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April 1993

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Investigation of Probable Causes of Cracking, Aircraft Weather Shelters, Kadena Air Base, Okinawa

by *Lillian D. Wakeley, Wayne G. Johnson, Patrick T. Harrington*
Structures Laboratory

WES

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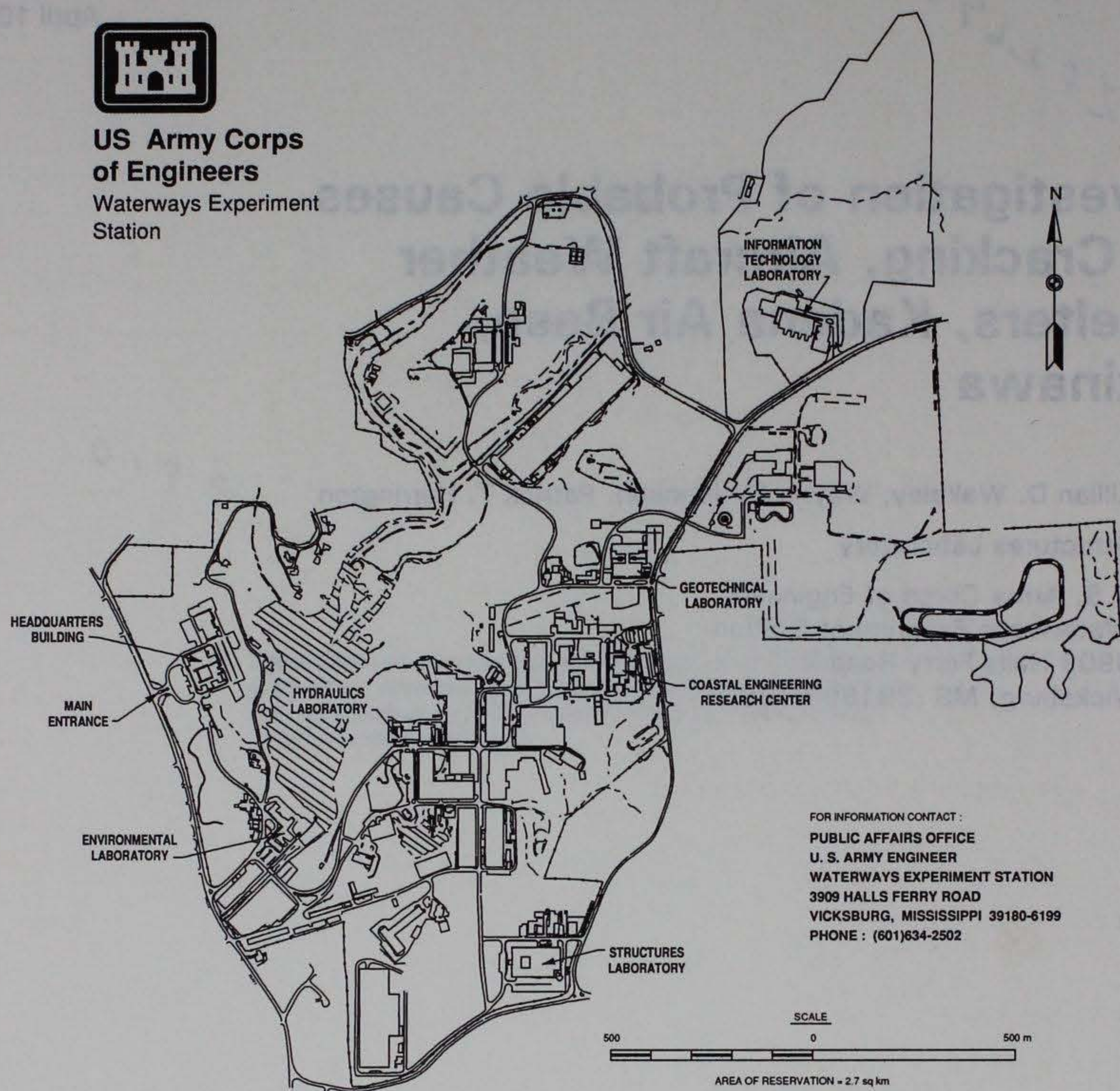
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**US Army Corps
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Preface

This report was prepared at the Structures Laboratory (SL), U.S. Army Engineer Waterways Experiment Station (WES), under the sponsorship of the U.S. Army Engineer District (USAED), Japan. Mr. Jim Cox, Chief, Construction Division, was point of contact for the USAED, Japan.

Dr. Lillian D. Wakeley, Concrete Technology Division (CTD), was the Principal Investigator. MAJ Patrick T. Harrington, CTD, was Project Coordinator. Mr. Wayne G. Johnson, Structural Mechanics Division (SMD), conducted the static and dynamic analyses and modeling. Others at WES who contributed to the study include Dr. Robert Hall, SMD, Mr. J. Pete Burkes, and Ms. Judy C. Tom, both CTD. This report was prepared by Dr. Wakeley, Mr. Johnson, and MAJ Harrington.

Mr. Kenneth L. Saucier is Chief, CTD. Mr. Bryant Mather is Director, SL, and Mr. James Ballard is Assistant Director, SL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Leonard G. Hassell, EN.

1 Introduction

At the request of the U.S. Army Engineer District (USAED), Japan, the U.S. Army Engineer Waterways Experiment Station (WES) investigated the causes of cracking in roof slabs of aircraft weather shelters at Kadena Air Base, Okinawa. Cracks had been observed on the upper surface of the roof slabs soon after the structures were built. With time, some of these cracks propagated through the slabs, appearing on the lower surfaces particularly near the valleys of the folded-plate structures. WES initiated studies to determine whether the cracks were related to materials, concrete mixture proportioning, field practice, static or dynamic loads, or some combination of these factors. Figure 1 shows a row of the shelters. Appendix A includes figures showing the underside of roof slabs and the geometry of beams and columns.

Sources of Data

The forensic investigations conducted by WES included data or information from the following sources.

- a. Discussions with Mr. Bruce Swafford, CEPOD, about his observations of the subject shelters and study of a video tape he provided showing cracks in the concrete structures with his commentary.
- b. Examination of 13 cores from these shelters, using petrographic and other forensic analytical procedures, to determine if cracking was attributable to concrete materials or mixture proportions or to chemical alteration of materials after placement. This work was requested specifically in a memorandum for Dr. Lillian Wakeley, Structures Laboratory (SL), WES, dated 4 Mar 92, from LTC Larry Talley, USAED, Japan (Okinawa Area Office). The subject of the memorandum was Petrographic Analysis Support. WES issued a Preliminary Findings Report following these laboratory studies (17 Jul 92; Appendix B).
- c. Study of Architect Engineer (AE) design reports, specifications, concrete test reports, as-built drawings, and crack maps as input to assessment of possible materials and structural contributions to cracking.

- d. Preliminary calculations to investigate possible structural sources of the observed cracking or foundation deficiencies.
- e. Review of the data with Mr. Jim Cox, USAED, Japan, during his visit to WES on 16 Apr 92. Mr. Cox provided more information about construction practices and other technical data during this visit.
- f. A site visit by WES researchers to Kadena Air Base during late July 1992. This trip included collection of visual information for the shelters, collection of active impact data using a calibrated hammer, and collection of "ambient" data for one bay of the structure. The latter phase included excitation of Bay 30 of the structure by an F-15 jet parked inside with the engine running (three tests) and taxiing of a jet through the structure (two tests). These activities were described in a trip report, included as Appendix A.
- g. Analysis of data collected during the July 1992 site visit and calculation of fundamental frequency of structures.

Organization of Report

The following chapters describe the factors considered as possibly having contributed to concrete cracking: Chapter 2 is a study of the concrete as a material, its components and proportions, physical appearance and properties, and any evidence for chemical alteration. Chapter 3, foundation and design factors, includes possible contributions by unusual loads and structural design and reinforcing. Chapter 4 focuses on the interplay of concrete materials properties with design and reinforcing. Chapter 5 presents an analysis of the possible effects of various static loads. Chapter 6 and Appendix A describe the dynamic tests conducted during the site visit.

2 Concrete Materials and Condition

The 13 cores received at WES for petrographic and forensic study were taken from roof slabs of shelter Bays 42 and 35. The core numbering system and brief descriptions are given in Table 1. The cores represented both cracked and uncracked portions of roof slabs as well as areas where water ponded on the roof. WES researchers requested cores from the ponded areas to compare with those from nonponding areas of the roof slabs. These provided samples of concrete with varying likelihood for chemical degradation to have occurred.

Physical Appearance and Properties

Cores were photographed and visual descriptions were recorded before the cores were subdivided for petrographic study or destructive tests. All cores were 102 mm long (4 in.), from 130-mm- (5-in.) thick roof slabs, so none of the cores represented the full depth of the slabs. Six cores were selected for detailed physical and chemical study: three from each bay, both with and without cracks and reinforcing. Cracks were not directly associated with reinforcing; that is, some cracked cores included no reinforcing, and some cores that included segments of reinforcing were not cracked. Reinforcing was corroded in only one core, where one reinforcing bar and one wire of the reinforcing fabric crossed the same crack. Cracks were roughly vertical in each core; that is, they ran approximately perpendicular to the slab.

Entrapped air voids were visible in the concrete of all cores. Three cores showed what appeared to have been water pockets on the undersides of aggregate particles and reinforcing strands. One core showed apparent segregation of aggregates, with no coarse aggregates in the upper 25 mm (1 in.) of concrete. The upper (finished) surfaces of two cores from Bay 42 had what appeared to be a topically applied coating. The distribution of these features is summarized in Table 2.

In all cores, the uppermost strand of reinforcing encountered in the core is 51 mm (2 in.) or more below the upper surface. The welded wire fabric, where present, is at least 25 mm (1 in.) up from the bottom of each core and, therefore, was 51 mm (2 in.) or more up from the lower surface of the slab. This evidence shows that depth-of-cover requirements were met. However, this arrangement puts the two layers of reinforcing less than 25 mm (1 in.) apart, and in the middle of the slab, given that roof slabs were only 130 mm (5 in.) thick.

Design strength for the concrete was 27.6 MPa (4,000 psi). Cores tested by the Pacific Ocean Division Laboratory all met this strength requirement (information provided by Mr. Swafford), so WES did not conduct tests of compressive strength.

Materials and Proportions

The average value for cement content (American Society for Testing and Materials (ASTM) C 1084 (1991c), six cores) was about 346 kg/cu m (584 lb/cu yd), which is higher than the 330 kg/cu m specified. Calculated values for cement content are given in Table 3.

The coarse aggregate was crushed limestone. Fine aggregate included crushed limestone and apparently metamorphic rock fragments. A coarse aggregate solid volume of 58 percent was called for in the approved mixture proportions. During our observations of slabs cut longitudinally from cores, we noted less coarse aggregate present than we expected to see in a structural concrete. The coarse fraction was missing from the upper 30 mm (3 cm) of core 920129 (Bay 35, ponding area). In all cores studied, coarse aggregate volumes calculated from point counts (ASTM C 457 (1991a)) were lower than the value provided in information about mixture proportioning. For the cores studied at WES, the volume of coarse aggregate ranged from 30.8 to 43.5 percent (Table 4).

The average value for volume of permeable pore space or voids in hardened concrete (ASTM C 642 (1991b), six cores) was 14.5 percent. This is higher than published values (13 percent or less) for volume of permeable voids of similar concrete with water-to-cement ratio (w/c) = 0.5 (Whiting 1988) and suggests that the w/c may have been higher than 0.5. Channels and large voids attributable to bleed-water collecting under reinforcing bars and coarse aggregates are visible (Figures 2 and 3) in three cores (Table 2). In addition to the 2.5 to 4 percent entrained air, up to 3 percent entrapped air was present (Table 4), and much of it was easily recognized large voids. The intended air content of the freshly mixed concrete as indicated by the approved mixture proportion was 4 percent. The core with the most obvious water channels along reinforcing bars also had the largest percentage of voids (by ASTM C 457 (1991a), > 7 percent), the most capillary porosity (observed during microscopy), a low cement content, and was cracked (Bay 35, core 118).

Chemical Alteration or Degradation

A combination of forensic laboratory techniques was used to study the microstructure and phase composition of the concretes. These techniques included polarized-light microscopy of petrographic thin sections, scanning electron microscopy (SEM) with energy-dispersive X-ray chemical analysis, and X-ray powder diffraction to identify crystalline phases present in the paste portion. The purpose of this effort was to determine if the concrete had been affected by deleterious chemical changes while in service. We looked for: presence of weak or expansive crystalline phases in the cement-paste portion of the concrete; evidence of attack by chlorides or other ions associated with wet coastal environments; carbonation along cracks or elsewhere; chemical interaction with aggregates; association between reinforcing bar locations and microcracking; and evidence of any other chemical degradation process.

Our study revealed no evidence that deleterious chemical reactions had caused or contributed to cracking. The crystalline phases present were those expected from normal cement hydration. The only unusual feature of the cement-paste portion of the concrete, studied in thin sections and by SEM, was the presence of a large amount of finely divided mineral matter ("fines"), as shown in Figure 4.

Aggregate surfaces were unreacted. We found only the expected background level of chlorides in paste, at aggregate surfaces, and along cracks. Ettringite, a phase that can cause cracking if it forms in a restrained condition, was minimally present on free crack surfaces (except where cracks were carbonated, as explained below); however, it was not in an amount or configuration to indicate it had caused the cracks to form. Ettringite appeared to have crystallized on free surfaces of preexisting cracks (Figure 5) and was not present in open pores in the paste (Figure 3).

Reinforcing strands were corroded where they were colocated with cracks but not elsewhere. From this we conclude that the reinforcing was not corroded before being used in construction. The corrosion observed was minimal, thus indicating that it occurred along cracks that had been initiated by some other mechanism and that the corrosion did not cause cracking.

The pattern of carbonation revealed more about the cracking than did any of the other techniques. Open crack surfaces were carbonated from the upper surface of the slab down to a depth of about 30 mm (1.3 in.) (Figure 6). From the surface to this depth, cracks had stepped carefully around coarse aggregate particles. Below 30 mm (1.3 in.), crack surfaces were not carbonated, and the cracks wandered both around and through coarse aggregate particles (Figure 7). Relative to the rate of crack propagation, carbonation was a slow process. Thus, the cracks have been open longer at the top of the slabs where the surface is carbonated and are younger downward.

The relationship between cracks and aggregates indicates the same trend. If the upper part of the slab cracked soon after placement, the paste would have been weaker than the aggregates; therefore, only the paste cracked.

Later, the concrete had gained strength, so that the strengths of the paste and aggregates were similar. Cracks then propagated without differentiating between them, leading to the lower zone of cracking through both paste and aggregates.

Crack Mechanism Indicated by Materials

It is likely that cracks were initiated early in the life of the structures by drying shrinkage. The following is background information about this crack mechanism, which is explained in American Concrete Institute (ACI) 224.1R-3 (1992). The volume of hardened portland cement changes with changes in moisture content. The combination of moisture-caused volume changes and restraint of the concrete--in this case, probably by beams and columns--causes tensile stresses to develop, and this can initiate cracks. Cracks then may propagate at much lower stresses than were required to initiate them.

Evidence for Drying Shrinkage Cracking

Several lines of evidence support the hypothesis that these cracks probably were initiated by drying shrinkage. According to field personnel, cracks were observed on the upper surfaces of roof slabs within the first few weeks to months after concrete placement. Cracks started appearing on the lower surface sometime later. During the study it was confirmed by petrographic techniques that cracks in the cores are older at the top (toward the upper surface of the slab) and younger into the depth of the slab. This is consistent with cracks having been initiated by drying shrinkage and later propagated by other stresses.

Additional evidence for probable drying shrinkage cracking is offered by mixture proportions. The proportions of the concrete mixture used in this construction made it susceptible to drying shrinkage cracking. The concrete was pumped upward in each column to encase a preexisting steel structure, and it appears to have been proportioned for ease of pumping. The same concrete mixture then was used to cast the beams and slabs in place. Specifically, the concrete in the cores of the study had: a relatively high fine aggregate content; fines in the paste portion; a w/c of 0.5 or higher; less coarse aggregate than was indicated in mixture proportioning information provided; and small coarse aggregates (apparently 19-mm (0.75-in.) maximum size). All of these factors made the concrete more pumpable, while at the same time making it more susceptible to shrinkage cracking. An increase in slump, from 7.62 cm (3 in.) to 10.16 cm (4 in.), was allowed. As indicated by field personnel, the roof slabs were cured using wet burlap.

Another line of evidence for drying shrinkage having initiated the cracks is the crack locations. Most cracks are located where the concrete would experience large stresses during curing: parallel to the column line and closer to the

trough than to the peak of each shelter roof. They initiated on the upper surfaces of the slabs, and these surfaces were not restrained by beams and columns.

Susceptibility to Drying Shrinkage

On the subject of concrete being susceptible to drying shrinkage, ACI 224R-41 (1992), Section 8.6.3 says in part:

...too often, to expedite pumping, the actions taken are those which increase drying shrinkage and resultant cracking: more sand, more fines, more water, more slump, smaller aggregate.

The concrete used in the aircraft weather shelters had all of these characteristics. Pumping *per se* is not harmful to fresh concrete. Large amounts of high-quality, even high-strength, concrete are pumped worldwide every day. However, factors that affect the long-term serviceability of a structure should be considered as important as those that make fresh concrete easy to pump.

3 Design Factors and Reinforcing

In the initial considerations of the causes of cracking, WES included the foundation of the structures as a possible contributor. However, the pattern of cracking in the shelters does not correspond to patterns known for foundation problems. There is no evidence of cracking of the base slab or cracking in the fire walls between certain bays of the structure, as would be expected if the foundation were deficient. Further, the cracking pattern is reasonably uniform over the entire site. Foundation problems generally result in localized structural problems. Following discussions with Mr. Cox and study of AE design documents and maps of crack patterns, it was concluded that further consideration of foundation characteristics would not be helpful. During the initial studies (before the site visit), the following structural factors were considered as possible contributors to cracking.

Unusual Load Condition

Discussions with area engineers revealed that the structures have been subjected to significant wind loads, both from daily fluctuations and from at least two typhoon-class storms between the time the structures were completed and the time of our study. However, the apparent wind speeds were smaller than the values used for design. Ambient conditions at Okinawa also cause structures to be exposed to daily temperature fluctuations of up to 27 °C (50 °F). Both winds and temperature changes could contribute to frequent load reversals.

Another source of nonconstant loads is the vibrations associated with jet engines as the planes move in and out of the shelters. The presence of cracking prior to use of the shelters guarantees that these vibrations cannot have initiated the cracks. However, these vibrations were considered a likely cause of propagation of cracks after they had been initiated by some other mechanism. Thus, they were the principal subject of our field measurements, described in Chapter 6. They were relatively small but significant enough to excite certain modes of vibration in the shelters to a level significant enough to record. Information collected does not suggest that the engine vibrations

are causing any significant damage to the structures, as is discussed in more detail further into this report.

Structural Design

A review of the design for location of reinforcing in the roof slabs and beams revealed no significant design deficiencies, although several questions were raised during the review. The design was reviewed by the original AE contractor (after construction) and by design engineers at USAED, Japan. For this reason, a thorough design review was not conducted by WES. AE responses to questions raised by USAED, Japan, about the design appear to have been adequate and reasonable.

Our main concern about the design is the use of thin slabs in association with deep girders at the column head. This is not an unusual practice for flat structures, but the folded-slab configuration complicates the load distribution for the aircraft shelters. The continuous slab design of the shelter roofs is analogous to a continuous highway bridge. This configuration causes the thin slab at the top of the section to be in direct tension throughout its depth. This situation exists because the neutral axis of the beam/slab system lies beneath the bottom of the slab. For this reason, any cracking due to flexure of the roof would be expected to be perpendicular to the plane of the slab and extend through it. As described in Chapter 2, cores taken through cracks in the slabs revealed that the cracks are approximately perpendicular to the plane of the slab. This gives credence to the argument that propagation of the cracking is primarily the result of flexure of the continuous-slab configuration.

Distribution of Reinforcing

Several changes were effected during construction of the weather shelters, after cracks were noted in the first nine shelters. The presence of a bend in the top steel layer of the roof slab at the column head contributed more flexibility to the joint of the roof slabs with the column support area. This detail was delineated as a problem in the inspection report of 1 Sep 89, a copy of which was provided to WES. When it was judged that these bends contributed to cracking in the first nine shelters, the bends were eliminated, and additional steel was added to the slab at the supports for the remaining structures. This is not considered to be a dangerous situation; however, the roof slabs for these first nine structures are inherently more flexible than the slabs for the newer structures due to the combination of this detail plus less total reinforcing.

After elimination of the bend and the increase in total reinforcing in the slabs, cracking still occurred. In general, the cracking became less severe and the cracking pattern changed somewhat after the modification. Figure 8 gives a comparison of cracking geometry for two bays of the structure as supplied to WES by USAED, Japan, personnel. It shows a distinct difference in

cracking geometry following placement of additional steel and elimination of the bend just described. Although this is a limited comparison, it appears that in the newer structures (with additional steel) the cracks have been forced farther up the slope. In some cases, the direction of the cracks has changed from parallel to the valleys to perpendicular to the valleys. This points to the flexibility of individual slab "panels" in the structure, but for the comparison given in Figure 8, this could also be the result of boundary conditions. Bay 35 has one "free" edge and one continuous edge while both edges of Bay 43 are continuous. Individual panel flexibility is brought about by the large width-to-thickness ratio of the panels (minimum of approximately 22:1) and the placement of steel with respect to the panel depth.

