STATEMENT OF CHARLES STOKES

1. My name is Charles Stokes. I am providing this statement, at the request of Mr. Joel Reynolds, for his submission to the Atomic Safety and Licensing Appeal Board. My statement rebuts the Pacific Gas and Electric Company (PGandE) defenses of the seismic design review as submitted to the Appeal Board as an attachment to an affidavit of R.C. Anderson, M.J. Jacobson, M.E. Leppke and L.E. Shipley dated March 5, 1984.

2. I have had only five days to prepare this statement, because I was occupied studying for and taking the Alabama bar examination. As a result, I could only analyze a sample of PGandE's responses, and my own points are illustative rather than exhaustive. I am only reluctantly submitting this statement now because Mr. Reynolds informed me that otherwise it could not be included with his reply brief in the licensing proceeding.

3. On balance, even a selected review demonstrates that PGandE's analysis cannot withstand scrutiny. In some cases there simply is not enough supporting documentation, explanation or analysis to support the conclusions offered. In other cases the conclusions are based on false statements and inaccurate assumptions.

4. Due to the severe time constraints, I have not been able to rebut all of the specious PGandE positions, nor to review and re-verify my own analysis to my satisfaction. Therefore, after reviewing, modifying, and supplementing this analysis, I will resubmit my statement as a notarized affidavit.

5. In the Introduction to the February 7, 1984 PGandE Letter No: DCL-84-046, under "2. Nature of Concerns," it is stated in Paragraph (b) that "discrepancies are of a minor nature and, when revised calculations or analyses were performed, all of the piping and supports fully met the licensing criteria and commitments." I have two questions in response: (1) How can PGandE be so sure that the above statement is true when in Paragraph (a) they admit that "discrepancies have been found in the small bore piping design work"? (2) Were the effects of torsion accounted for? The calculations that I performed, including torsion, failed about 50% of the supports (these have been redone; was torsion removed?), and a co-worker, in his affidavit, says that he was not allowed to include torsion. (*See* attached Affidavit (Exhibit 1).) He was a member of the Unit 1 team that is performing the present review. I will volunteer to review with the NRC a sample of the 110 supports recently reviewed by PGandE, both computer and hand-calculated.

6. In reply to PGandE's conclusion that there is no reason to believe similar concerns exist outside OPEG, it should be noted that OPEG was not very different from the home office of Bechtel or Westinghouse; all were under pressure to produce to meet schedule. If OPEG had problems with document control, how can one conclude without looking at the home office that it didn't have this problem also since the overall management was the same? After PGandE's long string of calculation errors, I question whether PGandE has now reviewed the calculations correctly. PGandE must demonstrate through a full review that

the calculations were based on the controlled documents listed below. Specific responses and challenges to illustrative issues are:

NRC Question: (Allegations 55 and 79, SSER 21) Gaps to reduce thermal loads (p. 5):

7. From PGandE's response, it seems obvious that they have reviewed thermal effects with blinders on. In a plant subject to seismic excitation, the only reliable anchors are those such as wall penetrations (which can vibrate depending on location) and anchors attached to walls, floors, or ceiling core or steel (they, too, can vibrate depending on fixity of structure). In effect, no anchor should be assumed completely in reduction of thermal load. For example, a large bore pipe is considered an anchor due to relative size. However, unless the large pipe itself is anchored close to the small pipe branch line, its location cannot be relied on over the life of the plant in establishing the thermal gaps to reduce loadings to other supports. To illustrate:

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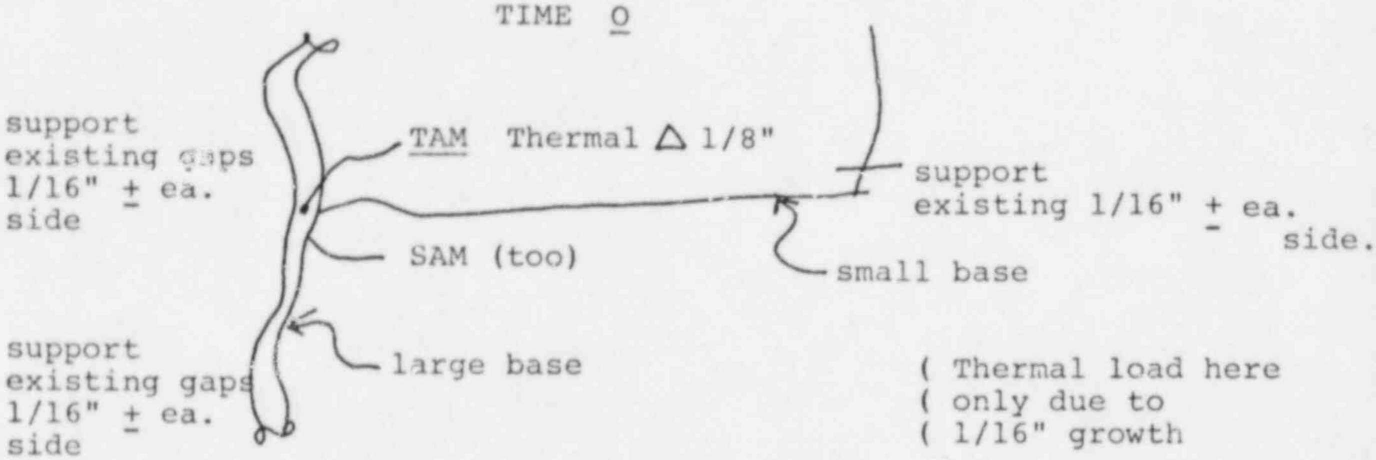
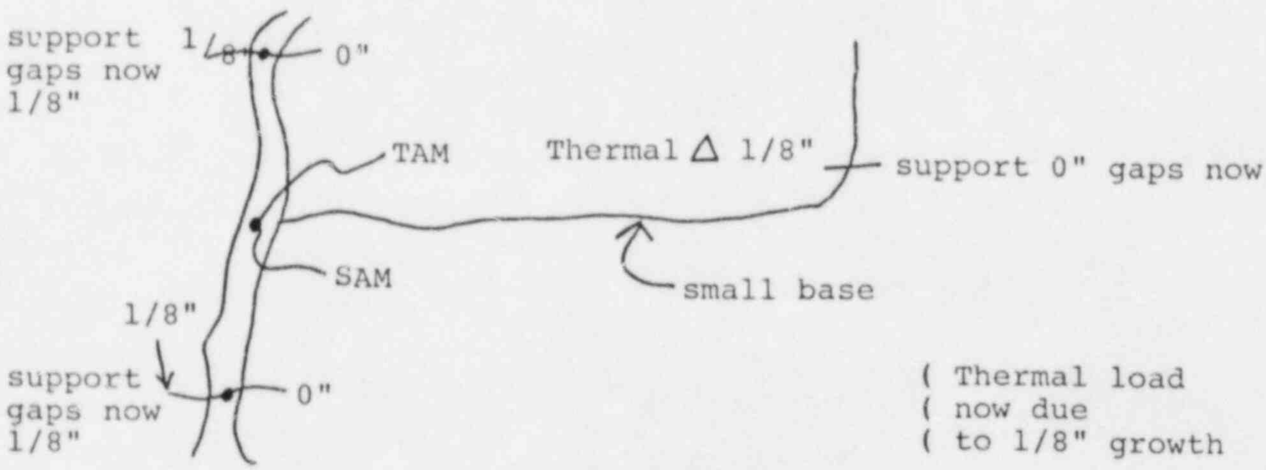
TIME O



