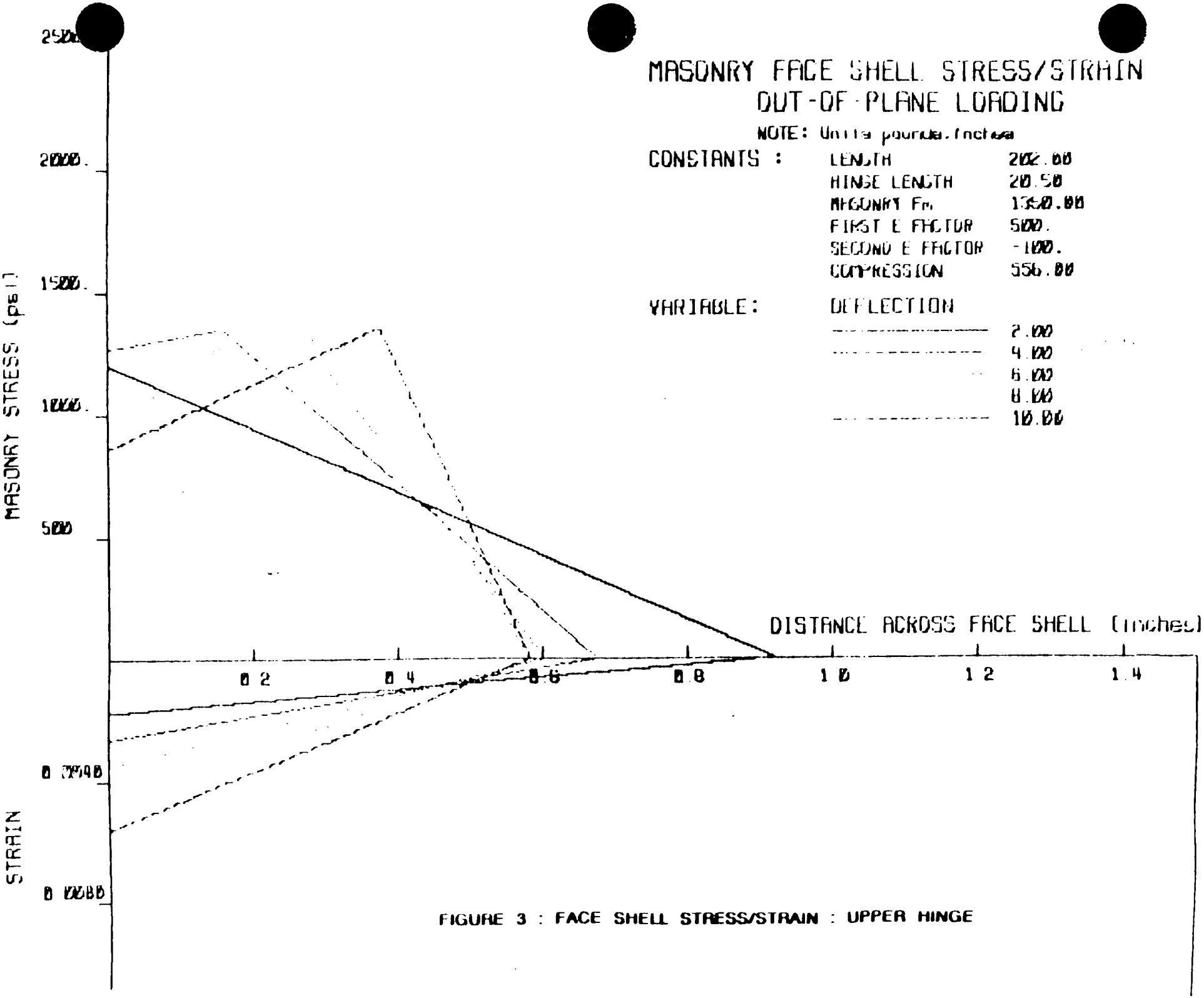


FIGURE 3 : FACE SHELL STRESS/STRAIN : UPPER HINGE

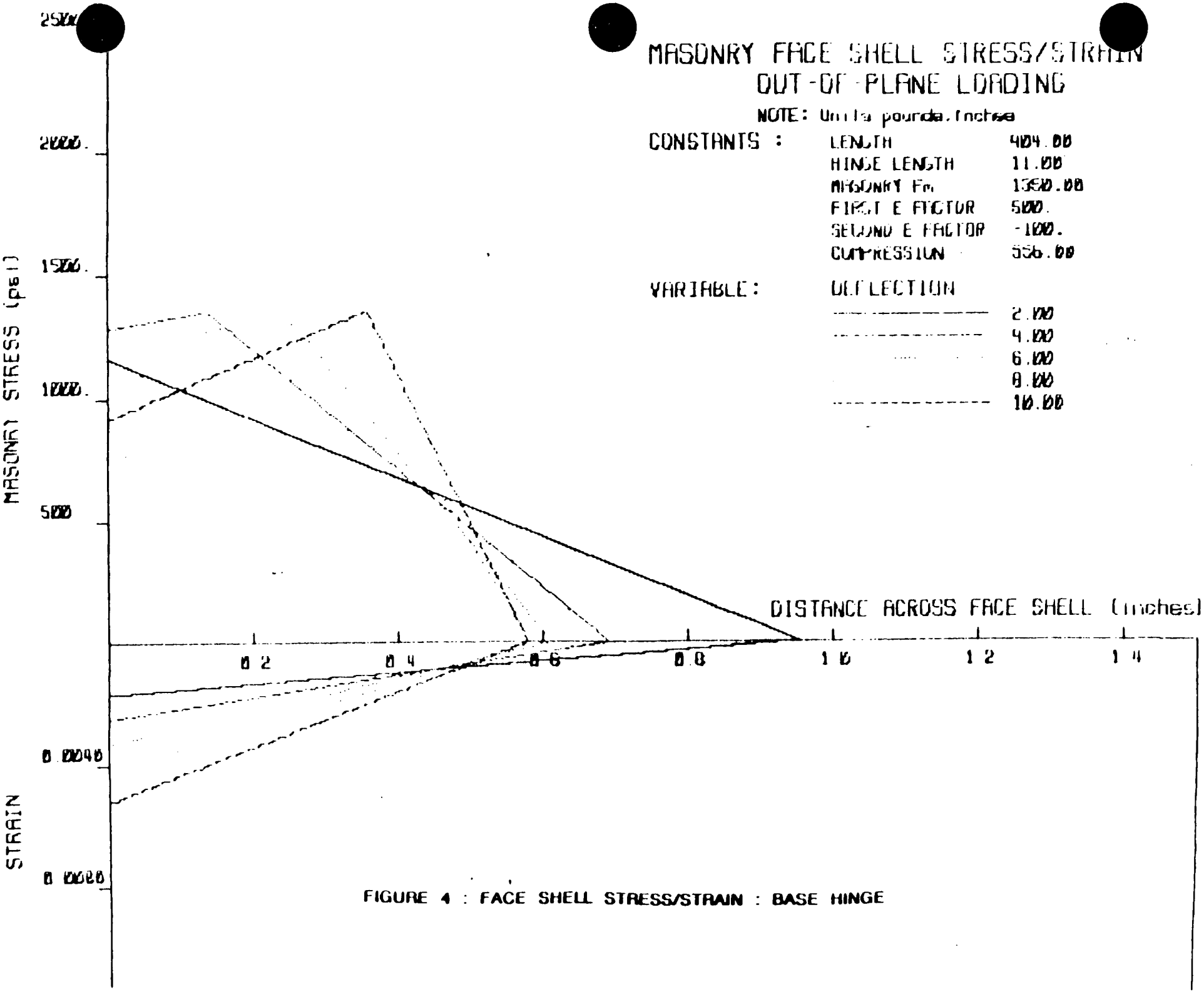


FIGURE 4 : FACE SHELL STRESS/STRAIN : BASE HINGE

4.5.5 Allowable Stress Limit

Using the allowable stress approach in the project criteria [1] for determining maximum load levels the governing parameters are steel stresses of 0.9Fy and masonry stresses of 0.85f'm. The example wall has a cracked section modulus of 0.398 for the steel and 22.078 for the masonry. This provides limiting moments of 14.328 lb-in/ft for the steel and 25.334 lb-in/ft for the masonry. The lower of these two values, 14.328, governs. This moment corresponds to a uniform load level in the wall of 34 psf.

4.5.6 Final Wall Evaluation

The final wall evaluation is then performed by listing the maximum load as governed by each parameter. For the example wall this is as follows:

PARAMETER	LOAD (psf)
Top Support Force (with 1.5 factor)	160
Steel Strain Ratios	
Base Hinge	> 85
Upper Hinge	> 85
Masonry Strains	
Base Hinge	> 85
Upper Hinge	> 85
Stability Limit	85

Based on these results the allowable load on this example wall would be 85 psf, governed by the the deflection at which the stability limit of the wall is reached. This is 2.5 times the maximum load computed on an allowable stress basis.

The points at which each criteria parameter are attained are shown on Figure 5. Also shown on this figure is the allowable stress limit. Features to be noted from this plot are as follows:

1. First yield occurs in the base hinge at a load level of 52 psf, approximately 50% higher than the allowable stress

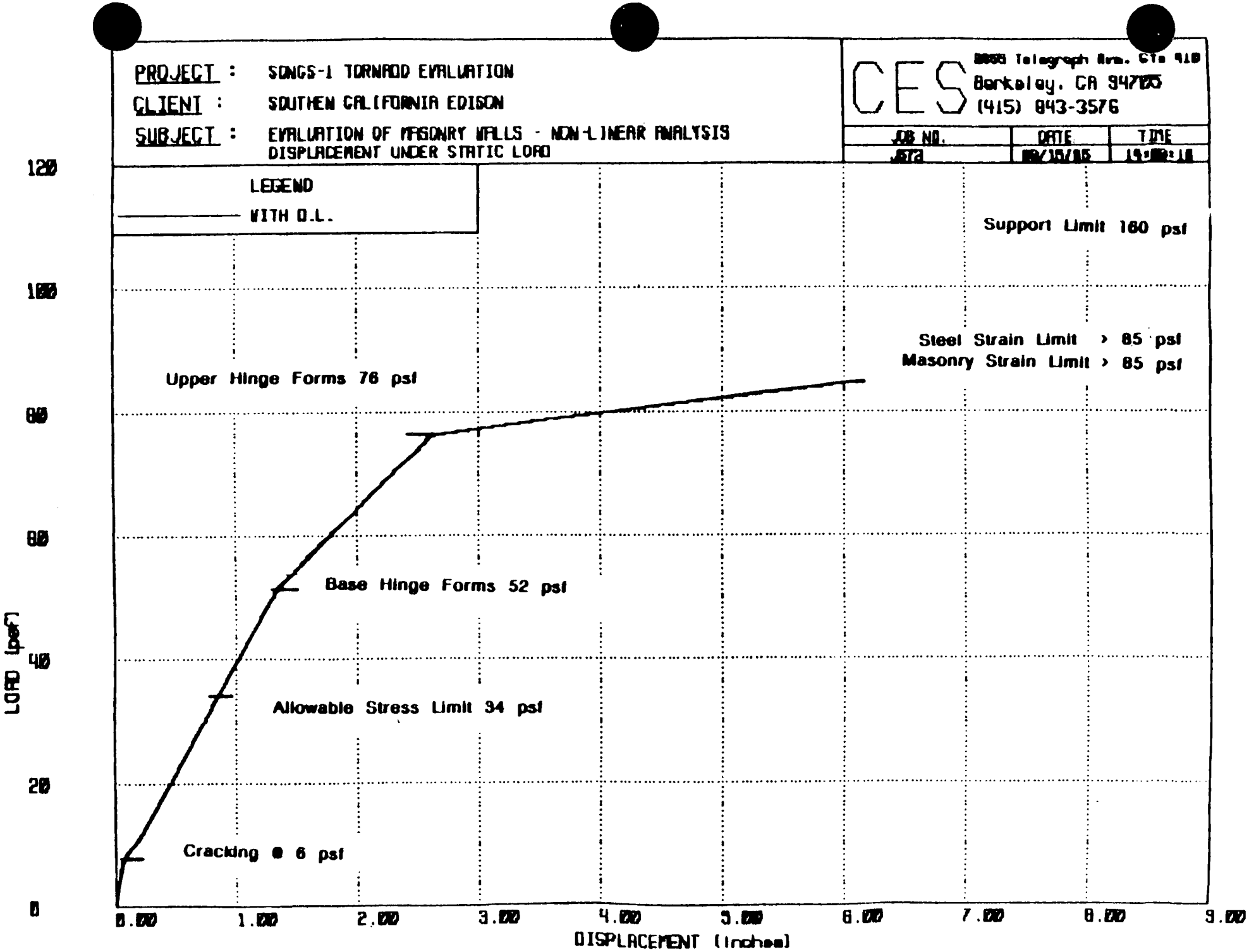


FIGURE 5 : LOAD LIMITS FROM ACCEPTANCE CRITERIA

value of 34 psf. The difference is caused partially by the 0.9 factor applied to the steel stress for allowable stresses and also by the effect of the dead load which is included in the non-linear analysis.

When dead load is not included first yield occurs at a load level of 42 psf. Without the capacity reduction factor in the allowable stress computation the load computed on an allowable stress basis would be 38 psf. about 10% lower than that computed in the analysis. This difference arises because the analytical model assumes that the center of compression is at the outer edge of the wall rather than 0.35" from the face shell as computed by the allowable stress method (where the neutral axis depth is 1.06"). This value is about 10% of the effective section depth (7.625"/2 = 3.8125") and accounts for the difference. The model then slightly overestimates the moment capacity but the actual neutral axis position would move closer to the outer face of the face shell as deflection increased (see Figures 3 and 4) and so this overestimation would be less at higher load levels.

2. After first yield the wall remains relatively stiff until the second hinge occurs, at a load level of 76 psf. The maximum load at this point is 2.2 times the allowable stress value. At this point a mechanism has formed in the wall and the stiffness drops off sharply.