As implied previously, another concern is the placement of steel relative to the thickness of the roof slab. Cores taken from the slabs confirmed that the steel layers were separated by a small distance relative to the slab thickness. The required cover depth of concrete was the driving factor for the steel placement design. The requirement was a 51-mm (2-in.) cover depth for the top layer of steel and a 38-mm (1.5-in.) cover depth for the bottom layer of steel. This depth of cover is good practice in coastal environments to deter corrosion of the reinforcing. In the thin roof slabs, however, the cover specifications resulted in an effective separation of less than 25 mm (1 in.) for the two layers of reinforcing steel. Although the slab was designed for this value and the design conforms to standards (thus, it is safe), this configuration results in flexible individual slab panels within the beam framework. This flexibility appears to have contributed to the cracking (Chapter 4).

Further evidence for the above argument is found in the changes in crack orientation following addition of steel at the column heads. In the original nine structures, almost all cracking is parallel to the valleys of the structure and is located near the valleys. Further, the cracks do not appear to extend across the girder areas. This points to excessive stresses in the individual slab panels brought about by negative moment loads at the column heads. The girders are sufficiently reinforced to resist cracking; however, the slab panels appear to crack due to direct tension over the column heads resulting from negative moment at these points (and possibly exacerbated by reversing loads). In the newer structures, it appears that many of the cracks have changed orientation, which indicates that the additional steel may have been adequate to handle the direct tension over the columns but not adequate to handle individual panel loads (probably dominated by wind). No additional steel was added in the orthogonal direction, thus cracking occurred in the weaker direction. For Bay 35, the change in crack orientation at the free edge could also be partially attributable to boundary conditions (Figure 8).

4 Interactions Among Concrete and Design Factors

As presented in Chapter 2, the concrete used in construction of the weather shelters was susceptible to cracking by drying shrinkage. Cracking of this type is likely to be distributed more or less uniformly over all of the slabs, as is the case here. Similar damage in virtually all bays suggests a problem with each individual slab (a slab is considered to be one-half of the roof for an individual bay). Drying shrinkage cracks are unlikely to extend the full depth of the slab. Given that the cracks propagated over time, extending through the slabs sometime after the cracks were first observed, it is probable that some other factor(s) contributed to crack propagation.

As discussed in Chapters 2 and 3, the most likely two factors contributing to cracking are concrete drying shrinkage and structural design. Either factor could explain the longitudinal cracking pattern in the valleys of the shelters. However, the available information implies that the cracks extend through the depth of the slabs. This is unlikely to have resulted from drying shrinkage alone and is most likely explained by shrinkage coupled with tension due to flexure of the slab/girder system.

One point of concern with the hypothesis that cracking was initiated by shrinkage is the presence of diagonal cracks at the corners of the slabs. This, at first glance, would not appear to be attributable to shrinkage. In general, this would likely be due to a structural design problem. However, no structural deficiency that would have led to this type of damage is obvious. Further, the fact that the cracking was displaced farther to the interior with the addition of diagonal reinforcement tends to provide evidence that diagonal cracking could have been initiated by shrinkage of the concrete.

The structure is very stiff in the direction parallel to the valley at the elevation of the valley. It is also very stiff in a direction transverse to the valley (across the gable) due to the frame system. The structure is less stiff in the longitudinal direction (parallel to the direction of entrance/exit) at the peak of the gable. Thus, shrinkage would first be expected to produce cracks parallel to the longitudinal direction at the peak or the valley and transverse to the longitudinal direction (up the gable) near the valleys. If the structure were

equally stiff in the longitudinal direction at the peak, shrinkage cracks would be expected up the gable slopes at one or both ends. However, since the structures are not as stiff at the peaks, it appears that the diagonal cracks are more or less the result of the transition from the stiff valley to the more flexible peak. The fact that the cracks, once begun, propagated with time and perpendicular to the plane of the slab is a strong piece of evidence supporting this hypothesis.

Not all of the damage can be attributed to cracks initiated by drying shrinkage. There is evidence of some cracking on the underside of the girders near the peaks. Tension in these areas would occur if the slabs were subjected to reversing loads.

5 Static Analysis of the Structures

Before the site visit, a simple finite element model was developed at WES to investigate possible structural weaknesses that could have contributed to the observed damage. The model consisted of only the roof portion of a two-bay system. It encompassed beams to model the beam/girder system coupled with shell elements to model the slabs. Linear-elastic material properties were assumed. Figure 9 shows the beam system with node numbers. A fine grid was necessary to define the problem. The beam portion of the model consisted of 548 beam elements. The slab portion consists of 960 four-node shell elements (Figure 10).

Seven loading scenarios were selected, based on our experience with failure analysis for other structures. These options (Cases 1 through 7 in the following discussion) were intended to reveal whether or not the cracking pattern could be related directly to a particular type of loading on the structure, ignoring nonstructural (i.e. materials) contributions to cracking. The cases selected dealt primarily with movements of structural support and included:

- a.* Static loading only.
- b.* Lateral movement of one edge away from the center.
- c.* Twisting of one exterior wall.
- d.* Settlement of a line of columns at an exterior edge.
- e.* Settlement of an exterior column.
- f.* Settlement of a line of columns along the valley.
- g.* Settlement of an interior column.

The model identified areas of greatest tensile stress and most likely cracking, assuming various displacements of up to 25 mm (1 in.) at certain nodes.

Case 1 is the simple static condition. It revealed larger stresses perpendicular to the valley on the top face of the shell elements and also near the peaks (Figure 11). The crack map of the Kadena shelters (as shown in Figure 8) indicates a stress distribution similar to that predicted by this case, which gave the best fit of those we considered. Stresses along the valleys of the real shelters probably are even greater than those indicated by this case, due to the presence of the flat segment joining the two slopes of adjacent bays (the model has a v-shaped valley).

Case 2 models an outward rotation of an exterior wall, which is a type of sway response. This model gives a good match for the observed cracking pattern. However, given the connected arrangement of the shelters, each bay is highly restrained from sway. Given that cracks are present in virtually all bays, this type of response probably does not explain the cracking. Likewise, Case 3 models twisting of one exterior wall of a two-bay system. This type of twisting would result in localized stresses, with cracking along the peak at one end of a bay and in the valley at the opposite end. Again, the structures show no evidence of having experienced this type of movement, given their crack distribution and connected arrangement. We considered the possibility that jet loads on the structures might produce this type of response. However, field measurements taken during our site visit (described in Chapter 6) imply that routine traffic loads are not large enough to produce stresses of this type.

Case 4 depicted a differential settlement problem. We judged this scenario to be unlikely given that floor slabs are not cracked and the damage is too uniformly distributed. Cases 5 and 6 also resulted in highly localized stress patterns, unlike the crack pattern of the shelters. Case 7 is similar to Case 5, giving highly localized foundation movement. The movement implied would involve several columns in a row, again giving a probable crack pattern that is not consistent with field observations.

Figures showing the stress patterns that would be generated in the two-bay system by these seven load scenarios are on file at WES and are available on request.

6 Dynamic Analyses and Field Measurements

WES also investigated dynamic properties of the shelters, looking for evidence that structural integrity or performance might be jeopardized by the cracks (and to verify the mathematical model). These investigations included calculations for the same two-bay model described before as well as investigation of one slab panel of the roof. Results of these calculations were compared with data from active tests: that is, collected during the site visit. Results from the model compared favorably with data taken during the active tests (the model was appropriate to the structures). In the following discussion, the term "global" refers to the entire two-bay structure (or its model equivalent), and "local" indicates only a single panel (or model).

Global Model

Calculated fundamental frequencies for the global model are relatively small. Figure 12 shows the lowest three calculated frequencies. Since it was not practical to collect data for the entire two-bay structure corresponding to the global model during our site visit to Kadena, the dynamic investigation concentrated on the local response for individual slab panels. Thus, the frequency range of interest was higher than would have been investigated for the entire structure.

There was evidence of one or more of the lower global modes present in some of the local data collected from a single slab, as is indicated in the low-frequency portion of the power spectrum shown in Figure 13. Excitation of the first global mode of response (Figure 12a) could result in cracking patterns similar to those observed in the structure. Cracking parallel to the valleys and at the underside of the girders near the peaks could be attributed to excitation of this fundamental global mode, as might be affected by wind loads. Unfortunately, the resolution for the lower frequency ranges was not adequate to determine if these global modes were being excited by ambient conditions. Also, the frequency range of interest for these global modes is at the low end of the effective range for the accelerometers used in the investigation. Therefore, the evidence for excitation of these global modes is inconclusive.

Tests of Individual Panels

The first three calculated fundamental frequencies and mode shapes for an individual slab panel (local model) are given in Figure 14. The boundary conditions for these calculations include rotations and displacements fixed for all nodes at the perimeter of the panel. Actual boundary conditions for the panels apparently varied slightly from this condition.

Nondestructive impact tests were conducted for three slab panels of Bay 30, defined in Figure 15. The grid of 17 points used for collection of data from each panel is shown in Figure 16. Tests were performed using an instrumented impact hammer and one accelerometer.

An undamaged panel (UP1) was selected to provide baseline data. Figure 17 displays the experimental fundamental vibration mode for this panel. The shape of the fundamental mode derived experimentally is strikingly similar to the calculated mode (Figure 14b). The fundamental frequency determined from test results was 46.25 Hz, compared to a calculated fundamental frequency of 45.65 Hz. Experimental results from modes 2 and 3 are shown in Figures 17b and 17c. The frequency values are not as close to the calculated values as they were for mode 1 (Table 5), but the patterns are still notably similar.

The damaged panels (DP1 and DP2) chosen for impact tests represented two of the more common crack configurations. One panel (DP1 in Figure 15) had a diagonal crack across one corner and penetrating the full depth of the slab. The same procedures were used for this panel as were used for the undamaged panel to define the mode shape. Figure 18 shows the first three experimentally derived modes for DP1. The results for this mode of DP1 appear to be identical to those of the undamaged panel. A minimal effect of the crack on structural behavior is indicated. The second and third modes for this panel compare very well visually with experimental results for UP1 and with calculated results.

DP2 is located as indicated in Figure 15. Damage to this panel consisted of a longitudinal crack running parallel to the long dimension of the slab and approximately 1.5 m from the valley edge of the panel. The experimental fundamental frequency for the first mode was 46.25 Hz, the same as it was for the other two panels. The first three mode shapes are presented in Figure 19. Table 5 gives a comparison of the first three frequencies for the calculations and test results.

Data were collected for five additional tests during aircraft activity within Bay 30. For these tests, a reference channel was maintained on the undamaged panel, and three other gages were used to collect the data sets. Three of these five tests involved the jet parked inside the shelter with engines running at normal maximum (described by the pilot as "revved to 80 percent"). Two additional data sets were recorded with the jet taxiing out of the shelter. Both of these situations are typical of everyday use of the shelters.

For each of the three revving tests, data were collected for a period of 30 sec while the engines were accelerated from idle to normal maximum. In addition to the data collected from the undamaged panel (UP1) for each test, the first test collected data from DP1, the second from a beam between UP1 and DP1, and the third from a column near DP1 on which the gage was mounted horizontally.

Figure 20 shows the power spectra for each of these three stationary tests, with data from the two channels open for that test (reference values plus test data). Each curve represents the averaging of 5,000 time segments of data, which reduces random noise significantly. As was true of results from the impact tests, these data indicate very similar dynamic properties for the damaged and undamaged slab panels and imply minimal effect of the crack on structural performance during everyday operating conditions.

The two taxiing tests compared UP1 first with DP1 and then with the beam separating UP1 and DP1. Power spectra are plotted in Figure 21, again comparing data from pairs of channels. The results are consistent with those of the previous tests: (a) very similar dynamic properties for all members and conditions tested; and (b) the expectation of structural performance from cracked slabs similar to that of the intact portions of the shelter-roof panels.

7 Conclusions

Although the concrete used in construction of the aircraft weather shelters met strength requirements, it was not resistant to cracking. The concrete appeared to be proportioned for ease of pumping into the column forms. While its proportions were appropriate for pumping, they also made it more susceptible to drying shrinkage cracking in the thin roof slabs. Although the information provided indicates that the roof slabs were cured with wet burlap, the present level of cracking suggests that the surface was not kept continuously wet for a long enough time after the concrete was finished.

Design and placement of reinforcing was driven by the requirements for depth of concrete cover over steel in a coastal environment. Again, the cover requirements were met. But in roof slabs only 130 mm (5 in.) thick, this resulted in both layers of reinforcing being compressed into the center of each slab (between 50 and 76 mm (2 and 3 in.) from either surface), diminishing their effectiveness.

Cracking was not initiated by unexpected loads or by vibration associated with jet engines. Cracks were visible in roof slabs very soon after the concrete hardened, before any severe weather on Okinawa, and before the shelters housed any aircraft.

The concrete has not experienced any deterioration from chemical attack, aggregate reactions, corrosion of reinforcing, or other environmental factors known to cause distress to concrete.

Cracks were initiated by drying shrinkage soon after concrete placement. Cracks are older at the top, so propagation of cracks through the full depth of the slabs required more time. Cracks propagated through the slab almost exclusively near the valley of each folded plate. Crack propagation was caused by factors unrelated to shrinkage.

Foundation settlement or movement did not contribute to the observed cracking. Evidence of foundation movement would be shown by cracks in floor slabs, vertical fire walls, or columns of the structures. No such damage was reported or observed.

Propagation of the cracks is attributed primarily to flexure along the column lines. This flexure may be augmented by reversing wind and thermal

loads on the shelters. Although propagation of existing cracks is likely to continue, appearance of new cracks is unlikely if use and load conditions remain the same.

As determined by onsite measurements, dynamic characteristics of an undamaged panel are basically the same as those of a cracked panel. Therefore, the shelters have not experienced significant structural deterioration.

No structural modifications appear necessary beyond those already effected. The membrane placed over the structures should slow the movement of moisture to the reinforcing steel through existing structural cracks. Deterioration of the reinforcing steel over time, ensuing stress on the panels, and loss of structural strength presents the most notable risk to long-term performance of the weather shelters.

8 Recommendations

Careful inspection of all bays of the structures should be conducted at regular intervals. Any evidence of deterioration should be investigated and evaluated for potential structural risk. This includes evidence of corrosion of reinforcing steel, as shown by staining on the undersides of slabs. The moisture retarding membrane over the structures should also be inspected regularly. (Some leakage of the membrane was observed during the WES site visit to Kadena. This was reported to USAED, Japan, personnel.)

A thermal analysis of the slabs could be performed to determine whether temperature effects are a factor in the cracking. Heat dissipation is a greater problem in the more massive beams than in the thin slab.

References

American Concrete Institute. (1992). *ACI manual of concrete practice*. Part 3, ACI 224R and 224.1R, Detroit, MI.

American Society for Testing and Materials. (1991). *1991 annual book of ASTM standards*. Philadelphia, PA.

- a. Designation C 457-90, "Standard test method for microscopical determination of parameters of the air-void system in hardened concrete."
- b. Designation C 642-90, "Standard test method for specific gravity, absorption, and voids in hardened concrete."
- c. Designation C 1084-87, "Standard test method for portland-cement content of hardened hydraulic-cement concrete."

Whiting, D. (1988). "Permeability of selected concretes." *Permeability of concrete*, ACI SP-108, D. Whiting and A. Walitt, ed., American Concrete Institute, Detroit, MI, 195-222.

Table 1
Core Samples Identification and Description

Core Sample	WES/CTD Number	Description
1	920109	Upper portion of Bay 42 roof slab, no cracks observed
2	920110	Same as core 920109
3	920111	Same as core 920109
4	920112	Lower portion of Bay 42 roof slab, crack observed
5	920113	Lower portion of Bay 42 roof slab, no cracks observed
6	920114	Valley portion of Bay 42, above girder No. 6, ponding area, no cracks observed
7	920115	Same as core 920114
8	920116	Upper portion of Bay 35 roof slab, no cracks observed
9	920117	Same as core 920116
10	920118	Lower portion of Bay 35 roof slab, crack observed
11	920119	Lower portion of Bay 35 roof slab, no cracks observed
12	920120	Valley portion of Bay 35 near column head, ponding area, hairline crack observed
13	920121	Same as core 920120

Table 2
Distribution of Macroscopic Features in Six Cores

Bay 42				Bay 35		
Core/Feature	109	112	114	116	118	120
Entrapped Air	X	X	X	X	X	X
Reinforcing	X	X		X	X	X
Crack		X			X	X
Carbonation		X			X	X
Water Channels	X			X	X	
Aggregate Segregation						X
Surface Coating		X	X			

Table 3
Cement Content of Cores as Determined by ASTM C 1084 (1991c)

Core Identification	Cement Content, lb/yd ³ (kg/m ³)
920111	577 (342.3)
920113	571 (338.7)
920115	599 (355.3)
920117	586 (347.6)
920119	558 (331.0)
920121	612 (363.0)
Average	584 (346.3)

Table 4
Air Voids and Solid Components by Volume as Determined by ASTM C 457
(1991a)

Material/Void Type	WES Core Number					
	109	112	114	116	118	120
	Percentage by Volume					
Coarse Aggregates	30.8	43.5	34.0	34.3	38.9	31.5
Fine Aggregates	31.5	26.1	29.9	28.6	25.6	29.4
Cement Paste ¹	33.2	24.8	30.5	32.1	28.5	34.9
Entrained Voids	3.0	3.1	4.4	2.6	4.0	2.9
Entrapped Voids	1.6	2.6	1.2	2.4	3.1	1.3
Total Voids ²	4.6	5.7	5.6	5.0	7.1	4.2

¹ The paste portion includes a notable amount of very finely divided mineral matter probably derived from the aggregates.

² Total of entrained plus entrapped air, which are counted separately and differentiated on the basis of void size and shape.

Table 5
Comparison of Calculated and Measured Frequencies

Mode	Computer Model (Hz)	UP1 (Hz)	DP1 (Hz)	DP2 (Hz)
1	45.7	46.2	46.2	46.2
2	77.0	85.3	89.6	89.0
3	107.7	109.3	106.4	114.8



Figure 1. Kadena Air Base aircraft shelters



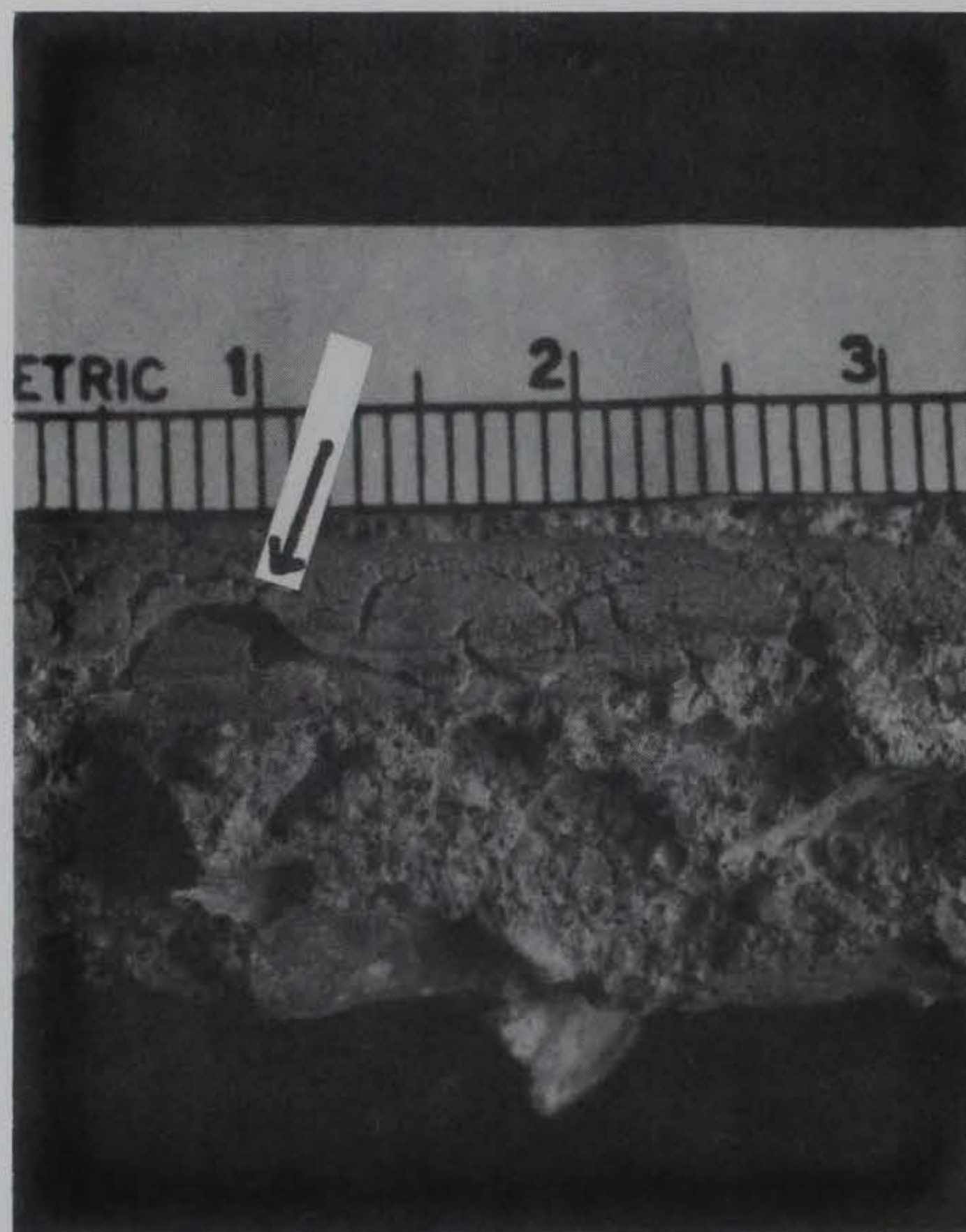


Figure 2. Broken surface of concrete core after rebar removed showing water channel, core 920109 (Bay 42, upper portion)