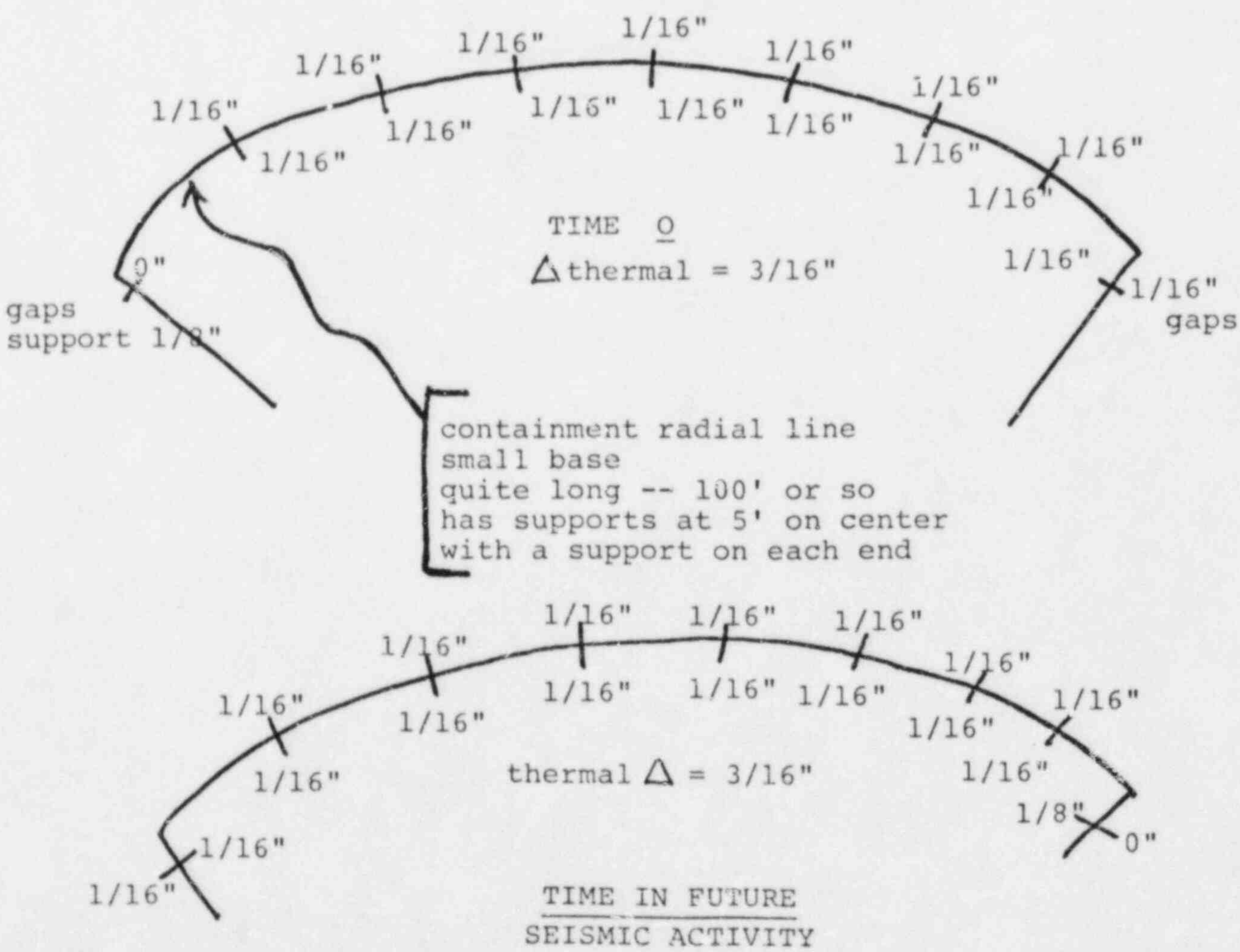
SAM (Seismic Anchor Movement)

TAM (Thermal Anchor Movement)

Time in Future
Seismic Disturbance



8. Depending on total conditions, use of gap may not be valid. Either it should not have been used to relieve load to small bore supports, or after every seismic disturbance these supports subject to increased load should be reviewed for gap and movement changes in location of TAM of large bore line. Also a similar effect occurs on a small bore line with a restraint on each end. If this line is quite long and the hold line is shifted due to seismic activity, the support at each end could be subjected to larger than designed-for loads. Example:



9. This may not be the worst case scenario. At time 0 the plant is cold. As it heats up, the line expands uniformly. This is because friction on the supports grows from the middle of the radial line out and produces balanced loading on each side of the center point until one end grows enough to encounter a restraint. The first end to hit is the right side after 1/16" growth; then this support in effect pushes or is pushed against by all friction loads on all supports as the line grows in the other direction of freedom. Time 0 on the right end is subjected to the sum of all friction force developed by internal supports. At some time in the future, during cold shut-down for refueling, a seismic disturbance occurs and the line shifts position. Expansion occurs as it did at Time 0, only now the left end is the restraint.

10. When gaps are used to relieve thermal load, there are certain requirements. I have never seen the load considered this way, with unequal placement or uncentered placement of the line in relation to gaps. The general assumption is that if there is 3/16" thermal expansion and there are a total of 3/16" gaps, then there is no thermal load to any support. This is not a conservative analysis, and I question whether or not the cases hypothesized above have been considered in the stress calculations and the resulting loads given to the support group.

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NRC Question: (Allegations 55 and 88, SSER 21) Use of different stiffnesses for the same rigid support in static and dynamic pipe analysis (p. 8):

11. Here, too, is the assertion that "these loads are derived from two totally different loading phenomena, one static (thermal), and one dynamic (seismic)." The same questions and concerns are raised here as in the use of gaps to reduce thermal loads. PGandE continues to state that after re-performing analysis that the licensing criteria are met. I question why a different method was used for their systems initially if a problem did not exist. These new analyses should be reviewed in depth by an outside party.