3. After the second hinge has formed deflections increase rapidly for small increases in load until the stability limit is reached at 85 psf. At this point the load is 10% higher than at the formation of the second hinge but the deflections are over twice as large.

In summary, most of the increase in load over that permitted by an allowable stress approach is caused by the inclusion of dead weight in the analysis and by the large increment in load between the time of first yielding and the formation of a mechanism. Beyond that point load increases are relatively small for a large increase in deformations.

5 DYNAMIC EFFECTS OF WIND AND PRESSURE LOADS

The non-linear methodology used for the tornado wind load and differential pressure evaluation assumes the load to be applied statically. In fact, wind loads are a dynamic phenomenon and the dynamic interaction of the load and the wall could modify the response. To examine this effect a wind load of 80 psf was applied dynamically rather than statically. The load was assumed to increase linearly from zero to a maximum value at 2 seconds and then decrease linearly to zero at 4 seconds. The magnitude of the load at its peak, 80 psf, is close to the limiting load of 85 psf determined for this wall.

A plot of the wall deformation versus time for the example is shown in Figure 6. Superimposed on this plot is the static displacement pattern determined as described earlier. This result shows that for a maximum load of 80 psf the dynamic response is similar to that of the static analysis. Maximum deflections differ by only a few percent, therefore, in this case the static analysis provides a close estimate of maximum response quantities.

This analysis indicates that for the extent of non-linearity present in this wall and the period of forcing assumption used dynamic effects do not increase response. For analyses in which the non-linearity is more marked or the forcing function has a different frequency this conclusion should be checked on a wall-by-wall basis.

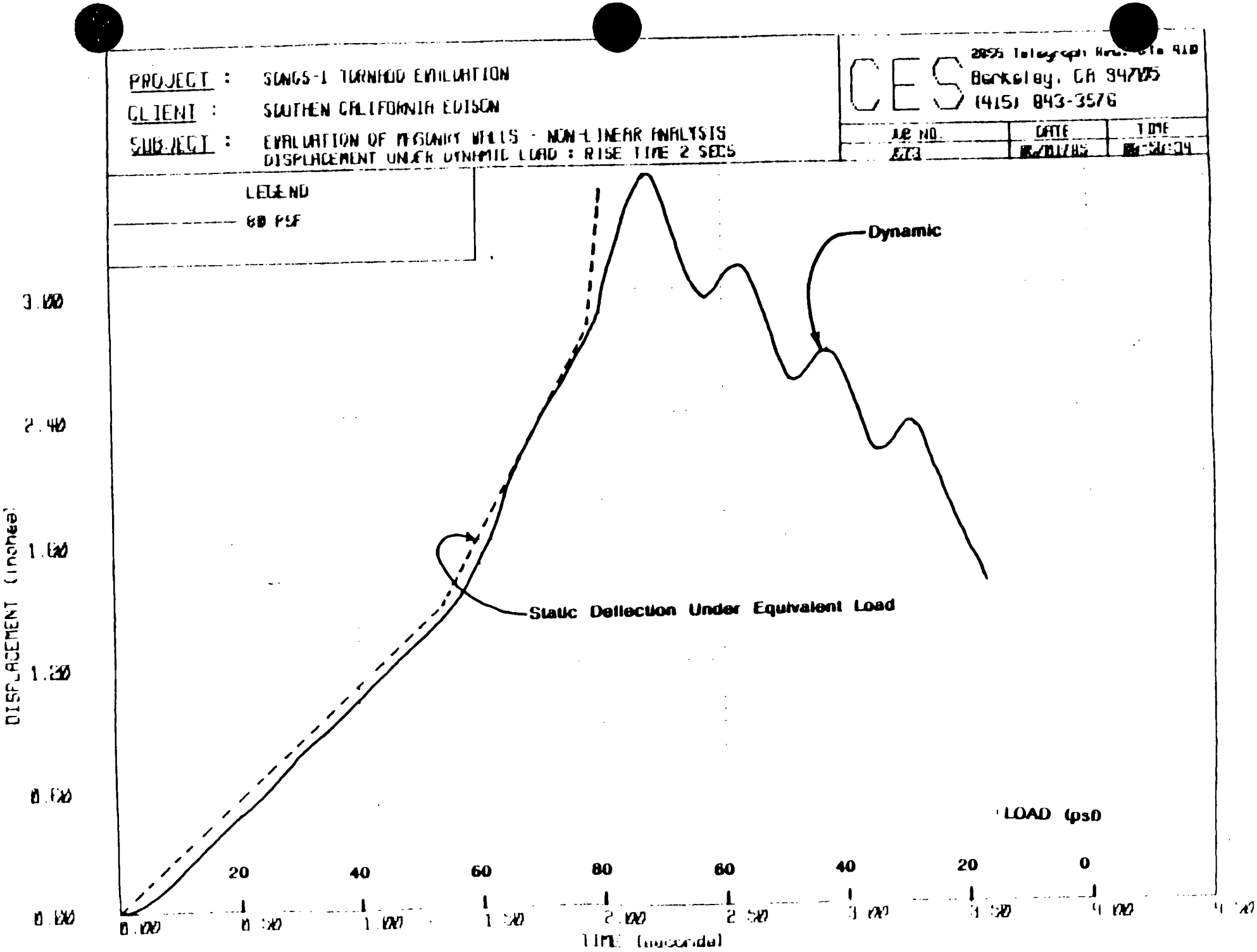


FIGURE 6 : DYNAMIC LOAD OF 80 PSF : 2 SECONDS RISE TIME

6 GLOBAL MISSILE EFFECTS

These criteria do not address the local effects of tornado missiles, i.e. damage caused to the portion of the wall directly impacted by a missile except as it affects the overall wall stability. The global effects are considered in the following two sub-sections, firstly on the structure itself and secondly on the overall integrity of the masonry wall.

6.1 Structural Evaluation

Global missile effects are the loads induced in the main structural system by missiles striking the masonry walls. Most data on missile impact effects addresses concrete and steel construction and no data is known for masonry construction. However, the upper bound force on a given portion of wall will be the capacity of the wall as computed in Section 4 for face loadings. The wall cannot transfer more force into the structure than can be resisted by the wall. Therefore, an assessment of the structural integrity under the impact of tornado missile loads may be made by using the maximum reaction loads at the time of wall failure. These loads may then be compared with the lateral loads from other sources for which the structure has already been evaluated, e.g. seismic. If these loads are less than other lateral loads no further evaluation will be required. If they are higher more detailed reaction loads may be obtained from the analysis procedures discussed in the following sub-sections.

6.2 Distribution of Impact Loads

The SONGS-1 walls are asssumed to be vertically spanning only because of the presence of control joints. For uniformly distibuted loads this enables a representative strip to be analyzed. However, for point loads it is necessary to determine an effective width over which the load may be assumed to be distributed.

As discussed in Section 6.3, some missiles will penetrate the wall and therefore reduce the load carrying capacity of the wall. The effective width should reflect this possible damage and so is developed separately for impact in the ungrouted and grouted regions in the following sub-sections.

6.2.1 Impact in Ungrouted Region

If a missile penetrates the wall in the ungrouted wall the damage will not reduce the wall capacity which is assumed to be provided solely by the grouted cells containing rebar. Therefore the full wall section may be used.