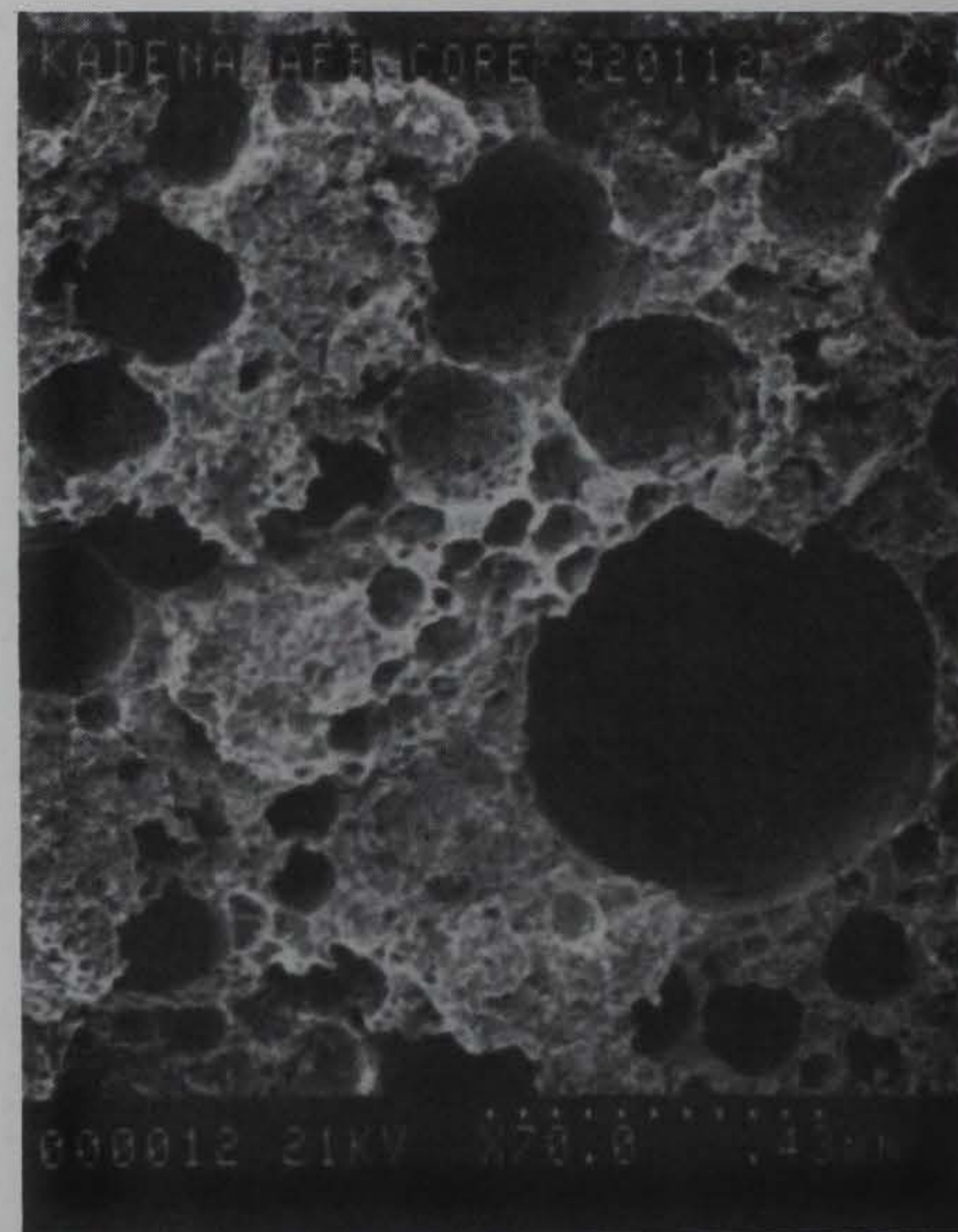


Figure 3. Electron micrograph of cement paste at aggregate interface showing open, porous microstructure; dotted line at bottom is 0.43 mm long

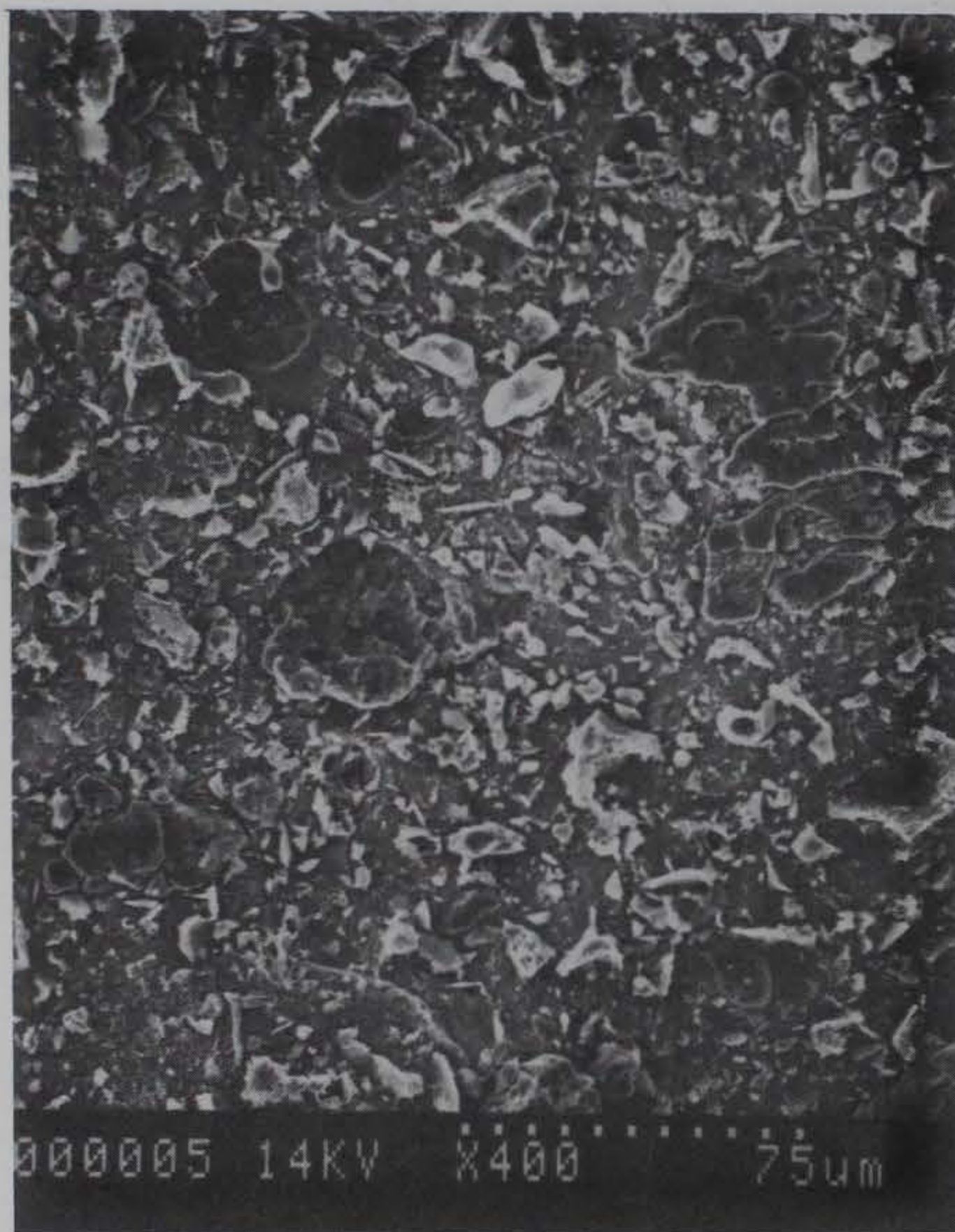


Figure 4. Electron micrograph showing finely divided mineral matter in cement paste; dotted line is 75 μm



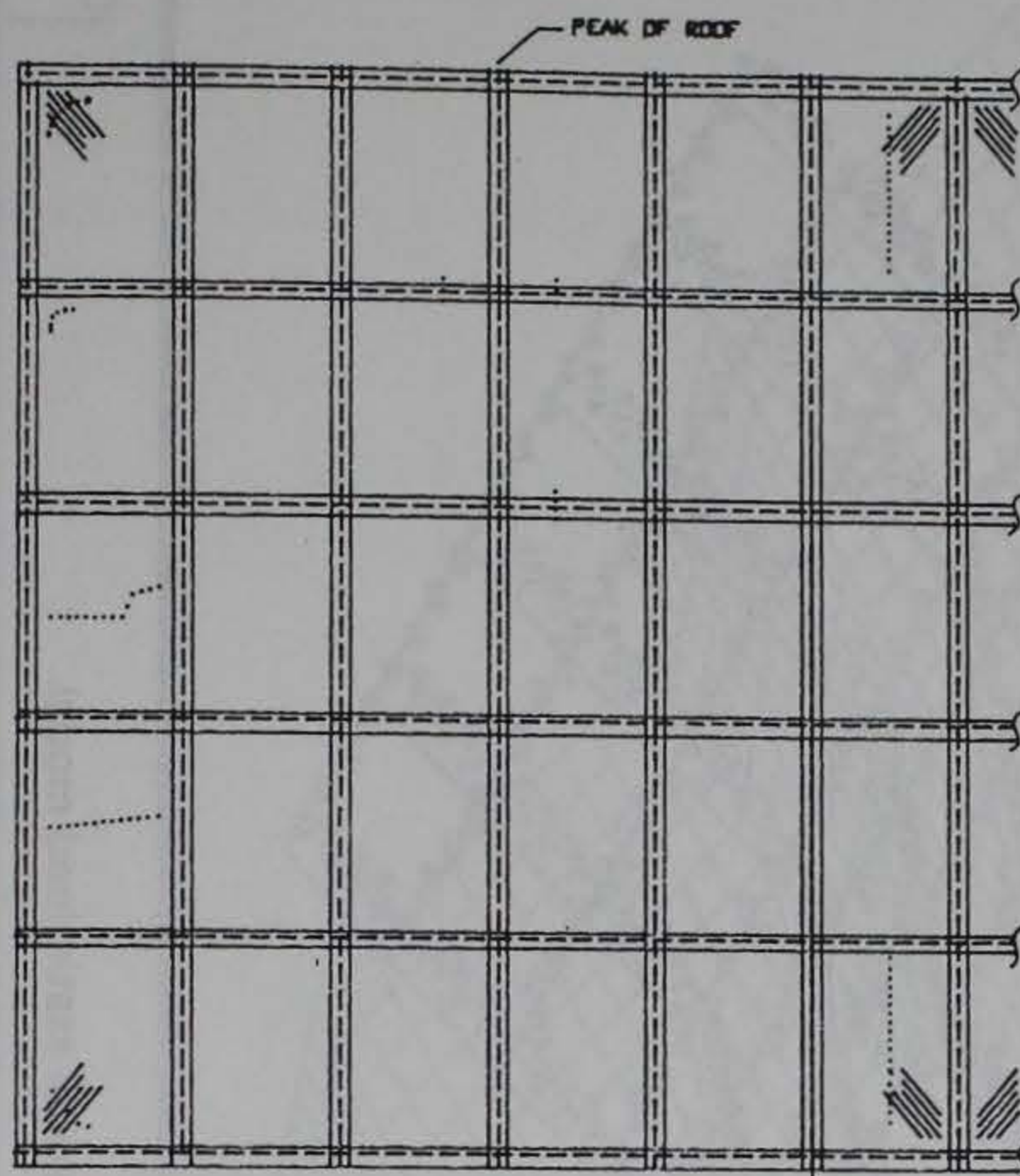
Figure 5. Electron micrograph of ettringite on crack surface below depth of carbonation; dotted line is 100 μm



Figure 6. Photograph of crack surface showing depth of carbonation along crack (light-colored upper part); lower noncarbonated surfaced reacted with phenolphthalein indicator (darker)

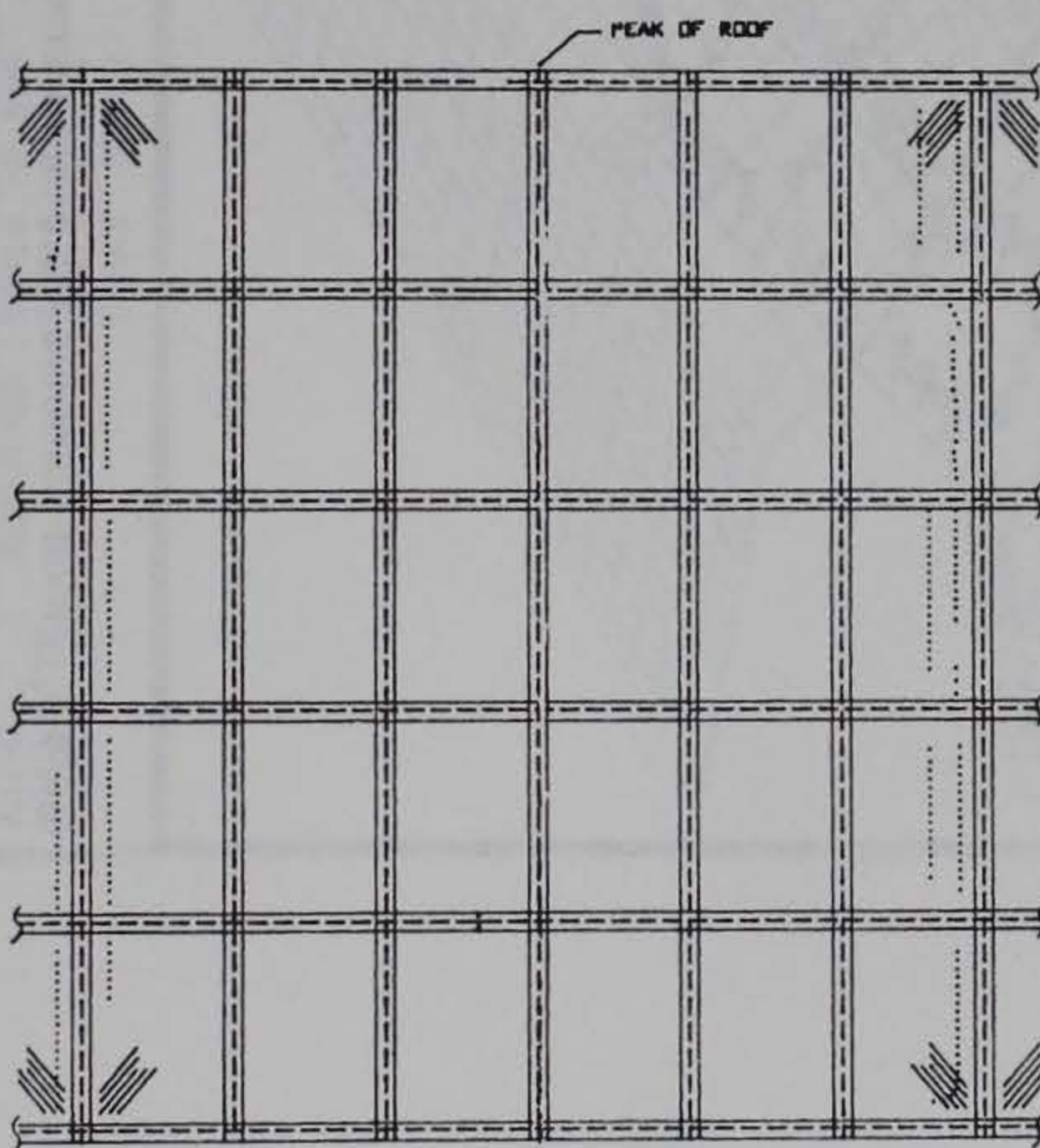


Figure 7. Photograph of concrete slab in reflected light showing crack going through aggregate particles (scale in millimetres)



a. Bay 35

..... INDICATES APPROXIMATE
CRACKING PATTERN



b. Bay 43

Figure 8. Plan view of Bays 35 and 43, showing locations of beams and cracks; peak of each folded plate is in the center of the bay