NRC Question: Computation errors and modeling deficiencies (p. 9):

12. I challenge the first line in PGandE's response, concerning engineering judgments. Many of the so-called "engineering judgments" were not those of the individual engineers performing the calculations, but were suggestions made by group leaders who claimed to only want to see how the hypothesized change would affect a support that had been failing to meet design requirements. Although they told those doing the work that the suggestion would not be used, when the results came back and the stresses were now acceptable, the engineer was either pressured into signing, or the calculation was given to another engineer who did not question the method used and just signed it off. I was given supports to perform the analysis; when I demonstrated that a support was deficient and returned it

to the group leader after it had been checked, I found that another engineer was performing the same calculation from scratch. This happened to other engineers also. Although the group leader acted surprised when the engineer discovered the same suggested calculation being performed by another engineer, to my knowledge every person in the Unit 1 squad from November 1982 to March 1983 was aware of this happening. In retrospect, I realize that this multiple assignment of the same support occurred so frequently as to be intentional. I remember one time the same support was assigned to three engineers simultaneously by the same group leader, only to be discovered near completion of all three. Due to the number of supports that I was finding on a preliminary basis to be inadequate, I felt that the reason for the multiple assignments was to see which results were the most favorable to passing the support. The others were thrown in the garbage can. This conclusion is based on the fact that no calculation package includes more than one original design.

13. In STRUDL modeling, possible errors by the engineer involved things like Beta angles, which were required to orient the members correctly; the determination of the proper Beta degree to be used in the model for structual steel angles; and especially unequal leg angles. Another modeling problem was that some engineers omitted the joint eccentricities where members are welded together. This could decrease the stresses, since by the omission of these eccentricities the torsional loads were reduced. Another problem in using STRUDL and hand calculations was the determination of "Ky," "Kz," "Lg," "LZ."

15. Another problem involved the evaluation of torsional stresses on the members of the supports. Some engineers use the "Torsional Analysis of Rolled Steel Section," published by Bethlehem Steel, which evaluates both the warping normal contribution to bending stress and shear effects. I am not sure where Bethlehem got the procedure, but the same method is developed in "Bending and Torsional Design in Structural Members" by C.P. Heins, published by Lexington Books (copyright 1975). I should note that the necessary projection for angles is not included in the Bethlehem data, nor is it completely developed in Heins' book. But the necessary factors can be found in other texts or calculated using analysis similar to that for structural channel shape in Heins' book. I used this method, and with the added shear stress and bending stress, many angles exceeded 1.0 in the interaction equation. The other method of torsion evaluation came from a book entitled, "The Design of Welded Structures" by Omer W. Blodgett, published by Lincoln Welding Foundation, in Section 2.10: "Designing for Torsional Loading." (See Exhibit 5.) This method is limited to shear stress. Some problems occurred between Table 1 (Torsional Properties of Various Sections) and Table 4 (Torsional Resistance of Frame and Various Sections). Table 4 was sometimes used incorrectly. Another problem with this method was that on page 2.10-8 the equation $\tau = \frac{Tt}{R}$ was used without considering equation $\tau_{max} = \tau(1+\frac{t}{4a})$. The $\frac{t}{4a}$ would have resulted in substantial increase to resulting stress if it were considered.

(See attached excerpt from STRUDL Manual (Table 14.1 -- Parameters used by the 1963 and 1969 AISC Codes (Exhibit 2)) and Quan (ECA) Memorandum to AISC Code Check Users (Exhibit 3).) These were almost never correct.

14. Other problems were common in both computer and hand calculations. The first resulted from the load case form. (See attached Stokes' Loading Cases for Hanger Form and HP 41C Program (Exhibit 4).) Two problems came out of this: (1) Teams of two were established early in the Project where one member checked the other's work and vice-versa. The individual teams resolved between them the correct way to fill out this form. Through discussions with other teams, we discovered that almost all had a different interpretation. On other design jobs, the checking was randomly assigned, so that the group inter-related and merged in practice. (2) The second problem was that typically all Load Cases A across were input to STRUDL or used in a hand calculation. In fact, there are more Load Cases (i.e., 1, 2, 3, 4, 5) than just A and B. In the case of an anchor support where FX, FY, FZ, MX, MY, MZ are filled in for cases A and B, adding all possible combinations of A and B under Case 1 will result in 36 possibilities. This number was never analyzed; it was only an assumed worst case. Had anyone analyzed this, he would have lasted at best a month before being dismissed for production reasons. The significance of this is that no one can guarantee that each support was verified adequately, except for the most simple cantilever single load (FX) or (FY) or (FZ).

16. Many times an angle would not pass with only the shear calculation per Blodgett. Since the Bethlehem method was more involved than Blodgett's, I resorted to a two-step analysis. I checked the angle using Blodgett and if the interaction was .75 or above, I would then check it using the Bethlehem method -- including the effects of warping normal (bending stress) contribution. This usually would exceed the interaction value of 1.0 and fail the angle. Other engineers did not do this because of management policy. (<u>See</u> Exhibit 1.) Other engineers and I felt that angles should be checked per AISC Section 1.5.1.4.66 for unbraced length. However, we were not allowed to nor was any method given to compute a reduced bending stress allowable.