To estimate the effective width a linear elastic analysis of a 32'-0" long section of wall 20'-0" high was performed. The model included only the grouted cells and therefore was essentially a grid of vertical columns at 32" centers and horizontal beams at 48" centers. Cracked properties of the wall were used. A point load was applied at mid-height and mid-length of the model and the moment distribution along the length of the wall examined. The distribution of maximum moments is as shown in Figure 7.

The total moment under the 2 kip point load is 8 kip-ft. The peak moment in any column is 1.643 kip-ft over 32", equivalent to 0.616 kip-ft/ft width. Therefore, an effective width could be assumed to be (8/0.616) = 13'-0".

This procedure is illustrative of the method which would be used for the SONGS-1 evaluation to obtain effective widths which would vary depending on the wall configurations and point of impact. The value of 13'-0" computed above is used in the examples of impact loads in the following sub-sections.

6.2.2 Impact in Grouted Regions

If impact occurred in a grouted region damage would occur to vertical and/or horizontal bond beams. The most severe damage would occur if complete penetration occurred under impact at the intersection of a vertical and horizontal bond beam. The maximum size of missiles determines the extent of the damaged region. Assuming a punching shear type failure mechanism the size of the damage zone would be equal to the width of the missile plus twice the wall thickness, i.e. 16". The minimum spacing of bond beams is 32" and so missiles up to 16" diameter would damage only a single bond beam.

To model this effect the analysis for the ungrouted region was repeated with 96" of vertical bond beam and a 64" portion of horizontal bond beam removed. The 2 kip load was then distributed to the four adjacent intersections, as shown in Figure 8.

The effective width was computed in a similar fashion as for the ungrouted region but in two different locations. At the top of the damaged region the applied total moment was 6.4 kip-ft and the maximum moment in the wall from the analysis 0.481 kip-ft/ft. At this section the effective width is then (6.4/0.481) = 13 feet, similar to that for the ungrouted region impact.

At mid-height of the damaged region where the total applied moment is 5.6 kip-ft the maximum wall moment increases to 0.769 kip-ft/ft, providing an effective width of (5.6/0.769) = 7.3 feet. Note that the width of the damage region, 5.3 feet, when added to the effective width of 7.3 feet, provides a total width of 12.6 feet, close to the effective width outside the damage region.

This procedure should be followed for actual SONGS-1 wall configurations. This result appears to suggest that the damage region may simply be considered as an opening in the wall although with no trimmer bars.