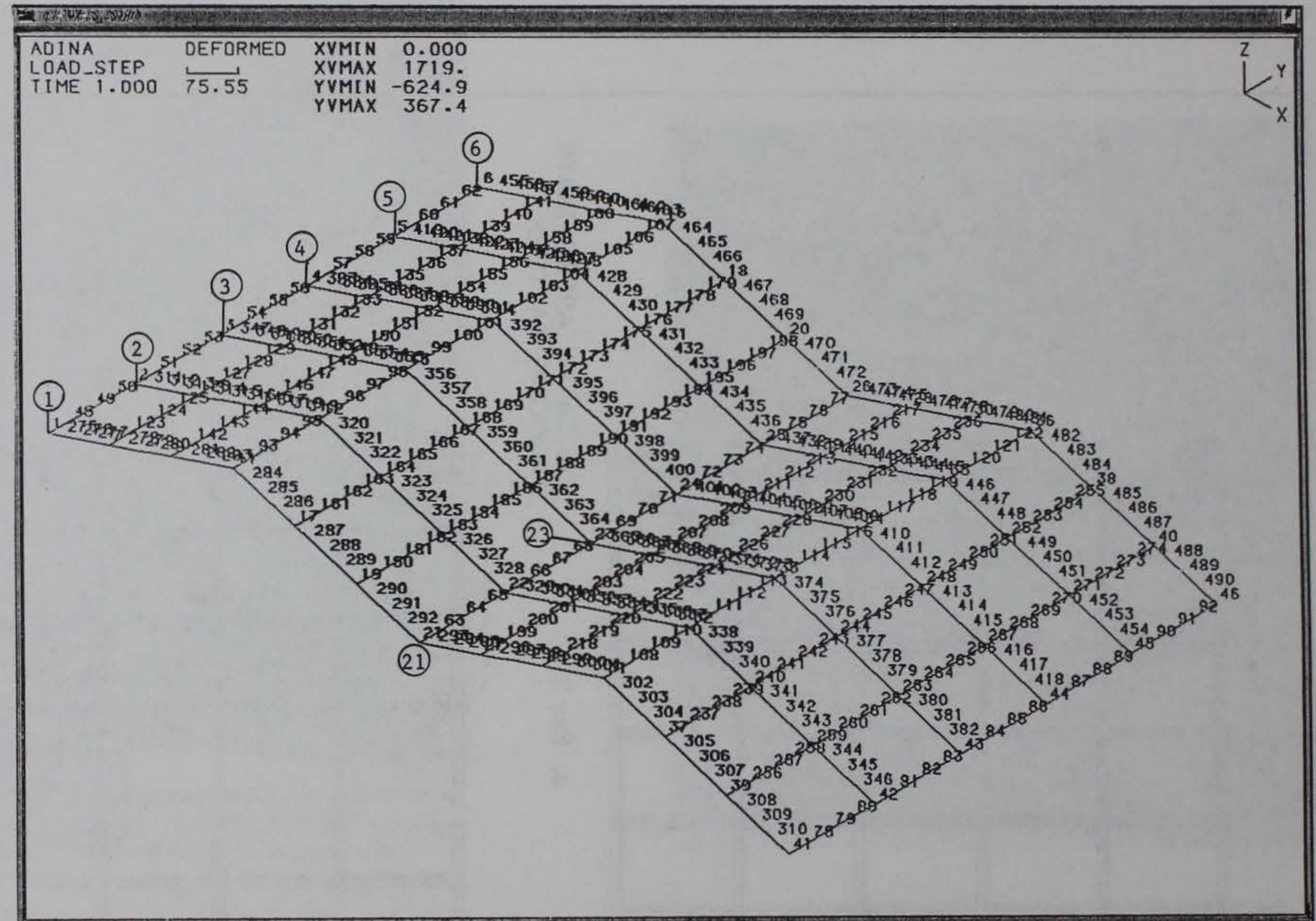


Figure 9. Node numbers for beam elements in global static-load model

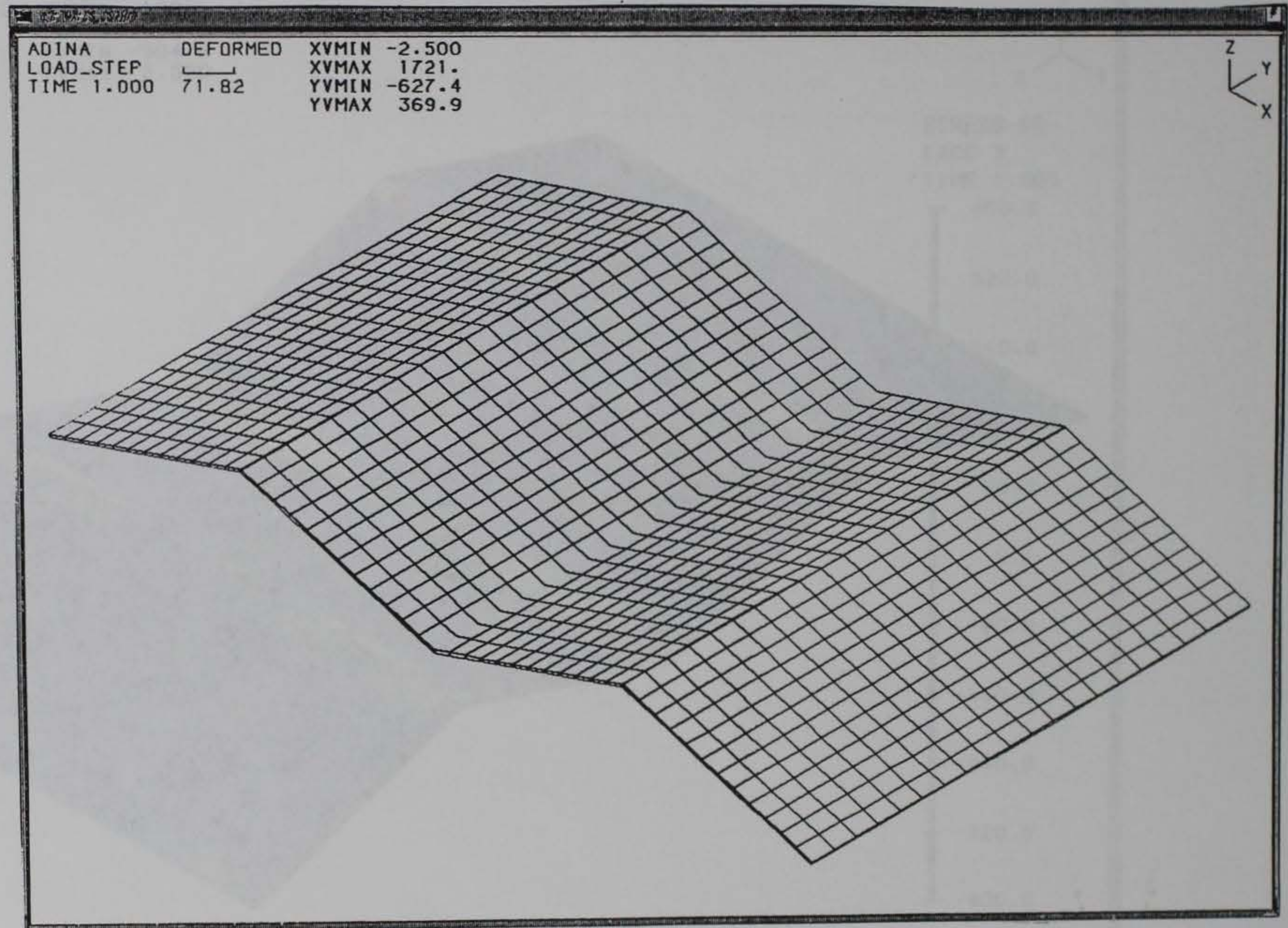


Figure 10. Shell elements of two-bay model

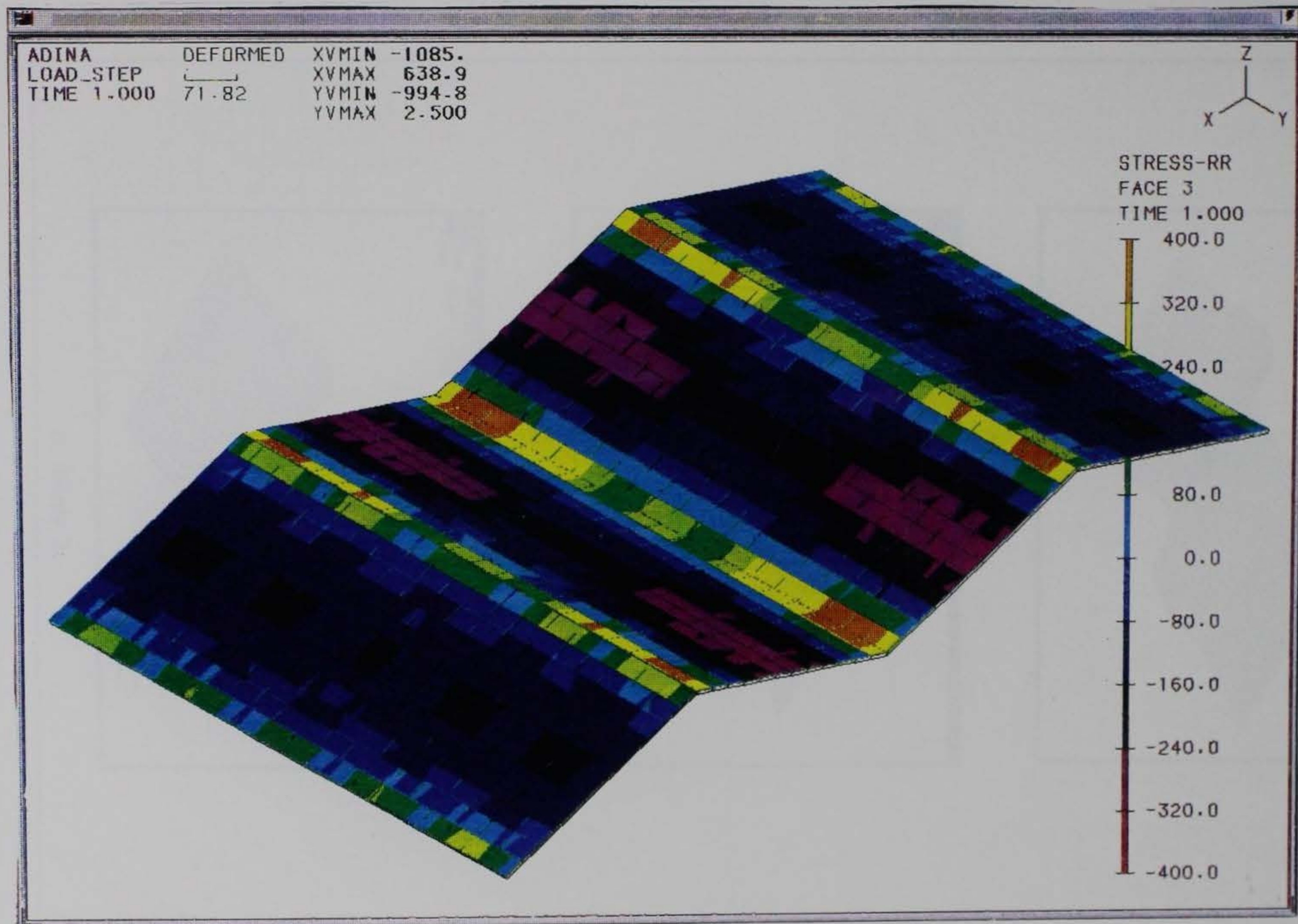
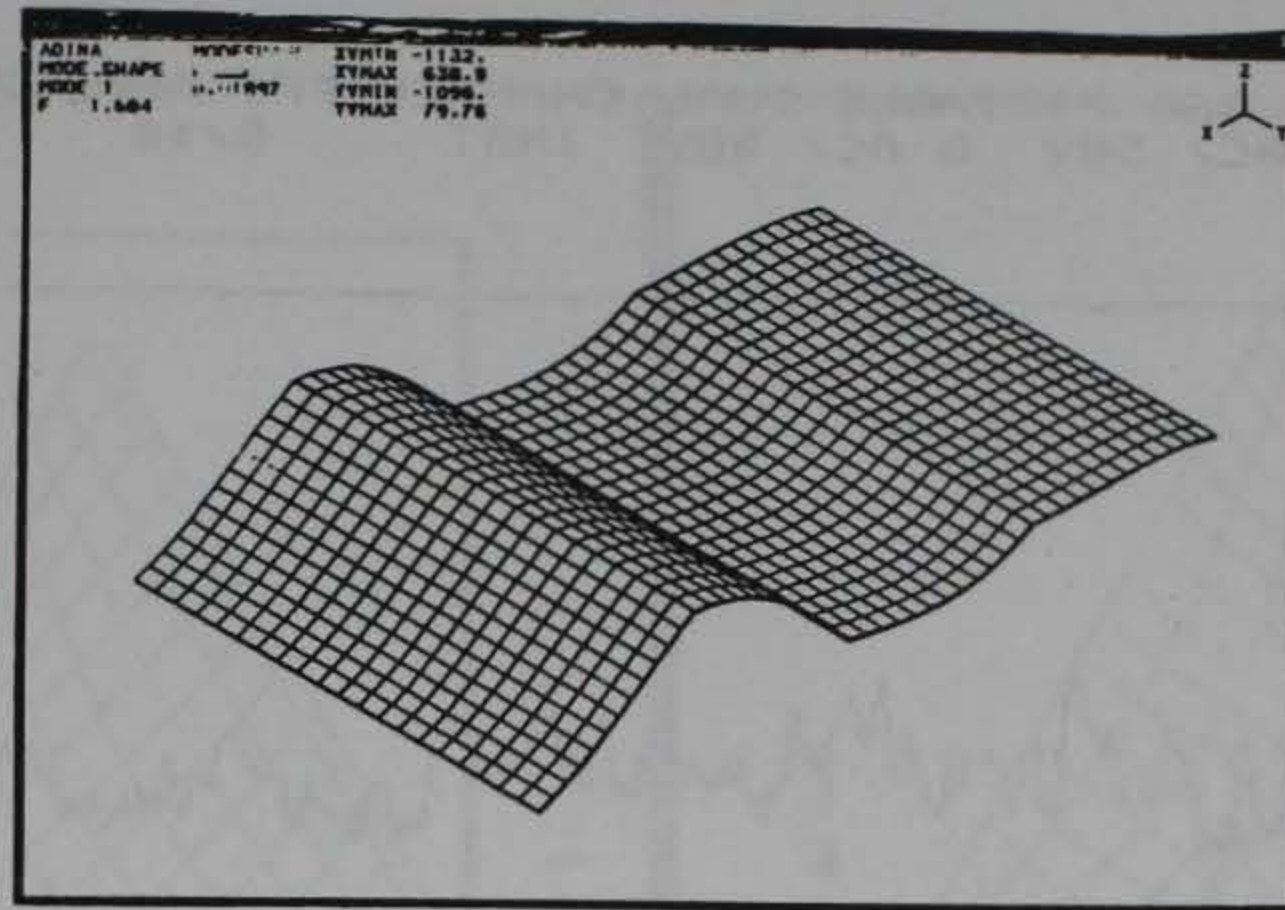
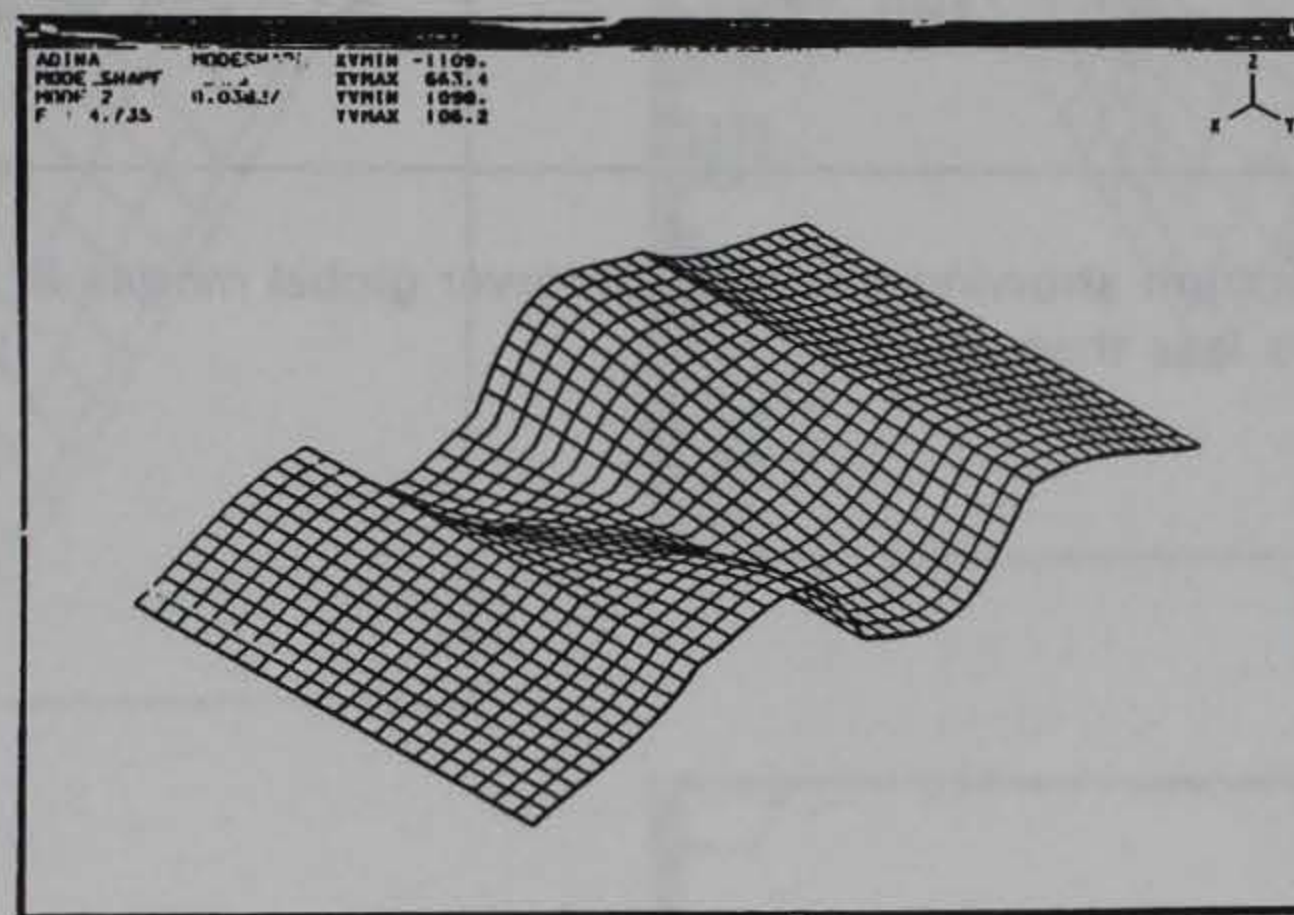


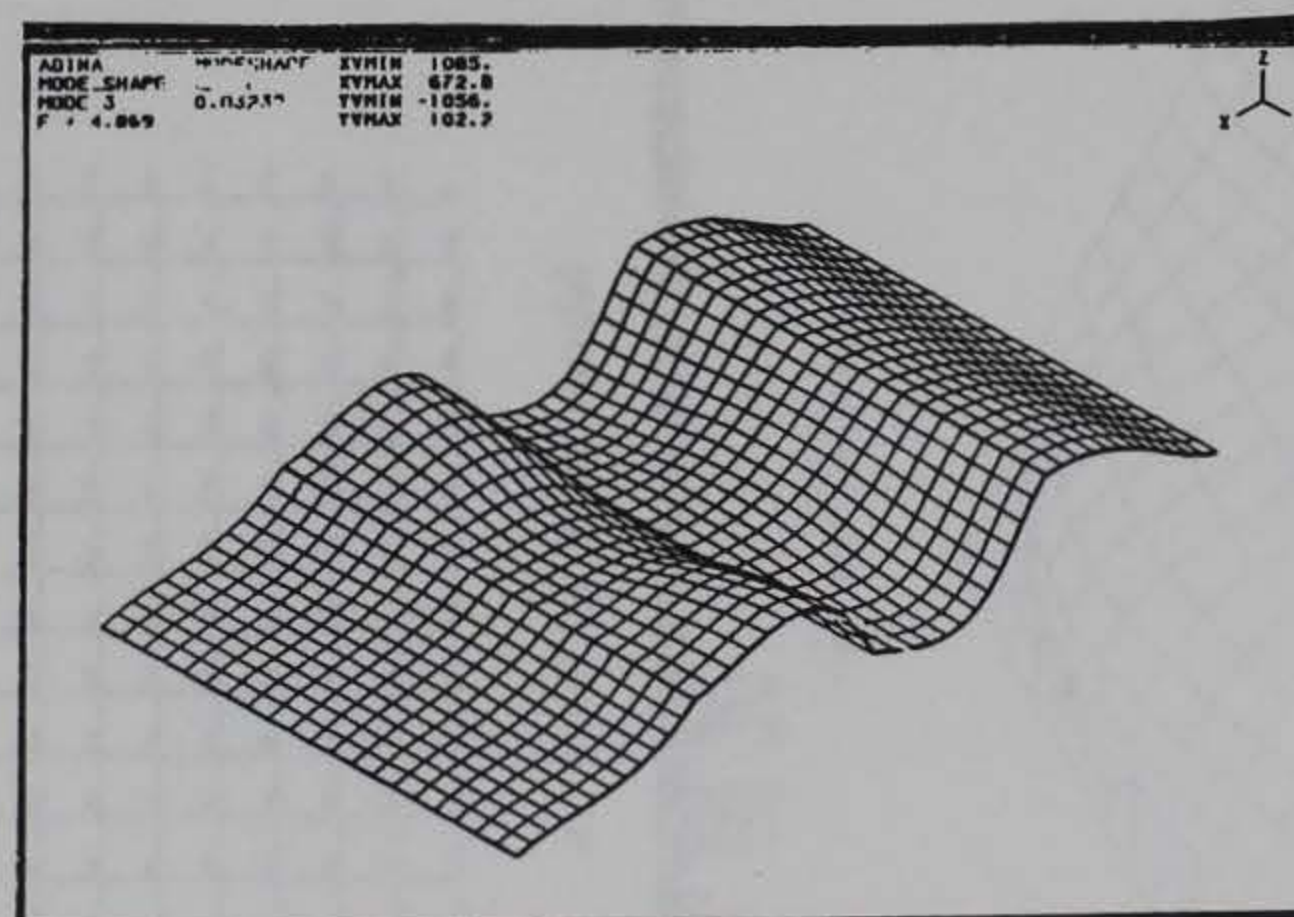
Figure 11. Stress distribution of two-bay model in simple static condition (Case 1)