17. It is impossible to determine whether an error originated with the designer by looking at the type of error. The engineer should be asked if his engineering judgment was used or whether it was a suggestion from a supervisor. I believe an additional cause of discrepancies was suggestions by the supervisors.

18. [illegible] age 11, Paragraph 3, PGandE concludes: "The fact that when the discrepancies were addressed the supports were accepted without modification substantiates the adequacy of the design process." It is my understanding that support No. 100-132 or another support did fail after being corrected. It is noted that six supports have not been finished. These could include the one that failed and continues to be analyzed.

19. It is also stated that "the methods and criteria were not modified for this evaluation." This implies to me two

possibilities: (1) all errors that have been found may still exist, and (2) things not included in the past still are not included, as described in Exhibit 1 and my earlier disclosures. I volunteer my services again to the NRC in reviewing a sample of the 110 packages.

NRC Question: (Allegation 88, SSER 21) Snubbers located adjacent to rigid restraints being inoperative (p. 16):

20. PGandE states that "It has been industry practice to ignore the dead bands when performing seismic analysis." I agree. However, generally, industry and manufacturer recommendations and good engineering practice also require that a snubber would not be used unless pipe movements required it and would not be placed close to a bilateral support unless it allowed sufficient pipe movement for the snubber to operate. In all plants and projects where I have worked, a snubber would usually be used with (a) a rigid support in one direction and snubber in the other direction, or (b) snubbers in two directions.

21. In addition, when using a snubber near a one-direction rigid support, close attention would be given to how the snubber and rigid restraint interfaced. In other words, a snubber would not be placed on the side of the rigid restraint where the pipe movement would cause the snubber clamp to hit the rigid restraint and restrict the axial movement. Most engineers issued the two packages (snubber and rigid) to the field together. Also, both packages would note that one should be considered in relation to the other on installation to prevent

interference problems.

22. Drawings on other projects and the old drawings on Diablo Canyon included the snubber movements so that someone in the field could catch any installation interference problems (Note: Originally in Unit 1 work, we included this data, but when someone decided it was unnecessary we were instructed to remove all movements.).

23. In no case would I use a snubber when the thermal displacement in that support direction was less than 1/16", which is typically an industry-used value. <u>Had these requirements been written into M9, there would be few dead band problems at Diablo Canyon</u>.

24. I have three concerns: (1) Why were these snubbers placed so close to bilateral supports and anchors? (2) In all cases where a snubber does not activate, was the stress analysis for that load case redone omitting the snubber? (<u>See</u> Snubber Displacement Chart (Exhibit 6).) (3) Has anyone reviewed the records to determine what was installed first: the snubber, the rigid restraint, or the anchor?

25. I think PGandE's summary of attachments is worth restating in different terms. Seven of fifteen snubbers do not lock up under Design Earthquake (DE) displacement, six of fifteen snubbers do not lock up under Double Design Earthquake (DDE) displacement, and four of fifteen do not lock up under Hosgri (Hos) displacement. Is it possible that 46% of all snubbers in the Plant are unnecessary? How much money will be wasted due to (1) engineering design, (2) material,

(3) construction, (4) re-evaluation, (5) removal, (6) possible risk to workers to perform removal if the plant is in operation?

NRC Question: (Allegation 89, SSER 21) Improper resolution of pipe interferences (p. 21):

26. When I was in Quick Fix for Unit 2, I deleted a support that was in the process of being installed when a Pullman field engineer brought this problem to my attention. Upon a visual inspection of the line configuration and support proximity, I questioned the necessity for adding a support at that location. I placed the support on hold for 24 hours until I could check with the stress group to see why it was being added and whether it was necessary. Upon locating the stress engineer, I was told that the pipe was resting on a piece of unistrut and that ME101 would not allow a dead load seismic restraint and that a support had been modeled in. This support was unnecessary, as loads to all supports were in the neighborhood of 10 pounds. The stress engineer should have requested the removal of the unistrut or its movement, so as not to interfere with the pipe. However, upon discussion, he agreed that it could be removed and told me the stress analysis would be corrected, and I agreed to void the design through Quick Fix to prevent its being installed.

27. In the last line of its response, PGandE states that "it would appear that this situation demonstrates good communication between Construction and Engineering, sound engineering practice, and a proper solution that resulted in a system that meets the design criteria." In fact, this "proper

solution" occurred only at my initiative, and I was later laid off for taking these kinds of initiatives. This kind of response cannot be assumed for other cases.