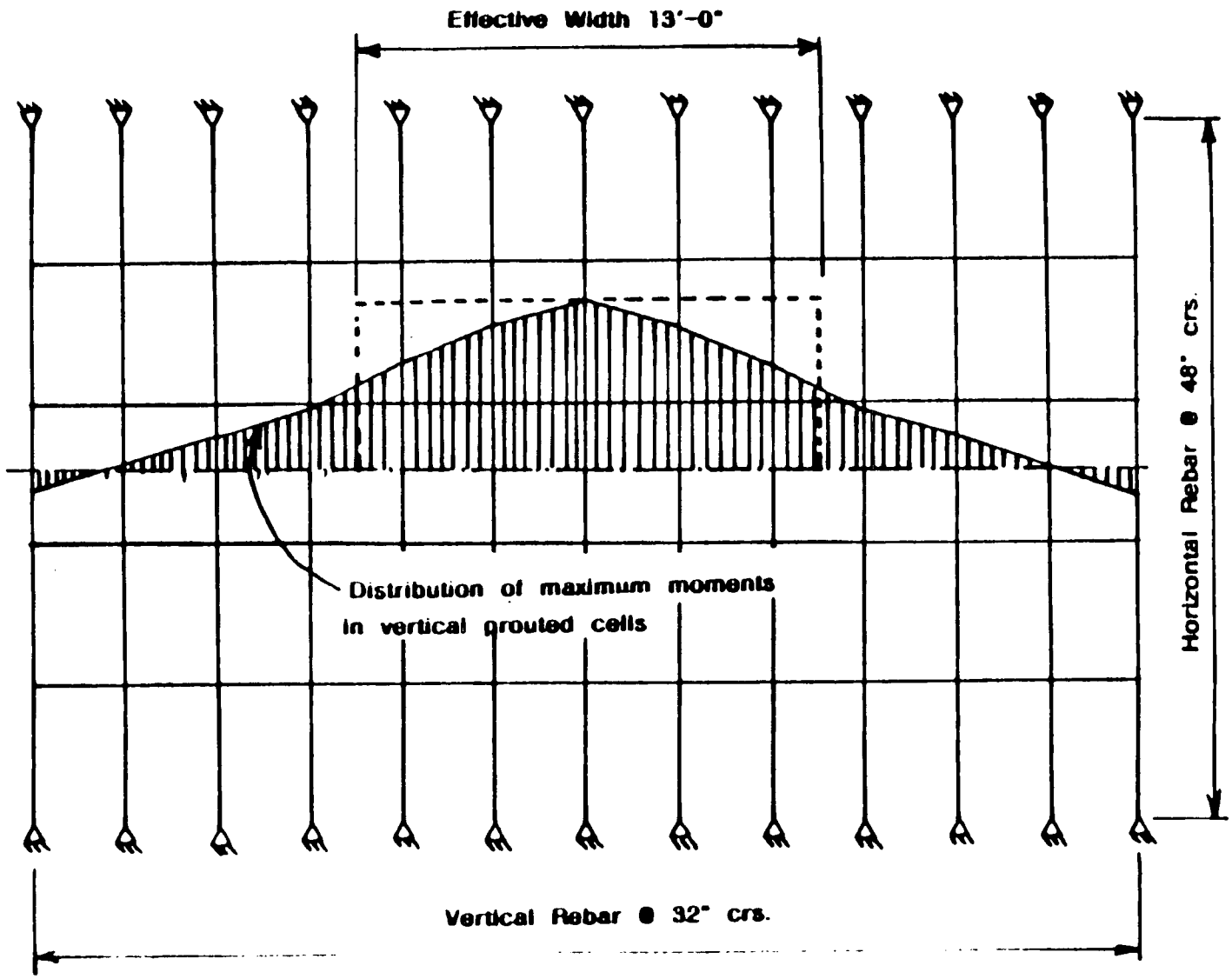


FIGURE 7 : MODEL FOR EFFECTIVE WIDTH DETERMINATION - UNGROUTED REGION

Distribution of maximum moments in vertical grouted cells

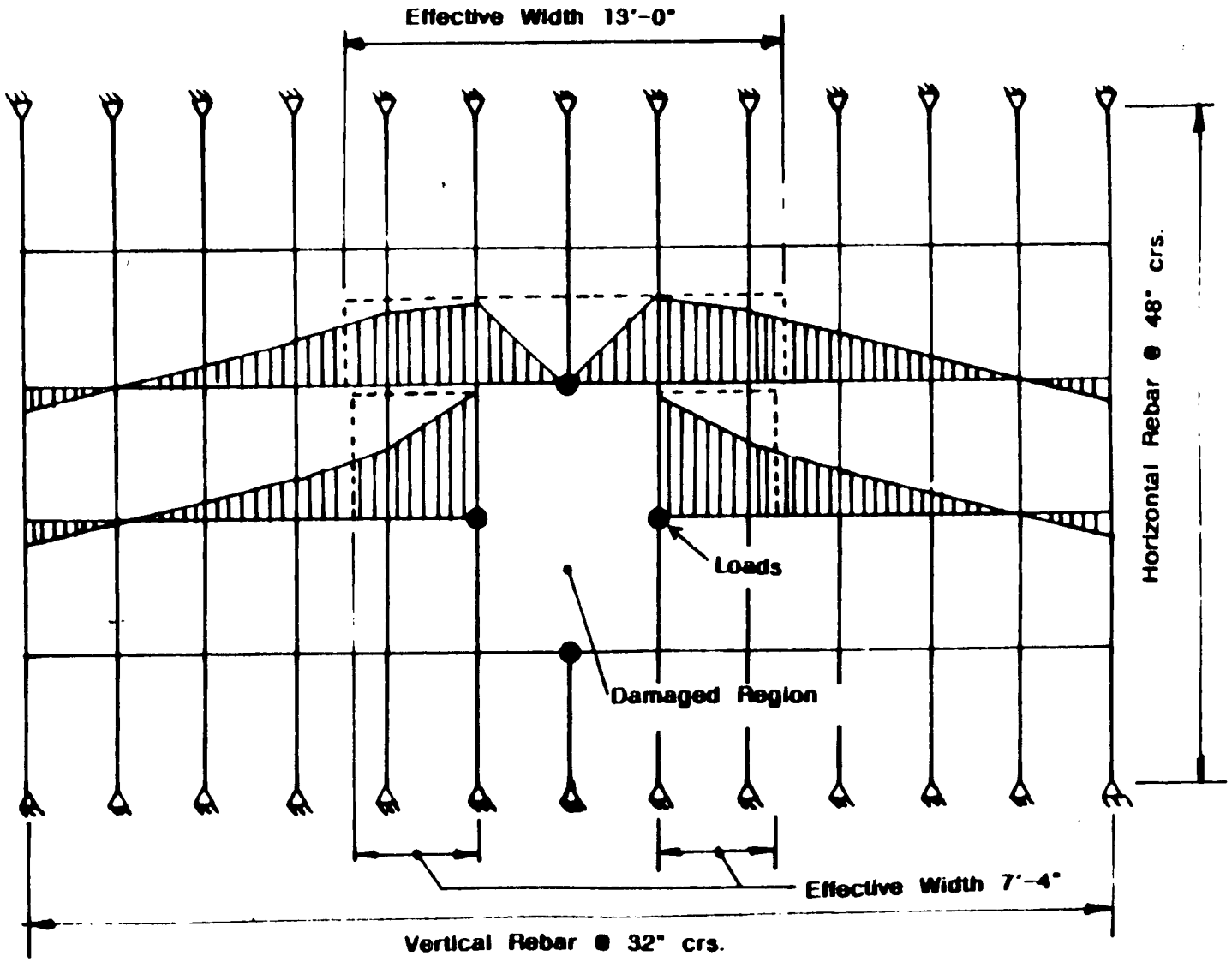


FIGURE 8 : MODEL FOR EFFECTIVE WIDTH DETERMINATION - GROUTED REGION

6.3 Overall Wall Integrity

The SONGS-1 masonry walls are typically reinforced at 32" centers vertically and 48" centers horizontally. At the rebar locations the masonry is grouted effectively forming a grid of small concrete beams and columns. At the typical spacing the area of grouted wall is about one-third the total area of wall. To assess the overall wall stability the case of missile impact must be considered both in the grouted region and in the ungrouted regions.

6.3.1 Impact in Ungrouted Region

In the ungrouted regions a small missile (e.g. steel rod) could strike between the block webs or at the web position. A larger missile (greater than 5" diameter) would strike at least one web. For the development of the criteria it has been assumed that the missiles have sufficient kinetic energy to penetrate the wall and one of three failure modes at the point of impact will occur:

1. Bearing failure will occur if the contact stress exceeds 0.95f'm.

2. One block or a group of blocks will be forced out of the wall if the mortar shear stress exceeds 1.7 times the square root of the prism strength.

3. A punching shear failure through the wall will occur if the shear stress around the perimeter of the missile exceeds 4 times the square root of the prism strength.

For each SONGS-1 missile configuration each of these modes should be checked and the perforation load will be the load associated with the lowest of these three values. The wall should then be analyzed for the effects of this load by one of two methods:

1. Dynamic Load Factor method, for example as given in Reference 6. This is only applicable for loads less than the load required to cause yielding in the wall

2. Time history analysis. The point load is applied as a constant force for a time interval computed from the impact velocity and wall thickness. This is described more fully in the following sub-section.

Using the failure modes listed above the maximum force to penetrate the SONGS-1 walls in the ungrouted regions has been computed to be approximately 1 kip for a 1" diameter missile (governed by bearing stress) increasing to about 30 kips for a 24" diameter missile (governed by punching shear). This load will be applied over a short time interval as determined by the velocity of impact. The time interval is approximately 0.006 seconds for a missile velocity of 100 ft/sec and 0.004 seconds for a velocity of 150 ft/sec. Because of this very small time of occurrence the dynamic load factor will be much less than unity

To assess the effect of impact on an ungrouted region the example wall was analyzed for an impact load of 40 kips modelled as a rectangular pulse with a duration of 0.006 seconds. This would be an upper bound for impact in the ungrouted region. The load was applied over the assumed effective width of 13'-0". A load of 40 kips corresponds to a uniform load of 183 psf over the entire wall area which is over twice as high as the maximum allowable static pressure determined for this wall.

As shown in Figure 9, the maximum displacement under this load was approximately 1.1", less than one quarter of the displacement occurring under the uniform static load of 85 psf. Therefore, the wall is capable of resisting much higher total loads when applied for a very short duration.