a. Mode 1



b. Mode 2



c. Mode 3

Figure 12. First three calculated mode shapes for global two-bay model

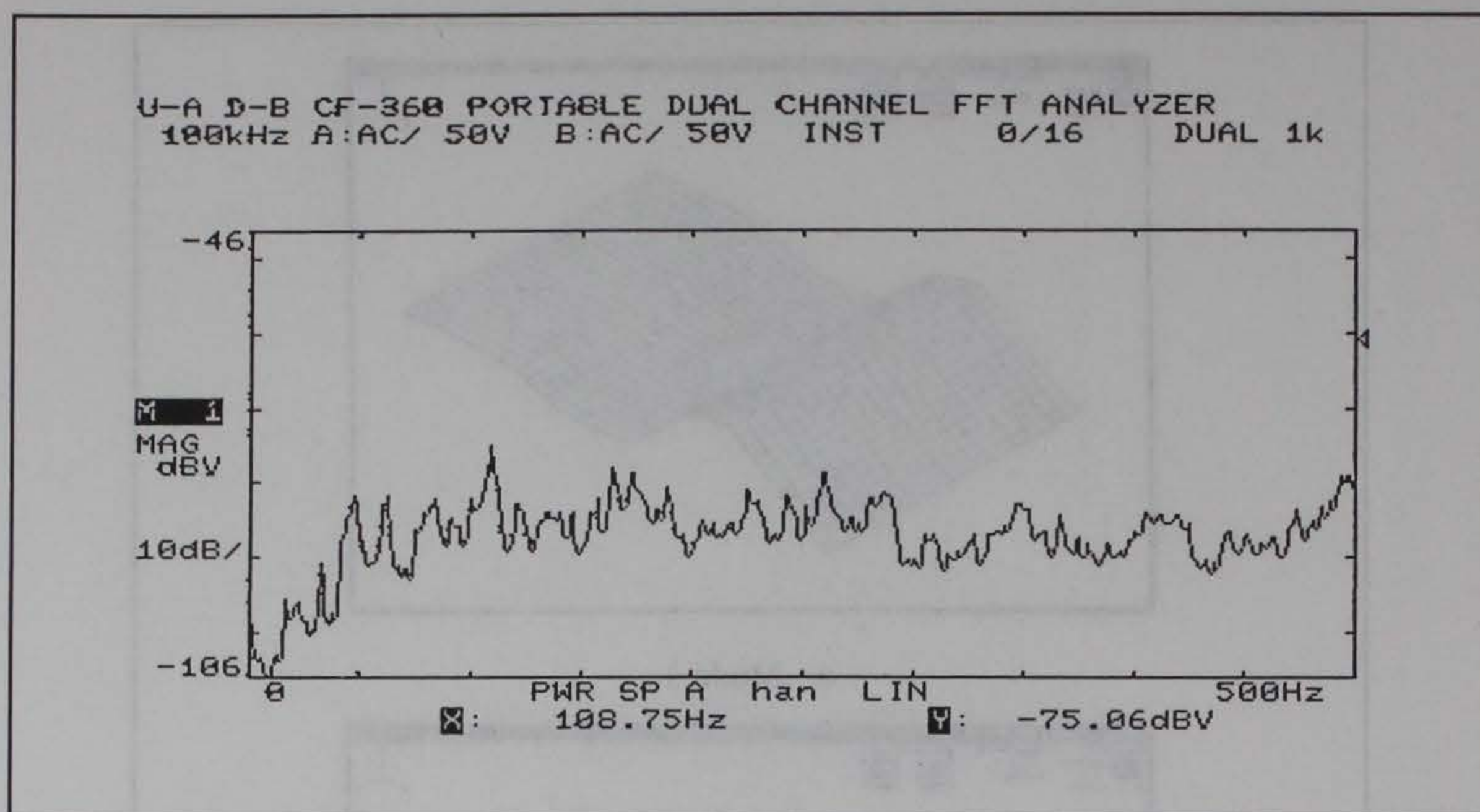
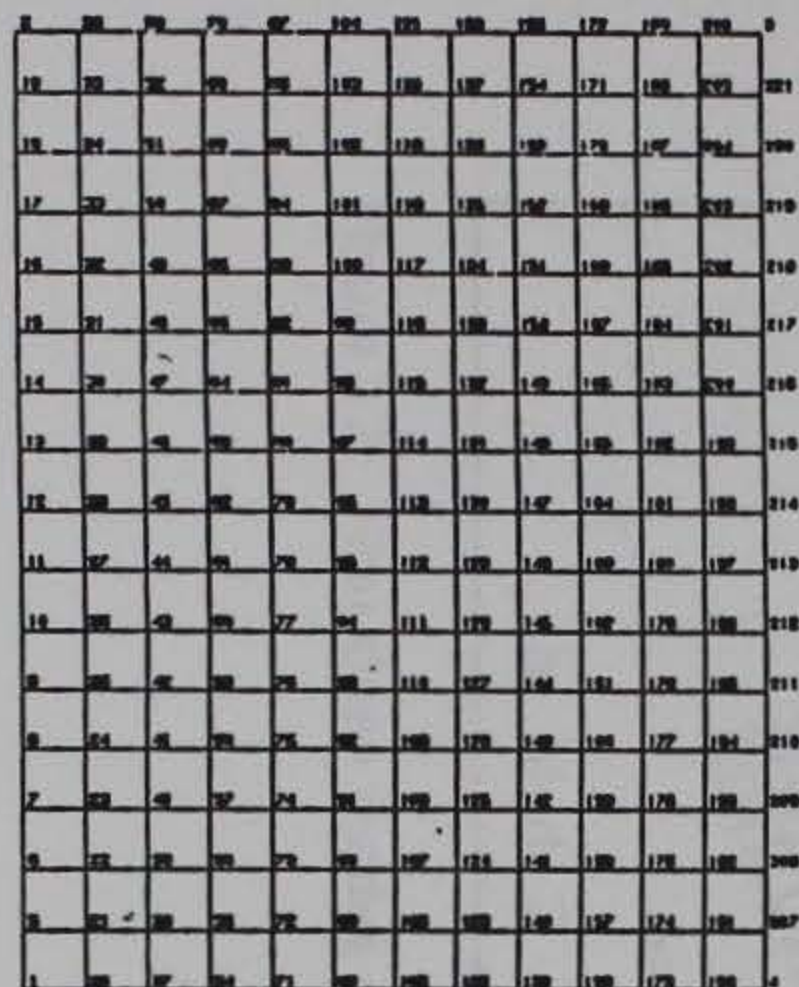
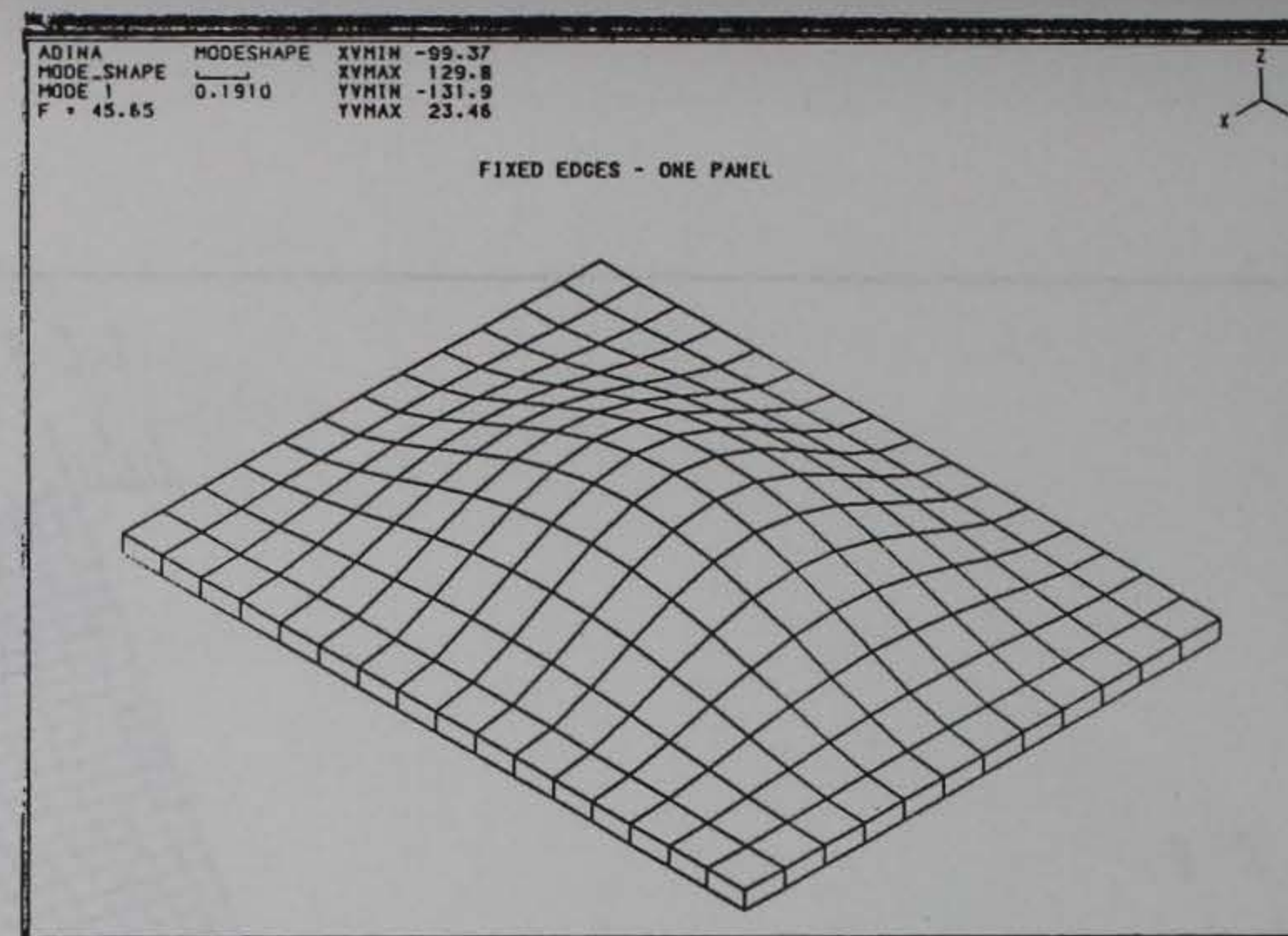


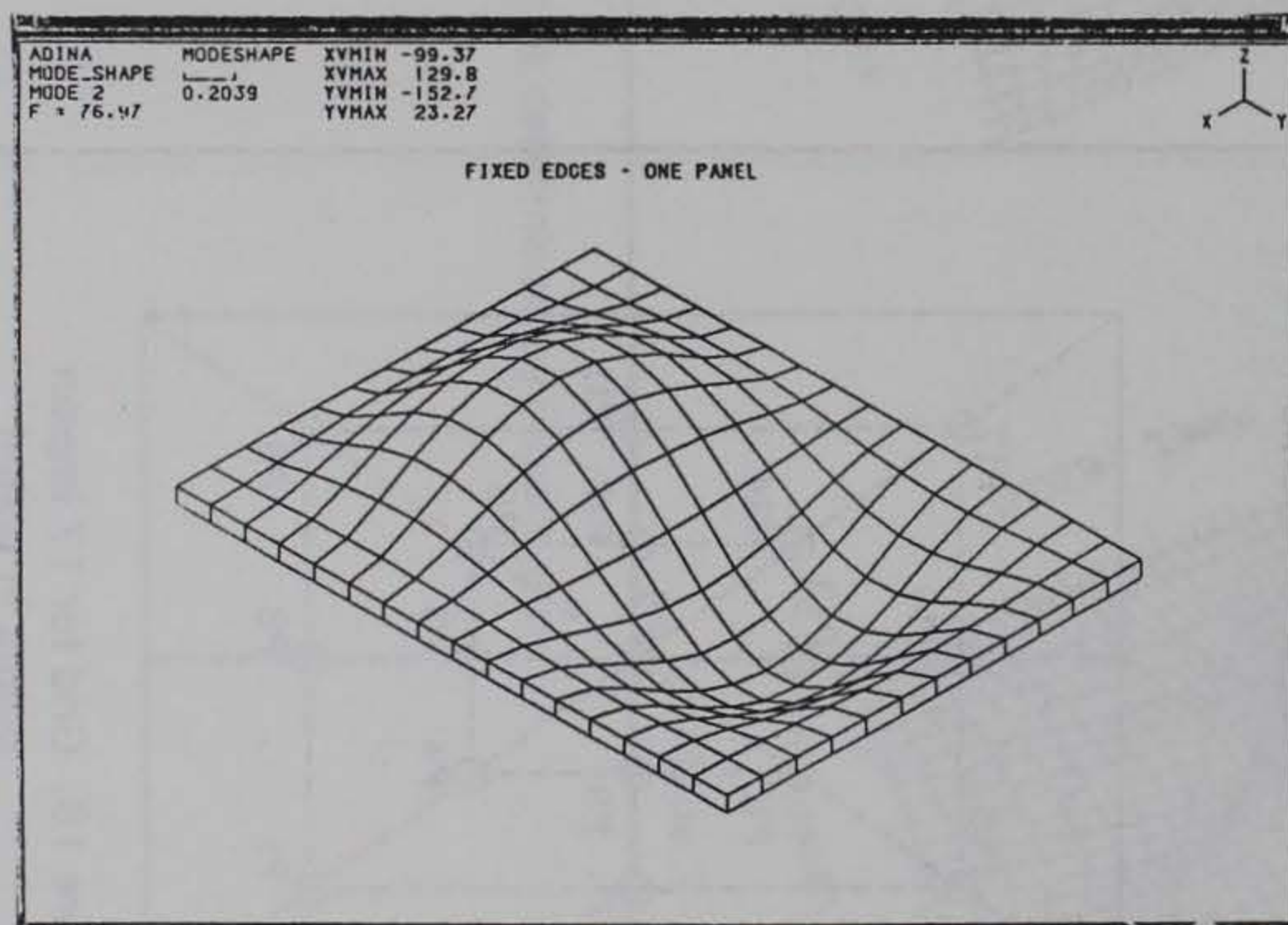
Figure 13. Power spectrum showing evidence of lower global modes in local data (peaks at frequencies less than 40 Hz)



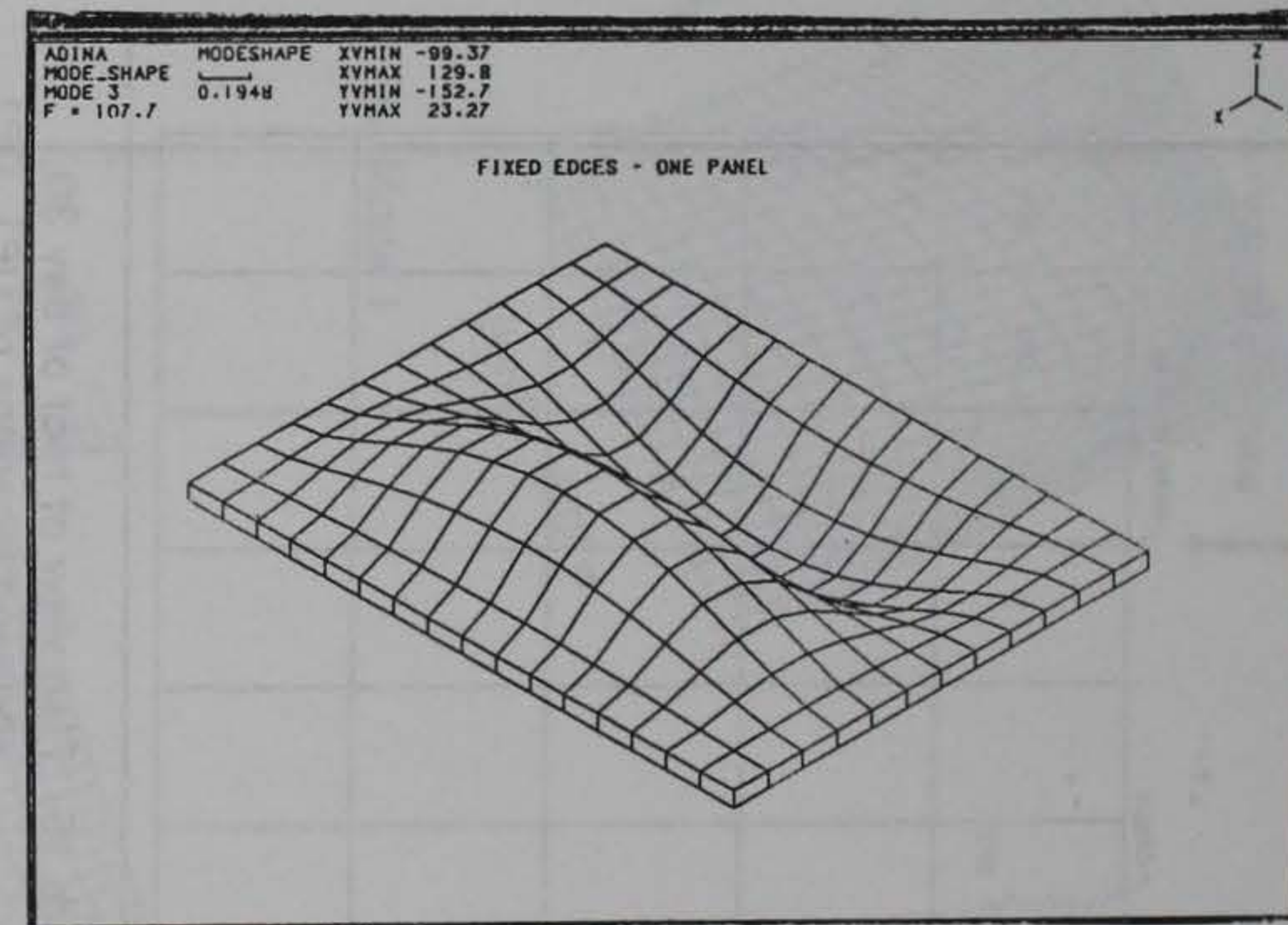
a. Grid for local model



b. Mode 1



c. Mode 2



d. Mode 3

Figure 14. Grid and first three theoretical mode shapes, data from one panel (local model)

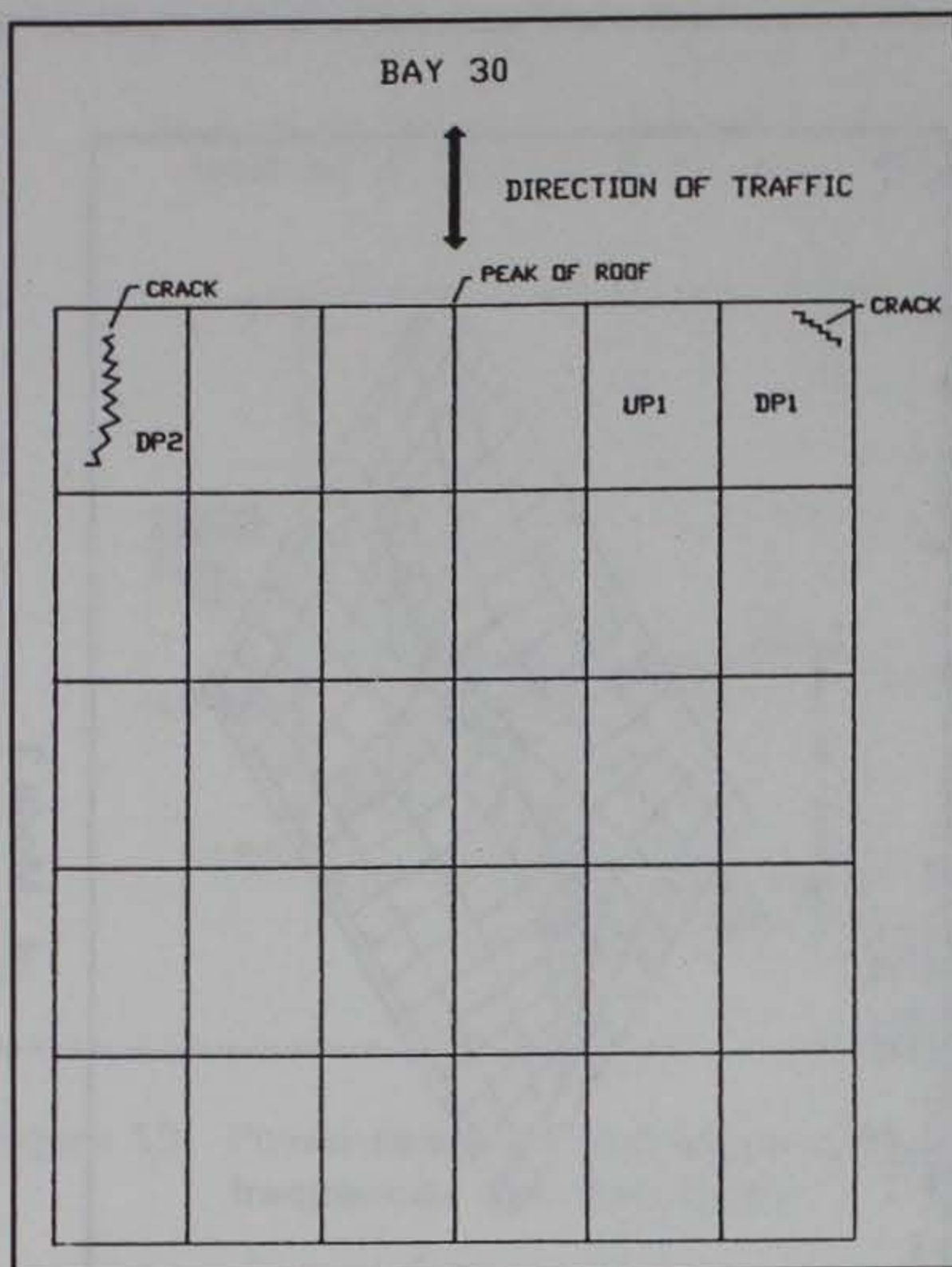


Figure 15. Plan view of roof of Bay 30, showing locations of UP1, DP1, DP2, and cracks; arrow marks roof peak and shows direction of aircraft traffic