NRC Question: (Allegation 79 and 88, SSER 21) Calculation of the load-carrying capacity of small bore piping support (p. 22):

28. PGandE states that "All final calculation packages are retained and permanently filed. There is no regulatory or other project requirement to retain the intermediate or interactive analyses." However, 10 CFR 50.34(b)(4) provides that "A final analysis and evaluation of design and performance of structures, systems, and components with the objective stated in paragraph (a)(4) of this section and taking into account any pertinent information developed since the submittal of the" "Preliminary Safety Analysis Report (PSAR)" (emphasis added). The problem is that PGandE's and Bechtel's final documents at Diablo Canyon ignore pertinent information developed in the design verification review. Vital data was not taken into account, incorporated or even referenced in the final calculations. It just disappeared. Consistent with standard industry practice, one would expect to find a steady progression to a more detailed, more technical, more expert calculation. This is in fact Bechtel's procedure or standard in practice at other plants, even though it may not be stated in writing.

29. Having worked in the nuclear industry with and for Bechtel, I can describe the company's and the industry's standard practice for the history of a support analysis. First, there is a preliminary calculation by the design engineer. He

may approach the problem using several proposed designs. These may be based on his knowledge and creativity or on others' knowledge and creativity obtained through discussions. In any case, a final approach is decided upon and calculations are completed by him. This analysis is then given to a checker (an independent reviewer). He will check technical points, Code sections relied on, math, ease of construction, and cost competitiveness compared to an alternative. He either agrees with the results as they are or suggests changes and returns the package to the design engineer. The design engineer then reviews the checker's comments. He may not agree, and then the designer and checker will have a discussion, usually coming to a mutual understanding. After the calculation is complete to the satisfaction of both the designer and checker, they sign it and the package is given to a supervisor for review and approval. Sometimes the supervisor (who should have greater experience) will ask for a complete redesign. The designer and checker then redo the calculation, sign it, and return it to their supervisor. He signs it. After his signature, the preliminary calculation becomes a final calculation package.

30. Later, new loads may be imposed due to a mistake being discovered. The calculation is then reviewed to see (1) if it is still acceptable; or (2) if it will require modification. The calculations described above are necessary as a basis for subsequent modifications. Even if the loads are of a preliminary nature, no need arises to remove the calculations showing non-compliance with Codes. In those cases, final loads can be compared against preliminary failing loads that are used

to determine if the support requires modification. In review of final laods against preliminary loads, in many cases an engineer need only compare loads and reduce previous calculated stresses as a percentage reduction of load. In others, the results may not be so easy and an engineer may redo some or all of the calculations. When doing a later review for load changes, many engineers do not review a previously checked calculation if in the past it was passing. However, if the previous calculation was failing, a complete review of the calculation would be necessary to see if errors had occurred that might be corrected and cause the support to pass before modifying it. (See attached example calculations on hanger 100-132 R-1 by both Gary Katcher and G.R. Shaw (Exhibit 7).)

31. With respect to Exhibit 7, I would like to make several points. PGandE stated that they have sharpened their pencil to prove the supports adequate now, even though they failed under preliminary loads. A careful comparison of the calculation of Gary Katcher and that of Shaw is instructive. Mr. Katcher's STRUDL model is considerably more detailed than Shaw's: (i) The cover sheet demonstrates that Katcher's version was performed before Shaw's; (ii) Katcher's includes more pages than Shaw's. Note on Katcher's three-sheet Summary his finding that base plates and anchor bolts failed; Shaw's didn't. Note also the sketches in Katcher's drawing that show the detail to which he resorted in investigating in the field the true configuration; Shaw used Katcher's sketches. Compare Katcher's load sheets load point by load point to Shaw's. They are identical. Both loads are the same, not more advanced as PGandE

has claimed to the NRC. Finally, compare calculations; Katcher's is more detailed than Shaw's.

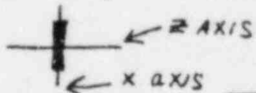
32. The only conclusion to be drawn is that Katcher sharpened his pencil while Shaw dulled his, unless the later model was a suggestion by his group leader to omit eccentricities or to introduce various other management-imposed inaccuracies. Also, I believe Mr. Katcher's work is a good, typical example of all the unused failing calculations that PGandE has admitted to throwing away.