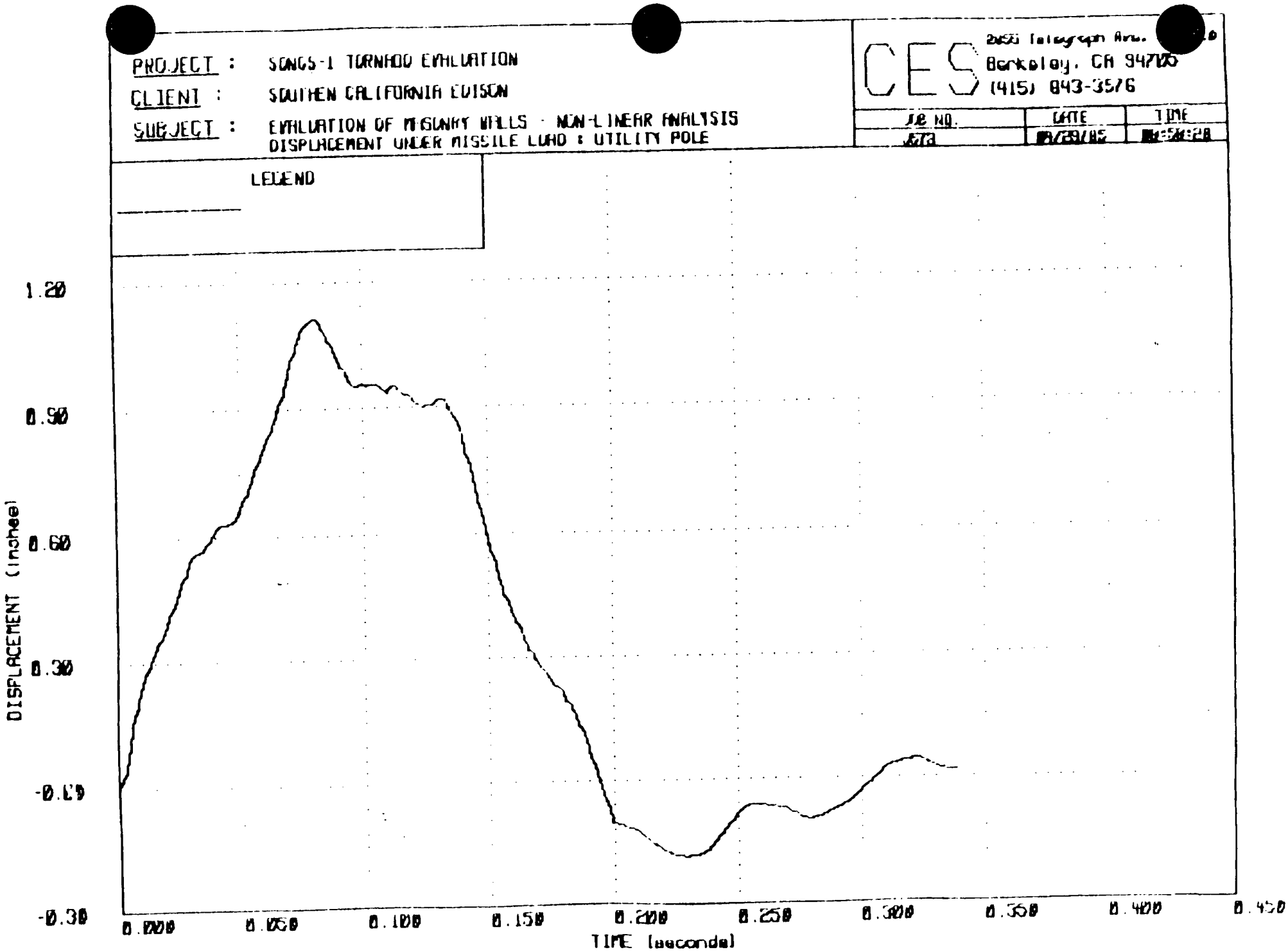


FIGURE 9 : RESPONSE TO 40 KIP LOAD 0.006 SEC. DURATION

6.3.2 Impact in Grouted Regions

In the grouted regions of the wall an upper bound force due to impact may be obtained by using formulas such as the modified NDRC formula which was developed for concrete surfaces. This is likely to overestimate the penetration resistance of the wall in these regions and therefore provide an upper bound force level for impact.

Note that the overestimation of resistance will be a conservatism as this will be used solely to assess the integrity of the walls under maximum forces. The separate problem of residual velocities of missiles passing through the walls will be governed by the previous case, impact in an ungrouted region.

The modified NDRC formula has been applied to two missiles, a steel rod at 229 ft/sec and a utility pole at 152 ft/sec. These are based on a tornado wind speed larger than that likely for SONGS-1 and will provide an upper bound to impact speeds.

By using the modified NDRC formula the steel rod would penetrate the bond beams approximately 3.5" and the utility pole would perforate them. For the steel rod it was assumed that the velocity reduced linearly from impact velocity to zero in a distance of 3.5". This implies a constant deceleration of 2792g over a time of 0.0025 seconds and therefore a maximum force of 2792 times the rod weight of 8.8 pounds, i.e. 24.6 kips.

A load of 24.6 kips over 0.0025 seconds is less severe than the most critical load for the ungrouted area of the wall as computed in the previous section and therefore the steel rod would not be a governing case.

The utility pole would have a penetration distance of 8.5", greater than the wall thickness of 7.625". For this load, a residual velocity of 62.7 ft/sec was computed (compared to 152 ft/sec impact velocity). It was assumed that the velocity decreased linearly over the wall thickness from 152 to 63 ft/sec, providing a constant deceleration of 467.6g. For an 1120 lb pole this provides a total force of 523.7 kips over a duration of 0.0059 seconds, the time taken to penetrate the wall.

A load of this magnitude, 523.7 kips, would produce a stress in the grouted area of over 3.5 ksi, obviously much higher than could be resisted by shear in the grout. Therefore, an upper bound to the maximum load may be computed beyond which the missile would perforate the wall. If it is assumed that the maximum punching shear stress is 179 psi and the shear area (two horizontal grouted blocks plus two grouted vertical cells) is 148.6 sq. in. the punching shear capacity is 26.6 kips. Added to this is the tensile strength of the four rebars which would have to fracture to allow perforation, 49 kips. The total upper bound strength is then 75.6 kips. This value would need to be refined for the individual SONGS-1 walls and some allowance made for dynamic overstrength but it is an approximate upper bound load.

To examine the global effects of such a load on the wall a dynamic analysis was performed using a 80 kips load applied as a rectangular pulse for 0.006 seconds on the example wall. In accordance with the effective width calculation the damaged area of the wall was assumed to include the loss of one vertical cell and one horizontal cell, reducing the effective width of the wall to 7.3'. The model was therefore modified to have properties (i.e. steel area and masonry face shell area) equivalent to a 7.3' width of wall in the region from 4 feet above the floor to 12 feet above the floor. Outside this zone the properties corresponded to an effective width of 13 feet.

The load of 80 kips corresponds to a uniform load of 365 psf, over 4 times the static load capacity. The response is as shown in Figure 10. Maximum displacement is 4.0 inches, about two-thirds the displacement limit imposed by the acceptance criteria.