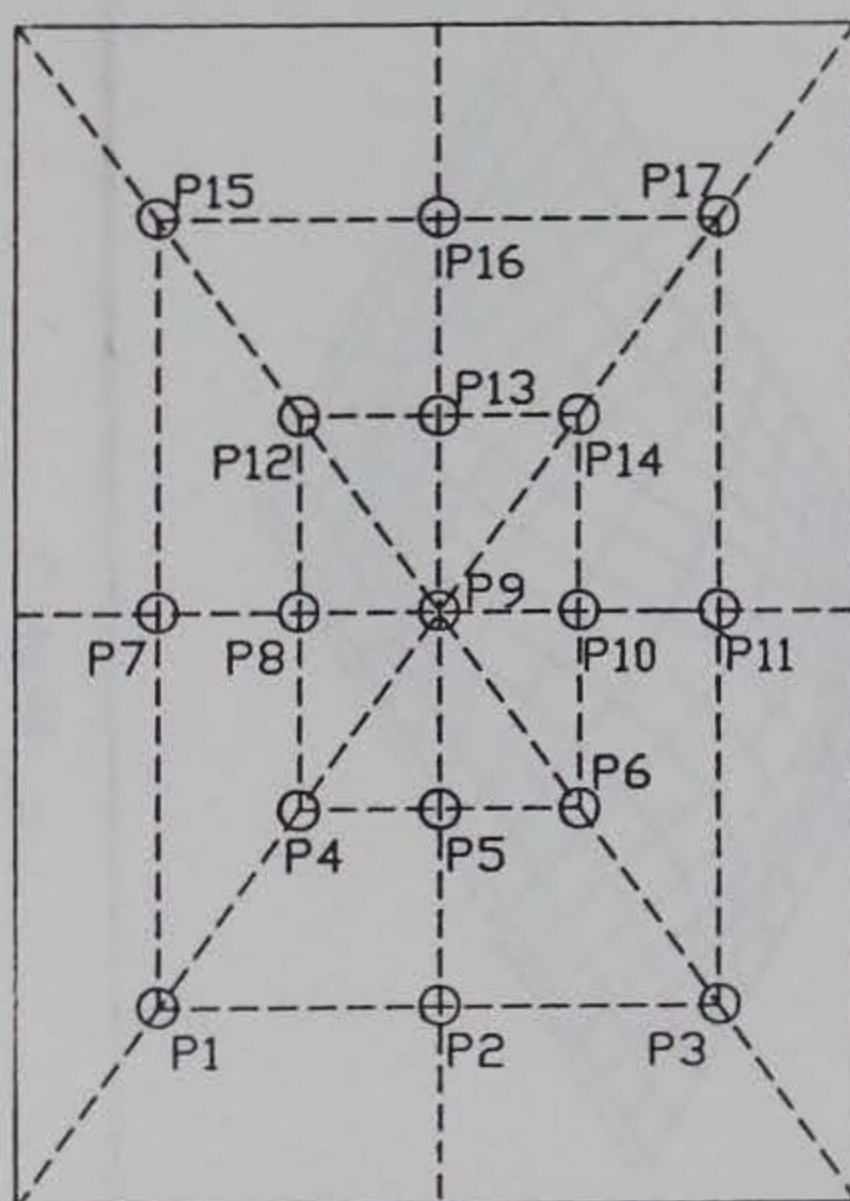


Figure 16. Grid for 17 impact points in tested panels

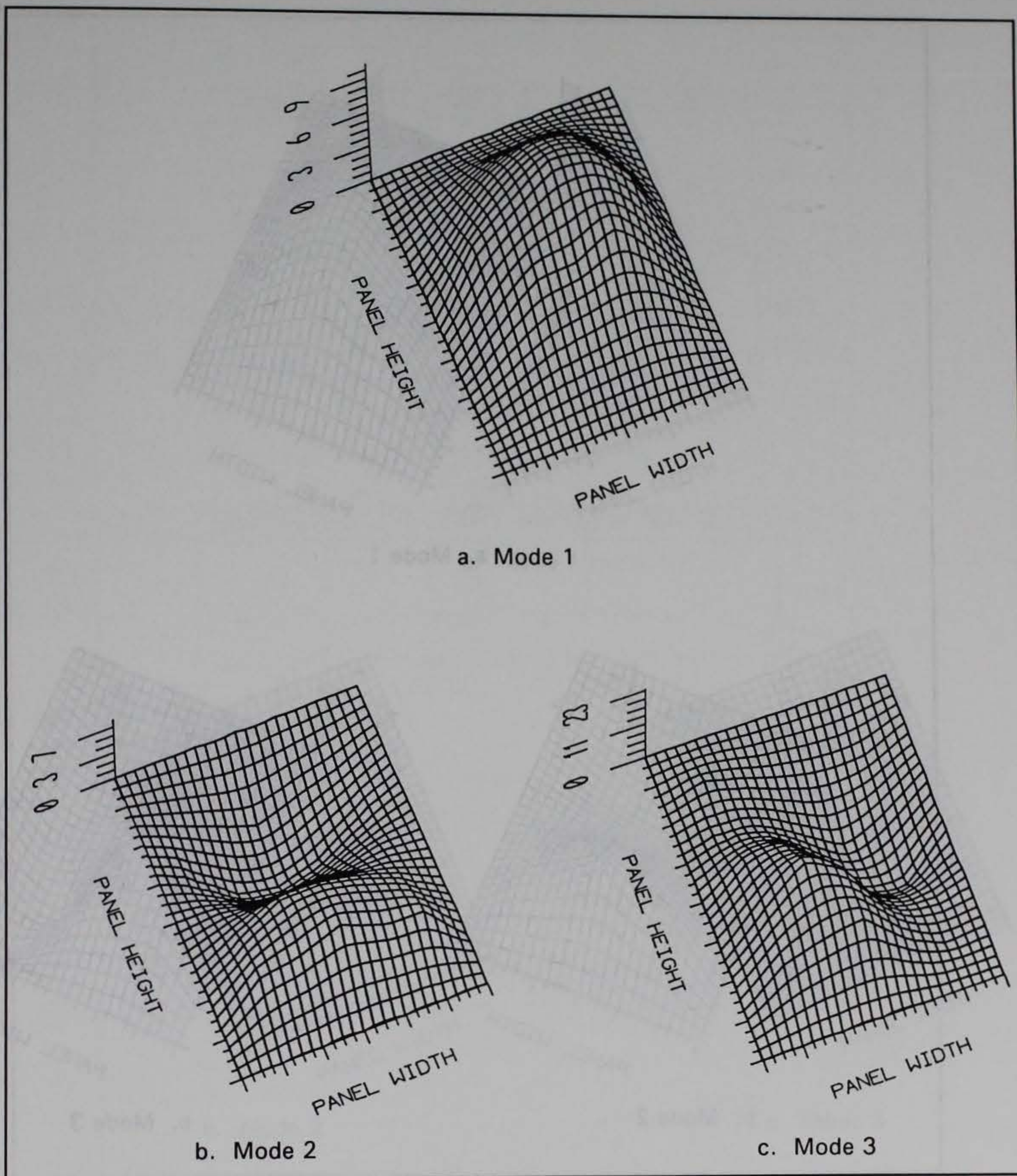


Figure 17. First three modes for UP1 as determined by impact tests

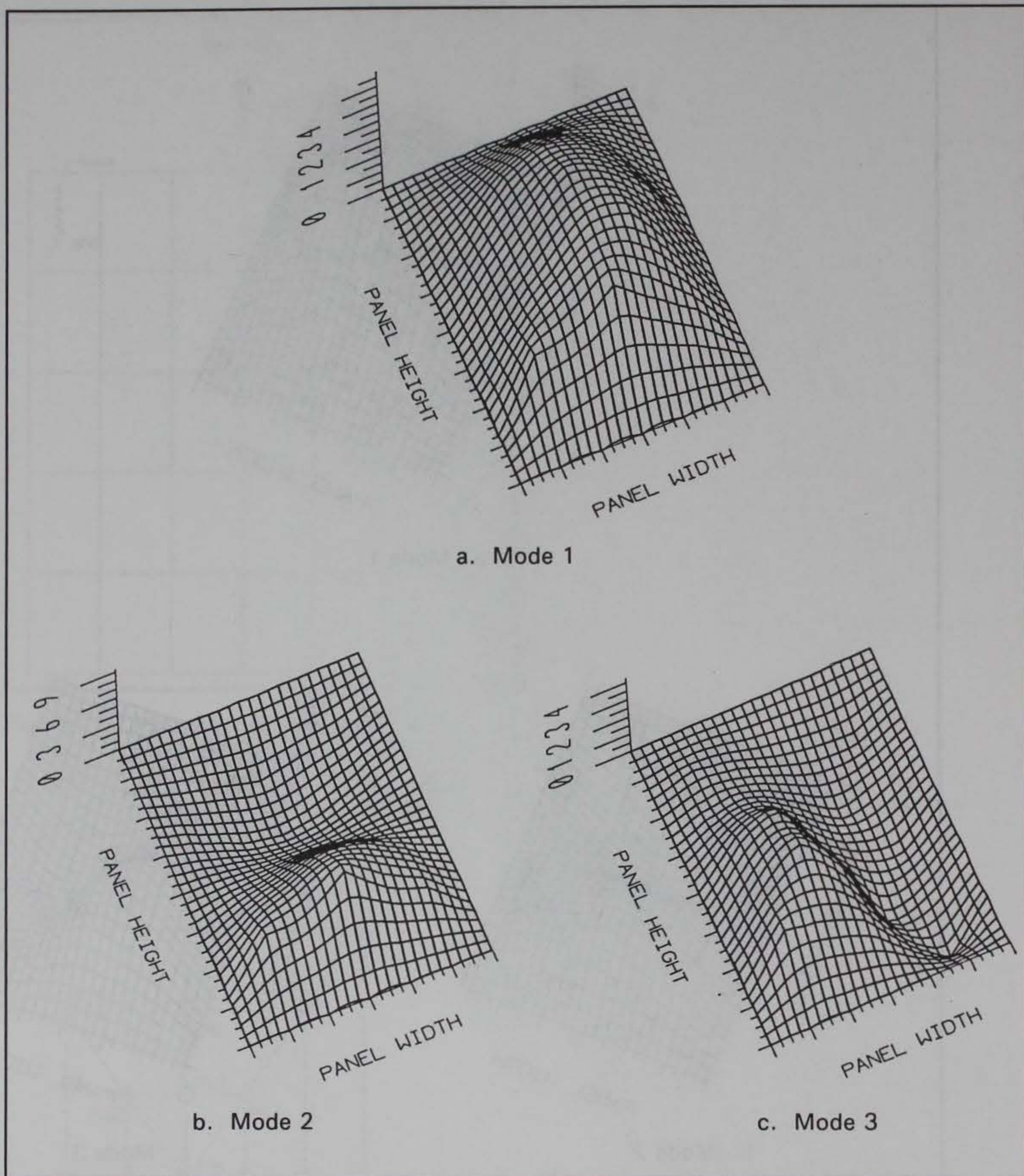


Figure 18. First three modes for DP1 as determined by impact tests

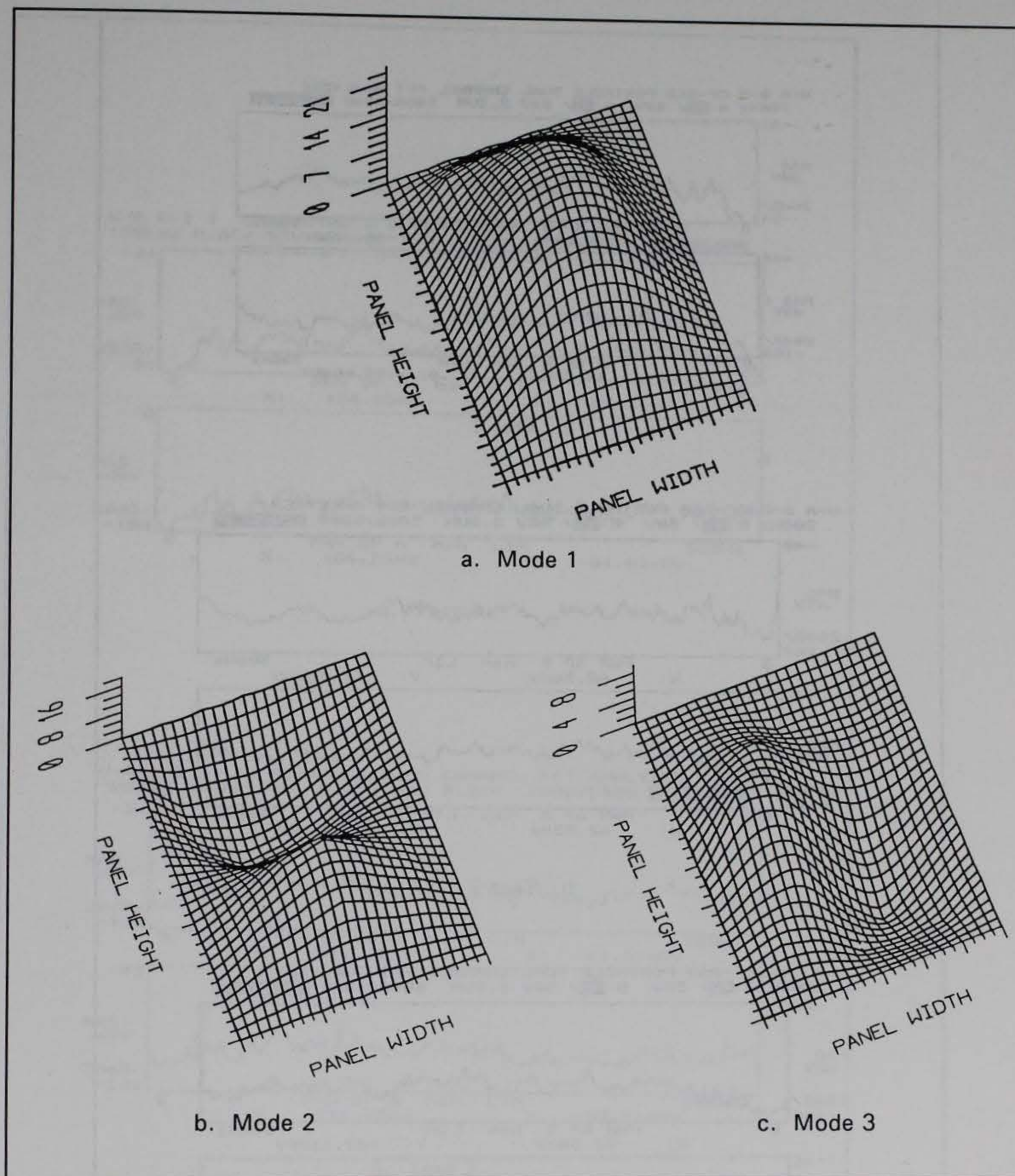
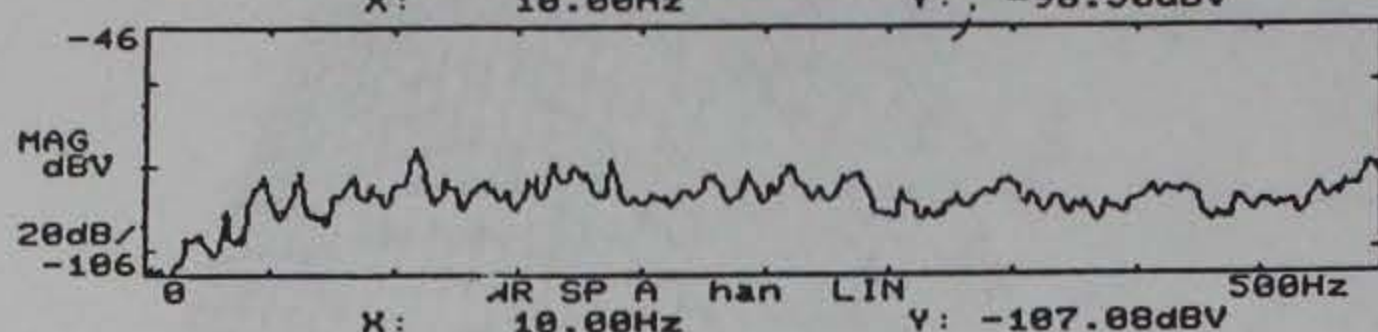
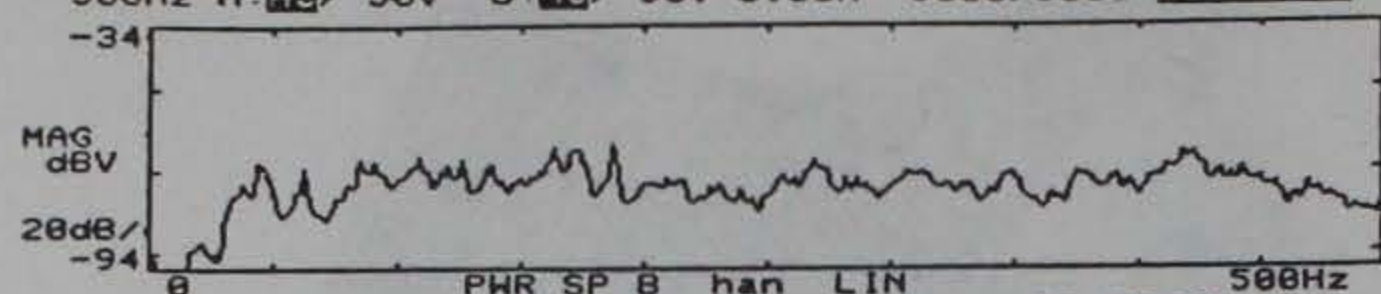


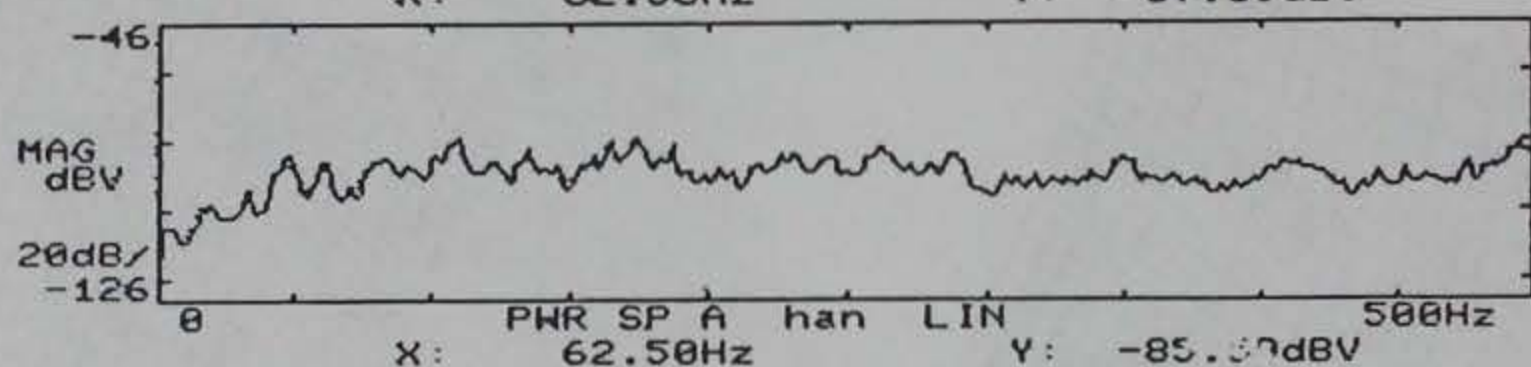
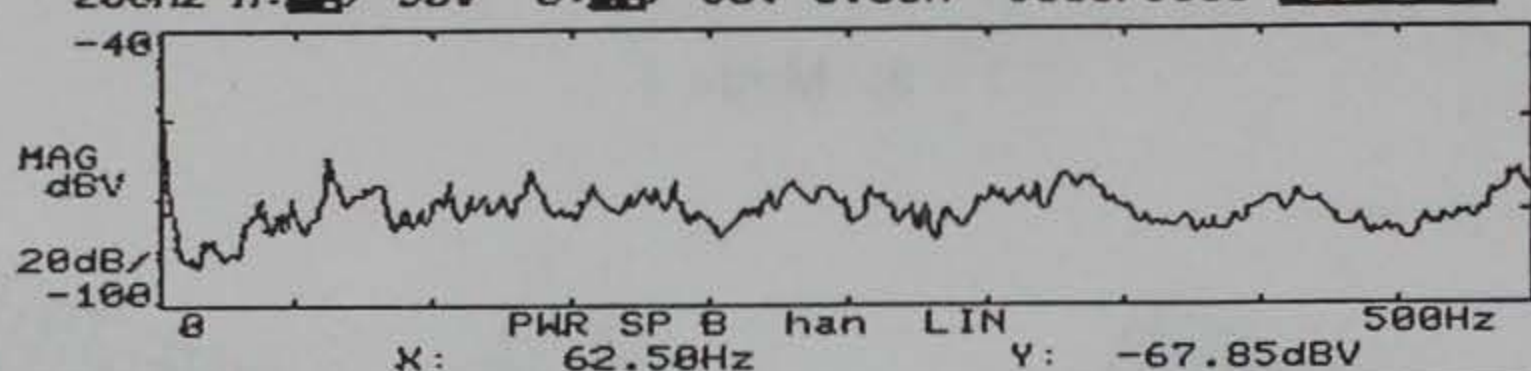
Figure 19. First three modes for DP2 as determined by impact tests

U-A D-B CF-360 PORTABLE DUAL CHANNEL FFT ANALYZER
500Hz A: 50V B: 50V S.SUM 5000/5000 DUAL 1K



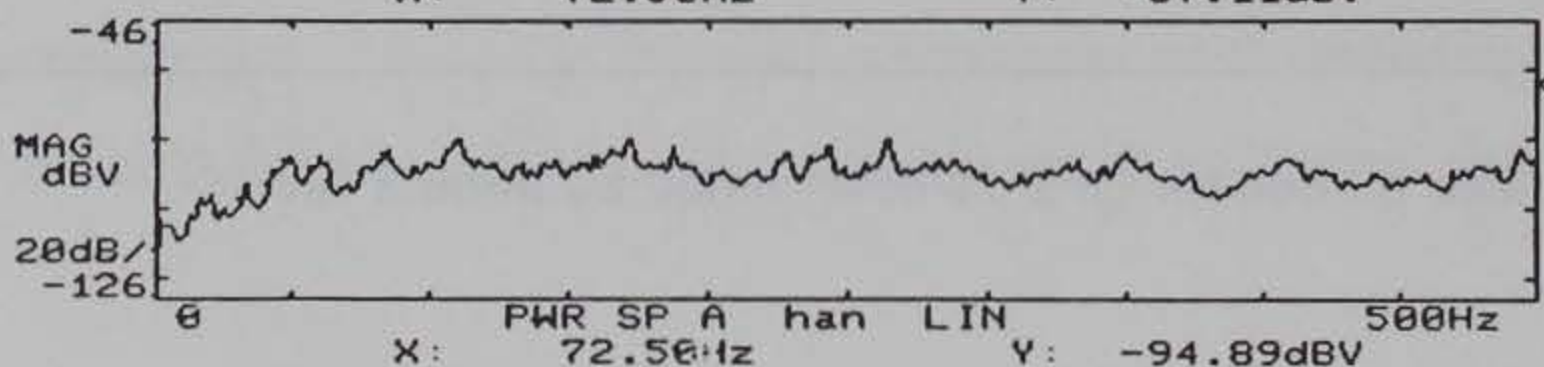
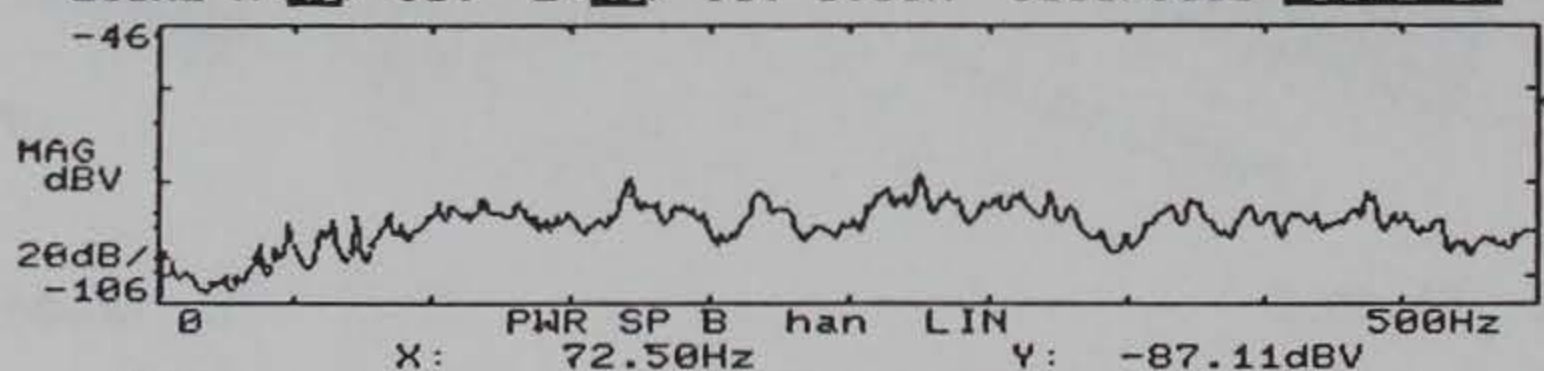
a. Test 1