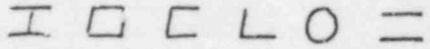
33. This comparison contradicts a number of statements made by DCP personnel at the December 15, 1983 meeting with the NRC Staff. One example: "[We] use more sophisticated techniques, more advanced techniques to see if it is possible that more detailed, more thorough, more sophisticated analysis can show that the pipe and supports in its existing configuration is acceptable." (Transcript, p. 9.) This was a false statement. Similarly, on page 11, second paragraph: "Even the logic of an implication that we intentionally mislead is faulty." "For anyone to suggest that we would risk all of this effort to save a support on a half-inch liner to keep from modifying a support on a 3/4 inch line is ludicrous in my mind." It would be ludicrous to me, as well, on the above-stated premises. But it was not so ludicrous if the modification would exceed the percentage determined by the Diablo Canyon Project to require expansion of the sample and thereby cause delay in the start-up of Unit 1. I have been told by supervisors that the cost per day to PGandE during any non-

operation amounts to about a million to a million and a half dollars.

<u>NRC Question: (Allegation 88, SSER 21) Joint releases for rigid connection (p. 23)</u>:

34. PGandE states that "no joint is completely 100% rigid." This is true for the figure 100%, since the loads transferred to a joint cause strains that stretch the material making up the joint. From any basic strength of materials or structural design text this can be shown. However, in many such texts, designs are postulated that for all practical purposes are 100% fixed.

35. In many instances, the joint is modeled so that no moment resistance is offered by the steel to which the member is attached. In structures, these connections would be, for example, column to beam with angle clips. However, in pipe supports, almost all joints are designed as moment connections, at least to carry the moments induced and calculated at the joint. Also, to my knowledge the only joint that would qualify for a moment release in any direction is a single line weld about the axis along its length. It would still have 2 moment resistance. Example: 

$M_x = \underline{0}$ $M_z \neq \underline{0}$.
All joints configurations such as  and others should not have joint releases used. Some computer programs allow that factor as an input for the joint, but these usually are no lower than .6 or 60% fixed. PGandE's response does not resolve the allegation or explain the use of joint releases for rigid connections.

NRC Question: (Allegation 85 SSER 21) U-Bolt allowables (p. 24):

36. Although PGandE's response mentions only section NF-3260 of ASME Section III, the section NF-3260 includes sections 3261, 3262, 3262.1, 3262.2, 3262.3, and 3262.4, and U-bolts come under sections NF-3261, NF-3262, and NF-3262.4 (component standard supports as defined in section NF-1214). Also relevant is section NF-3226.1 -- "Bearing Loads" -- which states, "(a) The average bearing stress for resistance to crushing under the maximum load, experienced as a result of design loading, test loading, or any seismic loadings, except those for which Level D limits are designated, should be limited to yield stress (Sy) at temperature, except that when the distance to a free edge is larger than the distance over which the bearing load is applied, a stress of 1.5 Sy at temperature is permitted." (Emphasis added.) (See ASME Section III, NF-3226.1, 3260 et seq. (Exhibit 8).) This section in effect requires that the support to the pipe not exceed the recommended bearing stress level.

37. I believe an accurate calculation would be based on the obvious premise that at the point of loading the pipe to the U-bolt, only a point contact occurs. It is obvious that any load applied on a point will have an infinite stress, which will cause the U-bolt to fail under this section. In B31.1, I should note Section 102.3.1(B): the "allowable stress values in bearing may be taken at 160% of tabulated value." Even this section will dictate that a U-bolt not be used.

38. In ASME section NF-1241.1 "Types of Component" -- standard supports are listed (U-bolt is not listed). Shoes,

lugs, rings, clamps, slings, straps and clevises are listed. These load-transmitting hardware typically have common characteristics. They are form fitting and all have width. They all spread the load over a larger area of pipe than a U-bolt. I understand that the use of U-bolts by many in the industry is justified on the grounds that they offer a simple installation of a cheap component. However, their use at Diablo Canyon is not supported by local bearing stress calculations. Note that even a component supplied as a catalog item should be chosen by the stress engineer to comply with all requirements of the Code selected as the design basis, whether B31.1 or ASME Section NF. I know many engineers fail to check bearing stress. At other plants, after I raised this point, management decided to replace U-bolts or pad load area so that bearing stress was acceptable.

39. In Paragraph (1) of its summary of conservatisms, PGandE states: "The test loads used in the equation of NF-3260 represent the lowest tension and side test loads found for 1/4 in. and 3/8 in. diameter rod U-bolts, respectively." To illustrate my point, results are summarized below from the U-bolt test data sheets:

1. Pipe Size 1/2"
   Rod 1/4"
   Force at .025 Displaceemnt:

   Run 1 = 1700 lb. (which failed)
   Run 2 = 2600 lb.
   Run 3 = 3500 lb.
   Run 4 = 2300 lb.
   Run 5 = 1800 lb.

2. Pipe Size 3/4"
   Rod 1/4"
   Force at .025 Displacement:

   Run 1 = 1900 lb.
   Run 2 = 900 lb.
   Run 3 = 1300 lb.
   Run 4 = 2000 lb.
   Run 5 = 1900 lb.