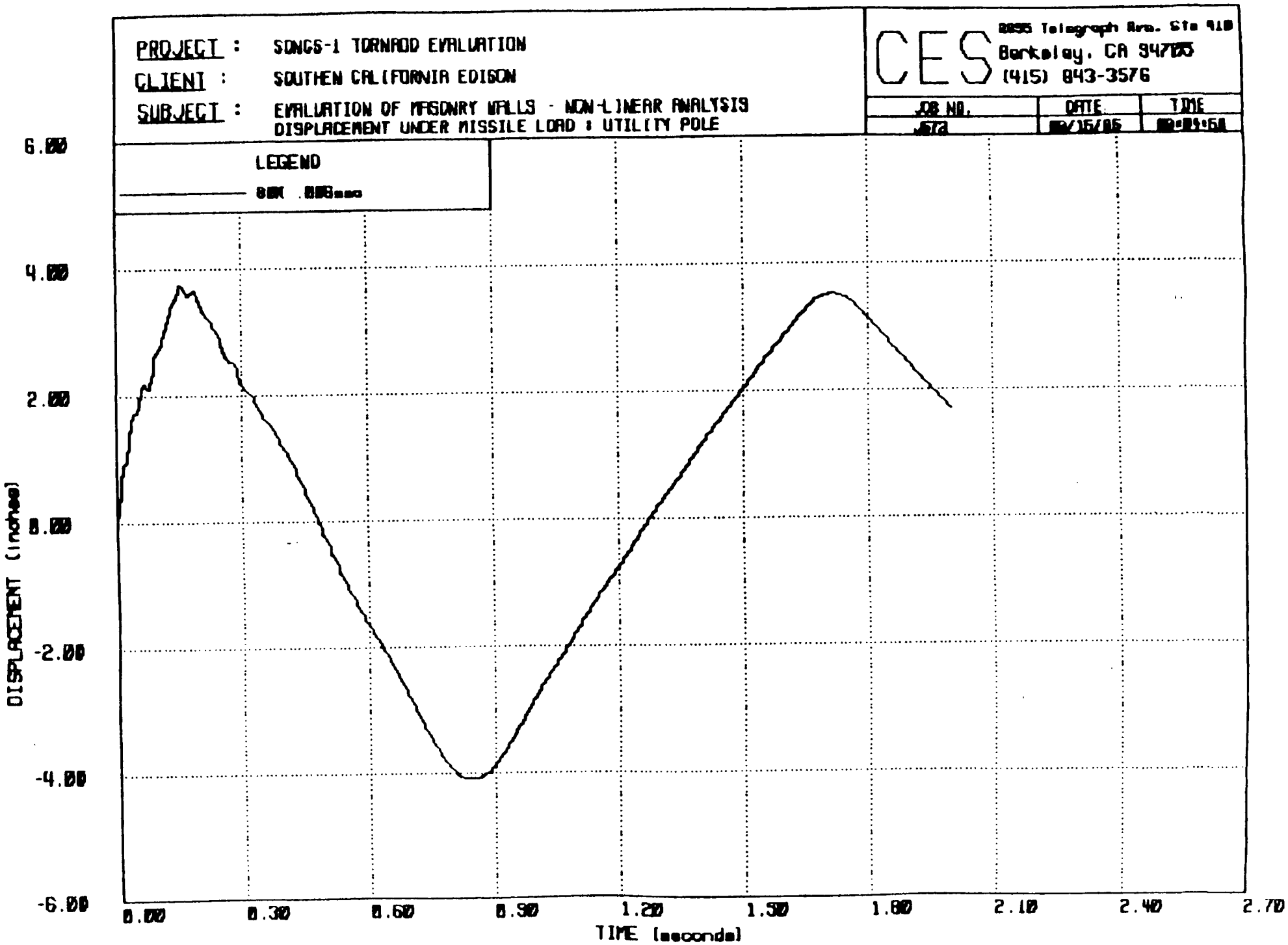


FIGURE 10 - RESPONSE TO 80 KIP LOAD 0.006 SEC. DURATION

7 COMBINED WIND, PRESSURE AND MISSILE LOADING

The Tornado Resistance Design Review Criteria (Reference 1) require that the following three combinations of tornado loading be considered for any structure or component:

1. $W_t = W_p$
2. $W_t = W_w + W_m$
3. $W_t = W_w + W_m + 0.5\ W_p$

where W_t is the total tornado load, W_w is the tornado wind load, W_m is the tornado missile load and W_p is the tornado differential pressure loads.

Section 4 has provided a procedure for evaluating the effects of wind and pressure loads which produce a uniform load over the area of the wall. The missile impact loads are dynamic point loads which are evaluated using the methodology described in Section 6. For load cases 2 and 3 above the uniform and point loads must be evaluated simultaneously. For response beyond the elastic limit the results cannot be superimposed and therefore a commined analysis is required where advantage is to be taken of the ultimate strength criteria.

The equivalent static load has been shown to produce satisfactory results for uniform wind and pressure loads but would grossly overestimate the effects of impact loads applied over a very short duration. Therefore the combined load case would be analyzed dynamically be combining the load specifications described in Section 5 for wind and pressure loads and in Section 6 for missile impact loads.

For a detailed wall evaluation, the steps to be followed for a tornado with a given probability of occurrence would be as follows:

1. Determine wind and pressure loads, allowing for shape factors, venting etc as appropriate.

2. Determine the missile characteristics for the appropriate missile subset, i.e. mass, area and velocity of impact.

3. For each missile, determine the maximum force level in the wall considering impact in the grouted and ungrouted regions.

4. Determine the most critical location(s) on the wall for missile impact, taking into account:

- Loss of wall strength for impact in grouted region.
- Loss of support capacity for impact at the top or bottom of the wall.

If the wall strength is more critical than the support strength (as for the example wall) the critical location will most likely be impact in the grouted region at the intersection nearest to mid-height of the wall.

5. Analyze the wall for the combined uniform plus point load.

6. Evaluate the wall in terms of the criteria.

8 EXPERIMENTAL DATA

The methodology presented herein is based on the available ductility of centrally reinforced masonry walls. This ductility has been demonstrated to exist in a number of static test programs, as discussed in Reference 3. Since Reference 3 was issued two further test programs have further demonstrated the available ductility, the SONGS-1 seismic tests and the ACI-SEASC lateral load tests.

1. SONGS-1 Seismic Tests

This test program was performed at the request of the NRC and so the NRC staff are familiar with the results. These tests demonstrated the available ductility and the capacity of the walls to maintain their integrity at displacement levels up to 12" under seismic type loadings.

2. ACI-SEASC Tests.

The ACI-SEASC Task Committee on Slender Walls tested a number of masonry and concrete walls under applied uniform face loadings (Reference 5). Of these tests, 3 were on 8" reinforced concrete masonry walls 24'-0" high. The load deflection curves obtained for these walls are reproduced in Figure 11. The maximum deflection in these walls was 14.5" and in none of the specimens did failure occur. Following are features of these tests pertinent to the SONGS-1 evaluation:

1. The loads were monotonically applied pressure loadings, similar to that for tornado.

2. Walls were of similar height to the maximum height of the SONGS-1 walls.

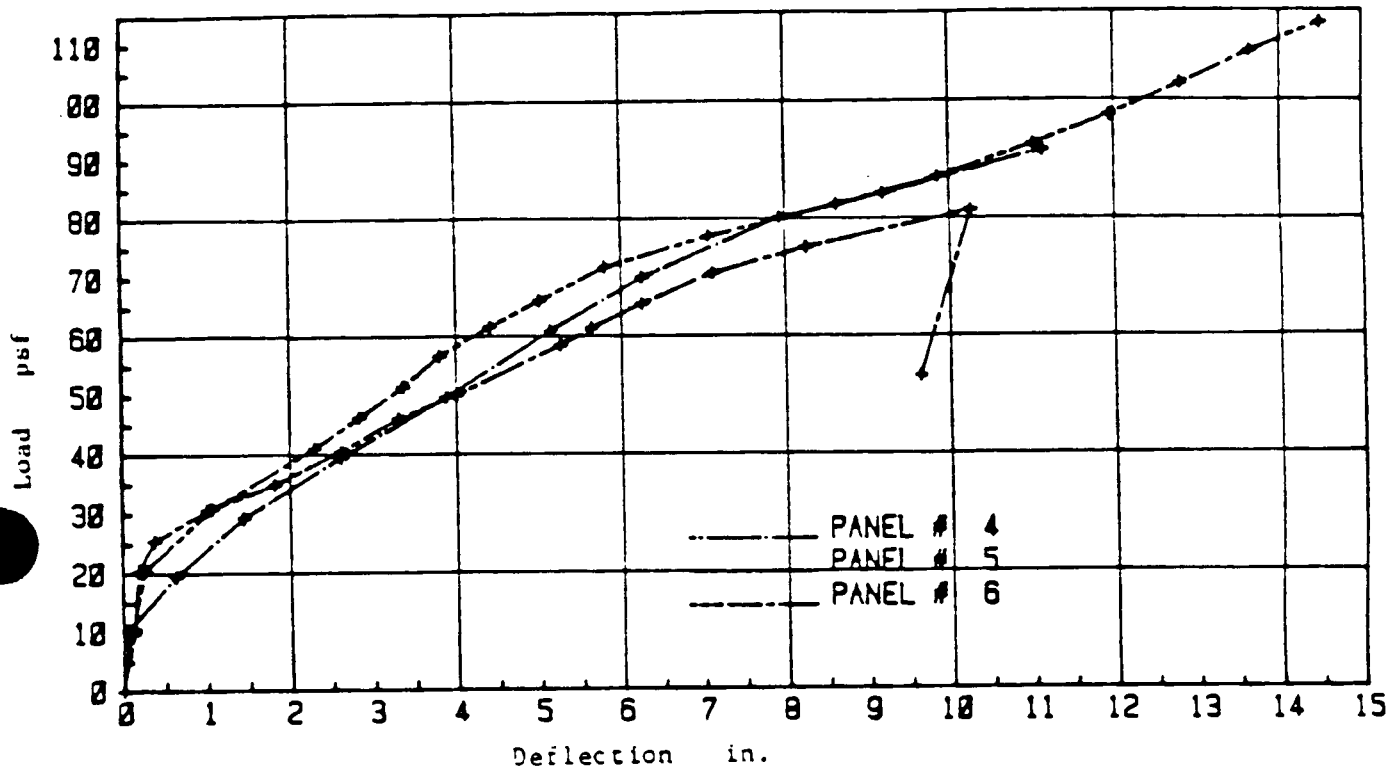
3. Based on allowable stresses as given in the SONGS-1 criteria, the allowable loads on these walls would be 52 psf. The actual yield load was 75 psf and ultimate load in excess of 110 psf.

4. The ACI-SEASC specimens were pinned at the base rather than moment connections as at SONGS-1.

Therefore, the increase in load between formation of first and second hinge was not present (i.e. a mechanism formed with the first hinge). Because of this the ratio between ultimate and allowable stress loads would be expected to be less than for SONGS-1. In fact, the ratio was 2.12, only slightly less than the 2.5 ratio computed for SONGS-1.

5. The ACI-SEASC reinforcing used higher strength, less ductile steel than the SONGS-1 construction. Even with this, no steel fracture or face shell distress occurred during the tests.

Therefore, the ACI-SEASC tests may be used as confirmation that increases over allowable loads do occur in reinforced walls as predicted by the analytical methodology.



Load - Deflection Curves, 8" Concrete Block Masonry

FIGURE 11 : LOAD DEFLECTION CURVE FROM ACI-SEASC TESTS

9 PRELIMINARY ASSESSMENT OF SONGS-1 WALLS

The example wall analyzed was a 16'-10" high wall with #5 rebars at 32" centers vertically. This does not correspond to any of the SONGS-1 walls. However, the overall response of this wall may be used to provide a preliminary assessment of the maximum tornado loads which the SONGS-1 walls are likely to be able to resist.

The example wall was able to resist a uniform pressure load of 85 psf. 2.5 times the pressure of 34 psf computed on an allowable stress basis. The increase in maximum loads at SONGS-1 could be expected to be similar and therefore the walls have been assessed to determine the maximum loads based on 2.5 times the allowable stress load.

For SONGS-1, representative walls would be 24'-0" high walls with #7 bars at 32" (Ventilation and Fuel Storage Buildings), 20'-0" high walls with #5 bars at 32" (Turbine Enclosure walls) and 16'-0" high walls with #5 bars at 48" (Reactor Auxiliary Building). For each of these wall types a preliminary assessment of the maximum wind speed has been made for two assumptions:

1. Assuming full wind load (shape coefficient of 0.8) plus one half of the differential pressures load, i.e. assuming that there is no venting.

2. Assuming full wind load (shape coefficient of 0.8) only, i.e. sufficient venting such that the differential pressure does not occur.

Under these two conditions the maximum wind speeds are as follows:

(1) With Differential Pressure

24'-0" wall	79 psf	146 mph
20'-0" wall	60 psf	127 mph
16'-0" wall	63 psf	130 mph

(2) No Differential Pressure

24'-0" wall	79 psf	196 mph
20'-0" wall	60 psf	171 mph
16'-0" wall	63 psf	175 mph

With wind load plus differential pressure these maximum loads correspond to a tornado with a probability of approximately 10^{-6}. If the differential pressure is neglected the wind loads correspond to a probability of 10^{-7} These probabilities however are for wind and pressure only and do not include missile effects. Therefore capacities would be reduced below these values. The extent of this reduction in indeterminate until the specific missiles are defined and combined analyses performed. Based on the the studies performed the reduction would likely be such as to increase the tornado probabilities listed above by one order of magnitude.

10 SUMMARY

Criteria and an analytical methodology for the tornado evaluation of the SONGS-1 masonry walls have been presented. The criteria are based on the development work performed for the seismic evaluation as is the method of analysis. It is shown that the maximum loads based on an ultimate condition are approximately 2.5 times the values obtained by the allowable stress approach. To obtain these load increases requires a method of analysis which traces the non-linear load deflection curve for the wall through cracking and yielding and which identifies points at which criteria limits are reached.

Example dynamic analyses have shown that for moderate levels of non-linearity no amplification over static results occurs. However, for very extensive non-linearity dynamic amplification may increase responses dynamic effects should be considered on a wall-by-wall basis as part of the evaluation.

For global missile effects means of developing maximum force levels have been presented for use when the SONGS-1 site specific missiles are defined. The maximum forces are based on upper bound wall capacities in both the ungrouted and grouted regions. Example analyses have shown that missile forces would be sufficient to perforate the wall and produce load levels several times as high as the static load capacity of the wall. However because of the very short time of occurrence of the missile loads the dynamic load factor is much less than unity.

Final wall evaluation requires a combined dynamic analysis for wind, pressure and missile loads. Until this is performed the final wall strength cannot be determined. However, based on the results obtained in example analyses performed to date the ultimate strength criteria appear likely to demonstrate wall capacities to resist tornadoes of approximately one order of magnitude lower probability of occurrence than would be obtained using the allowable stress criteria.

11 REFERENCES

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