U-A B-B CF-360 PORTABLE DUAL CHANNEL FFT ANALYZER
200Hz A: 50V B: 50V S.SUM 5000/5000 DUAL 1K



b. Test 2

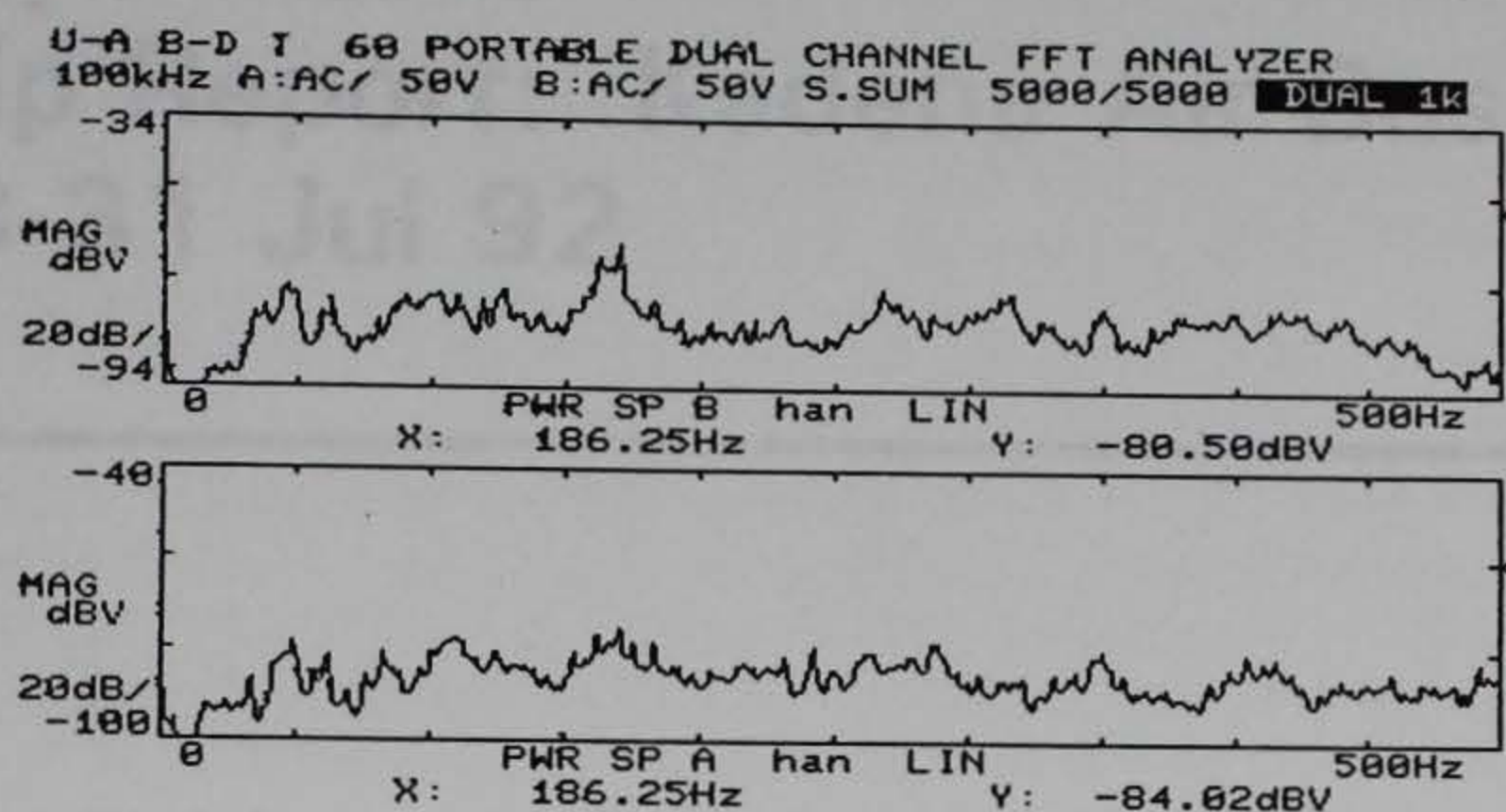
U-A C0-BCF-360 PORTABLE DUAL CHANNEL FFT ANALYZER
200Hz A: 50V B: 50V S.SUM 5000/5000 DUAL 1K



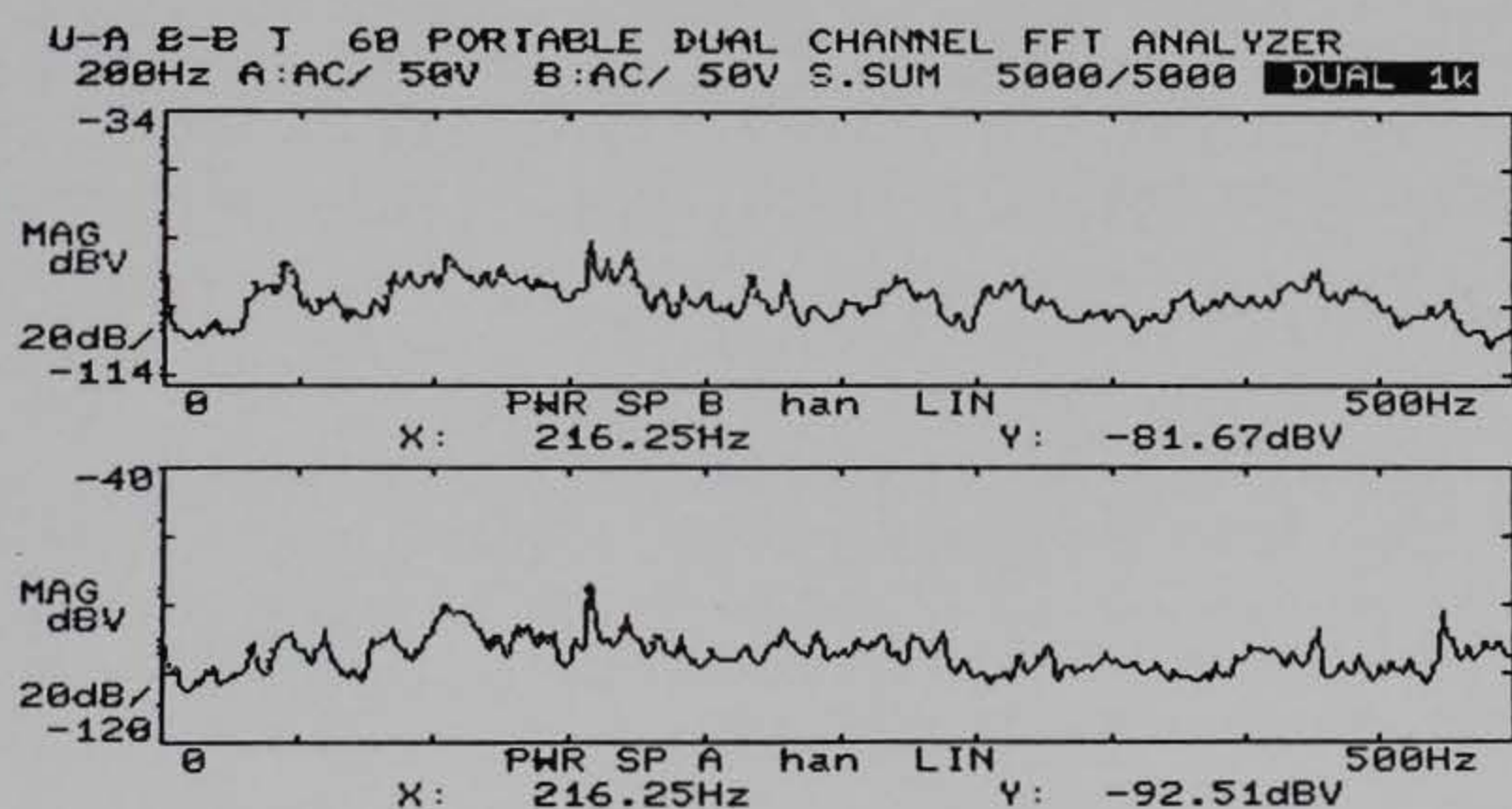
c. Test 3

Figure 20. Power spectra from three stationary tests (F-15, under power, parked in structure); data averaged from 5,000 time segments

Appendix A
T
28 Jul 92



a. Test 1



b. Test 2

Figure 21. Power spectra from two taxiing tests (F-15 moving through shelter); data averaged from 5,000 time segments

Appendix A

Trip Report: Kadena Air Base, 28-31 Jul 92

Corps of Engineers, USAE Waterways Experiment Station	Trip Report: Kadena Air Base, 28-31 Jul 92	Structures Laboratory 3909 Halls Ferry Road Vicksburg, Mississippi
Project: Concrete Failure Analysis, Kadena Air Base, Okinawa, Japan		Date: 17 Aug 92 Harrington

1. BACKGROUND.

a. From 25 Jul to 2 Aug 92, Mr. Wayne Johnson and CPT Patrick T. Harrington, representing the U.S. Army Engineer Waterways Experiment Station (WES), visited the Japan District. The primary purpose of this trip was to measure active impact, ambient, and jet blast dynamic loads on aircraft shelters located at Kadena Air Base, Okinawa. Jet blast load measurements were measured with one F-15 aircraft during normal flight line operations. Two other purposes existed for the WES visit to Japan during the same period, but this report addresses only the air shelter measurements at Kadena Air Base.

b. A site visit and measurements of dynamic response of structures were required to complete an investigation of damaged concrete located primarily in the roof slab panels of the shelters. These measurements contributed to our investigation of mechanisms for crack propagation through roof panels. The measurements were taken on 28 and 29 Jul 92. Coordination for access to the shelters and for use of one F-15 aircraft was provided by the Japan District, Okinawa Area Office, with Air Force officials at Kadena Air Base.

2. SCHEDULE OF EVENTS.

a. At 0830 on 28 Jul, WES representatives arrived at shelter number 30 to begin dynamic property measurements. During the visit measurements were recorded only on shelter 30. Time limitations of WES visitors and mission constraints at Kadena Air Base precluded data gathering from other shelters. Figure A1 shows the south end of the shelter structures. Figure A2 shows shelter 30.

b. On 28 Jul, active impact measurements were taken on two damaged panels. Three panels were measured for this condition, two with and one without cracking. Figure A3 shows the location of each roof panel in the structure measured for active impact with calibrated hammer. Panels 1 and 3 were damaged, panel 1 having a diagonal crack and panel 3 having a transverse crack. Figures A4 and A5 show panels 1 and 3, respectively. On 28 Jul, active impact data were collected on panels 1 and 2. Use of the calibrated hammer for measuring the active impact condition is shown in Figure A6. Before the end of work on 28 Jul, shelter 30 was instrumented for measurement of ambient jet blast conditions scheduled for 29 Jul.

c. Instrumentation consisted of an array of four accelerometers placed on two slab panels (1 and 2), one transverse roof beam (separating panels 1 and 2), and one vertical column (northeast corner of panel 1). Figure A7 shows installed locations for these accelerometers. On 29 Jul, dynamic response to jet blast conditions were measured first. An F-15 aircraft was positioned in the shelter with its engines operating at standard conditions for ground movement. Accelerometer responses from the vibrational forces induced on the shelter were electronically transferred and recorded. Jet blast conditions were measured with the aircraft stationary and with its movement in and out of the shelter. Figures A8, A9, A10, A11, and A12 show activities during measurements of jet blast loads. Ambient conditions were later measured with the same array of accelerometers.

d. Upon completing measurements of ambient and jet blast conditions, panel 3 was instrumented and then measured for active impact data. Figure A12 shows recording of calibrated hammer measurements on panel 3. At 1630 on 29 Jul, measurements for panel 3 were completed, completing all measurements on shelter 30.

3. DISCUSSION.

a. Visual observation of the aircraft shelters confirmed previous assumption that the preponderance of cracking occurred in roof slab panels. Few cracks were observed in roof beams, in vertical support columns, or in fire walls located every fourth shelter. This was based on observations of shelter 38 and immediately adjacent shelters. Continuous cracks were not observed in roof beams, vertical columns, or in fire walls.

b. We observed that an apparently bituminous material had been applied to the top surface of the shelters' roofs. The purpose of the material was to prevent water from penetrating through cracks in roof slab sections and, thus, potentially causing corrosion of steel reinforcement. However, during rain storms on 28 and 29 Jul, water was observed seeping through one crack in shelter 30, apparently penetrating both the bituminous material and portland-cement concrete roof slab. Figure A13 shows water seepage through a crack located in shelter 30.

4. CONCLUSIONS. WES visitors' ability to gather first-hand field data improves the final quality of the original study for the Japan District. The trip to Kadena Air Base was considered successful for this reason and because we were able to observe the entire structure and evaluate first-hand information previously provided in writing. Results and further conclusions of the study will be provided in a later document.



Figure A1. Kadena Air Base aircraft shelters



Figure A2. Kadena Air Base aircraft shelter 30

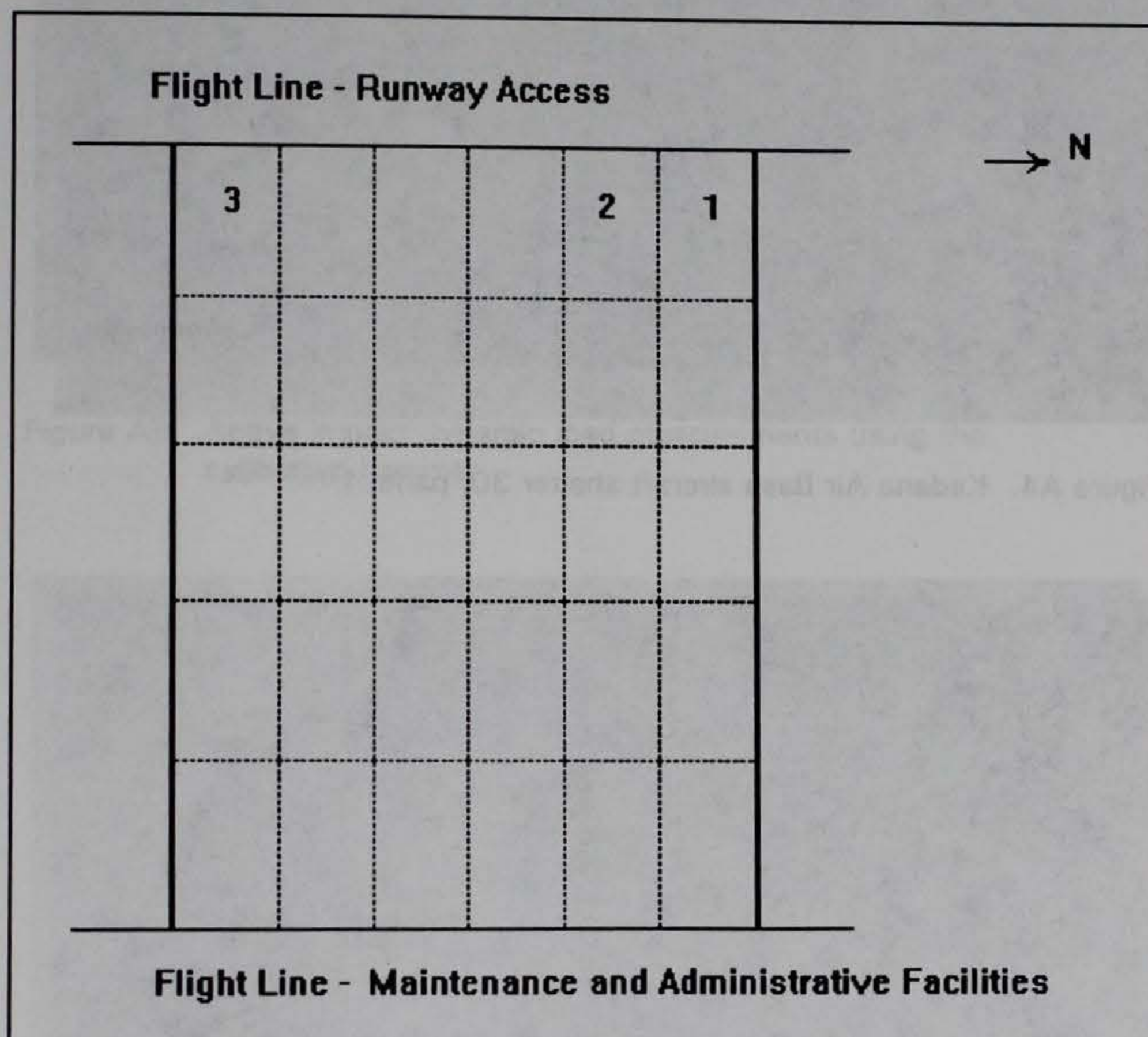


Figure A3. Schematic of measured roof panels, shelter 30



Figure A4. Kadena Air Base aircraft shelter 30, panel 1

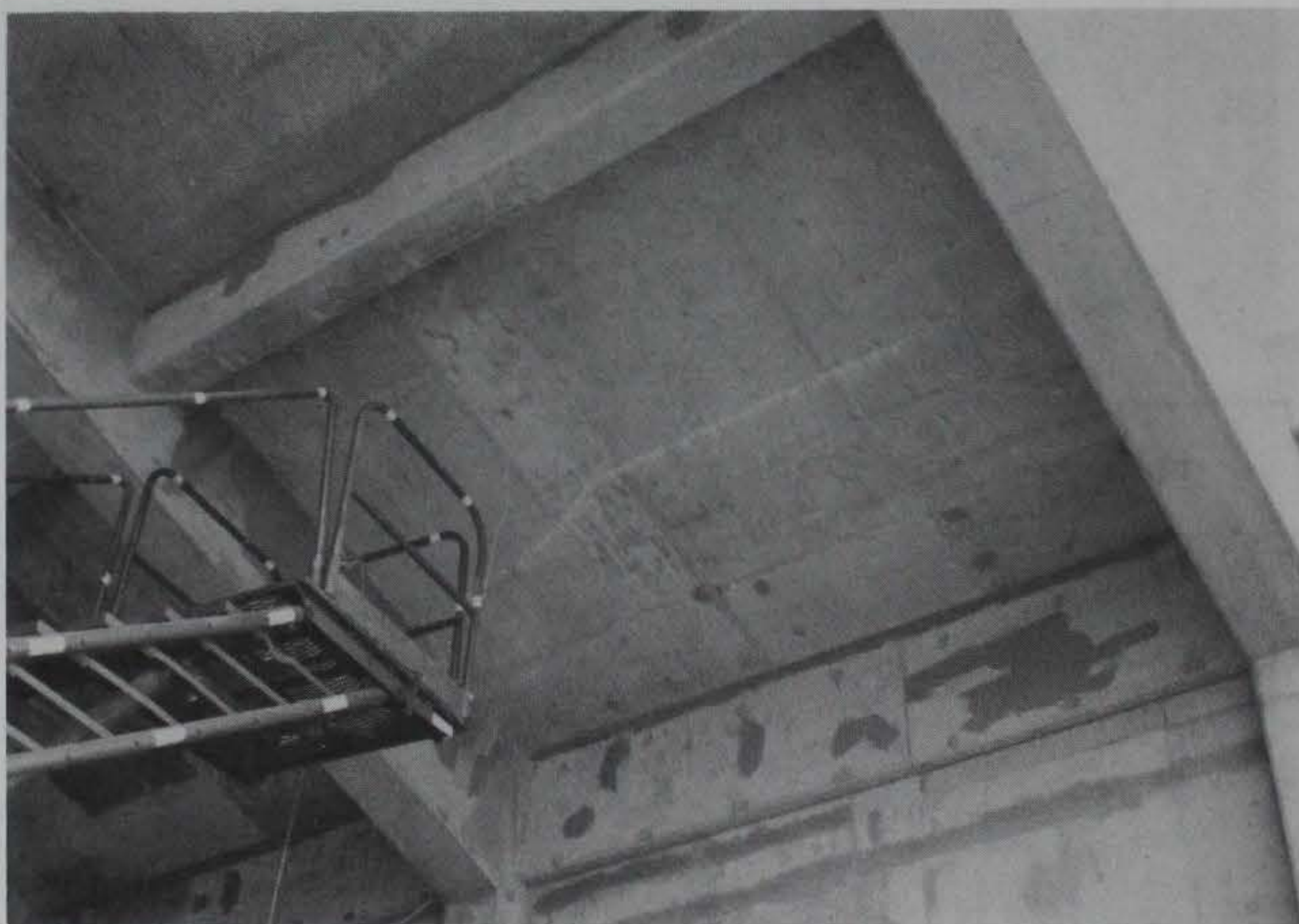


Figure A5. Kadena Air Base aircraft shelter 30, panel 3



Figure A6. Active impact dynamic load measurements using the calibrated hammer



Figure A7. Accelerometers for measurements of ambient and jet blast dynamic loads