3. Pipe Size 1"
   Rod 1/4"
   Force at .025 Displacement:

   Run 1 = 4000 lb.
   Run 2 = 2700 lb.
   Run 3 = 1900 lb.
   Run 4 = 3100 lb.*
   Run 5 = 1800 lb.** (1700 at .025")

   * Run 4 stopped for safety reasons
   ** Run 5 carried to .0265 in.

40. Thus, the low tension values are as follows: 1/2" ϕ pipe was 1800 lb. at .025"; 3/4" ϕ pipe was 900 lb. at .025"; and 1" ϕ pipe was 1700 lb.; 3/4" ϕ pipe with 1/4" rod size tension load using PGandE's failure point of .025 in. shows that the lowest failure is 900 lb. force. Inserting this as TL in Equations NF-3262.4 Level A Limits Load Ratio = TL x 1.0 (S or Fall/Sy) using S = 32.8 at 200°F KSI SU = 65KSI, the load rating for this U-bolt would be 454 lb. per Dwg. 049243 Sh 26 1/2 pipe 1/4" bolt tension load case 1&2 = 2000 lb. In short, PGandE exaggerated the strength by over four times.

41. PGandE's response does not explain how the data in the U-Bolt Test Program became 049243 Sheet 26, nor does it prove that the results are conservative. (*See* attached Sh 26 of 049243 and 1/4" Rod Data Sheets (Exhibit 9).)

42. I am at the disposal of the NRC for an in-depth look at the U-Bolt Test Program and the results of the data.

NRC Question: (Allegation 95, SSER 21) Angle-shaped structural members (p. 25):

43. I am reviewing documents supplied to the NRC and will also do my own research on references in this country to which I will submit an in-depth statement on applicability of Australian papers and any available U.S. information. My contention is that the use of a nuclear plant as a proving ground for a new design is not in the interest of public safety. As a licensed professional engineer, I believe the use of this information is premature until the profession in this country is able to assimilate and verify its reliability for the unique conditions at a particular nuclear plant, such as Diablo Canyon.

NRC Question Sample Size utilized for reverification (p. 28):

44. I was told, as were others, that a sample based on 5000 feet of pipe would be examined to justify the design of 25,000 feet; and that if 5% of these 5000 feet failed to meet criteria, then all 25,000 feet would have to be reviewed. Also, we knew of the generic categories of THERMAL, Seismic Anchor Movements, and Thermal Anchor Movements code break and active valves.

45. PGandE states in Paragraph 2 of its response that the sample was 5000 feet for 25,000 feet and, in Paragraph 4, that it later changed to end up with 5000 feet for 15,000 feet. This contradicts the statement that "the initial sample selected in the fall of 1982 remained the 'sample' throughout the small bore reverification program." It appears evident that when supports failed in the sample and justified a complete review of all

supports, PGandE reclassified those problems as generic rather than admit the need to review all supports.

46. If this statement is true, on the other hand, then the supports in the sample which were reviewed as generic should still be considered as sample supports. In that case, approximately 40% of the sample failed. This figure is based on the fact that the sample was used to justify 25,000 feet of piping originally, which later was reduced to 15,000 feet. The difference here, 10,000 feet, would have been determined to be generic. 10,000 feet divided by 25,000 feet is equal to 40%. I have told the NRC that I was failing about 50% of the supports. I believe the difference, 10%, may be due to the inclusion of torsion in my calculations. In any case, under these circumstances, the review program must be expanded to a full review of the additional 15,000 feet. The results of the work done are further in question, since the NRC Staff reported that nine out of twelve packages that they reviewed were unacceptable, due to one or more errors.

47. In conclusion, since time does not permit a complete rebuttal to PGandE's response, I would like to make one last point. In all of the responses I have read, no detailed calculation was included demonstrating that the issue raised through a specific example of a support has been accurately resolved and is no longer a problem. Based on previous practices and the false statements that I can identify through personal experiences, these responses cannot be accepted without a verifiable public record of supporting data. I would like to see copies of specific support calculation packages that I will

identify, with notes of problems originally discovered. These may be placed on public record, so that the quality of engineering work at Diablo Canyon can be reviewed by other interested parties.

47. I would like to restate that I am available to discuss with the NRC any of the issues relating to the subject matter of this Statement, to any earlier affidavits, or to any other matters concerning Diablo Canyon of which I am aware.

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I have read the above 25-page statement, and it is true, accurate and complete to the best of my knowledge and belief.

Charles C. Stokes

CHARLES STOKES

Dated: March 14, 1984.