Figure A8. Preparation for measurements of jet blast loading

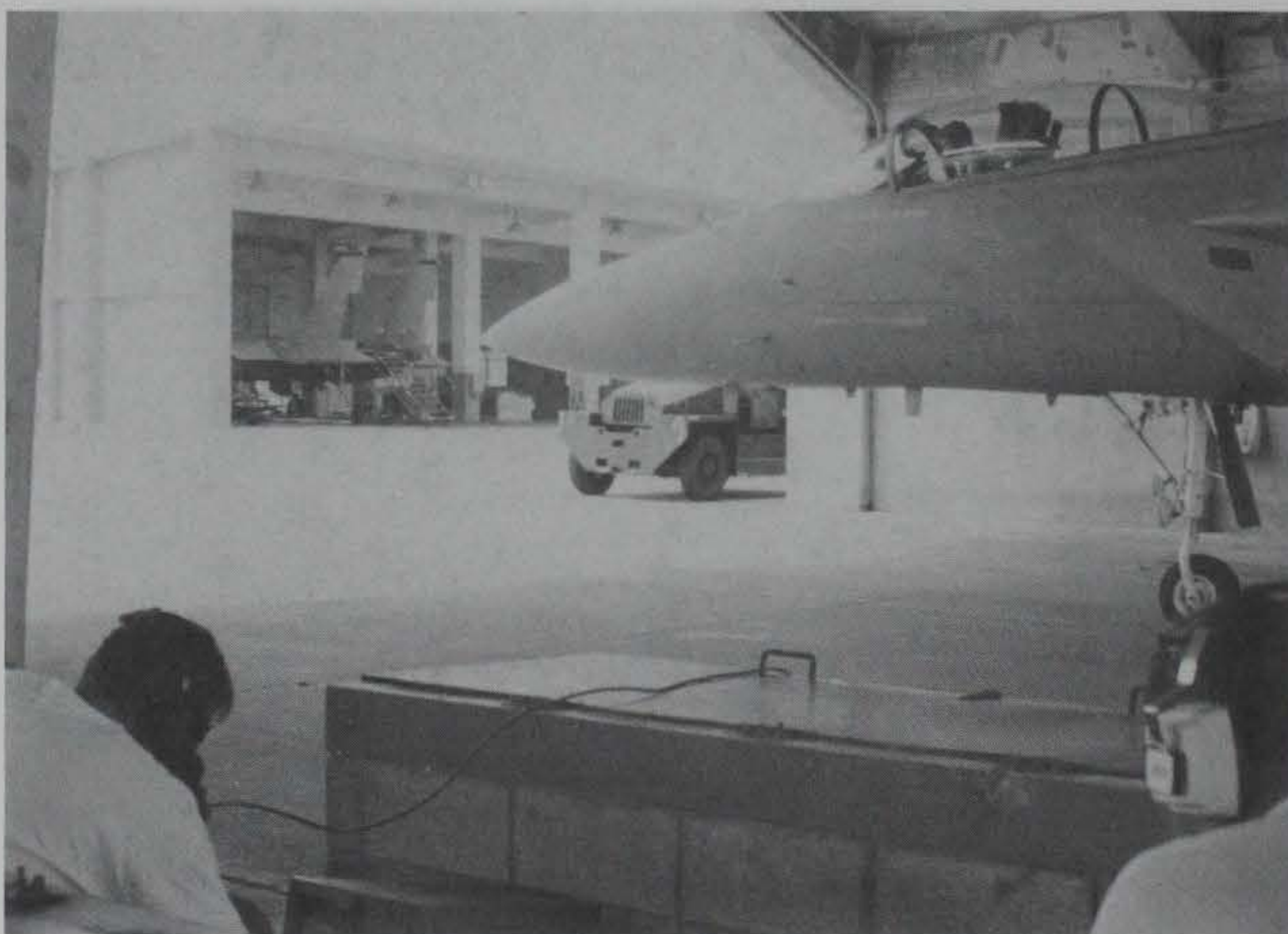


Figure A9. Preparation for measurements of jet blast loading



Figure A10. Data measurements for jet blast loading

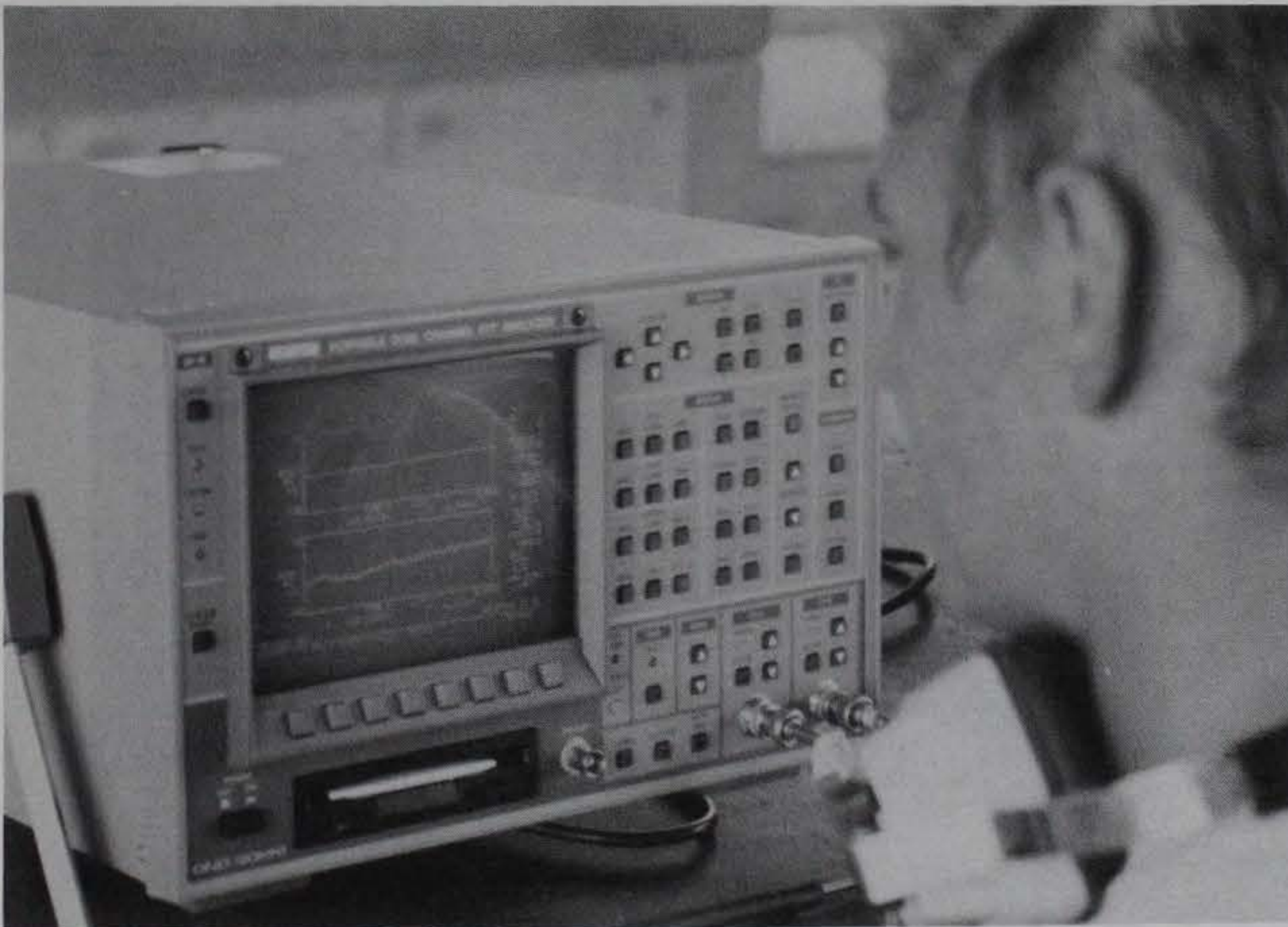


Figure A11. Data acquisition during jet blast loading



Figure A12. Gathering active impact measurements for panel 3

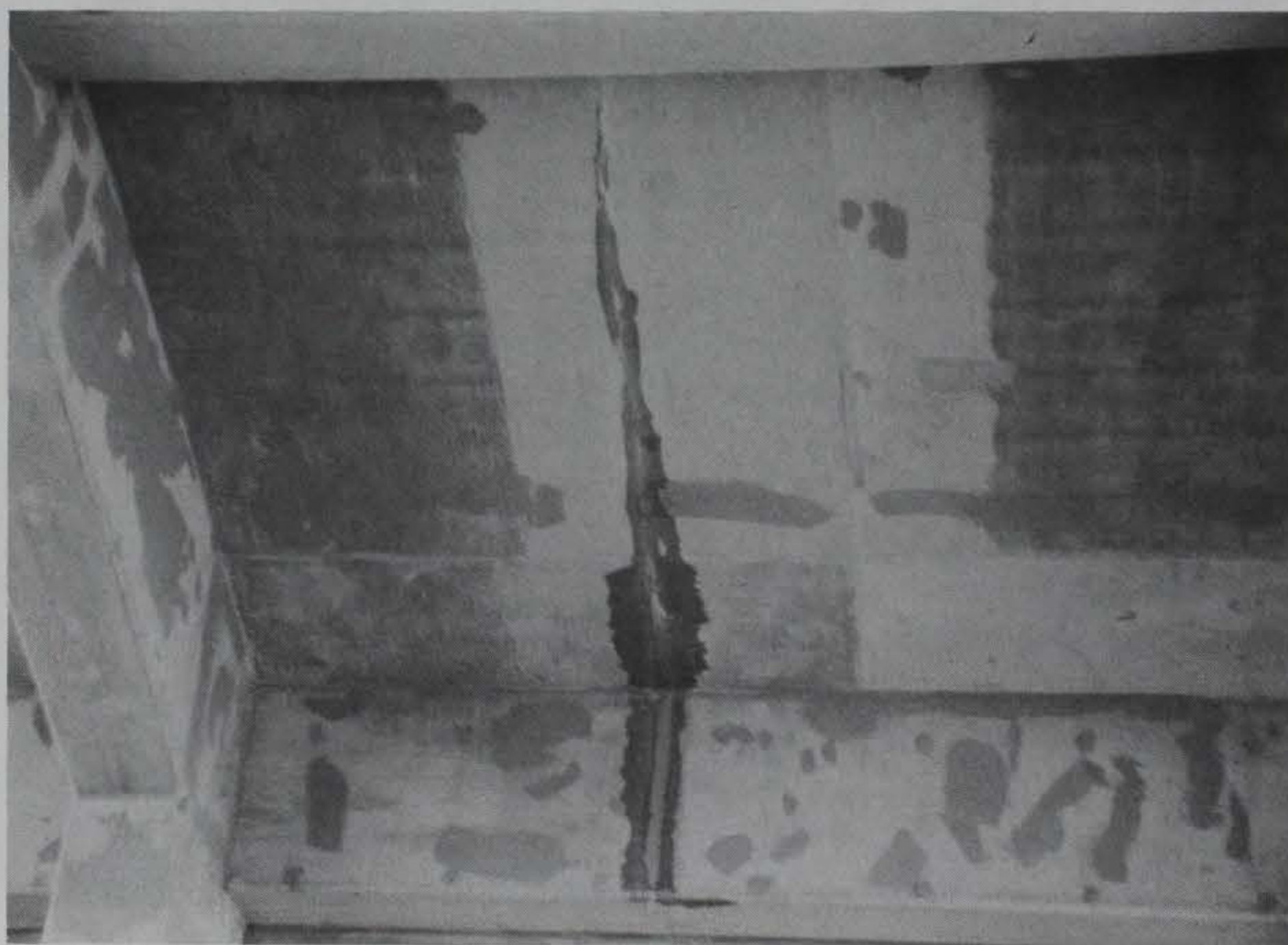


Figure A13. Water seepage, roof slab panel, shelter 30

Appendix B Preliminary Findings Report: 17 Jul 92

CEWES-SC-A (70-1r)

MEMORANDUM FOR Commander, US Army Engineer District, Japan, ATTN: CEPOJ-CD
(Mr. David Wu), Unit 45010, APO AP 96343-0061

Subject: Preliminary Findings, Causes of Cracking in Roof Slabs of Aircraft
Weather Shelters, Kadena AB

1. Reference Memorandum for Dr. Lillian Wakeley, CEWES-SC-A, dated 4 Mar 92, from LTC Larry Talley, CEPOJ-OA, subject: Petrographic Analysis Support; and subsequent memoranda from Mr. Jim Cox, CEPOJ-CD.
2. We studied 13 cores from the subject shelters. The enclosed report summarizes our conclusions about factors that are likely to have contributed to the observed cracking of shelter roof slabs, based on our laboratory studies of these 13 cores and of the supporting information provided by CEPOJ and CEPOD. Results of all tests and observations will be provided in our final report.

FOR THE DIRECTOR, STRUCTURES LABORATORY:

Encl

KENNETH L. SAUCIER
Chief, Concrete Technology Division
Structures Laboratory

CF (w/encl):
Okinawa Area Office, ATTN: CEPOJ-OA-U
(CPT Newman)

Preliminary Findings, Causes of Cracking in Roof Slabs of Aircraft Weather Shelters, Kadena AB

Probable Causes of Crack Initiation

1. We studied 13 cores from the subject shelters. Summarized below are our conclusions about factors that are likely to have contributed to the observed cracking of shelter roof slabs, based on our laboratory studies of these 13 cores and of the supporting information provided by CEPOJ and CEPOD. Results of all tests and observations will be provided in our final report. Our study revealed no evidence that deleterious chemical reactions within the concrete caused or contributed to cracking.

2. It is likely that cracks were initiated early in the life of the structures by drying shrinkage. The following is background information about this crack mechanism, which is explained in ACI 224.1R-3.¹ The volume of hardened portland cement changes with changes in moisture content. The combination of moisture-caused volume changes and restraint of the concrete -- in this case, probably by beams and columns -- causes tensile stresses to develop, which can initiate cracks. Cracks then may propagate at much lower stresses than were required to initiate them.

3. Several types of evidence support the hypothesis that these cracks probably were initiated by drying shrinkage:

a. Most cracks are located where the restraint is greatest, parallel to the column line, and closer to the trough than to the peak of each shelter roof.

b. Cracks were observed within the first few weeks to months after concrete placement. We confirmed by petrographic techniques that cracks in the cores that we studied are older at the top (toward the upper surface of the

¹ American Concrete Institute. (1992). *ACI manual of concrete practice*. Part 3, ACI 224R and 224.1R, Detroit, MI.

slab) and younger with depth in the slab, which is consistent with cracks having initiated by drying shrinkage and then propagated by other stresses.

c. The proportions of the concrete mixture used in this construction made it susceptible to drying shrinkage cracking. According to Mr. Cox, the concrete was pumped upward in each column, and it appears to have been proportioned for ease of pumping. Specifically, the concrete in the cores we studied had a relatively high sand content and a notable amount of material of particle sizes finer than sand (fines) in the paste portion, a water-to-cement ratio of 0.5 or higher, less coarse aggregate than was indicated in mixture proportioning information provided, and small coarse aggregates (apparently 0.75-in. maximum size), all of which contributed to its being pumpable while at the same time making it more susceptible to shrinkage cracking. Also, slump was increased from 3 in., as originally specified, to 4 in.

4. Reference ACI 224R-41, Section 8.6.3, which says in part: "...too often, to expedite pumping, the actions taken are those which increase drying shrinkage and resultant cracking: more sand, more fines, more water, more slump, smaller aggregate." The concrete used in the aircraft weather shelters had all of these characteristics. Pumping of concrete *per se* is not harmful to freshly mixed concrete. Significant amounts of quality concrete are pumped worldwide every day. However, the mixture should not be modified in harmful ways to accommodate the pump.

Specific Observations and Test Results

5. The average value for cement content (ASTM C 1084, of six cores) was about 580 lb/yd³ (345 kg/m³), which is higher than the 330 kg/m³ specified.

6. The average value for volume of permeable pore space or voids in hardened concrete (ASTM C 642, six cores) was 14.5 percent. This is higher than published values (13 percent or less) for volume of permeable voids of similar concrete with w/c=0.5 (Whiting 1988),¹ and suggests that the w/c may have been higher than 0.5. Apparent bleed-water channels were visible along the rebar in some cores. In addition to the 2.5 to 4 percent entrained air (4 percent was specified), up to 3 percent entrapped air was present, much of it as easily recognized large voids. The core with the most obvious water channels along rebar also had the largest air content (> 7 percent), the most capillary porosity (observed during microscopy), and the lowest cement content (Bay 35, core 118).

7. A coarse aggregate volume of 58.0 percent was specified. During our observations of slabs cut longitudinally from cores, we noted less coarse aggregate present than we expect to see and the coarsest fraction missing from

¹ Whiting, D. (1988). "Permeability of selected concretes." *Permeability of concrete*, ACI SP-108, D. Whiting and A. Walitt, ed., American Concrete Institute, Detroit, MI, 195-222.

the upper 3 cm of at least one core. Point counts gave lower calculated coarse aggregate volumes, ranging from 31 to near 44 percent.

Structural Considerations

8. Mr. Wayne Johnson, CEWES-SS-A, developed a mathematical model to investigate the likely damage to the structure from different loading scenarios. His analysis, coupled with study of the AE design report and a map of crack locations, confirmed that deficiencies in the foundation would not have caused the observed cracking.

9. Specifications for the shelter required 2-in. cover of concrete over the reinforcing from both the upper and lower surfaces of the roof slabs, both surfaces being open to ambient conditions. From information provided to us for this study (cores and drawings), we concluded that the slabs were constructed with welded-wire fabric 2 in. down from the upper surface and rebar 2 in. up from the lower surface. But the slabs are only 5 in. thick. This left about a 1/2 in. of clearance between the bottom strand of the wire and the upper edge of the rebar. Effectively, both types of reinforcing were in the middle of the slab. This is not a beneficial configuration for reinforcing slabs. Much of the cracking pattern appears to be a problem of individual slab compartments, extending over the beams in only very few locations. Once initiated, cracks could propagate readily through these effectively nonreinforced slabs.

10. Reinforcing strands are corroded where they are colocated with cracks, but not elsewhere. From this we assume that the reinforcing steel was not corroded before being used in construction. Also, the observed corrosion is minimal, indicating that corrosion occurred along cracks that were initiated by some other mechanism and that cracking was not caused by corrosion of steel.

11. Information provided by Mr. Cox and others indicates that the structures have been subjected to severe weather conditions in their relatively short life span, including large daily temperature fluctuations and hurricane winds. This information, coupled with the location of cracks, the locations of slab reinforcing, and a consideration of the nature of folded-plate structural geometry, suggests that the cracks, once initiated, could have propagated with reversing loads on the structure. WES engineers will explore this possibility during their site visit to Kadena Air Base later this month.

Initial Conclusions

12. Although the strength of test cylinders and cores exceeded the design strength and the cement content was higher than specified, the concrete used in construction of the Kadena shelters was not resistant to cracking. It appears to have been proportioned for ease of pumping in the columns, which

gave it proportions that made it more susceptible to drying shrinkage cracking in the thin and ineffectively reinforced roof slabs.

13. Cracks were initiated soon after concrete placement, most likely by drying shrinkage. Cracks then propagated through the slabs with time. Reversing loads on the folded roof, causing tension along the column lines, is the leading candidate cause for propagation of cracks through the slabs.

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13. ABSTRACT (Maximum 200 words) <p>The U.S. Army Engineer Waterways Experiment Station (WES) investigated the causes of cracking in roof slabs of aircraft weather shelters at Kadena Air Base, Okinawa. Cracking had been noted by the users beginning soon after the concrete portions of the shelters were constructed in 1989. Reinforcing was increased for shelters built subsequently, but the newer shelters also cracked. The WES investigated the potential causes of cracking attributable either to materials and mixture proportions of the concrete or to structural and materials response of the structure to conditions in the environment. Both groups of factors contributed to the observed cracking.</p> <p>The high water content, aggregate grading, and curing procedures made the roof-slab concrete more susceptible to drying shrinkage cracking, which apparently initiated most of the observed cracks. Subsequent propagation of cracks is attributed primarily to flexure along the column lines. The materials studies revealed no evidence of ongoing degradation of materials. Dynamic and static analyses likewise indicate that the shelters have not experienced significant structural deterioration.</p>				
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